

SOILS and ROCKS

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LECTURE

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The 9th Victor de Mello lecture: reflections and speculations

David William Hight^{1#} 

Lecture

Keywords

Localisation
Steady and critical state
Instability
Anisotropy of collapse potential
Fabric
Spatial variation in local void ratio
Moist tamped and sedimented samples
Chain reaction

Abstract

The paper reviews the published literature on some aspects of fabric and particle behaviour in cohesionless soils. It uses insights from Discrete Element Modelling and Microcomputed Tomography to speculate on reasons for the difference in behaviour observed between moist tamped and sedimented sands at the same global void ratio and stress state. It is suggested that inhomogeneities in the form of macrovoids in moist tamped samples trigger localisation, collapse and the development of a local scale chain reaction. It questions whether a similar sequence applies at the field scale and whether expansive partial drainage influences the rate of propagation of the chain reaction. The paper offers reasons why the use of critical state based on moist tamped samples to assess the stability of tailings, may not be the best approach, and suggests that identifying instability lines for sedimented samples at appropriate void ratios and principal stress directions, may be preferable. Critical state reflects behaviour at large strains where initial and evolving inhomogeneities affect identification of relevant void ratios; instability lines reflect behaviour at small strains where inhomogeneities probably initiate collapse. The paper emphasises the importance of spatial variations in local void ratios in loose material, the importance of anisotropy of collapse potential and of load-controlled rather than strain-controlled shearing.

1. Introduction

I met Victor de Mello in March 1977 when he spent time at Imperial College, prior to delivering his Rankine Lecture, a lecture that demonstrated his depth of practical and theoretical knowledge, his enthusiasm and his stamina. My recollection is that the lecture lasted 2 hours 20 minutes; the previous dry run, I understand, took 4 hours.

Unlike most of the previous de Mello Lecturers, I have not had the privilege of working with Victor, although we exchanged correspondence about my work at the Bothkennar soft clay test site in the early nineties. I have, however, had the great pleasure of working with and becoming a close friend of his son, Luiz Guilherme de Mello, more than ‘a chip off the old block’ from whom I have learnt much about his father. Of course, I also have the huge benefit of access to the amazing book on Victor (Figure 1), assembled by Luiz and his sister, Lucia. I also have the benefit of the anecdotes and Victor’s quotations presented by the previous Lecturers.

1.1 Reflections

The photograph on the cover of Victor’s book, showing him in reflective mood in his reflection in the window of the train carriage, first gave me the idea of ‘reflections’ for my title. Then I remembered the use of ‘reflections’ in the title

of Victor’s Rankine Lecture and in the title of John Burland’s First De Mello Lecture in 2008.

In his De Mello Lecture, Michele Jamiolkowski provides the following quote from Victor, ‘*We professionals beg less rapid novelties, more renewed reviewing of what is already there*’. Like Michele, I use that as the theme of this presentation. It gives me the excuse to go back to my Rankine Lecture in 1998 and, using research information published by others, reflect on some of the developments over the last 25 years, particularly in the area of particulate soil mechanics.

1.2 Speculation

Looking back at my Rankine Lecture, I now realise how much I speculated about the explanation of laboratory and field observations on the basis of fabric - now referred to as ‘*explaining macro observations on the basis of micro or particulate mechanics in multi-scale analyses*’. These speculations meant that I strayed unknowingly into areas occupied by specialists, to whom I apologise for the past and present.

My speculating days should be over because of developments in discrete element modelling (DEM) and microcomputed tomography, which eliminate the need for speculation. However, this lecture gives me the opportunity to speculate one last time, using some of the insights from

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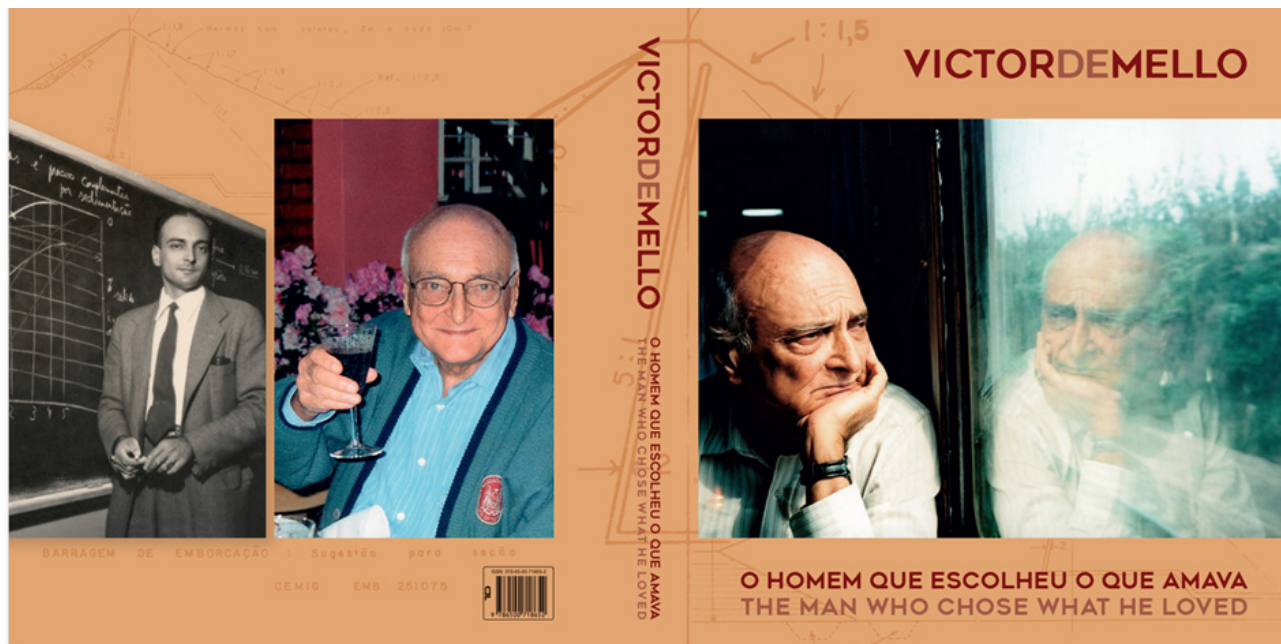


Figure 1. Reflections – inspired by the cover of Victor’s biography and the title of his Rankine Lecture.

a review of publications on DEM and tomography. I begin with this review before reflecting and speculating.

1.3 Context

Two quotations, one from Victor that is taken from his paper to the International Dam Congress in 1980, and one from Reid & Fanni (2022), provide some context for the presentation.

De Mello (1980): *“To sum up, I allow myself to say that in the case of water dams our experience has been so positive that the future invites the indispensable optimization of economics. On the other hand, when it comes to tailings dams, very justified are the concerns that, with time, the raises, and the multiplication of diverse cases, the invitation to unpleasantness and failures is open, and with a free ticket. We are only at the start, which favours us: let us do the re-considerations and revisions that the subject, and the gravity of concern, demand”.*

On the basis of the recent history of tailings dam failures, it seems that the necessary re-considerations and revisions have not been done, so what could be improved?

Reid & Fanni (2022): *“The critical state approach, based heavily on the use of the moist tamping (MT) reconstitution method, is finding increasing application in tailings engineering practice”.*

This statement invites the question – is this really the wisest approach?

In a setting of flow failures and tailings I will reflect and speculate on: localisation, critical state, anisotropy of undrained strength and instability in sedimented sands, void

ratio and its spatial distribution, moist tamped samples and chain reactions.

1.4 Definitions

Some definitions first, referring to Figure 2: the angle between the vertical and the direction of the major principal stress is alpha; the relative value of the intermediate stress, σ_2 , is expressed by the parameter b; in a triaxial compression test, alpha is 0 and b is 0; in a triaxial extension test alpha is 90 degrees and b is 1.

2. Discrete Element Modelling, DEM

Discrete element modelling, DEM, predicts the displacements and rotations of discrete elements which represent idealised rigid soil particles. It provides information on forces at particle contacts and so on the development of strong force chains/load columns. It enables explanations in terms of particulate/microscale behaviour to be put forward for macroscale observations. This has particularly been the case for laboratory tests on soils, providing reassurance on validity of previously controversial observations such as: phase transformation; the effect of the intermediate stress ratio, b, and of the major principal stress direction, alpha, as identified in the hollow cylinder apparatus (HCA); instability and critical state. DEM results support intuition and correct speculation.

DEM permits exploration of behaviour under conditions that are not achievable in the laboratory, particularly at critical state. It enables comparisons to be made between simulated

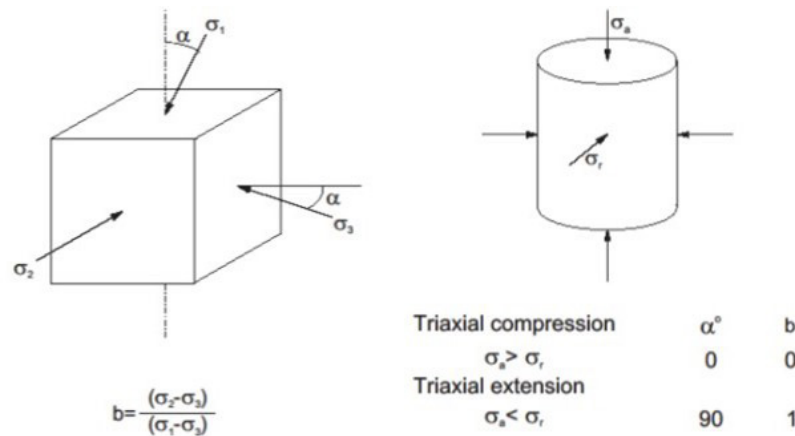


Figure 2. Definitions.

experiments on identical samples that use perfect boundary conditions and imperfect rigid and flexible boundaries that introduce boundary restraints; from these, information on strain localisation is obtained, supplementing that from microcomputed tomography.

Modelling realistic particle shapes is a challenge.

2.1 DEM - force chains

Force chains are networks of contacting particles that are relatively highly stressed (carrying forces higher than the average contact force) and aligned in the direction of the major principal stress (Figure 3). DEM findings confirm earlier investigations using photoelastic discs (e.g. Oda & Konishi, 1974; Drescher & de Jong, 1972).

As a material is loaded, these force chains collapse or buckle and new force chains are formed. A compact particle system, due to its highly redundant nature, quickly adapts to new loading directions by selecting new spatial distributions of favourably orientated contacts for transition of the forces in the strong sub-network. **This is presumably not the case in a loose particle system where buckling is more likely to lead to premature failure.** Key insights and speculations that are relevant to the theme of the paper are highlighted in bold for ease of reference.

DEM analyses show that the transition from sliding to rolling behaviour at the contact points and resistance to buckling increase as the friction assumed at the contacts increases.

2.2 DEM - force chains and the effect of b (O'Sullivan et al., 2013)

The value of b influences the force chains that develop because of the impact of the intermediate principal stress on the lateral support and resistance to buckling it provides to the force chains that are aligned along the minor and intermediate stress directions. This is illustrated in

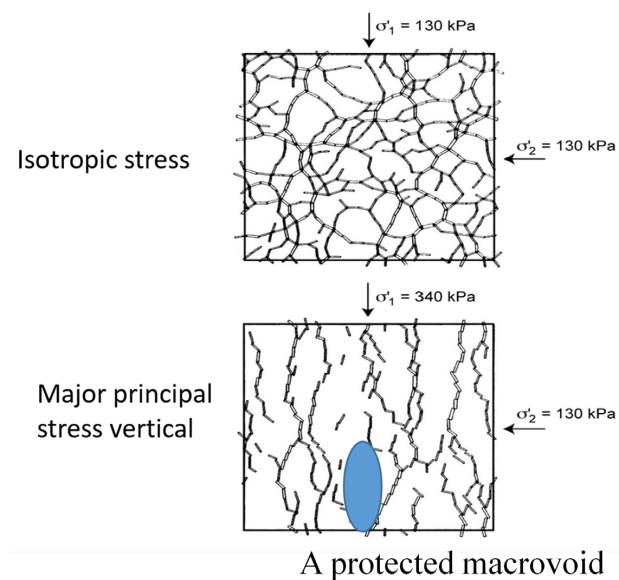


Figure 3. Force chains in the direction of the major principal stress. 2D DEM. Modified from Dobry & Ng (1989).

Figure 4 from Barreto (2009) which shows the force chains that develop with different values of b . The main force chains, that transmit the majority of the load and are aligned along the major principal direction, are supported by secondary force chain structures that stabilise the system.

2.3 DEM - the quasi-steady state and phase transformation

In the 1970s, learned professors were instructing bewildered research students to repeat undrained triaxial compression tests on loose materials that showed what we now refer to as Phase Transformation (Ishihara et al., 1975) or Quasi Steady State – these results did not fit into the understanding of soil behaviour at that time.

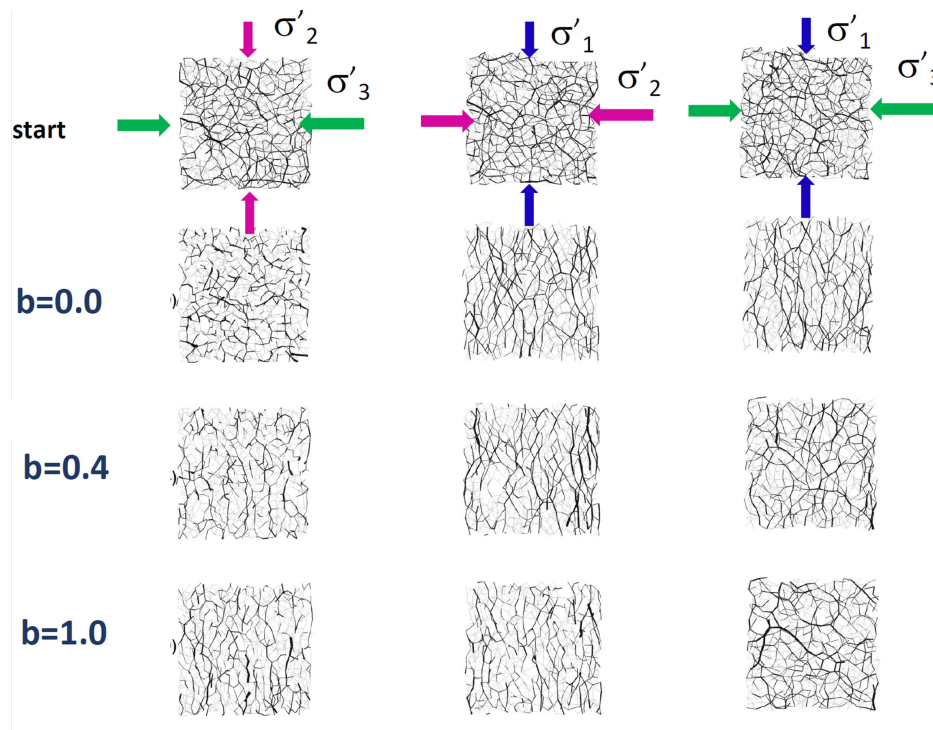


Figure 4. Dependence of force chains on the value of b . From Barreto (2009).

The 2D DEM study by Yang & Dai (2011) showed that the quasi-steady state reflected real behaviour rather than a test-induced phenomenon. It is a transitional state, in which spatial rearrangement of discrete particles, sheared under the constant-volume condition, leads to the average number of contacts per particle decreasing to a minimum value of around 4 before increasing gradually and after that to an approximately constant value at large deformations associated with the steady state. At the steady state deformation continues at constant volume, constant mean effective stress, constant shear stress and constant velocity.

3. Microcomputed tomography, XCT

X-ray microcomputed tomography imaging allows the evolution of the three-dimensional microstructure/fabric of a small sample of sand to be followed while it deforms, by measuring the attenuation of X-rays that pass through the sample. Individual grains can be distinguished in time-lapse 3D images and the full 3D kinematics of all the grains (i.e. sliding, rolling etc.) can be measured. Current limitations relate to specimen size and complexity of the loading system. I shall refer to the insights that it has provided on strain localisation and particle movements during 1D compression and during shear.

Figure 5a shows a typical image obtained using XCT to observe deformations of a dry non-cemented uniform angular sand (Hostun HN31 sand $D_{50} = 0.328$ mm) under 1D oedometric compression loading at 2500 kPa; 3D translations

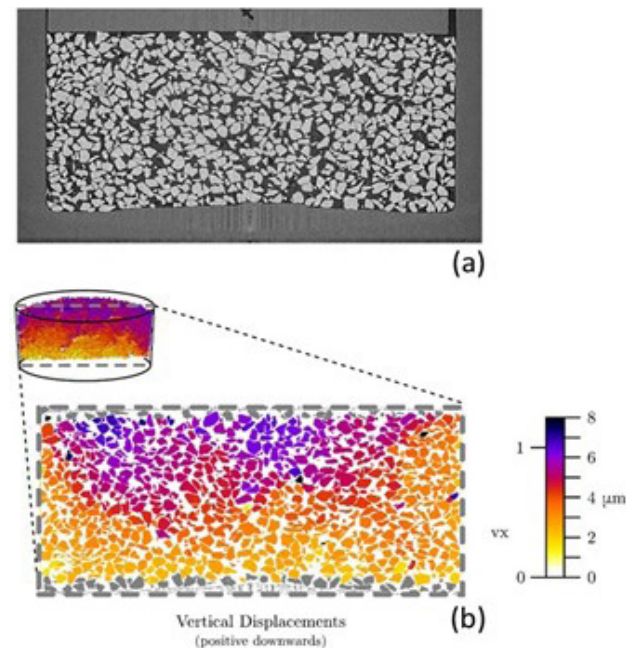


Figure 5. (a) vertical slice before creep phase; (b) vertical displacements of grains over the creep phase. Modified from Ando et al. (2019).

and rotations of grains were tracked. Figure 5b is a vertical slice through the sample in which grains are coloured according to the vertical component of their displacement during on-going creep. **Note the variation in pore size and shape, the range**

of particle shapes, the variation in displacements on any horizontal surface and the boundary effects.

3.1 Microcomputed Tomography: particle movements during shear of natural fine sand (Garcia et al., 2024)

XCT provides extremely valuable insights into particle displacements during shear, including, importantly, natural sands. The following description is taken verbatim from Garcia et al. (2024) who performed triaxial compression of a sample from a shoal of fine sand. In this description, as usual, coordination number is defined as the average number of contacts per particle.

“Up to the mobilisation of the peak strength, the grains undergo a relatively small amount of rotation. The volumetric compression during this pre-peak phase manifests itself by increases in the grain coordination number throughout the entire shoal sample. The initial volume-change behaviour turns to dilation as the grains begin to rotate in the zone of the eventual shear band and the sand approaches the peak strength. Following the peak stress, the shear band develops through a mechanism of highly localised grain rotations. Volumetric expansion continues during the strain-softening phase and shear band development, and the number of contact points between adjacent grains and areas of each contact within the shear band decreases. At the same time, the grains orient toward a unique critical state shearing

fabric characterised by the long grain axes aligning parallel to the direction of shearing and pointing on the order of 15° to 20° above the shear plane”.

3.2 Microcomputed tomography and particle rotation

The development of particle rotation as shearing proceeds has been very neatly captured in XCT experiments on an angular sand undergoing triaxial compression (Alshibli et al., 2015). Figure 6 shows that some particles rotated more than 20 degrees from the loading direction, with a higher rotation about the short diameter of the particle than its long diameter.

Of particular interest to the theme being developed in this paper is the observation that there was “[...] a high variation in local dilatancy distribution where some particle groups exhibited high dilatancy angles while others showed negligible dilation or contractive behavior”. In more detail, some particle groups “[...] show a drastic change from one strain increment to another which is caused by neighbor particle interactions that very often force a collapse of a large void between particles which results in a generation of a high volume change in another particle group”. “[...] very angular sand particles exhibit a heterogeneous spatial distribution of particle translation, rotation and dilatancy angle”.

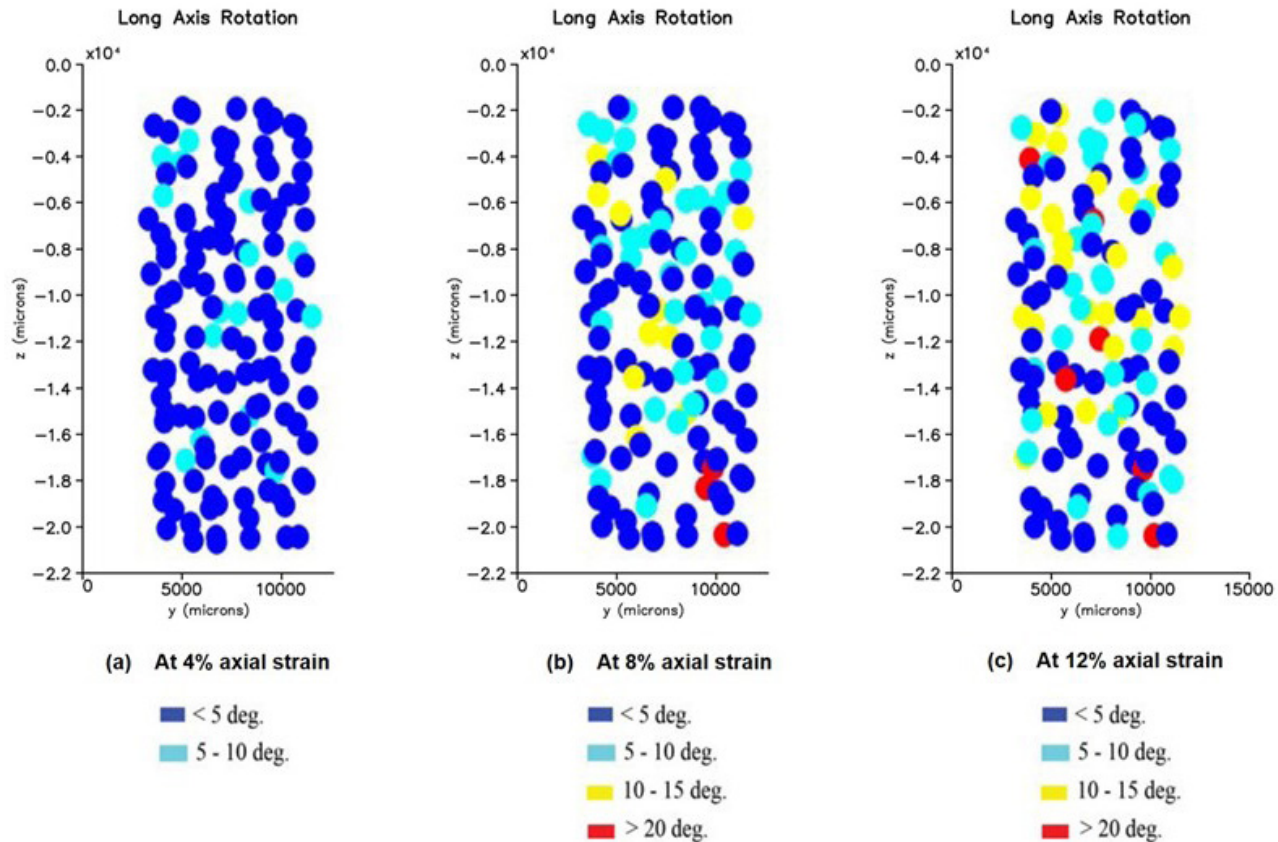


Figure 6. Cumulative values of orientation of particle long diameter measured from the direction of the global major principal stress. Modified from Alshibli et al. (2015).

3.3 Microcomputed tomography and validation of DEM

Pan et al. (2023) present the results of triaxial compression tests on green lentils prepared with different inclination angles at deposition, and so with different initial anisotropic states, as the anisometric particles settle with their short axis parallel to gravity. The tests were run with X-ray tomography imaging.

DEM simulations of the triaxial tests were run with particles in a thin layer near the top and bottom walls restricted from horizontal displacement to simulate the constraint caused by the frictional caps and adjacent membrane in the triaxial experiments. By simulating the particle shape, initial state, (including similar distributions of local void ratio and boundary conditions of the physical experiments, and with careful selection of contact parameters, good agreement was found between DEM predictions and experimental observations, in terms of macro scale stress-strain response, meso-scale strain localization and micro-scale fabric anisotropy evolution. Examples of the similarity of incremental deviatoric and volumetric strains are shown in Figure 7.

4. Localisation of strains

The results in Figure 7 provide perfect examples of strain localisation in cohesionless soils undergoing triaxial compression in tests with end restraint, and illustrate the value of both microcomputed tomography and DEM in investigating the phenomenon.

4.1 Localisation in laboratory tests.

Desrues et al. (1996, 2018) make the following important and profound statements on the subject of localisation, based on their work with microcomputed tomography imaging of triaxial compression tests on dry sands:

“[...] strain localisation is unavoidable and is ineluctable and omnipresent when sand is sheared”.

“[...] all other tests analysed so far, show the same pre-peak structuration of the deformation process, and this in a variety of different experimental conditions, initially dense and loose specimens, slenderness ratios of 1 or 2, with or without end lubrication”.

“[...] localization structures inside the specimens were clearly revealed by the density maps in the dense case. Conversely, in the loose specimens it was difficult to observe localization structures, because the density in the localized strain zones did not change significantly”.

“[...] Even when taking all possible precautions to perform the “cleanest” (i.e., most mechanically perfect) triaxial tests, which appear to have no external sign of strain localisation, the ability to look inside deforming specimens revealed complex structures of localised changes of porosity whose integrated kinematics appear externally as a uniform deformation [...]”.

Strain localization in triaxial tests can occur in different localization patterns depending on test conditions, such as end restraint, platen rotation, eccentricity in vertical alignment, loading directions and mixed boundaries. The use of lubricated ends and 1:1 sample height appears to delay the onset of localisation in triaxial compression tests.

Figure 8, taken from Pan et al. (2023), compares the incremental deviatoric strain distribution in DEM specimens

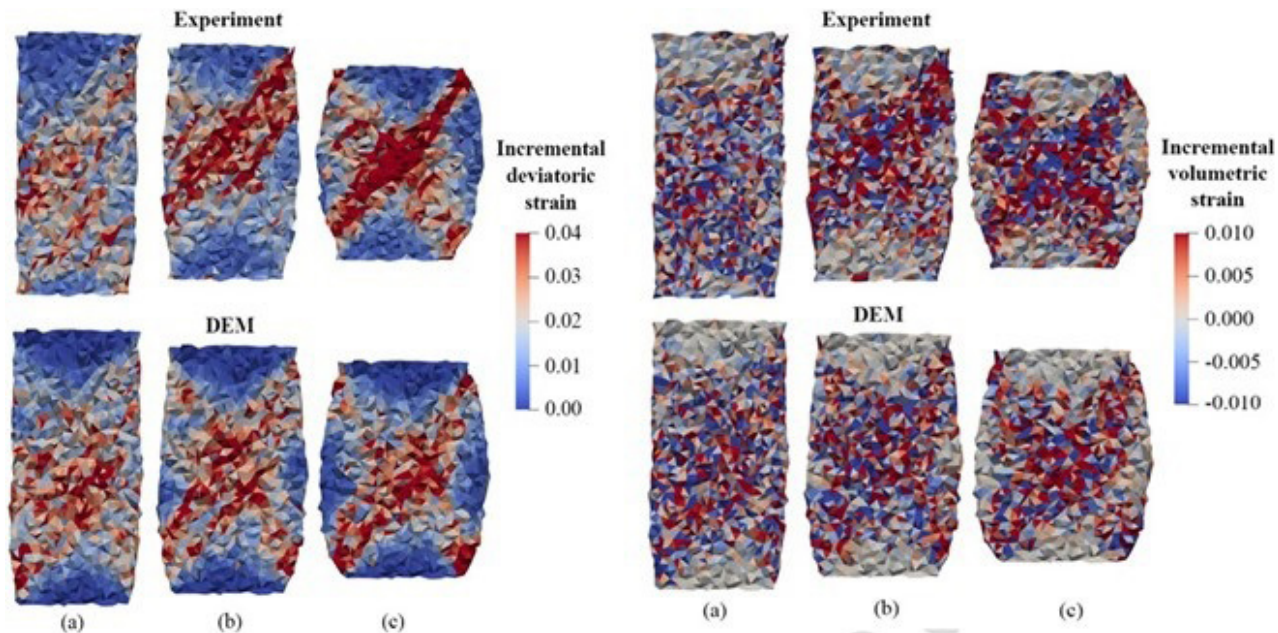


Figure 7. Experiment and simulation: local incremental deviatoric strain and local incremental volumetric strain in $\delta = 60^\circ$ specimen between axial strains of (a) 4 to 5%; (b) 14 to 15%; (c) 29 to 30%. Modified from Pan et al. (2023).

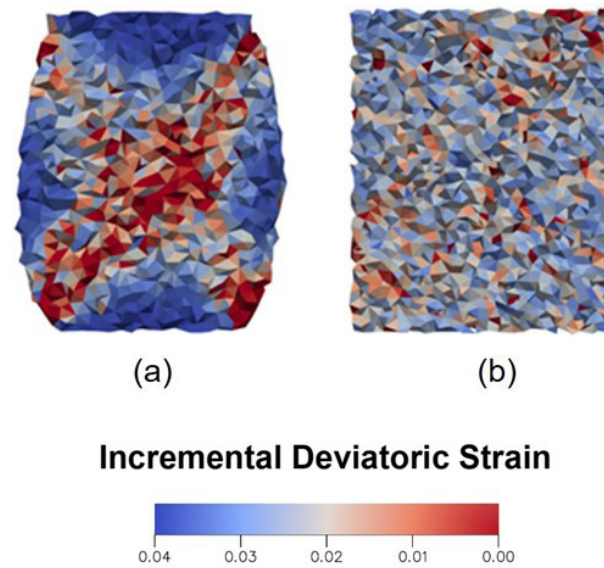


Figure 8. Incremental deviator strain distribution for the $\delta = 60^\circ$ specimen between 29% to 30% axial strain with (a) ‘force line’ boundary; (b) rigid wall boundary. Modified from Pan et al. (2023).

between 29% to 30% axial strain, one with flexible lateral boundaries and one with rigid wall boundaries. Pan et al. (2023) explain that “[...] *explicit conjugate shear bands are observed with the flexible boundaries, whereas for the rigid wall boundary, the strain field is diffuse even at large axial strain*”. The initial stiffness is increased and the peak strength is reduced with rigid wall boundaries and this is mostly attributed to the difference in the constraint enforced on particles near the boundaries.

The changing constraints on particle movement when moving from triaxial compression to plane strain compression, increasing b from zero, appears to result in shear banding becoming visible externally at an earlier stage in the shearing.

Localisation in hollow cylinder tests is dealt with later after introduction of relevant terms.

4.2 Localisation triggers

Santamarina & Cho (2003), who also regard localisation of strains to be omnipresent in particulate materials, consider localisation to be promoted by: “[...] *drained dilation, contractive tendency in undrained loading, imposed strain non-uniformities, boundary conditions, heterogeneity in situ or in laboratory prepared specimens*”; and to be triggered “[...] *by cavitation in undrained dilative shear, platy particle orientation, bond failure in cemented soils, menisci failure in unsaturated soils, and possibly particle crushing*”. **Heterogeneity in situ or in laboratory prepared specimens includes initial and evolving spatial variations in void ratio.**

Grain rotations appear to be key for detecting the onset of localisation.

4.3 Implications of strain localisation

- Strain localisation in triaxial compression tests involves bands of 10 to 15 grains width, which dominate the material’s macroscopic response and lead to global failure.
- Localisation means that tests cannot be analysed as uniform elements. The concept of strain as an average quantity is invalid.
- According to experimental results by Saada et al. (1999) using a digital image processing technique, when the global shear strain is 8%, the local shear strain in the shear band is 160%. “*Therefore, localization in drained tests restricts the determination of large strain volume data but facilitates the determination of the large strain strength*” (Santamarina & Cho, 2003).
- Measured parameters become specimen-size and boundary condition dependent.

4.4 Localisation in dilatant soils

- In dense soils, dilatancy concentrates in the shear band, causing weakening in drained and in slow undrained tests when pore water is drawn into the shear band. **In fast undrained tests pore water migration does not occur and undrained strengths are higher** (Sandroni, 1975).
- In drained tests involving dilation, decreasing stability is accompanied by reducing coordination number (Yang & Dai, 2011).
- Strain localization prevents the global dilation of a dense specimen.

4.5 Localisation in contractant soils

So, localisation does develop in loose contractant soils but is difficult to observe and is self-healing in drained and slow undrained shearing, when locally higher pore pressures can dissipate out of the shear band. **Under rapid undrained loading, the shear band cannot drain, straining is concentrated in the band and strength is reduced.**

4.6 Localisation in the field as opposed to the laboratory

Strain localisation in contractant and dilatant plastic clays are readily observed in the field. This is not the case

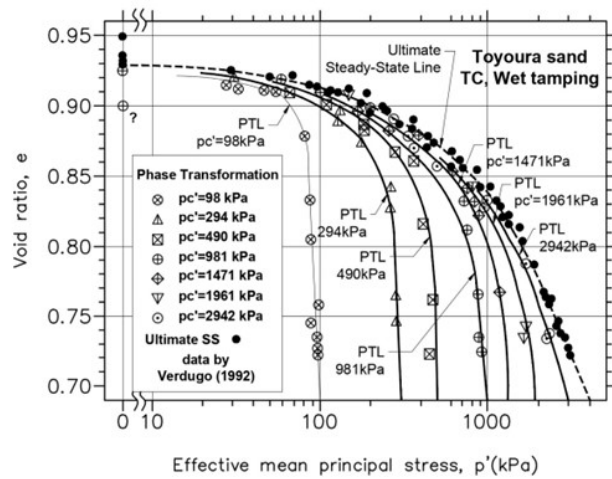
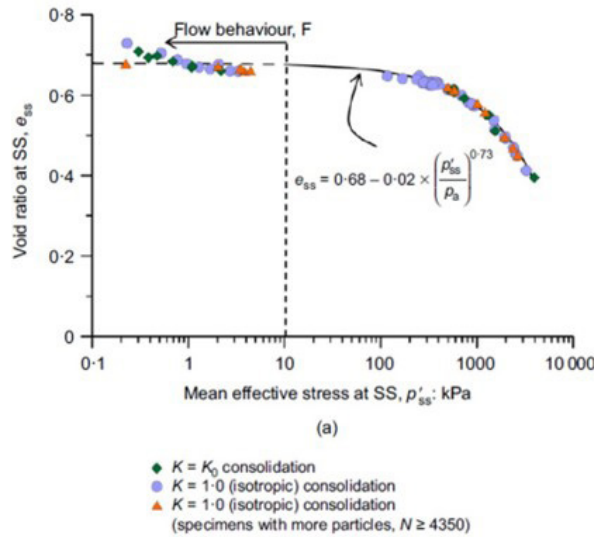


Figure 9. Phase transformation and ultimate steady state in triaxial compression. Modified from Verdugo & Ishihara (1996).



in cohesionless soils and there are few examples of in situ localisation being reported. This begs the question, is localisation in a laboratory test representative of localisation and progressive failure in situ?

5. Steady/critical state

5.1 Steady state/critical state. Is it non-linear in e-log p'?

Steady state was defined herein as the ultimate state achieved under undrained conditions, whilst critical state was defined as the ultimate state achieved under drained conditions. This distinction does not appear to have been retained in the literature and the terms will be used interchangeably herein.

One of the most complete investigations of steady state and phase transformation in undrained triaxial compression of sands (Toyouka sand) was carried out by Verdugo (1992) and reported by Verdugo & Ishihara (1996). Verdugo's work, refer to Figure 9, showed clearly that **in e-log p' space the steady state was non-linear, with marked curvature of the steady state line (SSL) at low stresses.** This level of non-linearity has also been found in investigations of the Stava tailings (Carrera et al., 2011) and has been confirmed in DEM studies, see, for example, Figure 10 from Nguyen et al. (2017).

Verdugo also showed that phase transformation became less likely to occur at high global void ratios and high stresses. Note that Verdugo prepared his samples using 'wet tamping', and Carrera et al. by 'moist tamping', with comparisons using wet and dry pluviation.

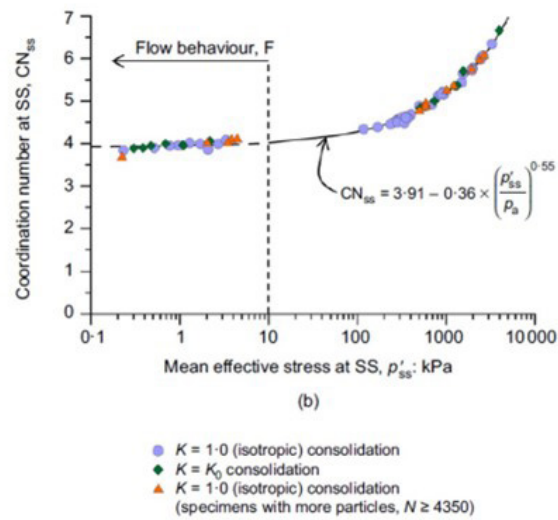


Figure 10. Steady-state data points obtained from isotropic and Ko-consolidation simulations: (a) e-log p' space; (b) CN-log p' space. Modified from Nguyen et al. (2017).

5.2 Steady/critical state. Does it depend on b ?

Force chains have been shown to exist at critical state, see, for example, Figure 11 from Zhao & Guo (2013). On the basis of the discussion put forward in Section 2.2, on the dependence of force chains on b , **It is reasonable to assume that critical state depends on b , as suggested by Chu & Wanatowski (2008), refer to Figure 12. It is also reasonable to assume that there is simultaneous dilation and contraction at critical state as force chains continuously form – collapse – and re-form.**

5.3 Steady/critical state. Is it attainable? - Localisation

It follows from the discussion on localisation that a critical state cannot be defined using boundary measurements of volume change after the development of shear bands. Using microcomputed tomography, however, Desrues et al. (1996) showed that it was possible to demonstrate that a critical state developed in the shear bands in triaxial compression tests on sands (Figure 13), stating: ‘Critical state is particularly well-verified inside the observed localised band of deformation, rather than the average of a complex strain field.’

To minimise the effects of localisation, several authors recommend that the most reliable test to determine critical state parameters should be on homogeneous contractive specimens subjected to drained shear. In these, localisations are self-healing.

5.4 Steady/critical state. Is it attainable? - Layering

Many sedimented cohesionless soils with a range of particle sizes, including tailings, are layered. Figure 14a, from Yoshimine & Koike (2005), illustrates a layered or

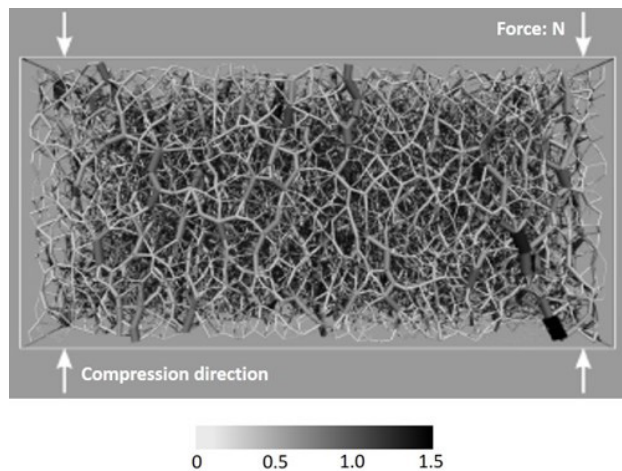


Figure 11. Force chains at critical state in DEM simulation of triaxial compression of a medium dense sand. Modified from Zhao & Guo (2013).

stratified fabric, which when reconstituted in the laboratory, will have a totally different fabric, two examples of which are shown in Figures 14b and c.

Using a well-graded but clean sand that was sieved and separated into four components with different particle size ranges, Yoshimine & Koike (2005) created layered specimens by sedimenting each component successively. Figure 15a illustrates the resulting fabric. Uniform homogeneous specimens having the same grading were also prepared (Figure 15b). Undrained triaxial compression tests were run on both types of specimens and the steady state and phase transformation lines were established for each (Figure 15c).

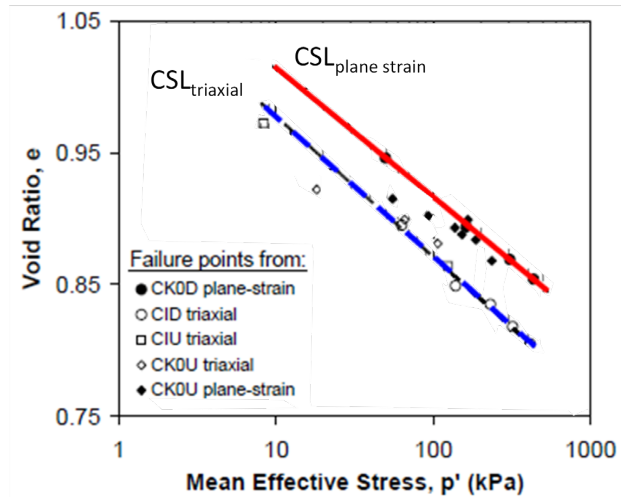


Figure 12. Undrained triaxial and plane strain compression of moist tamped Changi sand (CSL: critical state line). Modified from Chu & Wanatowski (2008).

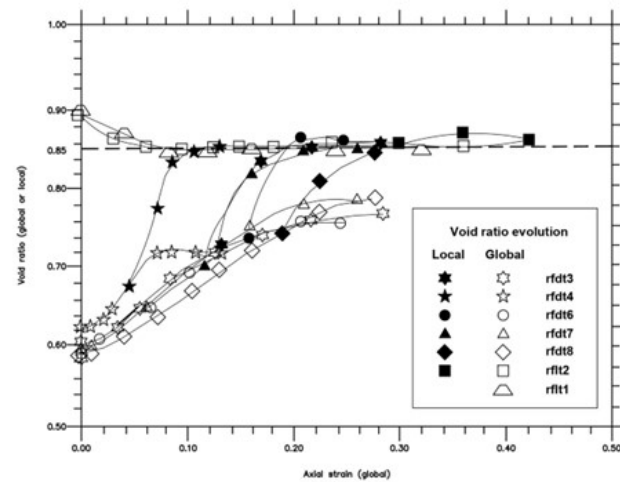


Figure 13. Global and local evolution of the void ratio in loose and dense Hostun RF sand specimens submitted to axisymmetric triaxial test under 60kPa effective confining pressure. Modified from Desrues et al. (1996).

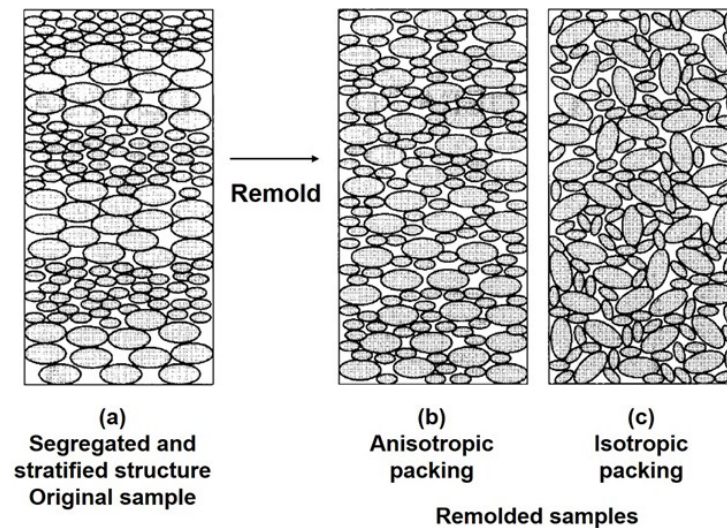


Figure 14. Illustration of different soil particle packing. Modified from Yoshimine & Koike (2005).

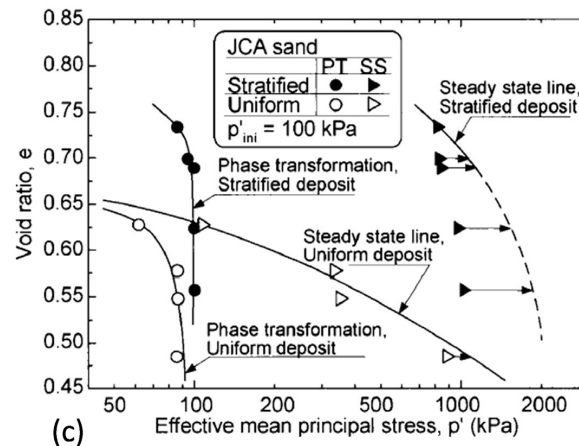
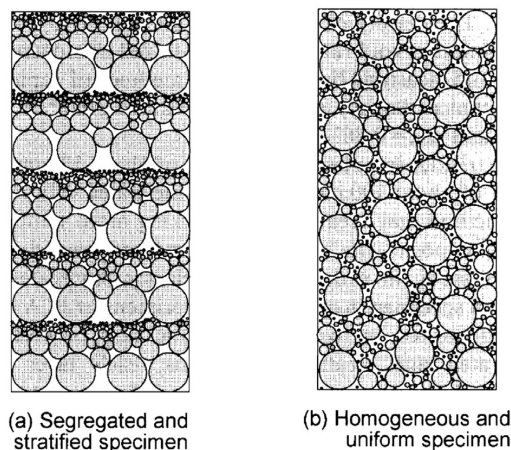


Figure 15. Illustration of sand fabrics (a) layered/stratified; (b) homogeneous and uniform; (c) phase transformation and steady state in undrained triaxial compression tests on uniform and stratified sand. Modified from Yoshimine & Koike (2005).

Despite being of the same overall particle size distribution, these lines occupied very different positions in e - $\log p'$ space for the two fabrics.

Obliteration of initial fabric at large strains/displacements in triaxial or plane strain tests is not always feasible, particularly in layered soils, hence the introduction of a class of soils referred to as 'transitional soils'.

5.5 Steady/critical state. Is it attainable? – Bedding surfaces

Pre-existing bedding surfaces in clays and cohesionless soils with platy particles, dominate behaviour, including behaviour under varying principal stress directions. Two examples are shown in Figure 16. **Once sliding on**

the bedding surface takes over during shear, mixing and removal of initial fabric cannot take place.

5.6 Steady/critical state. Does it depend on α ?

At critical state, an anisotropic fabric can develop. Elongated/platy particles align relative to loading direction (Figure 17), as do load columns (Figure 11), even in shear zones. If the loading direction is changed at critical state, i.e. if there is principal stress rotation and change in α , particles must re-orientate and the soil must undergo volumetric deformation to reach a new critical state. Is this feasible? Pre-existing particle alignment can dominate subsequent behaviour, consider the effect of bedding, referred to above.

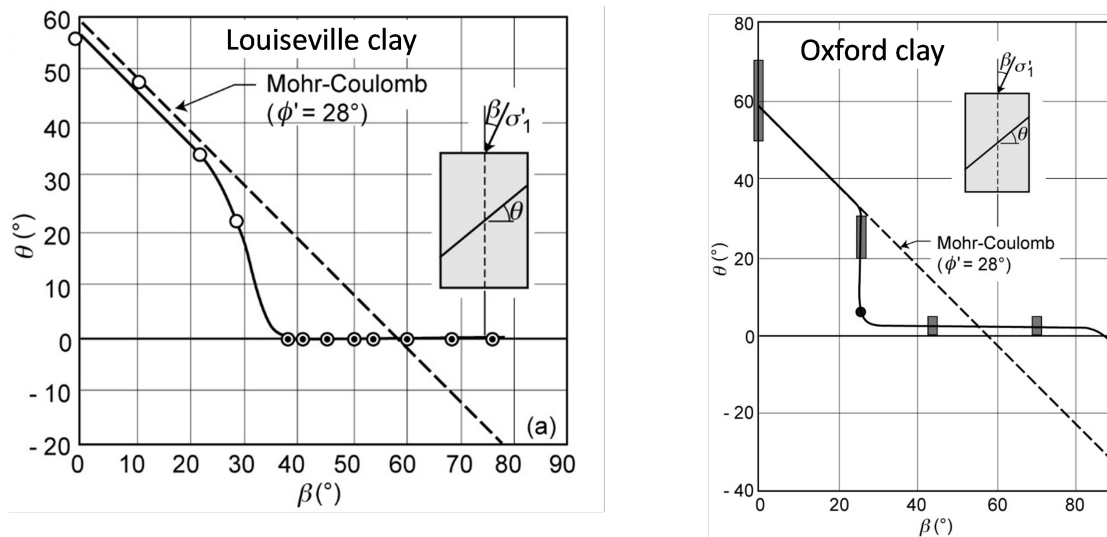


Figure 16. Inclination of failure plane with the horizontal, q , versus the angle of major principal stress axis rotation, β . Modified from Leroueil et al. (2024).

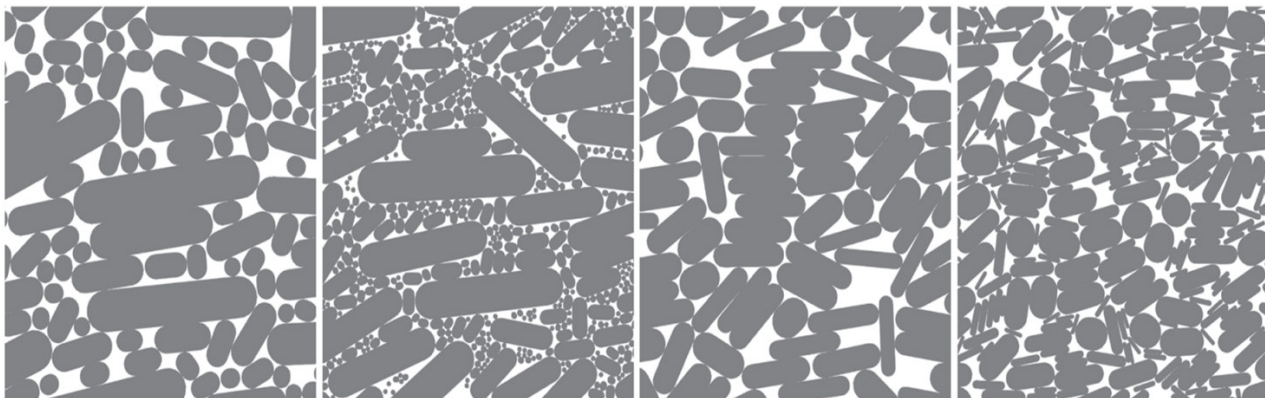


Figure 17. Orientation at 400% shear strain of particles of different shapes and size ranges. Simulations performed using the discrete-element approach known as contact dynamics. From Carrasco et al. (2023).

5.7 Steady/critical state. Does it differ between triaxial compression and extension?

Laboratory testing specialists have concluded that steady state is different between triaxial compression and triaxial extension. Referring to Figure 18, Vaid et al. (1990): “[...] while a single steady state line is found for compression loading, extension loading yields several lines, each characteristic to a given deposition void ratio. All the extension lines lie to the left of the compression line in void ratio - effective stress space. Thus at a given void ratio, steady state strength is smaller in extension than in compression, the difference increasing as the sand becomes looser”.

The dependence of critical state on b would suggest there could be a difference, and the fact that fabrics will be different

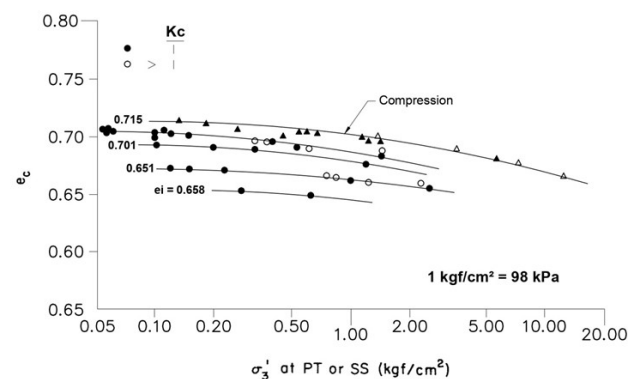


Figure 18. Steady state and phase transformation lines for Ottawa sand in triaxial compression and extension. Modified from Vaid et al. (1990).

in terms of particle orientation strengthens the argument. However, localisation in triaxial tests is rife, as confirmed by XCT, and XCT is suggesting, surprisingly, that within the shear bands the same critical state is reached in triaxial compression and extension tests; refer to Figure 19 from Salvatore et al. (2017). **Whether critical state is different between triaxial compression and triaxial extension remains an open question.**

5.8 Steady/critical state. Is it rate dependent?

Displacement controlled shearing is usually adopted in laboratory testing on the basis of its simplicity and that it provides data on the post-peak behaviour, strain softening, of the soil. However, that is post peak behaviour under displacement control, which is not necessarily the same as post-peak behaviour under dead or maintained loading, the form of loading which usually applies in field instability problems. Vaid et al. (1999) consider that: ‘**Only an inertial system can capture the true strain softening response of sand,**’ and that ‘**steady state deformation in an inertial loading system occurs at constant acceleration not constant velocity.**’

A critical difference between the two forms of loading is the ultimate/steady state strength available. Two sources of rate dependency at high rates of shear, particularly those associated with load control/maintained load, can be considered:

- Time for pore water migration into or out of shear bands when there is localisation. In contractant soil, no time for pore water migration out of the shear band leads to reduced undrained strength, as concluded earlier.
- Collision of particles at high displacement rates leads to increased spacing of particles and a reduction in their number of contacts (see Iverson & Denlinger, 2001).

Figure 20 presents a comparison of undrained plane strain tests on very loose Changi sand under deformation and load-controlled shear: (a) stress-strain; (b) effective stress paths; (c) deviator versus time. **The ultimate strength after collapse under maintained load, although not well defined, is more than halved compared to the strength measured under deformation control.**

Speculating on the basis of the findings by Yang and Dai described above, it seems reasonable to expect that behaviour at phase transformation could be affected by this difference in the form of loading and so in the rate of displacement; **above a certain global or local void ratio, the phase transformation seen under displacement control may not apply under load control.**

6. Instability

6.1 Tolerance to undrained disturbance under simple shear

Flow sliding in micaceous sands during construction of river training works on the Jamuna River, Bangladesh, was one of the focuses of my Rankine Lecture. Investigating these flow slides, and identifying a slope angle that would prevent further slides, led us to the concept of ‘tolerance to undrained perturbation/disturbance’, which involved defining a line in stress space relevant to DSS testing, above which there was no tolerance to undrained disturbance (refer to Figures 21 and 22). Direct simple shear, DSS, testing was selected on the basis of its simplicity, particularly for running constant volume tests, and for its greater relevance to conditions in a slope, rather than triaxial compression. In our naivety, we referred to this line as a collapse line, having previously suggested the existence of a collapse surface when examining the effects

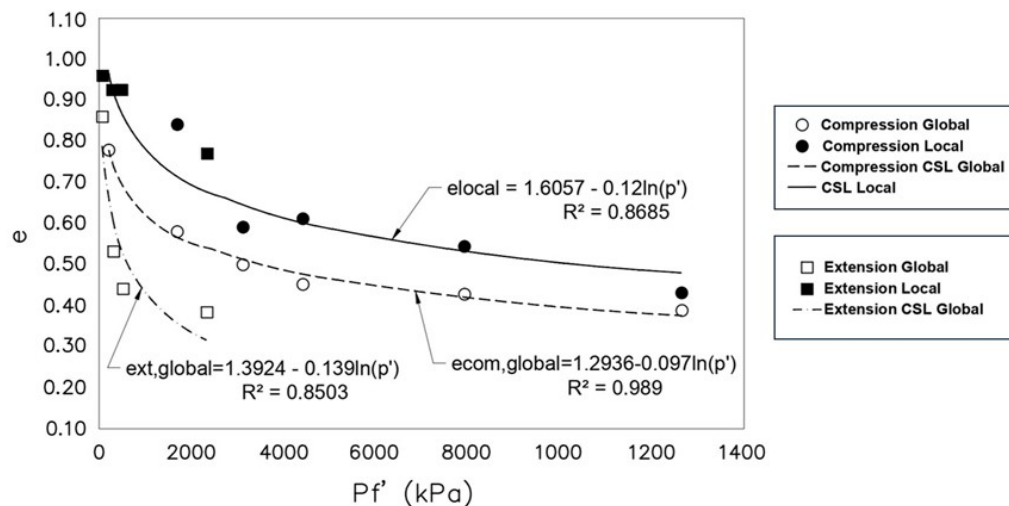


Figure 19. Global and local volumetric states at the end of triaxial compression and extension tests. Modified from Salvatore et al. (2017).

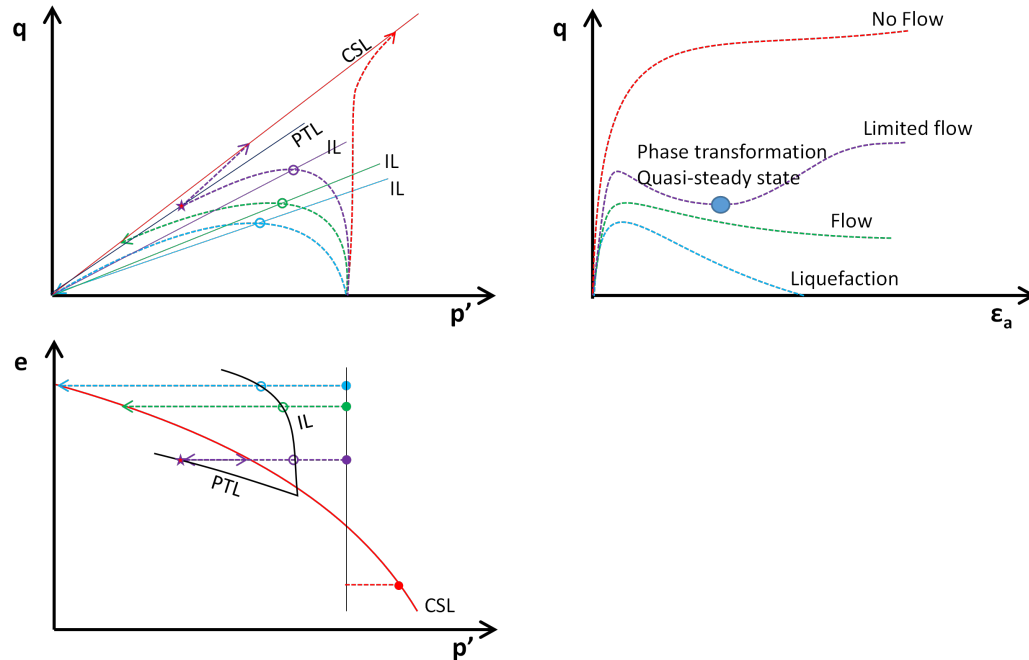


Figure 23. Patterns of behaviour in undrained triaxial compression. IL=instability line, PTL=phase transformation line (CSL: critical state line).

of principal stress rotation (PSR) on an anisotropic sand (see later). Inadvertently we had entered the world of instability, a topic which, I later realised, was being fiercely debated in the nineties and subsequently.

6.2 Instability/collapse in triaxial compression

The phenomenon of instability, sometimes now referred to as diffuse instability, is the initiation of a sudden increase in strain, before the full frictional strength of the soil has been mobilised. In undrained shear, behaviour is brittle beyond the point of instability. Under load-maintained conditions, strains develop extremely rapidly, and it seems reasonable to refer to this as collapse. In Figure 23, showing behaviour in undrained triaxial compression, I distinguish between flow and non-flow conditions, restricting the term liquefaction to the case of zero strength at large strains. According to Wang et al. (2016), “Once the material is in a liquefaction state, the material is overall in a “semi-suspended particle” regime, in which the coordination number is associated with random transient contact between particles instead of load-bearing structure”.

To assist in the explanation of the collapse of a hydraulically placed subsea sand berm, which occurred at Nerlerk in the Canadian Beaufort Sea, Sladen et al. (1985a, b) had introduced the concept of a ‘collapse surface’ in loose sedimented sands. The ‘collapse surface’, or more correctly a ‘collapse line’, was found by drawing a line through the peak points on effective stress paths from isotropically

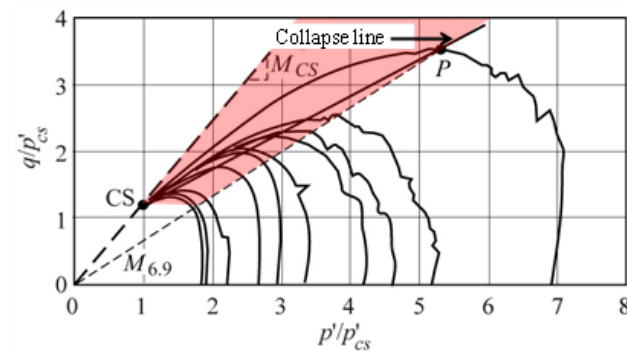


Figure 24. Undrained triaxial compression tests on isotropically consolidated loose Leighton Buzzard sand showing the collapse or instability line, the critical state (CS) and the zone of potential instability (shaded pink). Modified from Leroueil et al. (2009) and Sladen et al. (1985b).

consolidated undrained, CIU, triaxial compression tests on specimens prepared at the same post consolidation void ratio by a form of moist tamping, and connecting this to the steady state or critical state point. Vaid & Chern (1985) used the term ‘the Flow Liquefaction Surface (FLS)’ to refer to a similar surface to the collapse surface. Experiments show that the peak strengths fall on a straight line in stress space. An example of a ‘collapse line’ for loose Leighton Buzzard sand is presented in Figure 24.

Lade (1993) explored these ideas, and the cause of the Nerlerk failure, referring instead to static instability and to

instability lines, IL, which, in Lade's case had been identified in CIU triaxial compression tests on Valgrinde sand samples prepared by moist tamping (Bjerrum et al., 1961). Lade (1999) summarised his views in a landmark paper.

Chu et al. (2003) and Chu & Wanatowski (2008) carried out a comprehensive investigation of instability lines, running CIU triaxial and plane strain compression tests on moist tamped samples of Changi sand. Their tests illustrated how the slope of the instability line varies with void ratio (Figure 25a). Interestingly, Chu & Wanatowski (2008) found that the relationship between the slope of the instability line and void ratio was not affected by whether the specimen was isotropically or K_0 consolidated or whether tested in triaxial or plane strain compression (Figure 25b).

Chu et al. (2003) and Leroueil et al. (2009) introduced a framework which made use of instability lines and critical state to anticipate the response of loose and dense sands to drained and undrained perturbations in slopes. This included the response to reducing mean effective stress at constant shear stress (CSD test), a common situation in slopes undergoing infiltration or rising groundwater level. The framework for loose sand, our interest here, is illustrated in Figure 26.

6.3 Summary re instability in triaxial compression

Features of soil behaviour that emerge from these findings for triaxial compression are summarised below.

- **Instability occurs before there is full mobilisation of shearing resistance.** Lade (1993): “*Instability occurs inside the failure surface and so is not synonymous with failure, although both lead to catastrophic events*”. Lade (1999): “*In loose sands and sensitive clays in which the pore pressures increase monotonically during shear, the maximum stress difference is reached before the maximum effective stress ratio The condition of maximum stress difference does not correspond to a true failure condition, but rather to a condition of minimum stress difference at which instability may develop inside the true failure surface*”.
- Lade (1999): “*Instability is not produced along a particular slip surface, but rather in a volume of soil within the slope, and classical slope stability methods cannot be used to produce a factor of safety.*” ‘*Instability can potentially be triggered in any region*

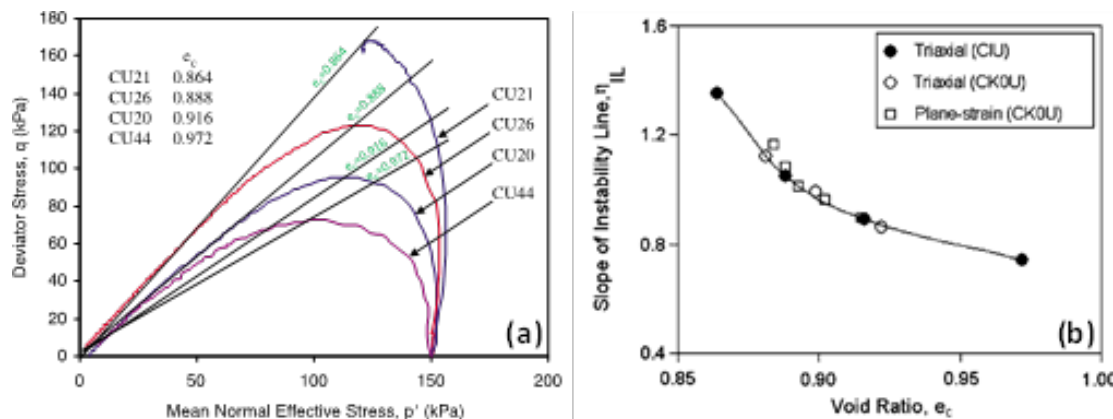


Figure 25. Instability lines and their dependence on void ratio. Typical loose sand behaviour in undrained triaxial compression. Modified from Chu & Wanatowski (2008).

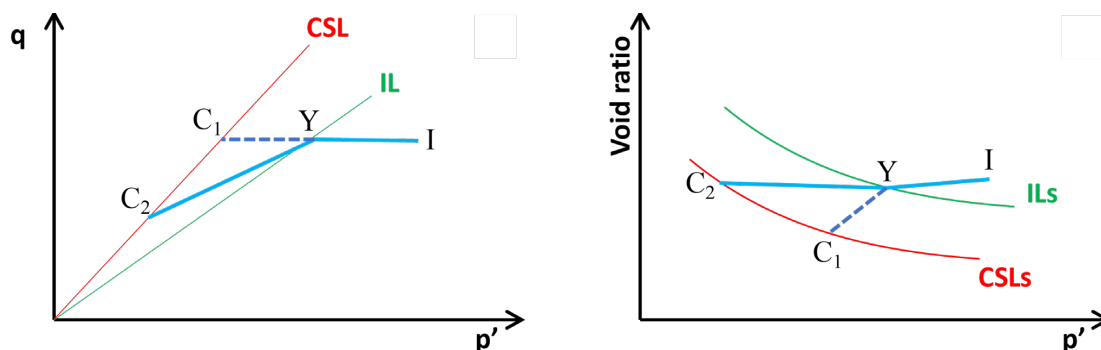


Figure 26. Schematic illustration of instability conditions for loose sand along a CSD path. Modified from Leroueil et al. (2009). (CSD: drained constant shear).

of the slope in which the stress states are above the instability line”.

- **For a granular material to become unstable, the state of stress must be located on or above the instability line, in a zone bounded by the IL and the critical state line, CSL, and referred to as the zone of potential instability or instability zone** (Figure 24).
- Instability can occur under drained, undrained and partially drained conditions.
- **Under undrained and partially drained conditions, only very loose sand can become unstable** and this can lead to static liquefaction or collapse if the driving shear stress is and remains significantly higher than the ultimate or critical state strength.
- **Drained instability in both loose and dense sands only occurs when there is a reduction in the mean effective stress** that may result from a decrease in total mean stress (e.g. an excavation) or from an increase in pore pressure (e.g. rising groundwater level).
- The difference between undrained and drained conditions is illustrated using the framework in Figure 26. When a loose sand is sheared along a constant deviator stress path, starting from point I, instability occurs at point Y, on the IL. If the pore water pressure can dissipate freely (i.e. under drained conditions), the stress path will eventually reach the failure state at point C1, on the critical state line. which is also the failure line for loose sands. During this process, large axial and volumetric strains will develop, and the void ratio of the soil will decrease. If the pore water pressure cannot dissipate, the stress path will move towards the critical state associated with its current water content, i.e. C2, under undrained conditions.
- **A triggering mechanism for collapse is one that causes the pore pressure to increase faster than it can dissipate when the stress state is in the instability zone or has been brought to the instability line.** Once initiated, collapse can propagate progressively and rapidly, particularly into areas where the stress state lies in the instability zone. Triggers for the flow slides in the micaceous sands at Jamuna included drained and undrained excavations by dredging, which were accompanied by principal stress rotation, and cyclic loading during storms which increased pore pressures.
- Strains to reach the instability line are small, making it difficult to use displacement monitoring to predict incipient instability. **The small strains probably explain the insensitivity to the value of b when comparing triaxial and plane strain compression tests** (Figure 25b). Again, because strains are small, and so before particle rotation, the instability line is not affected strongly by localisation.

7. Anisotropy of undrained shear behaviour, including instability, of water and air pluviated loose sand

7.1 Hollow cylinder apparatus tests

In 1998 I presented Figure 27 from Professor Shibuya's 1985 thesis, showing the anisotropy of undrained behaviour in sedimented, isotropically consolidated Ham River sand, sheared with different orientations of the major principal stress to the vertical, α , in the hollow cylinder apparatus (HCA); these tests were run with the intermediate principal stress ratio, b , being equal to zero. 1998 turned out to be a vintage year for data from tests in different hollow cylinder apparatuses on different sedimented sands, with different values of b – see Figure 28 for data from Yoshimine et al (1998) on Toyoura sand and Figure 29 for data from Uthayakumar & Vaid (1998) on Fraser River sand. There was also an HCA based paper by Nakata et al. (1998) on Toyoura sand at different relative densities and confining stress levels. The pattern was consistent, and, reassuringly, the same pattern has been captured in more recent HCA studies and in DEM experiments (e.g. Yamada et al., 2023; Farhang & Mirghasemi, 2017).

7.2 Anisotropy of instability and brittleness

The similarity between the observed undrained behaviours in different clean and sub-angular sands and different testing equipment is striking:

- **Sedimented sands are non-brittle when the major principal stress is vertical ($\alpha = 0$ degrees).** This is consistent with the findings in triaxial compression tests on sedimented sands and contrasts with the extreme brittleness that is found in the same sand at the same global void ratio when prepared by moist tamping.
- **Sedimented sands become brittle when sheared with the major principal stress rotated away from the vertical, i.e. from the direction of deposition ($\alpha > 15$ to 30 degrees).** This is perfectly logical when considering the fabric that develops during sedimentation (e.g. Oda, 1972) and the **anisotropic load transfer framework that develops in sands, having the ability to protect macrovoids** (Figure 3), until principal stresses are rotated.
- **Peak undrained strengths at the point of instability reduce with increasing values of α .**
- Strains to peak are small for all values of α .

The anisotropy observed in these HCA tests is that existing after sedimentation and so before any subsequent stress or strain history. It could be indicative of the anisotropy

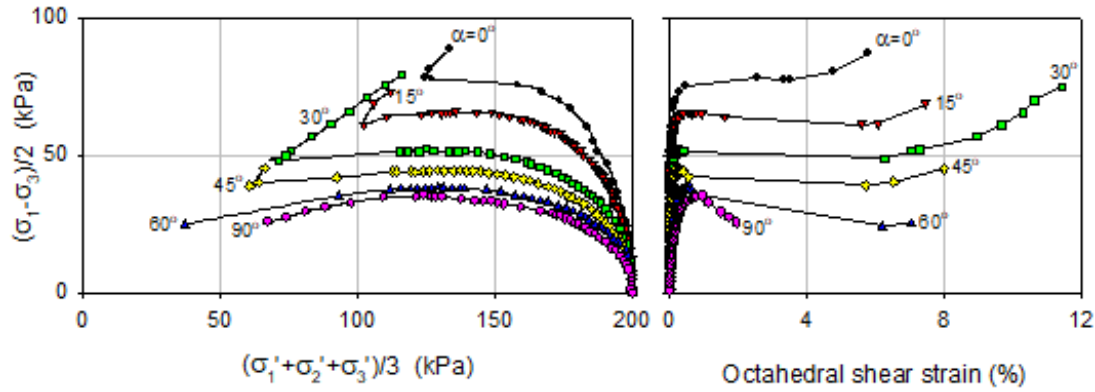


Figure 27. Undrained behaviour of water pluviated Ham river sand in HCA tests, $b=0$, α varying. Modified from Shibuya (1985).

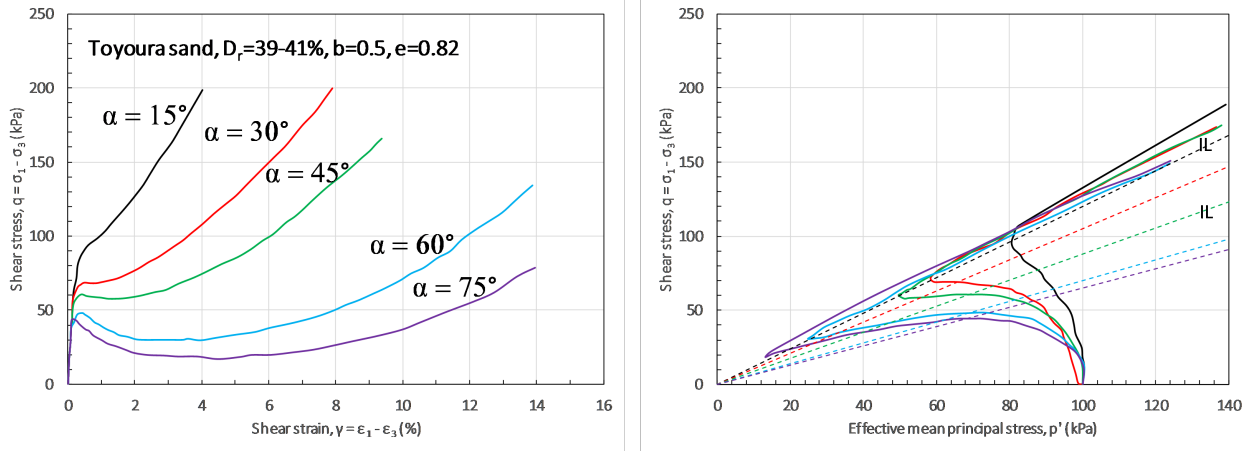


Figure 28. Undrained behaviour of water pluviated Toyoura sand in HCA tests, $b=0.5$, α varying, $D_r=39-41\%$. Modified from Yoshimine et al. (1998).

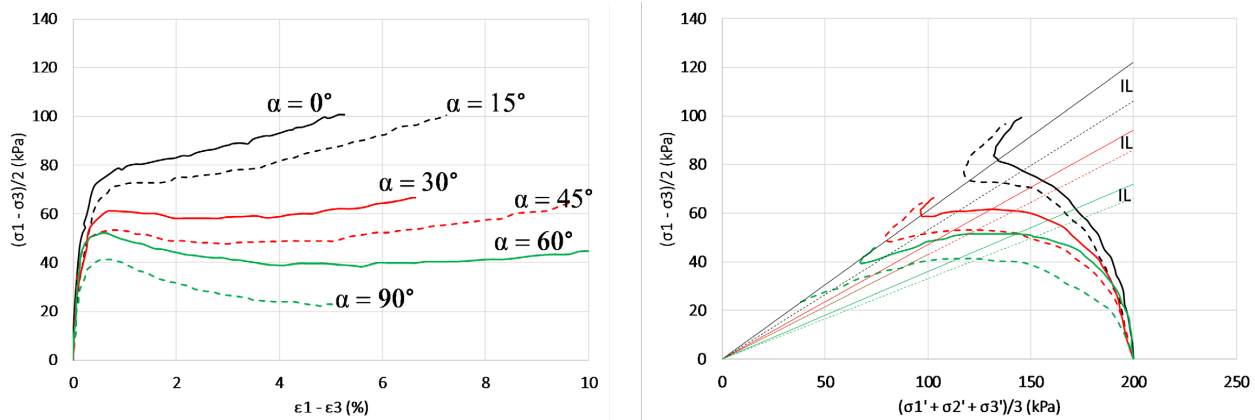


Figure 29. Undrained behaviour of water pluviated Fraser River sand in HCA tests, $b=0$, α varying, $D_r=30\%$. Modified from Uthayakumar & Vaid (1998).

of relatively recently sedimented silty sand tailings but one must consider the potential impact of fines.

Stacking the effective stress paths from CIU HCA tests on sedimented Ham River Sand in q - p' - e space (Figure 30),

the initial anisotropy of the sand can be portrayed, and the resulting surface can be used to identify when collapse could occur under rotation of principal stresses at constant deviator stress. Slices through this surface at constant α , can, with

the addition of the relevant instability line and critical state line, be used to define the instability zone and to anticipate when instability could develop, for example as in a CSD test at different or constant alpha values (Figure 30).

7.3 Localisation in HCA tests

Beyond the point of instability, HCA samples of loose sand develop spiral shear bands, removing any possibility of identifying critical states at different values of alpha.

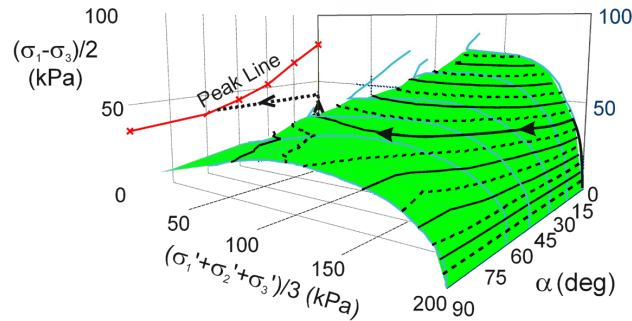


Figure 30. Undrained principal stress rotation at constant t leading to instability.

Localisation is inevitable in view of the non-uniformity of stresses across the wall of the HCA sample, when torsion is applied, or different inner and outer pressures are maintained.

In HCA tests run wholly under stress control, contractancy after the point of instability initiates a runaway failure that can probably, at sufficiently low mean effective stress, over-ride phase transformation.

A DEM investigation of localisation in HCA tests was described by Li et al. (2015). Li et al. (2015) demonstrated in their DEM, that ‘a dominant shear band is initiated in the vicinity of the peak strength and fully develops as the loading moves toward the critical state. ...The inclination of the shear band appears to depend on both friction angle and the dilation angle.’ In this case, the peak strength was determined at a strain of 0.3% to 0.45%.

This is good news in that up to the strains at which instability occurs, conditions in the hollow cylinder sample are reasonably uniform (Figure 31).

7.4 Anisotropy of instability lines

For the reasons of localisation explained above, the critical state in HCA tests cannot be defined and so the instability line cannot be connected to the critical state. To illustrate the dependence of instability on principal stress direction,

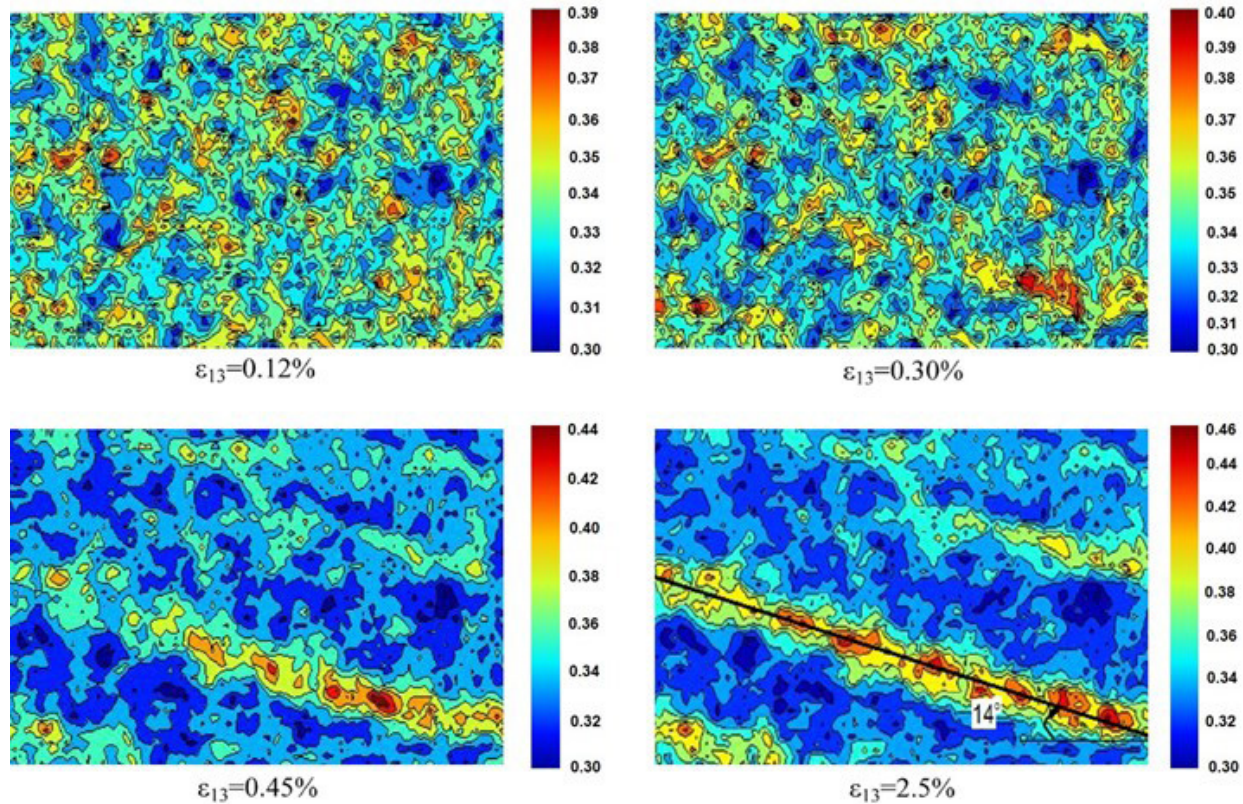


Figure 31. Localisation in a hollow cylinder sample with alpha at 45 degrees. Porosity evolution at increasing deviatoric strain. Peak strain between 0.3 and 0.45%. Modified from Li et al. (2014).

instability lines have been added to Figures 28 and 29, passing through the origin and the peak point of the undrained effective stress path. Adding instability lines in this way shows that the IL inclinations reduce as α increases. The fan of instability lines for different α values is similar to the fan for different void ratios in triaxial compression (Figure 32).

Extending our summary of the features of instability to other than triaxial compression we can say:

- **Instability line inclination varies with void ratio, shearing mode and direction, α .**
- **Zone of potential instability increases with increasing α and global void ratio.**
- **Instability can also be triggered by undrained rotation of principal stress directions at constant or increasing shear stress.**

8. Spatial variation in void ratio, specimen preparation, localisation, instability, partial drainage and a chain reaction

8.1 Void ratio

Void ratio is the average ratio of void space to the volume of solid particles in a specimen or mass of soil. Void ratio cannot convey the fabric of the soil, in particular the particle arrangement in terms of the distribution of the different particle sizes. Figure 33 makes this point, showing different particle arrangements at the same nominal void ratio.

The problem with void ratio is that it is a global measure of density of packing, yet in natural and reconstituted soils, local void ratios can and do vary widely. In this context, local void ratio of a particle is defined as the ratio of the volume of the void space associated with the particle divided by the volume of the particle. My attention was drawn to this shortcoming in void ratio by the painstaking work published by Jang & Frost

(1998) from whom I took Figure 34 and included it in the Rankine Lecture. The work was unable to deal with loose sands because it depended on being able to infiltrate the specimens with resin, but without disturbing their packing. The results showed clearly: the variations in local void ratio in graded sand; the dependence of that variation on the method of specimen preparation, in this case moist tamped and air pluviated specimens; and the fact that the scale of variation, higher in moist tamped specimens, increased with increasing global void ratio.

One of the by-products of the developments in observing individual particles in specimens, in this case micro-computed X-ray tomography, is confirmation of this variation in void ratio. Two examples from the literature are shown in Figure 35, both illustrating the wide range of local void ratio.

8.2 Dependence of macro behaviour of loose sands on method of specimen preparation. (How can samples prepared in different ways to the same global void ratio be dilatant and stable or contractant and unstable?)

There are many examples of the difference in undrained shear behaviour of loose sands which have been prepared to the same global void ratio, but in different ways, and consolidated to the same stresses. The differences between the behaviour of specimens prepared by moist tamping and by pluviation, wet or dry, are illustrated in Figure 36; Figure 36a shows the different stress strain data from constant volume simple shear tests on Syncrude sand, and Figure 36b shows the different stress-strain and effective stress paths in triaxial compression tests on Hostun sand.

The differences are superficially ascribed to differences in fabric, but which aspects of fabric? Note that differences are seen at small strains, at instability, and at large strains, approaching critical state. The moist tamped specimen collapses and flows, with no sign of phase transformation

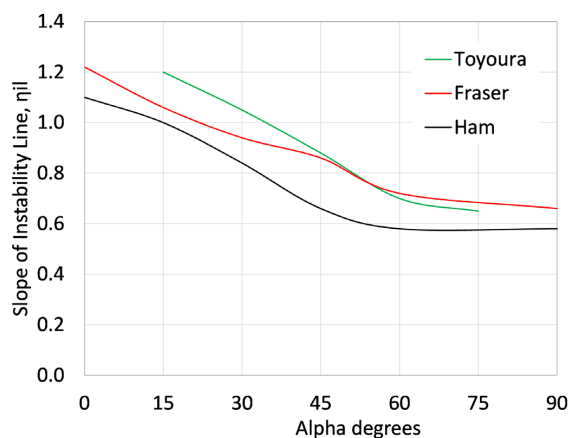
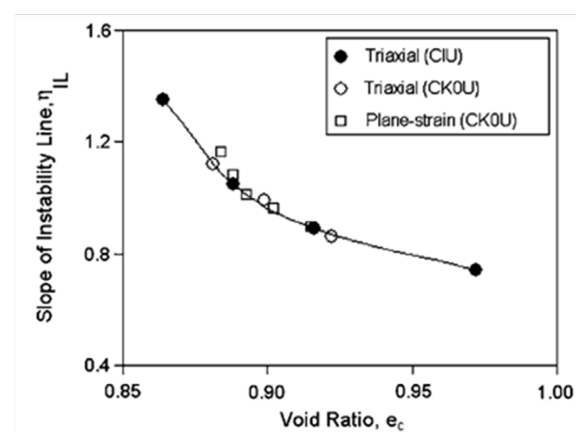


Figure 32. Slope of instability lines at different α and void ratio.



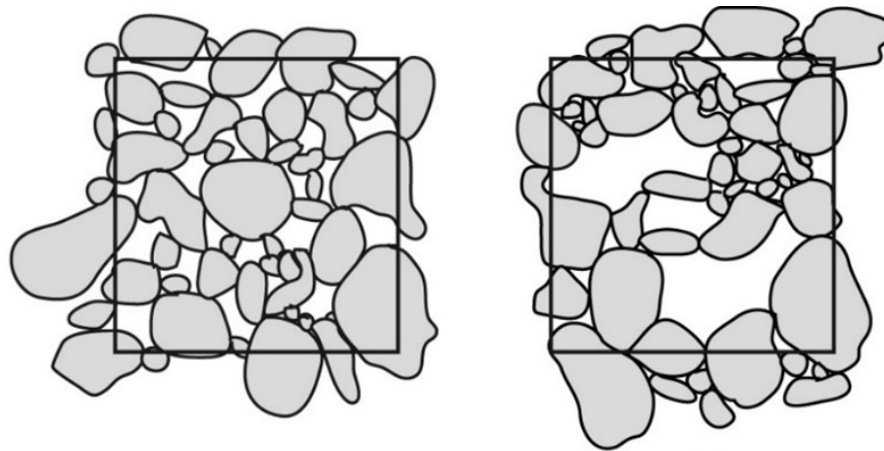


Figure 33. Illustration of potential ranges in the arrangement of identical particles at the same relative density.

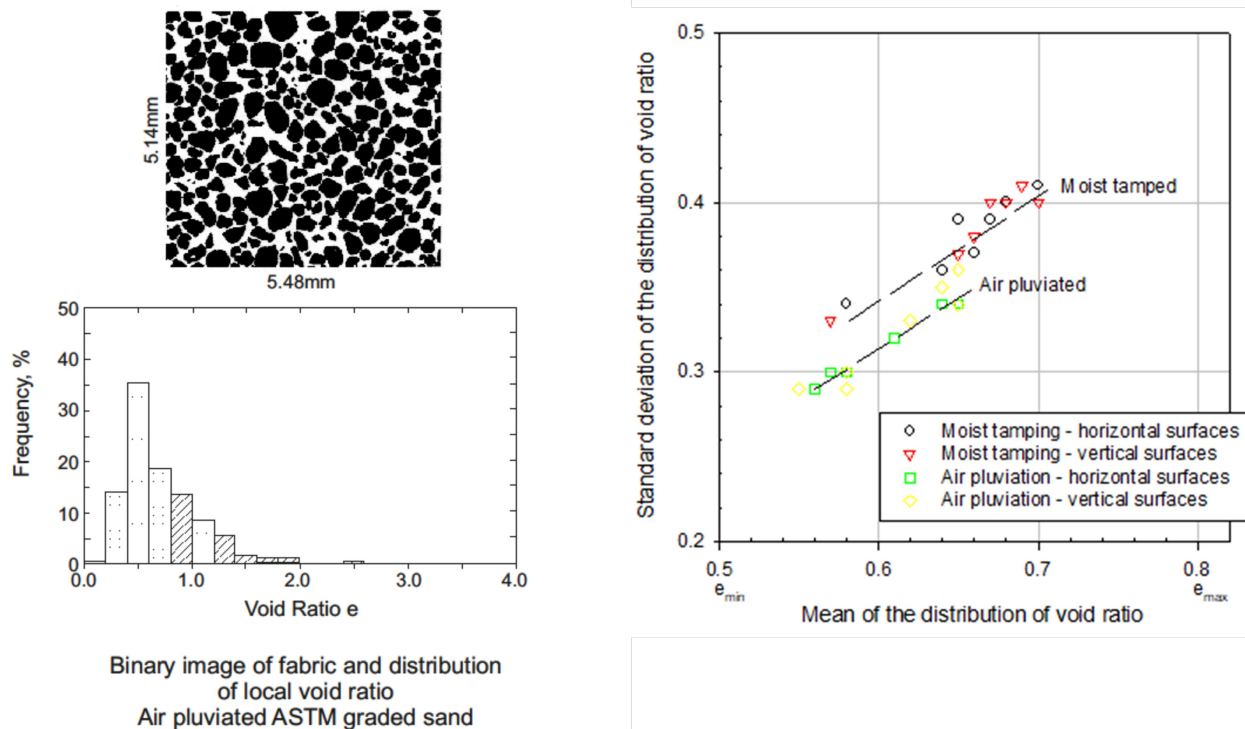


Figure 34. Relation between mean and standard deviation of local void ratio distributions. Modified from Jang & Frost (1998).

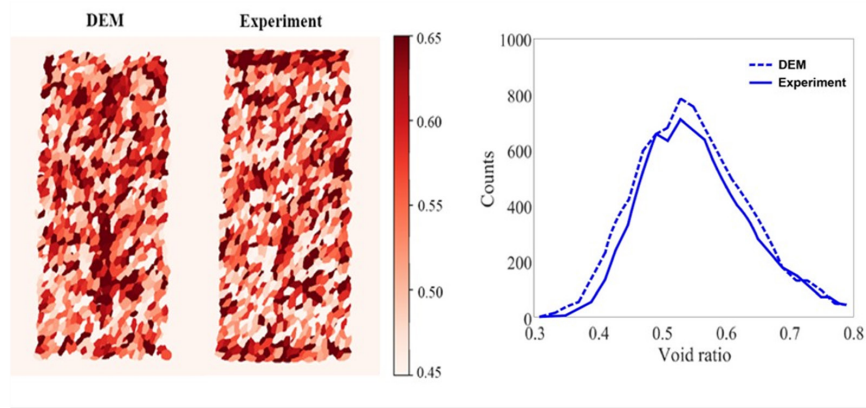
and so no suggestion that it will reach the same critical state as that to which the pluviated specimen is heading.

Benahmed et al. (2004) provide valuable insight into the mesofabric of the moist tamped sample of Hostun sand. Based on the photographs in Figure 37a and b and recognising the clustering of particles encouraged by menisci forces, they suggest the conceptual picture of void distribution shown in Figure 37d. Work on localisation described above tells us that inhomogeneities of this form can trigger localisation and that behaviour in a shear zone determines observed macro/global behaviour.

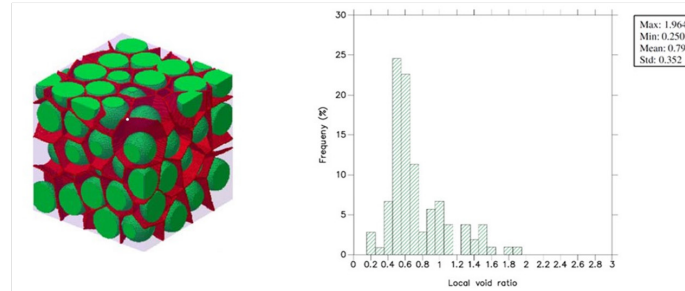
8.3 A chain reaction

A chain reaction inside the specimen, triggered by an initial local collapse, can be envisaged, referring to Figure 38:

- The zone having a high local void ratio, e_1 , is within the instability zone and so collapses under loading, generating local excess pore pressures. This is localisation at a particle level.
- Pore pressures migrate under drained, partially drained or expansive drained conditions, discussed below, to neighbouring clusters.

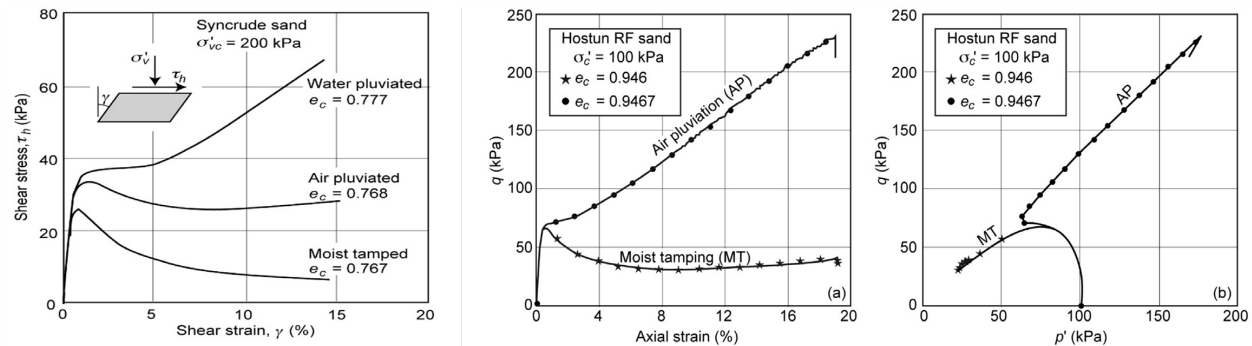


Comparison of initial state void ratio distribution between experiment and DEM for the 60 degree specimen. Modified from Pan et al (2023).



Distribution of local void ratio in porous media systems from 3D X-ray microtomography images. Modified from Al-Raousha and Alshibili (2006).

Figure 35. Spatial distribution of local void ratios.



(a) Undrained simple shear, Syncrude sand. Modified from Vaid et al (1995).

(b) Undrained triaxial compression, Hostun sand. Modified from Benahmed et al (2004).

Figure 36. Effects of specimen preparation.

- Increase in pore pressures in the e_2 zone, under constant or increasing shear stress, and local undermining, moves the state into its instability zone and so that zone collapses.

It follows that **the behaviour of the moist tamped specimen is determined by its spatial variation in void ratio and the presence of macrovoids which trigger localisation and the formation of shear bands** The shear

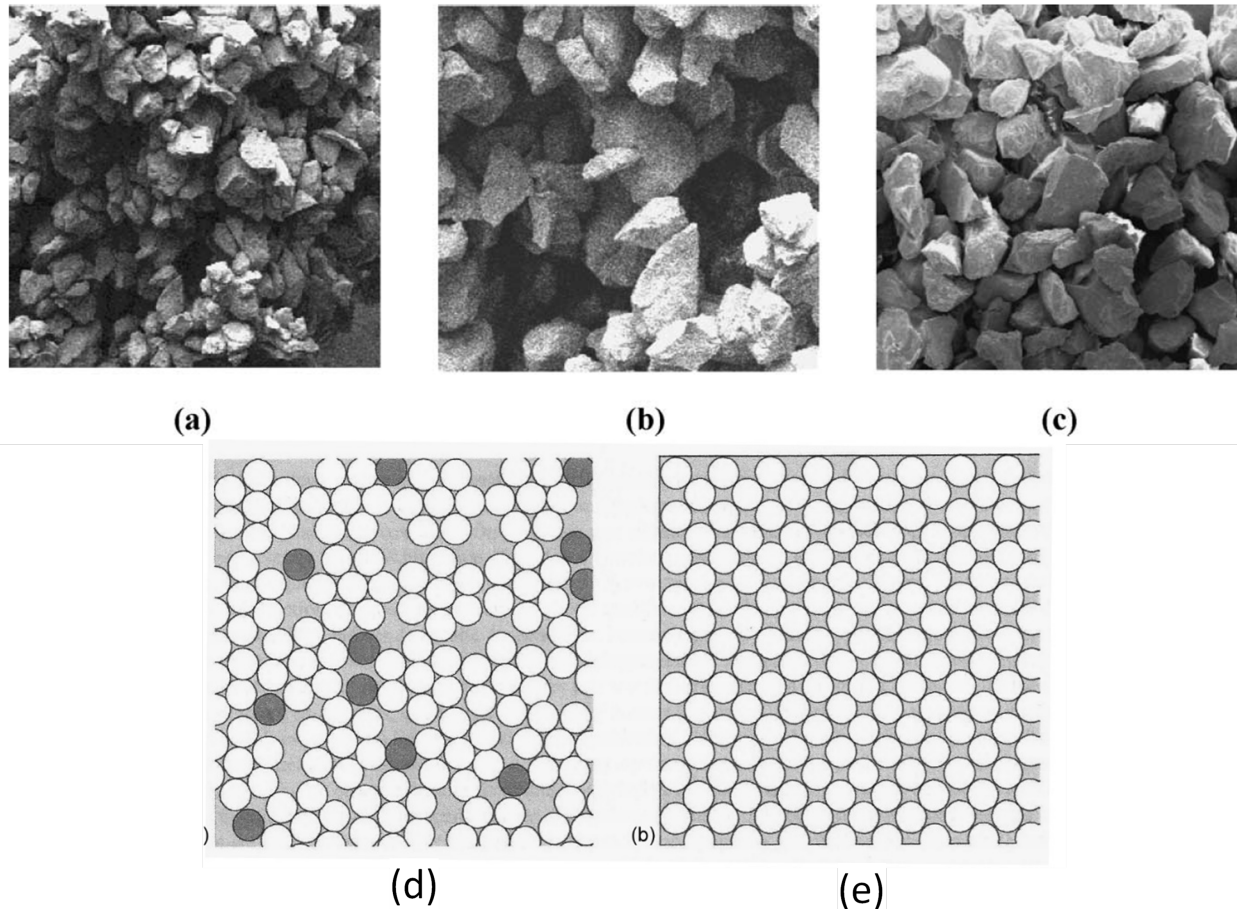


Figure 37. Photographs of moist tamped (a and b) and air pluviated (c) samples. Conceptual bidimensional representation (d) aggregated fabric (e) homogeneous fabric at same void ratio. Modified from Benahmed et al. (2004).

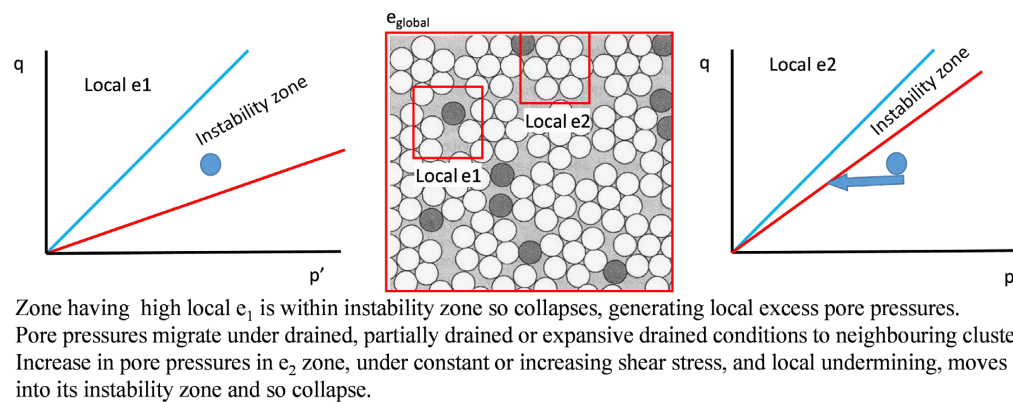


Figure 38. Chain reaction in globally undrained moist tamped sample with spatially varying local void ratios. Illustrative figure.

bands control the macro behaviour and the local void ratio in the shear band is high enough to be consistent with flow failure in the critical state concept (Figure 39).

The putative development of shear bands in loose moist tamped samples is confirmed by Cao et al. (2020) who observed

banding in the moist-tamped samples, while bulging was observed in the middle portion of the air-pluviated samples.

The large spatial variation in local void ratio in moist tamped samples and the presence of macrovoids at low relative densities, are confirmed by Ni et al. (2021), who

compared the fabrics of most tamped and air pluviated samples of Toyoura sand at different relative densities (Figure 40). The more uniform distribution of voids in the air pluviated samples at all relative densities is consistent with the findings of Jang & Frost (1998).

Ni et al. noted that the long axis direction of the particles was distributed isotropically in the moist tamped samples at all relative densities but in the air pluviated samples the dominant particle orientations were anisotropic

and varied with relative density, see Figure 41. The oriented particles in the inherently anisotropic specimens form strong contacts in their long axis direction and so provide a support frame in the early stages of shearing, which restrains the contraction of the specimens (Figure 42; Chen et al., 2020). **Anisotropy of fabric in the air pluviated sample explains its resistance to continuing vertical loading and why it does not contract while the moist tamped sample does** (Figure 36).

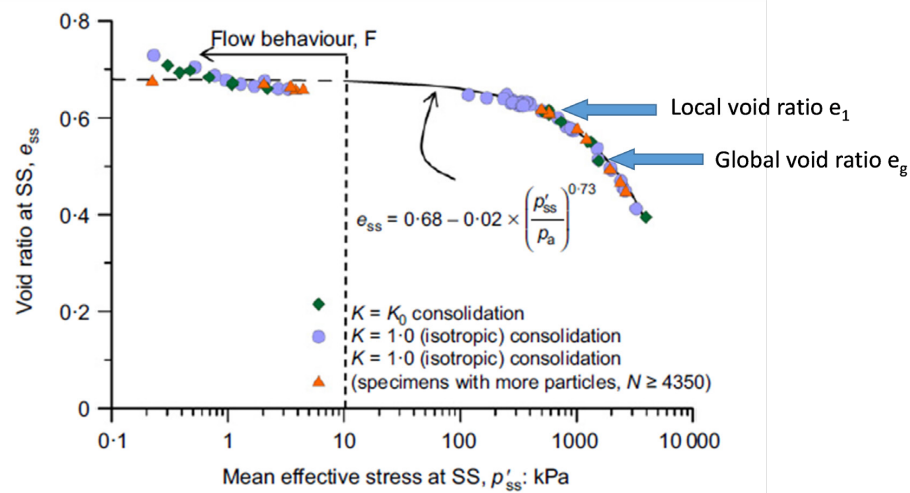


Figure 39. Local void ratio e_l initiates chain reaction and is consistent with CS predictions. Illustrative figure.

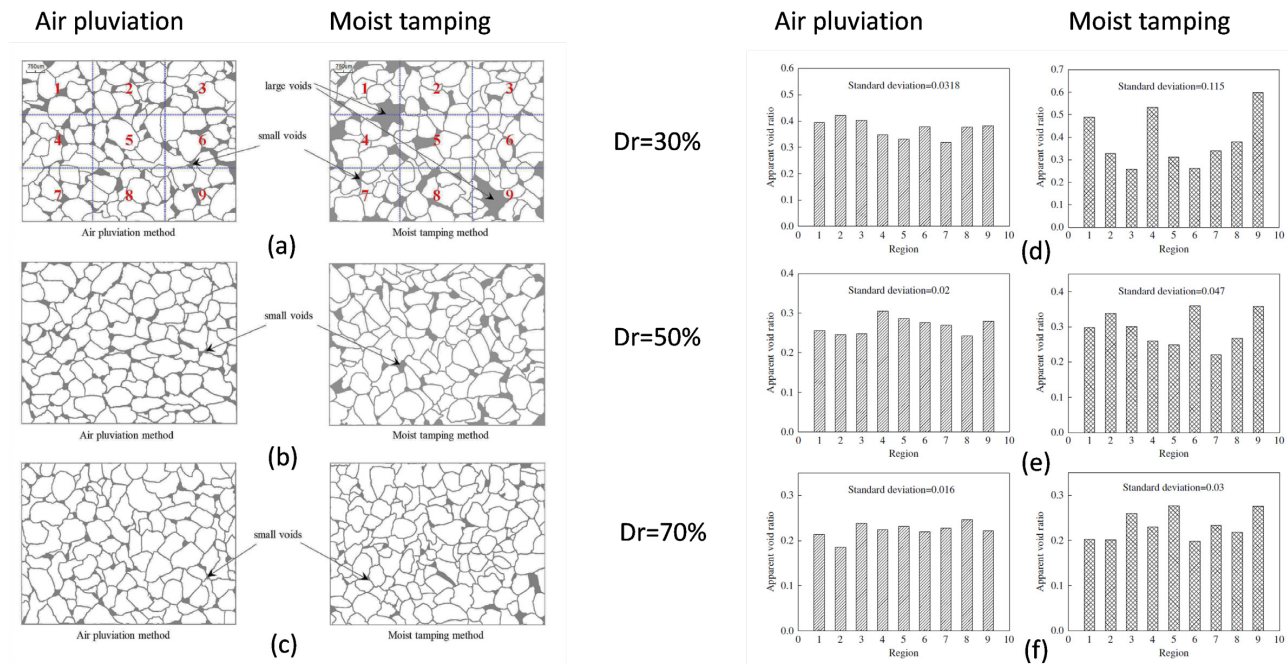


Figure 40. Moist tamped and air pluviated samples of Toyoura sand at different relative densities: (a), (b) and (c) zoning for investigating apparent local void ratio; (d), (e) and (f) apparent local void ratio distributions at different regions. Modified from Ni et al. (2021).

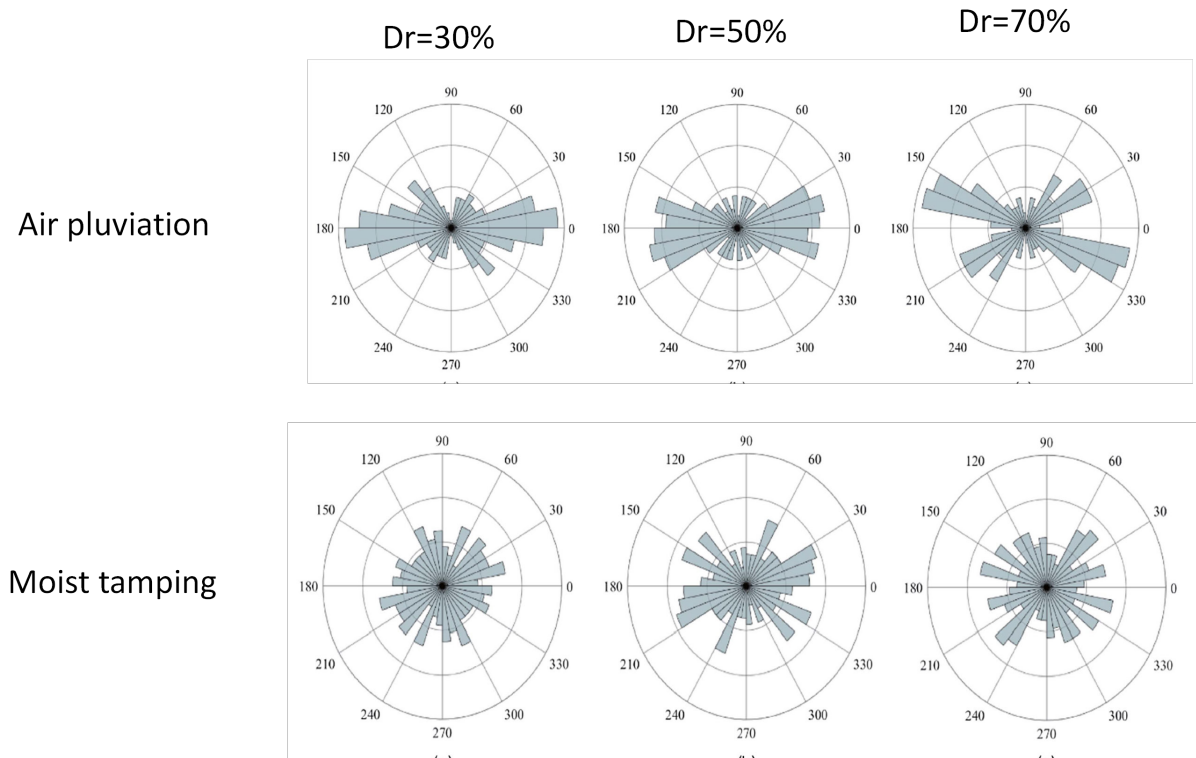


Figure 41. Particle orientation in moist tamped and air pluviated samples of Toyoura sand. Modified from Ni et al. (2021).

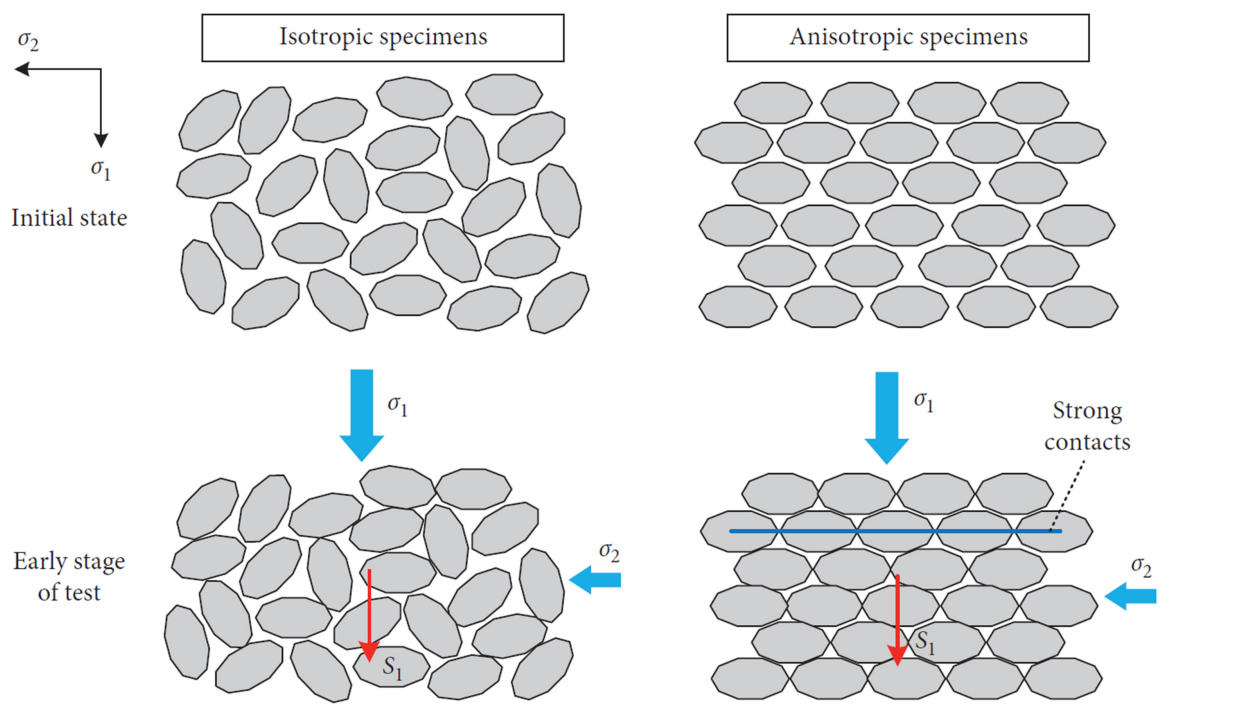


Figure 42. Isotropic and anisotropic packing. The oriented particles in anisotropic specimens form strong contacts in their long axis direction and provide a support frame in the early stages of shearing, which restrains the contraction of the specimens. Modified from Chen et al. (2020).

8.4 Expansive partial drainage and a potential role in the chain reaction and its rate of propagation

The concept of expansive drainage appears to originate from the comments of Peters (1991), quoted by Lade (1999), as “[...] even dilating sand may become unstable: If a rate of volumetric expansion were imposed on an element of granular material and this rate exceeded the rate of expansion exhibited by the material, then the effective confining pressure would decrease and the element would become unable to sustain the current, applied shear stress. Thus, a rate of volume change (whether positive, zero, or negative) imposed on a material element could cause it to become unstable depending on the rate of volume change exhibited by the soil itself (compression or dilation)”.

These comments are consistent with findings from strain path testing reported by Chu et al. (1993, 2003). In strain path testing, volume expansion can be imposed on a specimen to simulate the process of forced water infiltration into a soil element. This appears to be referred to as expansive drainage, or more correctly expansive partial drainage. Under expansive partial drainage, as opposed to contractive partial drainage, water inflow may contribute to the development of excess pore-water pressure which is additional to any shear-induced pore pressure and can result in a more onerous condition than undrained loading.

Lashkari & Yaghtin (2018) state: “It is a common belief that soil behaviour under any possible stress path in a triaxial apparatus is bounded by the behaviours found under undrained and fully drained triaxial (i.e. CU and CD) tests as extremes for shear strength and volume change behaviours. However, that hypothesis may only be valid when soil is homogeneous and pore-water pressure is uniformly distributed across the soil element. The load-carrying structure of real soils and the earthquake induced excess pore-water pressure are generally heterogeneous and thus this presumption is questionable”.

Figure 43, from Lashkari & Yaghtin (2018) capture the hierarchy of strengths and behaviours, including that for expansive partial drainage.

Expansive partial drainage would appear to be able to contribute to the spread of a local collapse and possibly contribute to its rate of propagation of tailings dam failures and other flows.

8.5 Chain reaction at specimen and field scale

The speculative chain reaction inside the moist tamped specimen, with its large spatial distribution of local void ratio and the presence of macrovoids, is no more than a scaled-down version of the spread of instability in a slope with a large region of potentially unstable soil, described by Lade (1999): “The slope is perfectly stable as long as the soil remains drained. An undrained perturbation initiates instability. Once a local zone of instability has been created, the resulting pore pressure buildup will propagate and enlarge the unstable region in the slope. The instability initiated in the potentially unstable region is self-sustaining, i.e. the material is not dependent on any further outside perturbations, and it consequently exhibits unconditional, run-away instability”.

Instability under undrained conditions is more likely in a slope where conditions are load maintained, so collapse or crossing the instability surface can occur under global, and even local, undrained conditions.

9. Final thoughts on moist tamped versus sedimented samples

Samples prepared by moist tamping (wet tamping) are often tested when investigating the behaviour of sands and sands with fines, and, as we learnt from Reid & Fanni (2022), when characterising tailings. Why does the use of moist tamped specimens continue to be used to investigate the behaviour of hydraulically-placed soils such as tailings, when water-sedimented specimens provide a much better match of fabric?

Clustering in moist tamped samples results in non-uniform void distribution and the exaggerated presence of macrovoids.

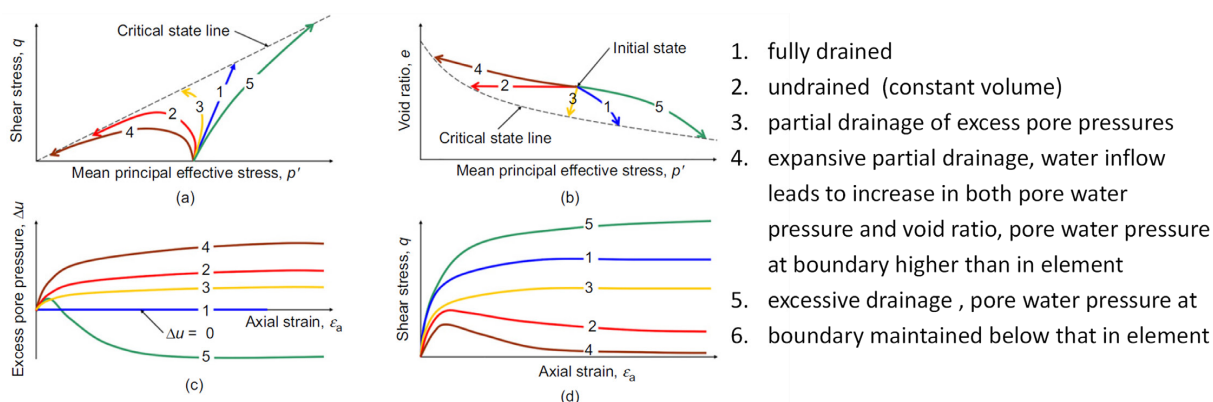


Figure 43. Schematic illustration of various patterns of loose sand behaviour in triaxial compression: (a) shear stress plotted against mean principal effective stress; (b) void ratio plotted against mean principal effective stress; (c) excess pore water pressure plotted against axial strain; (d) shear stress plotted against axial strain. Modified from Sivathayalan & Logeswaran (2008) and Lashkari & Yaghtin (2018).

As a result, the collapse potential is also exaggerated and localisation and collapse can occur in triaxial compression. The relevant void ratio at critical state is uncertain.

Sedimented clean sands do not generally show a potential to collapse when tested in triaxial compression but show a marked anisotropy of collapse potential when sheared after or during rotation of the principal stress directions away from the vertical, as in hollow cylinder tests, referred to earlier. This anisotropy of instability results from the preferred vertical orientation of contact normals during deposition and the development of vertical load carrying columns during overburden loading which bypass the macrovoids, until principal stresses are rotated sufficiently from the vertical.

The fact that specimens prepared by sedimentation do not show collapse in undrained triaxial compression, simply demonstrates the critical importance of anisotropy in instability problems in general.

10. Some final thoughts on fabric, void ratio, heterogeneity, localisation, critical state, and instability

10.1 Fabric and void ratio

Cohesionless soil behaviour is determined by stress state, stress or strain history, stress or strain path, initial and evolving arrangement of particles and pore spaces (fabric including density of packing), and characteristics of the particles, including the range of particle sizes and their shapes and strengths.

Excluding residual soils, the arrangement of the particles and void spaces at the time of formation of a soil is determined by the form, rate and environment of deposition, the particle size distribution and the characteristics of the particles being deposited.

The forms and environments of deposition include those that occur naturally, those associated with construction filling, and those involving specimen preparation in the laboratory. Methods of specimen formation in the laboratory may or may not attempt to simulate soil formation in situ.

Fabric changes after formation depend on the initial fabric itself and the post-depositional changes such as stress or strain path, time effects such as creep, and changes in environment that might lead, for example, to cementation.

Void ratio is a measure of density of packing, but like other parameters defining the state of a soil, such as water content, degree of saturation, density, it is at best an average value. Global void ratio is only a part of a complete description of the fabric of a soil. The arrangement or fabric of relevance includes the distribution and location of the different particle sizes, the orientation of particles and aggregates and their contacts, distribution and shape of void spaces, for each particle the number of contacts with supporting and non-

supporting particles, and the presence of discontinuities. Much attention has been paid to the quantification of soil fabric in the last 25 years.

The importance of the spatial distribution of local void ratio at high global void ratios, and the presence of macrovoids, have been illustrated in the paper and both need to be taken into account when explaining observed behaviour and predicting future behaviour.

Macrovoids can exist in natural or laboratory prepared soils, depending on depositional environment or specimen preparation method. Macrovoids close first during consolidation and collapse first during shear. It has been suggested herein that macrovoids contribute to a chain reaction responsible for the spreading of collapse.

In an under-filled situation, 'fines' can lead to the presence of macrovoids, depending on the size disparity of fines and host sand, their shape and their location within the particle arrangement or load carrying framework. Are the fines acting as: fillers of voids, simply reducing void ratio but with little effect, other than on permeability; as stabilisers in the load carrying framework; as disruptors in the framework; all as speculated on by Leroueil and Hight (2003). Because of the dependence on their location, neither void ratio nor relative density can provide satisfactory indications of instability when smaller particles are present within a host sand.

10.2 Heterogeneity and localisation

Many, possibly most, natural and man-made cohesionless soils are not homogeneous. There is heterogeneity of local void ratio, particularly at high global void ratios, and heterogeneity of the load carrying structure. Evolving fabric modifies anisotropy and heterogeneity. Both the initial and stress induced heterogeneities trigger localisation which leads to increased heterogeneity of shear and volume strain, and pore pressure. Localisation can trigger a chain reaction.

There appears to be little information on localisation in cohesionless soils in situ and whether the kinematics of localisation in the field match those in the laboratory tests.

10.3 Critical state

The attraction of critical state is that at large strains it is considered that the effects of stress history and initial fabric are obliterated, so critical state depends only on current stress state, density of packing (global void ratio) and particle characteristics. As discussed, difficulties arise in practice because of layering, bedding and localisation, and there is uncertainty over the dependence of critical state on stress path, including the effects of b and α .

Inherent and evolving spatial variations in local void ratio give rise to uncertainties over the local void ratio determining the critical state, for example the local void ratio in a shear band. This difficulty would appear to be more

acute when combining a critical state approach with the use of loose moist tamped samples.

Localisation results in a rate effect on strength at critical state, apparent when comparing load and strain-controlled conditions.

10.4. Instability

An instability approach, determining the variation in inclination of instability lines with void ratio and principal stress direction in sedimented sands and silty sands, would appear to have a key role to play in design and analysis. The zone of potential instability increases with α and global void ratio. Reaching the instability surface determines whether flow will or will not occur. Identifying this zone is more valuable than attempting to identify post collapse strengths.

The attraction of instability lines is that instability in sedimented sands occurs at small strains, prior to major particle rearrangement and so before ϕ' is mobilised and before localisation is dominant.

In design, the aim would be to keep stress states below the local instability zone, as we did at Jamuna. Lade (1999) advised as follows: “*Analysis of instability requires determination of the current effective stress states in the slope and recognition of their locations relative to the instability line. Instability can potentially be triggered in any region of the slope in which the stress states are above the instability line. The analysis procedure can only indicate the potential for collapse. A trigger is required to initiate the actual collapse*”.

In the zone of potential instability, the soil will be stable if drained conditions are maintained, so the design process requires a risk analysis of potential triggers that will lead to undrained or partially drained conditions.

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Though I have never met Professors Lade or Vaid, I appreciate hugely the insights they have provided in their publications on instability and the testing of sands

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.

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ARTICLES

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Influence of the addition of carbon nanotube on the physical behavior of a lateritic soil from the southwest Amazon

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Article

Keywords

Lateritic soil
Carbon nanotubes
Soil improvement
Nanotechnology

Abstract

In this work we evaluate the physical-mechanical behavior of lateritic soil with addition of carbon nanotubes. The soil was collected in a commercial deposit located in Rio Branco – AC and later characterized through particle size tests, X-Ray Diffraction, X-Ray Dispersion Spectroscopy and Tropical Compressed Miniature essay. The dispersion of nanotubes in solution was carried out and the size of the nanoparticles was verified using Dynamic Light Spreading - DLS, Zeta Potential and *PDI*. Three percentages of additions (0.05%, 0.1% and 0.2%) of carbon nanotubes were evaluated and compared with the control group according to the parameters of Atterberg limits, real density, dry density maximum and optimum humidity. The results indicate that the soil has a clayey behavior with a medium texture, with the presence of clay mineral kaolinite in its composition and silicon and iron oxides. The dispersion of carbon nanotubes reached particles with an average hydrodynamic diameter of 68.9 nm and Zeta Potential of -24.87 mV and *PDI* of 0.231, characterizing a solution as moderately dispersed and kinetically stable. The results of the liquidity limit and plasticity tests showed a reduction of 10 and 13%, respectively, with the addition of carbon nanotubes, while for the parameters of plasticity index, real specific mass, maximum dry density and optimal moisture, they did not show significant variation. Carbon nanotubes interact with soils with lateritic behavior, and further studies are essential to better understand the mechanisms behind this interaction.

1. Introduction

One of the biggest challenges in carrying out infrastructure or paving works is related to finding soil deposits with low quality geotechnical properties, such as low resistance and high deformability, requiring geotechnical engineering to apply soil improvement techniques that minimize the problems of settlement and stability that are typical of these soils (Silva et al., 2020).

Among the fine soils widely used in bases and sub-bases of pavements, soils that present lateritic behavior stand out, where despite their natural state presenting low support capacity, when compacted they acquire high resistance, a factor that, together with its abundance in Brazilian territory, justifies its use in urban paving works (Arruda, 2020).

However, despite its properties enabling its use in the construction of traffic routes, soils with lateritic behavior may require the use of improvement techniques to increase resistance due to the most varied loads that the highway must withstand (Silva & Castro, 2020). Located in the extreme southwest of the Brazilian Amazon, the state of Acre is geologically characterized by a sedimentary basin with a thick horizon of fine material, occupying almost the entire surface of the state, where about 70% of the soil in the region is composed of clayey soils, which has low support capacity and high expansion in the presence of water (Acre, 2010). According to the AcreBioClima Environmental Studies and Services Group (Acre, 2010), the state has monthly rainfall indices above 200 mm, factors that make it difficult to build roads with the minimum resistance indices adopted by conventional methods, such as the *CBR*.

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Soil chemical stabilization is the improvement of soil geotechnical properties, notably strength, durability and deformability, and usually occurs through the addition of cementitious materials to the soil in situ (Corrêa-Silva et al., 2020). Among the main types of stabilizing agents, lime, Portland cement, asphalt, emulsions and acrylic polymers can be mentioned (Machado et al., 2017). However, the significant environmental impacts associated with the production of these elements bring a global concern (Eyo et al., 2020). With this, nanotechnology has been proving to be a promising area, since the use of nanomaterials, in addition to being feasible from a technical point of view, is also proving to be an economical alternative, due to the very low amount of nanomaterials needed in the soil to achieve improvement in the geotechnical parameters (Karumanchi et al., 2020).

Among nanomaterials, carbon nanotubes (CNT) are considered one of the most promising in the construction industry. CNT's have excellent properties, such as high tensile strength, high modulus of elasticity and excellent electricity conductor (Taha et al., 2018). In addition, these nanomaterials are already widely available on the market, with good quality products and relatively low prices, which favors their use for research (Taha & Alsharef, 2018). Compared to other types of cementitious materials used for soil stabilization, CNTs have proven to be an ecological alternative, highlighting their high potential for absorbing biological contaminants or those associated with bodies of water, due to their high specific surface area (Alobaid et al., 2022).

Despite this scenario, there is still little research related to the use of nanomaterials in soils to verify improvements in geotechnical properties, and even less for the investigation of these materials in soils with lateritic behavior, making this research a pioneer in this area of knowledge. The objective of

this work is to verify the influence of the addition of carbon nanotubes in a lateritic soil of the southwest region of the Amazon, aiming at improvements in the physical properties.

2. Materials and Methods

2.1 Description of study place

The State of Acre is in the extreme southwest of the Brazilian Amazon, between latitudes 07°07'S and 11°08'S and longitudes 66°30'W and 74°WGr. Its territorial surface is 164221.36 km², corresponding to 4% of the Brazilian Amazon area and 1.9% of the national territory (Acre, 2010).

Among the geological units, the Solimões Formation occupies about 85% of the territory of Acre, and this formation originated from sediments coming from rivers from the Cretaceous period (Acre, 2010). Dystrophic Red-Yellow Argisols and Latosols dominate the eastern tablelands of the State, with deep, well-drained, leached soils, low natural fertility and the occurrence of lateritic crusts. Such characteristics result from the fact that the mineralogical composition of the Solimões Formation consist of clays with a high expansion content due to moisture variations (Shinzato et al., 2012).

2.2 Collection of samples

The lateritic soil samples come from a commercial deposit located on BR 317, km 15, coordinates 10° 06' 29.37" S and 67° 38' 39.66" W, Rio Branco – AC, Brazil, as shown in Figure 1.

After collection, the materials were stored, homogenized and sealed at room temperature at the Soil Laboratory of the Federal University of Acre (UFAC).

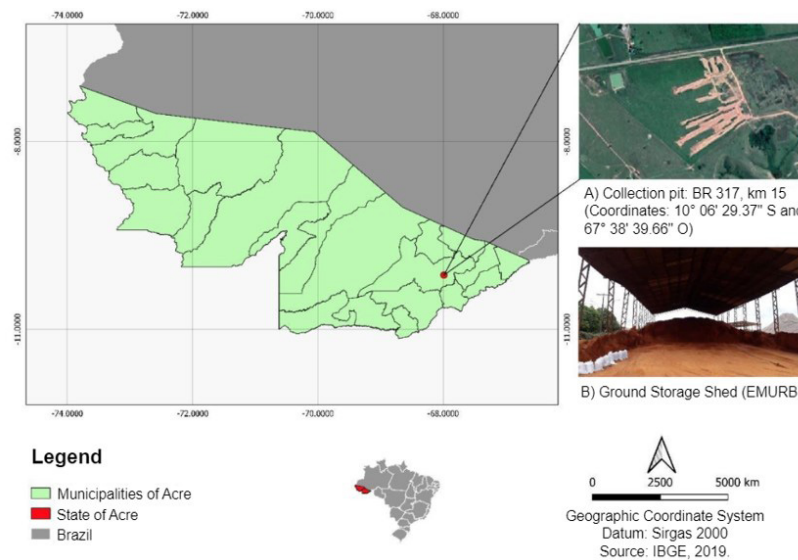


Figure 1. Location of the lateritic soil removal deposit.

2.3 Soil characterization

2.3.1 Granulometric analysis

To determine the dimensions of the lateritic soil particles, as well as their percentages, the granulometry test was carried out using sieves and sedimentation in accordance with NBR 7181/16 (ABNT, 2016). The granulometric analysis by sieves was carried out at the UFAC Soil Laboratory, while the granulometric analysis by sedimentation was carried out at the Technology Foundation of the State of Acre (FUNTAC).

2.3.2 MCT test

To verify the lateritic behavior of the soil, the MCT test was carried out, according to the DNER ME-258/94 and DNER ME-256/94 (DNER, 1994d, 1994e) standards, which are the Mini-MCV and mass loss by immersion tests, respectively. The MCT methodology is used to distinguish the behavior of soils from tropical regions, especially those soils with considerable particle sizes smaller than 2 mm (Lima et al., 2020). The following coefficients are calculated: c' (grading parameter that differentiates sandy and clayey materials), d' (parameter of the compaction curve that is directly related to the maximum density) and P_i (parameter that quantifies the degree of soil laterization). These parameters are necessary to calculate e' , which is the parameter that classifies the soil as being lateritic or non-lateritic. All steps were carried out at the FUNTAC Soil Laboratory.

2.3.3 Chemical analysis

The chemical composition of the lateritic soil sample was carried out using an energy dispersive X-ray system (EDX), in the forensic laboratory of the Civil Police of the State of Acre. The equipment used was a Shimadzu EDX 720. EDX is an essential accessory in the study of microscopic characterization of materials, as it allows immediate identification of the chemical composition (Araújo, 2022). When the electron beam falls on a mineral, the outermost electrons of the atoms and the constituent ions are excited, changing energy levels. When they return to their initial position, they release the acquired energy, which is emitted at wavelengths in the X-ray spectrum. A detector installed in the vacuum chamber measures the energy associated with this electron. As the electrons of a given atom have different energies, it is possible, at the point of incidence of the beam, to determine which chemical elements are present in that location and thus instantly identify which mineral is being observed.

2.3.4 Mineralogical analysis

To carry out the mineralogical analysis of the existing clay fraction in the lateritic soil studied (fraction less than

0.002 mm), the X-ray diffraction technique was used, using the X-Ray Diffractometer equipment. The test was carried out at the Department of Geoscience of the University Federal of Amazonas, with equipment model Shimadzu XRD 6000®, using X-ray tube with Cu K α radiation, wavelength of 0.15418 nm. The configuration used was a monochromator with slits (1, 1, 0.30), at 40kV and a current of 30 mA and a speed of 2000 deg/min. The identification of clay minerals present in the soil was performed using the Match! software. When characterizing clay minerals, the use of the X-ray diffraction technique becomes even more indicated, since the high quartz content of the sample and its ease of orientation result in well-defined peaks of great intensity in this crystalline phase, which helps in the identification of phases (Albers et al., 2002).

2.4 Preparation and characterization of carbon nanotubes

2.4.1 Obtaining carbon nanotubes

The CNTs used in the research are of the multiple wall type (MWCNT), and were commercially obtained from the company CNTCompany, located in South Korea, and the product in question is called CTube120.

2.4.2 Dispersion of carbon nanotubes

Carbon nanotubes are very small particles with a very large specific surface, which makes them tend to attract and agglomerate (Cwirzen et al., 2008). To take real benefits from its potential, it is necessary to make a good dispersion of the particles. The methodology used in this work was adapted from Taha et al. (2018) and Rafael (2017).

First, a 0.15% Tween 80 surfactant solution was prepared to promote better solubilization of the nanotubes in the aqueous medium. The use of the surfactant is justified because it has the property of reducing the interfacial tension between two components of different affinities (Leite, 2020), in the example water and nanotubes, forming a homogeneous mixture. The solution was placed on a magnetic stirrer for 2 h, and then the carbon nanotubes were added at a concentration of 0.01% in relation to the solution, as shown in Figure 2.

However, just inserting the nanotubes into the solution is not enough, as the nanotubes tend to agglomerate in the dispersion. Therefore, an ultrasonic homogenizer brand Sonoplus Bandelin, model UW 2070, equipped with a titanium rod was used. Rafael (2017) verified that the increase in temperature produces negative effects on the dispersion of MWCNT, and with that, the solution was stored in a container containing crushed ice, to control the temperature during the test. Ultrasound was used with a frequency of 20 kHz and power of 500 W for 10 minutes, as shown in Figure 3.

After the process of dispersing the nanotubes through the ultrasonic homogenizer, the amount of soil necessary for

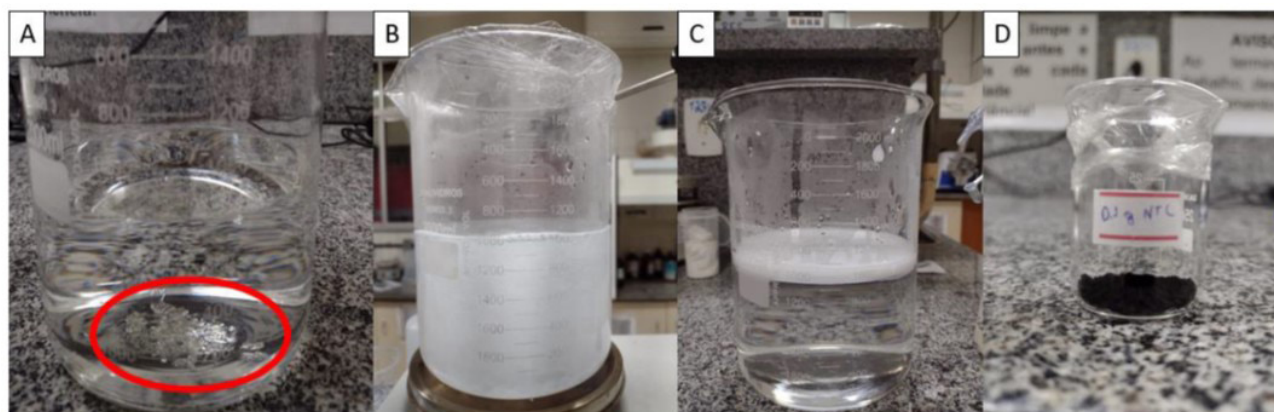


Figure 2. Solution for carbon nanotubes dispersion. (A) Solution with Tween 80 surfactant (before the shaker); (B) Solution during magnetic stirrer; (C) Solution after 2 h period on shaker; (D) Nanotubes used in the solution.

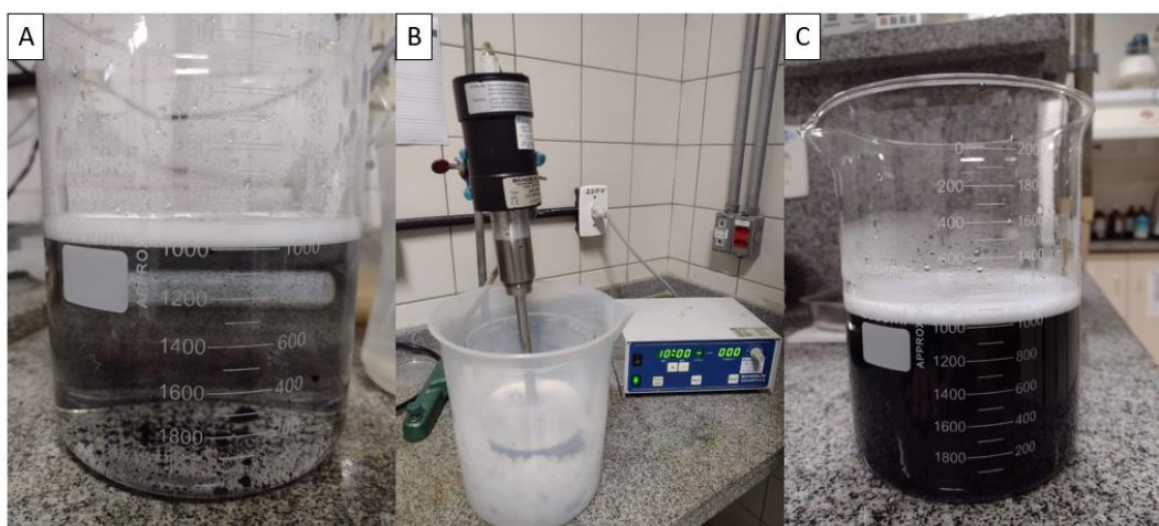


Figure 3. Dispersion process of carbon nanotubes. (A) Addition of the nanotubes to the solution before the ultrasound; (B) Solution in a temperature-controlled container for use in ultrasound; (C) Solution after 10 minutes in the ultrasonic homogenizer.

carrying out each test was added inside the container with the solution, the magnetic stirrer was used for 15 minutes and then the material was taken to an oven at 100 °C, for complete drying of the water. Finally, the dry material was slightly crumbled in a mortar with the aid of a mortar hand and separated into identified flasks referring to the test to be carried out with the appropriate percentages of carbon nanotubes for this research, as shown in Figure 4.

2.4.3 Average size, Polydispersity Index (*PDI*) and Zeta Potential

The particle size after the carbon nanotube dispersion process, the polydispersity index and the Zeta Potential were determined with Zetasizer Nano ZS 90 equipment (Malvern Instruments; Malvern, UK), performed using the Dynamic Light Scattering (EDL) technique. This technique is widely used for evaluating the dimensions of particles in suspension, because when the particle is illuminated by a

light source, it will scatter light in all directions, and this technique consists of analyzing the intensity fluctuations of the scattered light at a certain angle, which provides information about the particle movement and, consequently, its mean hydrodynamic diameter (Nomura et al., 2014).

In addition, through the *PDI* and the Zeta Potential, it is possible to evaluate the quality and stability of the solutions. In an Eppendorf tube, a solution was prepared with 100 μ L of each sample together with 2 mL of ultrapure water, and the solution was stirred for approximately 2 min. 900 μ L of each solution were removed for analysis of particle size, *PDI* and Zeta potential. The analyzes were made from 10 runs with 3 reading cycles and a scattering angle of 173° and a temperature of 25 °C. The average particle diameter and the *PDI* were calculated from the particle size distribution and the Zeta Potential values were determined by measuring the electrophoretic mobility, expressed in mV.

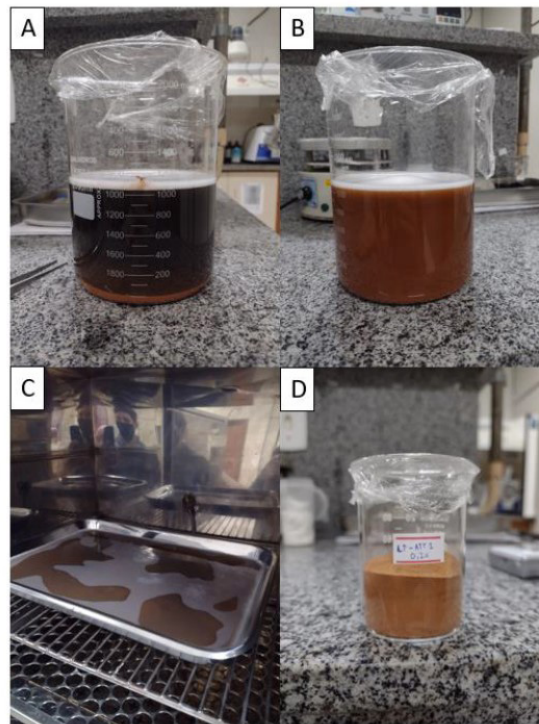


Figure 4. Process of preparation of samples used for physical and mechanical tests. (A) Addition of soil to the solution, before the agitator; (B) Soil solution + nanotubes after the shaker; (C) drying the solution in an oven; (D) Sample to be used in physical.

2.5 Statistical and experimental design

Different percentages of carbon nanotubes (0, 0.05, 0.1 and 0.2%) were added to the soil samples, as proposed by Alsharif et al. (2016), and subsequently evaluated and compared the physical properties of the soil with the addition of carbon nanotubes compared to the original soil.

For these tests, a factorial experiment installed in a completely randomized design, in triplicate, was used, and the data obtained in the mechanical tests were statistically analyzed according to the normality of the errors and the homogeneity of the variances of the data, and if assumptions were met, it was carried out an analysis of variance (ANOVA) and Tukey's test with 5% probability for comparison between means, as shown in Table 1. The homogeneity of variances was performed using the Prophet® software, while the normality of errors and ANOVA were performed using the Sisvar® program.

2.6 Physical and mechanical soil tests

To carry out the liquidity limit test, the recommendations of the DNER ME 122/94 (DNER, 1994b) standard were followed. As for the plasticity limit test, it was performed according to the DNER ME 082/94 standard (DNER, 1994c). To determine the plasticity index, the averages of the liquidity limits were subtracted from those of the plasticity limits. The Atterberg limits tests were performed with the samples

Table 1. Statistical and experimental design of the research.

Treatments	Repetitions		
	I	II	III
Group 01 (0% of MWCNT)	G1.1	G1.2	G1.3
Group 02 (0.05% of MWCNT)	G2.1	G2.2	G2.3
Group 03 (0.1% of MWCNT)	G3.1	G3.2	G3.3
Group 04 (0.2% of MWCNT)	G4.1	G4.2	G4.3

from the control group and the samples with the addition of carbon nanotubes. For the determination of the Atterberg limits, approximately 200 g of material (air-dried) passing through the #40 sieve (0.42 mm) was separated. Atterberg limits define the consistency and behavior of cohesive soil at different water contents. These limits are important as they are related to the parameters of contraction, expansion and settlement of the soil (Rehman et al., 2020).

To determine the maximum dry density and optimal moisture, the recommendations of the DNIT 164/2013 (DNIT, 2013) standard were followed, which provides for compaction using unworked samples, using normal compression energy. To determine the real specific mass of the lateritic soil before and after the addition of different percentages of carbon nanotubes, the recommendations of the DNER ME 093/94 (DNER, 1994a) standard were followed.

3. Results and discussions

3.1 Soil characterization

3.1.1 Granulometry

Through the granulometry test, it was possible to obtain the granulometric curve of the soil, as shown in Figure 5.

Through Figure 5, it is possible to see that this is a soil with more than 50% of its composition consisting of fine particles (silt and clay), being composed mainly of clay. From the percentages of each soil component, it is possible to classify the soil as a medium texture soil (Santos et al., 2018). Texture is one of the main indicators of soil quality, since it influences the dynamics of adhesion and cohesion between soil particles, as well as the dynamics of water in the soil (Centeno et al., 2017).

3.1.2 MCT test

Through the MCT test, it was possible to determine whether the soil in question has a lateritic behavior or not. Thus, according to Figure 6 and after the tests performed, the soil was classified as sandy lateritic.

According to the Unified Soil Classification System (USCS), the soil used in the research was determined to be between clay or silt of low compressibility, which corroborates the statement that the soil is lateritic, with non-plastic behavior and non-expansive, according to Figure 7.

In the HRB classification system, the soil was classified as a clayey soil of the A-7-5 group, presenting general behavior as a weak to poor subgrade. However, it is important to point out that the use of techniques considered traditional for soil classification is not suitable for tropical soils, given the peculiarities inherent to this type of soil (Nogami & Villibor, 1991).

3.1.3 Chemical analysis (EDX)

Figure 8 shows the chemical composition of the soil, obtained through the EDX technique. The main constituents of the soil are the minerals silicon, iron and titanium, which are present in the form of oxides. The elements Si and Fe are considered the most important chemical elements of lateritic soils (Borba, 1981). This collaborates, together with the results obtained in the DRX, that the analyzed material has the characteristics of a lateritic soil.

3.1.4. Mineralogical analysis (DRX)

The X-ray diffractogram of the soil used in the research is shown in Figure 9. Although there are well-defined peaks at $2\theta = 26.66^\circ$ and $2\theta = 20.86^\circ$, these peaks correspond to the quartz minerals present in the sample (Hazen et al., 1989). The peaks of $2\theta = 50.12^\circ$, 22.15° and others correspond to the kaolinite clay mineral (Gruner, 1932). Kaolinite has a

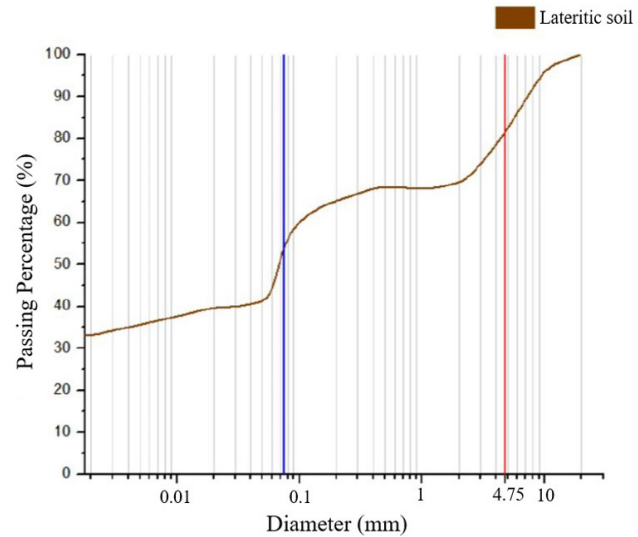


Figure 5. Graph of soil granulometry.

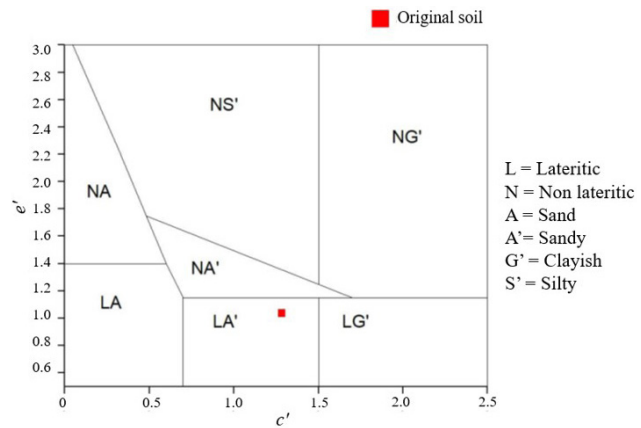


Figure 6. MCT classification abacus with soil results used in research.

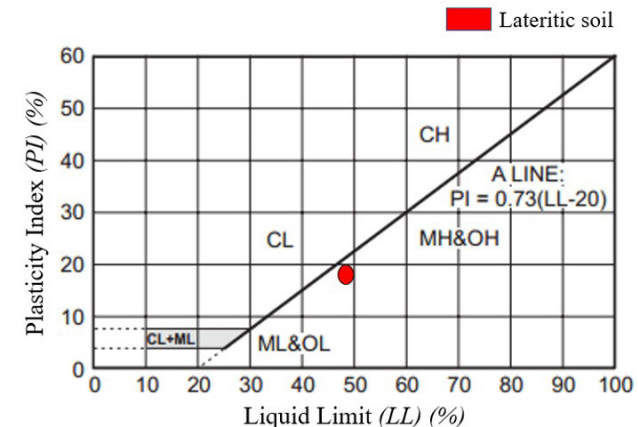


Figure 7. Plasticity chart with soil classification.

lamellar structure formed by the regular stacking of layers 1:1, in which each layer consists of a sheet of silicon and oxygen tetrahedrons and a sheet of alumina octahedrons with basal spacing of approximately 0.7 nm and of chemical

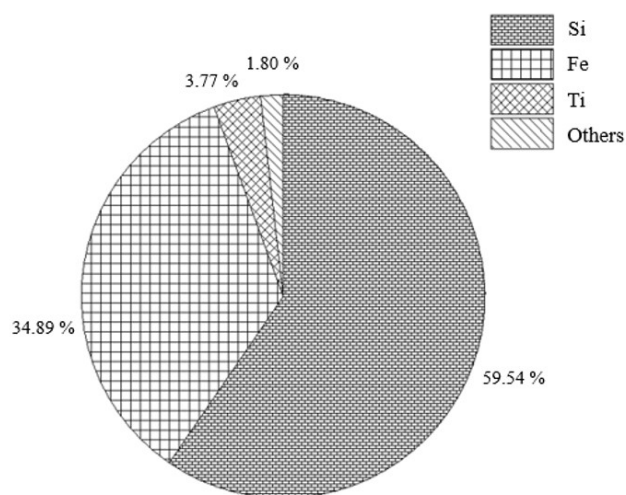


Figure 8. Soil Chemical Composition.

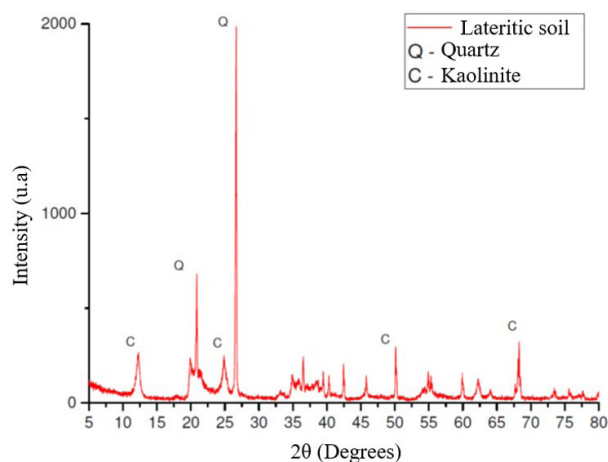


Figure 9. Lateritic soil X-ray diffractogram.

composition $\text{Al}_2\text{O}_3 \cdot 2\text{SiO}_2 \cdot 2\text{H}_2\text{O}$ (Scorzelli et al., 2008). This mineral clay, due to its low expansion potential, is indicated for use in impermeable barriers, in addition to having less water dispersion (Borba et al., 2019).

3.2 Carbon nanotubes

3.2.1 Characterization of nanotubes

The properties of the nanotubes are shown in Table 2. Images of carbon nanotubes through scanning electron microscopy are shown in Figure 10.

3.2.2 Dispersion of carbon nanotubes (average size, *PDI* and Zeta Potential)

Through the DLS, it was possible to analyze the average hydrodynamic diameter of the nanoparticles dispersed in the solutions. The result is shown in Figure 11, where particles with an average hydrodynamic diameter of 68.9 nm were

Table 2. Properties of carbon nanotubes (CTube 120).

Property	CTube 120
Appearance	Black Powder
Carbon Purity (%)	≥ 97
Average diameter (nm)	20
Length (μm)	1-25
Metal oxide (%)	≤ 3
Apparent density (g/cm^3)	0.030-0.050
Specific surface (m^2/g)	150-250

obtained, satisfactorily meeting the requirement that the particles remain in the nanometer scale after dispersion. The results show better results than research done with other types of surfactants, as shown in Table 3.

It is possible to notice that the solution becomes more homogeneous with low levels of surfactants, followed by a relative trend of stability as the concentration increases. This variation can be divided into two states: one where the surfactant is limited and another where there is an excess. At low concentrations, there is not enough surfactant to protect all particles present in the solution. Consequently, small oil droplets collide with each other, which increases the particle size. The dimensions of the particles, in this state, are defined by the type and concentration of surfactant present in the solution (Yang et al., 2013; Zhu et al., 2019). The surfactant used in the research, Tween 80, is highly recommended to obtain homogeneous solutions and highly ordered nanoparticles, due to its hydrophilic character (Hamza et al., 2020).

Figure 12 shows the graph of the Zeta Potential of the solution. The average Zeta potential obtained in all experimental runs was around -24.87 mV and the *PDI* obtained was around 0.231. In general, $PDI < 0.1$, $0.1 - 0.4$ and > 0.4 represent highly dispersed, moderately dispersed and colloidal solutions, respectively (Zaib et al., 2020). The closer to 0 mV the Zeta Potential value, the greater the tendency for the solution to agglomerate, while values close to -40 mV represent solutions with good stability (Salopek et al., 1992; Clogston & Patri, 2011). These results indicate that the solution appears as moderately dispersed, and the Zeta Potential far from 0 mV indicates that the particles present in the solution present enough repulsion to keep it stable.

3.3 Physical tests

3.3.1 Atterberg limits

The results of the Atterberg limits tests are shown in Figure 13. First, the homogeneity of the data variances was calculated using the Bartlett test and the normality of the errors using the Shapiro-Wilk test, where the hypothesis of difference in variances and abnormality in errors was rejected. With that, we proceeded with the ANOVA.

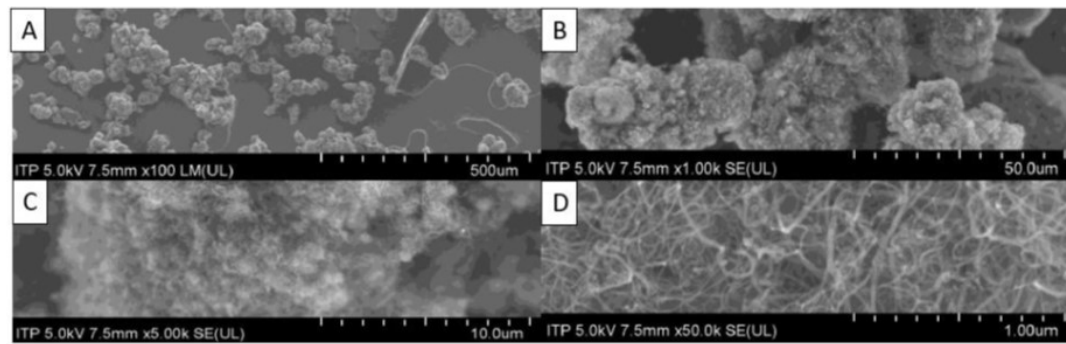


Figure 10. Scanning Electron Microscope (SEM) image of carbon nanotubes. (A) 20x magnification; (B) 200x magnification; (C) 1000x magnification; (D) 10.000x magnification.

Table 3. Mean hydrodynamic diameter obtained from MWCNT in solutions with different surfactants, with 0.01% MWCNT.

Author	Surfactants	Average diameter (nm)
Figueiredo et al. (2015)	Glycerox 0.5%	197.2
	Glycerox 1%	167.6
	Glycerox 3%	175.2
	Amber 4001 0.5%	521.45
	Amber 4001 1%	322.85
	Amber 4001 3%	316.8
Rafael (2017)	Pluronic F-127 0.03%	198.4
	Policarboxilato 0.05%	221.1
	SDBS 0.03%	206.9
	SODOCO 0.03%	207.1
	Triton X-100 0.1%	174.0
	SDBS 0.015% + Pluronic F-127 0.015%	191.3
	SDBS 0.015% + Policarboxilato 0.25%	200.2
	SDBS 0.015% + SODOCO 0.015%	205.1
	SDBS 0.015% + Triton X-100 0.05%	188.0
	SODOCO 0.015% + Pluronic F-127 0.015%	191.0
Moura (2015)	Amber 2006 0.1%	354.9
	Amber 2006 0.5%	431.1
	Amber 2006 1%	514.9
	Amber 2006 2%	816.0
	Amber 2002 0.1%	424.0
	Amber 2002 0.5%	304.1
	Amber 2002 1%	399.9
	Amber 2002 2%	623.9
	Amber 2001 0.1%	550.0
	Amber 2001 0.5%	548.0
	Amber 2001 1%	871.6
	Amber 2001 2%	985.0

From the results obtained, the addition of CNTs to the soil showed a significant difference in the parameters of liquidity limit and plasticity limit, while for the plasticity index there was no significant difference. The percentage of addition that proved to be the most efficient was 0.2%, presenting a reduction according to the percentage variation in relation to the initial value of about 10% in the *LL* and 13% in the *PL*, followed by the percentages of 0.1%, which

showed a reduction of 15% in the *PL* and 0.05% with similar values for *LL* and *PL*.

Atterberg limits play an important role in soil identification and classification. These parameters may indicate some possible geotechnical problems that may arise in the soil, such as potential for expansion and workability (Alsharef et al., 2016). Reductions in the Atterberg limits are indicators of soil improvement (Majeed & Taha, 2012).

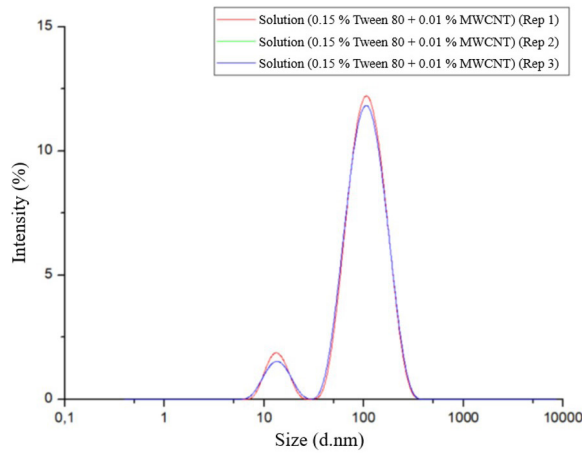


Figure 11. DLS with the dispersion solution with 0.15% Tween 80 and 0.01% MWCNT.

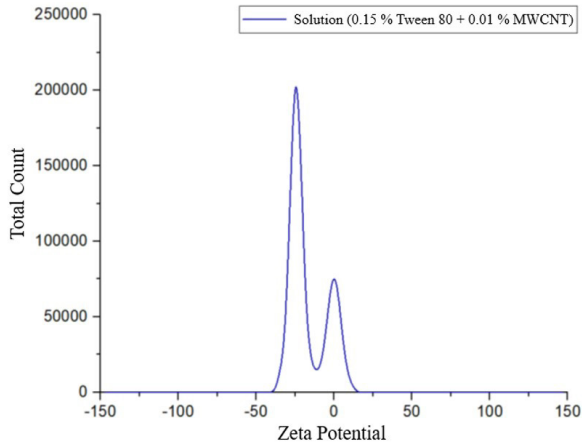


Figure 12. Graph of the Zeta Potential of the solution with 0.15% Tween 80 and 0.01% MWCNT.

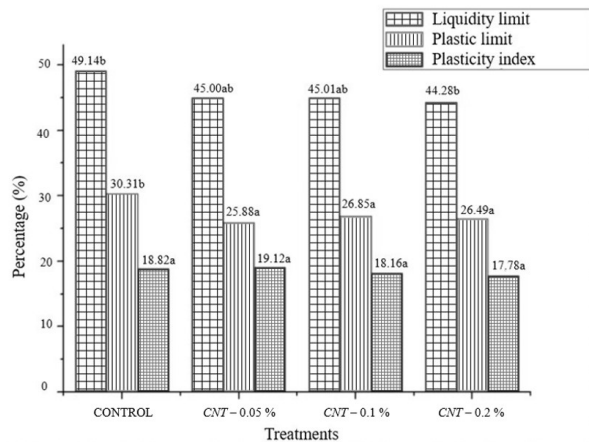


Figure 13. Averages of the Atterberg limits for the samples (control groups and with different additions of CNTs). Obs: Groups with the same letter do not differ significantly in relation to their means, according to the Tukey Test ($\alpha = 5\%$).

A possible explanation for the reduction of these indices is that the carbon nanotubes form a coating layer around the clay lamellae, which causes a suppression of its reactivity, reducing the possible absorption of water from the soil (Krishnan & Shukla, 2019).

The result of soil classification according to the Unified Soil Classification System is shown in Figure 14. It is possible to see that, even after changes to the Atterberg limits, the soil maintained its non-plastic and non-expansive behavior characteristics. This behavior is advantageous, as an increase in Atterberg limits is related to a greater potential for soil expansibility (Mearek et al., 2022).

3.3.2 Real specific mass

The results of the real specific mass tests are shown in Figure 15. First, the homogeneity of the data variances was calculated using the Bartlett test and the normality of the errors using the Shapiro-Wilk test, where the hypothesis of difference in variances and abnormality in errors was rejected. With that, we proceeded with the ANOVA.

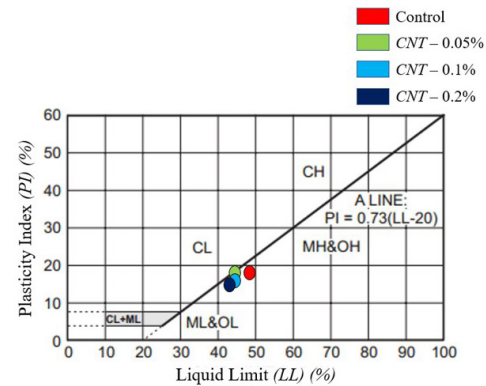


Figure 14. Sample results with CNTs according to USCS.

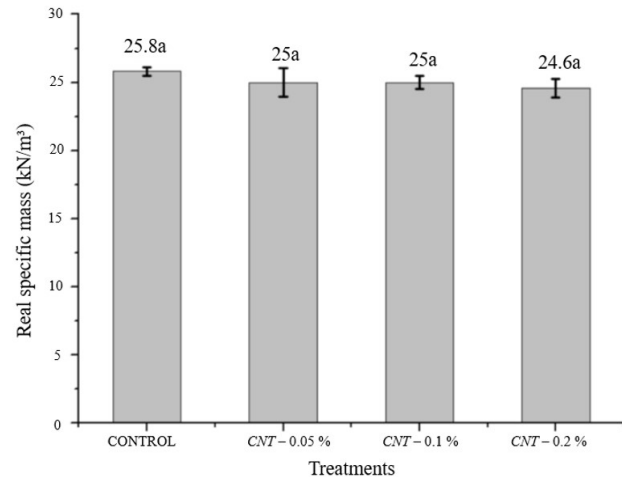


Figure 15. Averages of the real specific mass of the samples of the control groups and with different additions of CNTs. Obs: Groups with the same letter do not differ significantly in relation to their means, according to the Tukey Test ($\alpha = 5\%$).

Although there was a reduction in the mean real specific mass as the percentage of CNTs in the samples increased, this reduction was not significant. This reduction is expected, due to the low density of CNTs, in addition to meeting the results obtained by Taha & Alsharef (2018). However, due to the non-significant result, it is possible that higher percentages of addition of carbon nanotubes in the soil could lead to relevant results, due to the tendency of reduction as the percentage of CNTs increases, observed in Figure 15.

3.3.3 Maximum dry density and optimum moisture

The homogeneity of data variances was calculated using the Bartlett test and the normality of errors using the Shapiro-Wilk test, where the hypothesis of difference in variances and abnormality in errors was rejected. With that, we proceeded with the ANOVA. The results of the mean maximum dry density and optimal moisture of the samples from the control group and with different additions of CNTs are shown in Figure 16.

The values of maximum dry density showed an increase in the percentages of addition of 0.05% and 0.1% of CNTs, while there was a reduction in the values of optimal moisture in the observed treatments. However, there was no significant variation in both maximum dry density values and optimal moisture values. The increase and decrease in the values of maximum dry density and optimal moisture, respectively, are in line with those obtained by Alsharef et al. (2016), which also showed a small variation in these values of 2% and 6%.

The increase in density can be caused by the rearrangement and consolidation of soil particles promoted by filling the nanoparticles in the voids present in the specimen (Krishnan & Shukla, 2019). The reduction in the optimal moisture content can be explained by the formation of nanocomposites that fill the voids in the soil, occupying the water space, where the ion exchange mechanism is the main phenomenon that maintains the connection between clay minerals and

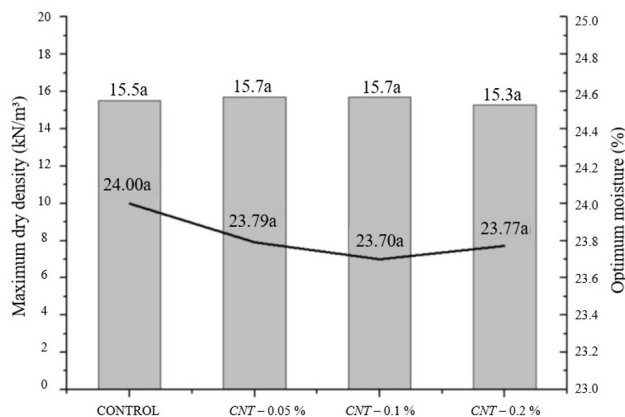


Figure 16. Means of maximum dry densities and optimal moisture of soil samples (control groups and with different additions of CNTs). Obs: Groups with the same letter do not differ significantly in relation to their means, according to the Tukey Test ($\alpha = 5\%$).

carbon nanotubes (Azzam, 2014). However, despite these mechanisms, the results obtained here do not demonstrate that there was a significant change in these parameters, so performing treatments with different percentages can lead to values with a significant difference.

4. Conclusions

The granulometry test revealed that the soil has more than 50% of its weight passing through the #200 sieve, being predominantly composed of silt with 24.76% and clay with 33.05%, being able to classify the soil as being a clayey soil of texture average. The MCT methodology allowed classifying the soil as a lateritic sandy type. Through the granulometry results and the Atterberg Limits, the soil was classified as low compressibility clay/silt in the USCS soil classification system and A-7-5 in the HRB system, classifying it as a clayey soil with weak behavior as subgrade. The X-Ray Diffraction analysis allowed us to identify the clay minerals present in the soil, where kaolinite was the main component of the soil, and together with the chemical test through EDX it allowed us to identify the main chemical compounds present in the soil, with predominance for silicon and iron. The results obtained from these tests corroborate the statement that the soil used in the research was a soil with lateritic behavior.

The Dynamic Light Scattering measurements showed that solutions with the dispersion of carbon nanotubes resulted in an average hydrodynamic diameter of the particles of 68.9 nm, a result considered to be of better quality compared to other studies, and the Zeta Potential was -24.87 mV and *PDI* of 0.231. These results confirm that the dispersion was carried out properly and that the solution is considered moderately dispersed, with a good tendency towards kinetic stability, where it was possible to reach the nanometric scale of carbon nanotubes in the solution. These factors collaborate so that the results of the mechanical and physical tests are reliable.

According to the physical tests, the addition of different percentages of carbon nanotubes showed a reduction of 10 and 13%, respectively, in the parameters of liquidity limit and plasticity limit, and these results indicate a tendency for improvement in the soil, since as the liquidity limit decreases, so does soil compressibility, among other variables. However, despite the variation in the mean values of real specific mass, maximum dry density and optimal soil moisture, it was not possible to observe a significant difference in the results of these parameters after the addition of different percentages of carbon nanotubes in comparison with the samples. control group soil. Running these tests with different percentages of nanotubes can lead to results with significant variation, so further studies are needed to investigate these variations.

Advances in MWCNT synthesis methods have resulted in a substantial decrease in their cost, which opens several possibilities for their use in different fields. In the present study, the percentage that proved to be most advantageous was 0.2% of carbon nanotubes, which represents a very

small amount in relation to the soil, which leads to low implementation costs. However, special attention must be paid to the techniques for dispersing carbon nanotubes before mixing, as it plays a fundamental role in the correct incorporation of nanomaterials into the soil.

Through this work, it is possible to state that carbon nanotubes interact with soils with lateritic behavior, and that further studies are essential to better understand the mechanisms behind this interaction. During the research, it was not possible to find in the scientific literature works that use carbon nanotubes in the region of Acre or even in the North of Brazil, so that this work constitutes a pioneer in the investigation related to the addition of nanomaterials in soils of the region, establishing it as a scientific basis for further research with the use of nanoparticles in soils in the region to have a scientific-technical basis.

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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

Marcelo Victor de Assis Moraes: conceptualization, data curation, formal analysis, investigation, methodology, visualization, writing – original draft. Marcelo Ramon da Silva Nunes: methodology, resources, validation, writing - reviewing and editing. Carlos Drumond do Nascimento Moraes: methodology, investigation, visualization, writing - reviewing and editing. Ricardo Ribeiro do Nascimento: conceptualization, formal analysis, methodology, project administration, investigation, supervision, validation, writing – review & editing. Anselmo Fortunato Ruiz Rodriguez: conceptualization, funding acquisition, methodology, project administration, resources, supervision, validation, writing - reviewing and editing.

Data availability

All the data and models that support the findings of this study are available from the corresponding author upon request.

List of symbols and abbreviations

c' Parameter that differentiates sandy and clayey materials

d'	Parameter of the compaction curve that is directly related to the maximum density
e'	Parameter that classifies the soil as being lateritic or non-lateritic
ANOVA	Analysis of Variance
CBR	California Bear Rating
CH	Highly plasticity inorganic clays
CL	Low plasticity inorganic clays
CNT	Carbon Nanotubes
DLS	Dynamic Light Scattering
DNER	National Department of Highways
DNIT	National Department of Transport Infrastructure
DRX	X-Ray Diffractometer
EDS	Energy Dispersive Spectroscopy
EDX	Energy Dispersive X-ray System
EMURB	Municipal Urbanization Company of Rio Branco
FUNTAC	Technology Foundation of the State of Acre
HRB	Highway Research Board
LA	Lateritic Sand
LA'	Lateritic Sandy
LG'	Lateritic Clayish
LL	Liquid Limit
PL	Plastic limit
MCT	Tropical Compressed Miniature
MCV	Moisture Condition Value
MH	High plasticity silt
ML	Low plasticity silt
MWCNT	Multi Wall Carbon Nanotubes
NA	Non lateritic Sand
NA'	Non lateritic Sandy
NG'	Non lateritic Clayish
NS'	Non lateritic Silty
OH	High plasticity organic clay
OL	Low plasticity organic clay
PDI	Polidispersivity Index
Pi	Parameter that quantifies the degree of soil laterization
PI	Plasticity Index
SDBS	Sodium Dodecyl Benzene Sulfonate
SODOCO	Sodium Deoxy Cholate
SDS	Sodium Dodecyl Sulfate
USCS	Unified Soil Classification System
UFAC	Federal University of Acre

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Analysis of the modification of piping channels on kaolinitic clayey samples in the pinhole test

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Article

Keywords

Dispersivity
Pinhole test
Piping
Internal erosion
Double hydrometer test

Abstract

Dispersivity is a severe pathology that occurs mainly in clay soils and is usually catastrophic in geotechnical structures susceptible to this damage. Hundreds of dams worldwide have failed due to quality problems, mainly by piping in the body, foundation, spillway, culvert, and other peripheral structures. The pinhole test is currently considered the most accurate test for detecting the dispersivity of clay soils. However, it presents problems when objectively evaluating the dispersivity of a material due to the qualitative nature of the estimation of results. In particular, the methodology for determining turbidity has been identified. This document studies different piping paths in the sample, which *a priori* may be more realistic than the single path in the current test. A kaolinitic clay, widely studied through index and mineralogical tests, is used as the base material. Regarding the detection of dispersivity, a specialized test package was used to reduce the uncertainty of the results. Natural samples were analyzed using ASTM D4647-13. A modification of the pinhole test was proposed based on the imposition of additional artificial channels. The results revealed that this modification can make the test more realistic because when the dispersive front advances in the soil, it does not travel along a single path but instead looks for different erosive paths. The details of this assertion are discussed throughout the paper.

1. Introduction

In dispersive materials, the interstitial structure contains many exchangeable sodium ions that affect the segregating behavior of the soil. The sodium molecules act as dispersing agents that increase the thickness of the diffuse double layer. Under saturated conditions, the clay assemblages, often called “tactoids”, repel each other, undergo lamellar deflocculation, and transform into individual colloidal suspensions (Zorluer et al., 2010). This water-induced removal process creates internal erosion and tubular formation (piping), the degree of which is a function of sodium content, mineralogy, textural chemistry, dissolved salt level and pore size distribution (Zourler, 2003). In simpler terms, dispersivity occurs in cohesive soils when the repulsive forces between particles exceed the attractive forces, facilitating the segregation phase and movement in the suspension. Soils with significant dispersive spectra generally have low permeability, porosity, and bulk density (Ouhadi & Goodarzi, 2006).

Several traditional and modern approaches have been proposed to optimize the standard experimental procedures for identifying dispersive clay. Generally, conventional laboratory

index tests such as visual categorization, gradation, specific gravity or Atterberg limits do not allow for deeply defining the internal erosion suitability of soil (Belarbi et al., 2013). There are three tests most frequently performed to determine the numerical framework of dispersivity: the crumb test (Emerson, 1967), the pinhole test (Sherard et al., 1976a) and the Soil Conservation Service (SCS) laboratory dispersion test (sometimes called double hydrometer test) (Decker & Dunnigan, 1977), commonly used in combination to obtain more reliable results. However, there are many critical empirical tests and adaptations of chemical nature (Ladd, 1960; Heinzen, 1976; Coumoulos, 1977; Forsythe, 1977; Sargunan, 1977; Jones, 1981). Similarly, Shoghi et al. (2013), Abbaslou et al. (2016), and Singh et al. (2018), provide a detailed summary of many of these measurement techniques.

Through preferential flow paths, the pinhole technique indicates the development of tubular formations in dispersive materials with high and low sodium ion content and soils with liquefaction potential. In some cases, the Pinhole method has a variable degree of suitability for identifying dispersive soils (Reeves et al., 2006; Ismail et al., 2008) or tunneling processes (Vacher et al., 2004). However, the

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shortcomings of pinhole test for being qualitative and not effectively identifying accurate soil dispersion (Jermy & Walker, 1999) have generated numerous refinement proposals that provide a more quantitative measurement rather than the typical assessment based on visual examination. Maes (2010), evaluated the susceptibility of the pinhole mechanism to internal pipe routing and the effect of tunnel development on internal soil structure resistance across different hydraulic pressures.

This study proposes modifying the pinhole test by the imposition of multiple tubing channels on the cross-section of kaolin samples. The number of induced channels is presented under two different configurations to explore the effects on the dispersivity paths. The results will be analyzed mainly by comparing the mechanics of the conventional pinhole test with the modification proposed in this investigation.

2. Background

As mentioned in the abstract, many dams worldwide have failed due to piping problems. In the US alone, 20% of dams have revealed incidents related to internal erosion driven by seepage. This pathology has often been identified in hydraulic-geotechnical structures such as levees, dikes, dams, embankments, and spillways, caused by different aspects such as animal burrows, roots of some plant species, fissures or by some intrinsic condition of a soil susceptible to this anomaly (ASDSO, 2023). In the rest of the world, most cases are divided between overtopping and quality problems, covering 80% of the total cases. Of these events, 58% of quality problems are related to piping in the body or foundation. Data about piping are not shown when it is due to dispersivity or erodible soils detonated by an external factor (Zhang et al., 2007).

Caldeira (2018) mentions that the erosive (dispersive) path has a retrogressive tendency because the detachment of particles at the path's end reveals the pathology's triggering mechanism. Such detachment is exacerbated through the porous medium when the critical gradient is greater than the threshold that the material physically supports, according to its intrinsic properties. The gradient necessary to initiate erosion must be very high in fine soils that cross the sieve 200 and have a plasticity index greater than 7. The opposite is true for non-plastic soils (NP), particularly with a plasticity index of less than 7. Schmertmann (2001) reported that the minimum gradient generating the detachment in the soil is very low, of the order of 0.08. While the speed required for longitudinal scour in a dispersive path is between 40 and 90 times that for piping processes. It is essential to understand this process, according to *ibid.* Schmertmann (2001), as a detachment condition in zones of effective stress and, therefore, of zero shear strength (vide Figure 1).

This aspect can be understood in Figure 1, which shows the onset of erosion for non-plastic soils according to the value discussed in the previous paragraph. As the hydraulic

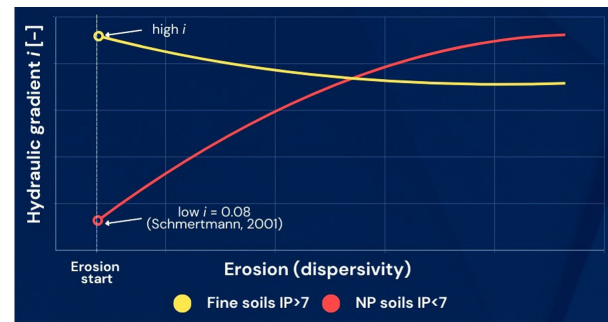


Figure 1. Hydraulic gradient versus erosion.

gradient increases, erosion is assumed to increase in these soils. In contrast, a high hydraulic gradient – i – is required to initiate erosion in fine soils. The hydraulic gradient should remain constant or, at most, decrease slightly to maintain constant erosion in such soils.

The dispersive mechanism of cohesive soils is complex. Some researchers, such as Wei et al. (2007), rely on acidity theory, using the pH value to explain the reasons for dispersivity. Mineral and cation theories are sometimes applied to answer this behavior in fine-grained soils (Wang et al., 1999). Although many of these theories have conceptual bases, they still need to be completed in application fundamentals, and to ensure proximity to the intrinsic behavior of dispersivity, numerous experimental archetypes are developed. The pore water-soluble cation test developed by Edgar (1991) and the exchangeable sodium percentage test (Sherard et al., 1973) follow execution protocols similar to the criteria for dispersion potential classification. Fan et al. (2013), propose a quantitative method that interprets the results of those tests. Systems similar to SCS with different limits were patented by Gerber & von Maltitz (1987) and Walker (1998). Likewise, Muttuvél (2008) modeled an analytical device for the simulation of internal erosion and piping processes, which incorporates the stress-strain characteristics of the soil and validates the results employing uniaxial tensile tests. Other studies (Camapum de Carvalho et al., 1999; Camapum de Carvalho & Gitirana, 2021) have evaluated in tropical non-plastic soils that discontinuity in particle size distribution can strongly influence tubified erosion.

More current equipment, such as the Jet Erosion Test (Hanson & Cook, 2004), the Hole Erosion Test (Fell & Wan, 2002; Wan & Fell, 2004a, b) and the Slot Erosion Test (*op cit.* Fell & Wan, 2002), allow modeling of internal tubular erosion rate and shear stress using the measured flow rate and hydraulic gradient. Modifications of these devices and simple numerical methods to analyze the collected data are extensive (Lim, 2006; Farrar et al., 2007; Bonelli & Brivois, 2008; Mercier et al., 2012; Karamigolbaghi et al., 2017; Lüthi, 2011; Lüthi & Millar, 2011; Marot et al., 2011; Regazzoni & Marot, 2018). It has recently been possible to determine the potential for pipe formation through true triaxial testing under a broader range of confining stresses and hydraulic gradients (Richards & Reddy, 2010). Tomlinson & Vaid (2000) developed a test to

assess the effect of uniaxial stress on the piping phenomenon. Valdes & Liang (2006) adjusted the performance of industrial filters in various modes of internal tubular behavior of soils.

The pinhole test, as an indicator for dispersive soils, was first expounded by Sherard et al. (1976b) to distinguish and refine the understanding of dispersive, sodium ion-rich, fine-grained, highly erodible soils. The test procedure described in ASTM (2013a) is based on extensive testing and observational experience, so it is not intended to be used as a quantitative test capable of accurately measuring subsurface erosion rates (Figure 2). This method is discussed extensively by Maharaj (2010) and Maharaj & Paige-Green (2010), and numerous approaches are intended to inform the procedure of this test for dispersive problem-solving (Leonards et al., 1991; Tosun, 2000; Botschek et al., 2002a, b; Vacher et al., 2002; Batog et al., 2007; Nadal-Romero et al., 2011a). In all these reports, the susceptibility of various soil types to dispersion or tubular ingrowth is reported through physicochemical parametric measurements before and after the test.

The evident importance of the interaction between electrical conductivity and exchangeable sodium in the numerical description of clay dispersion and piping (Turner et al., 2008) has exposed the pinhole methodology to modifications in order to address its operation under quantitative principles. Also, Arulanandan & Heinzen (1977) used a rotating cylinder and weight loss to measure erosion within the Pinhole device. In the same way, Zhang (1981) proposed discriminating soil dispersion by the molar rate of sodium ions in the pinhole test. Fan et al. (2004) adopted the pinhole mechanism for different test-weighted values, which provided a more reasonable and reliable synthetic discrimination method to isolate soil dispersion (Chen et al., 2017). Similarly, Rahimi & Abbasi (2008) developed a metal disk with a short conical inlet tube that prevents erosion inside the pinhole device.

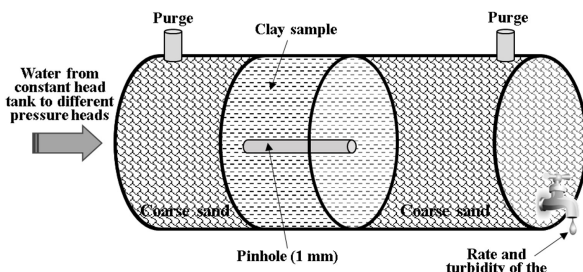


Figure 2. Schematic of the chamber housing the sample in the pinhole test.

Table 1. Physical properties of kaolin.

Sample	G_s (.)	Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)	Hydraulic conductivity (cm/s)	Specific weight γ (g/cm ³)
1	2.58	42.2	24.3	19.7	0.000043	1.50
2	2.56	42.2	25.2	17.0	0.000041	1.57
3	2.53	43.0	24.3	18.7	0.000038	1.58
Variation (%)	1.0	1.0	2.0	7.0	6.0	3.0

3. Materials and methods

3.1 Material characterization

The study was carried out with kaolin, a hydrated aluminum silicate. The particle size distribution (PaSD) was obtained using the ASTM D7928-1 standard procedure under hydrometric processes. Figure 3 shows the particle size curve for kaolin. The test used a deflocculant (sodium hexametaphosphate) to obtain values for particles with diameters less than 0.002 mm. An essential aspect of the PaSD of this material is the intrinsic contour of the curve, which extends primarily from 0.075 mm to 0.002 mm, indicating the dominance of a silty soil.

Table 1 summarizes the clay's physical properties and classification parameters, analyzed considering the coefficient of variability between samples.

The use of X-Ray fluorescence (XRF) spectral data was carried out in order to know the chemical compositions of the minerals present in the clay. On the other hand, the changes in the mineralogical information of kaolin were focused using X-Ray diffraction (XRD) through the positions of the basal reflections. The data provided in Figure 4b show that alumina and silica oxide are present in significant proportions, while the other compounds are present in trace amounts. Excluding the quantitative estimation phase, the XRD patterns indicate the presence of quartz, kaolinite and illite as the main minerals (Figure 4a).

3.2 Proposed modification to the Pinhole test

The pinhole test alteration includes adding four tubing channels in modification type I and eight for modification type II, arranged crosswise in the x-y and x'-y' plane with the center at the main hole, as shown in Figures 5 and 6.

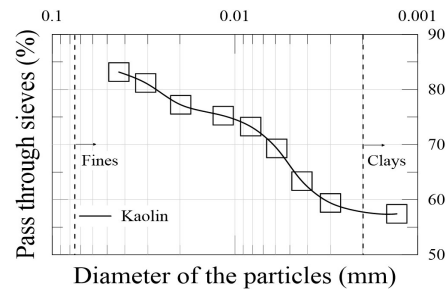


Figure 3. Particle size distribution of the kaolin.

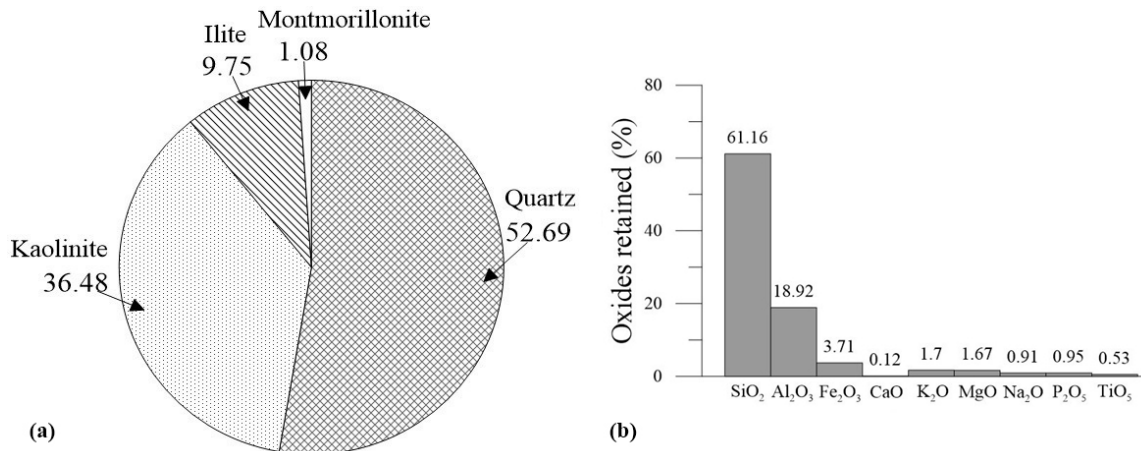


Figure 4. Results of: (a) XRD; and (b) XRF.

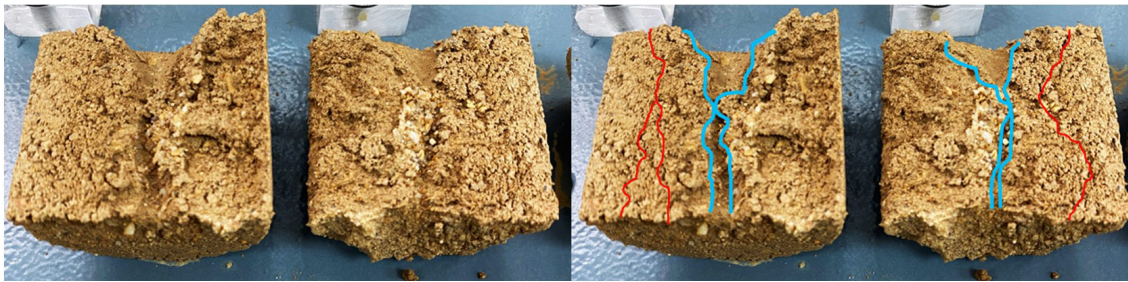


Figure 5. Different natural and induced paths in the pinhole sample.

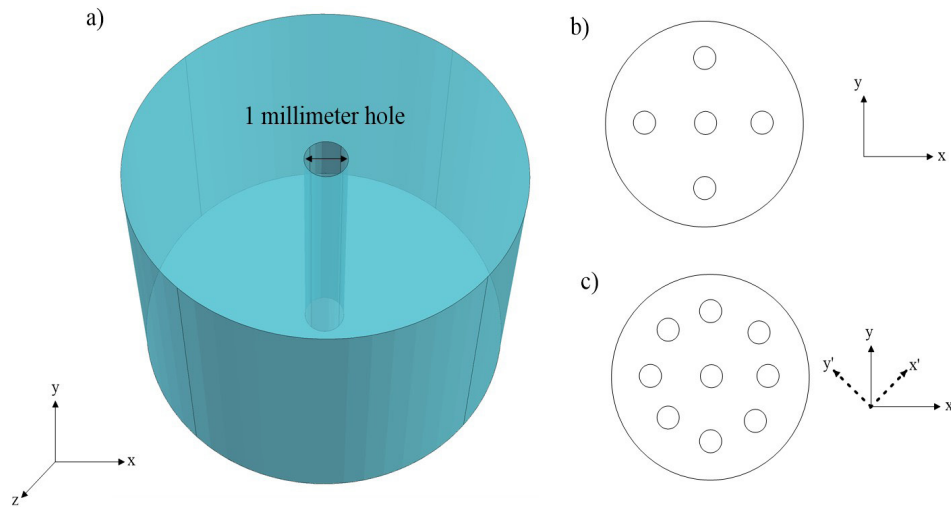


Figure 6. (a) Pinhole apparatus chamber; (b) Modification I; (c) Modification II.

The flow injection is performed through the induced conduits using a 1 mm diameter pipe. It is important to describe that the distance between the eccentric holes and the sample chamber is at least 10 mm. The objective of this proposal is to evaluate the dispersive potential of the soil, using a more realistic approach to internal piping behavior in compacted materials, since generally, erosion processes co-occur in numerous channels parallel and perpendicular to the flow and are not directed through a single circulation pathway as established by the original test.

As seen in Figure 5, a sample of the pinhole test reveals a cross-section made at the end of the test showing a blue line representing the erosive path induced by a 1 mm diameter probe before starting the test. Although other piping channels were not induced, they are identified in the sample. Therefore, modification of the test by imposing new pinholes is justified, as illustrated in Figure 7.

Inducing various artificial conduits allows for obtaining more approximate dispersivity values and testing whether including these new channels affects the mechanics of the

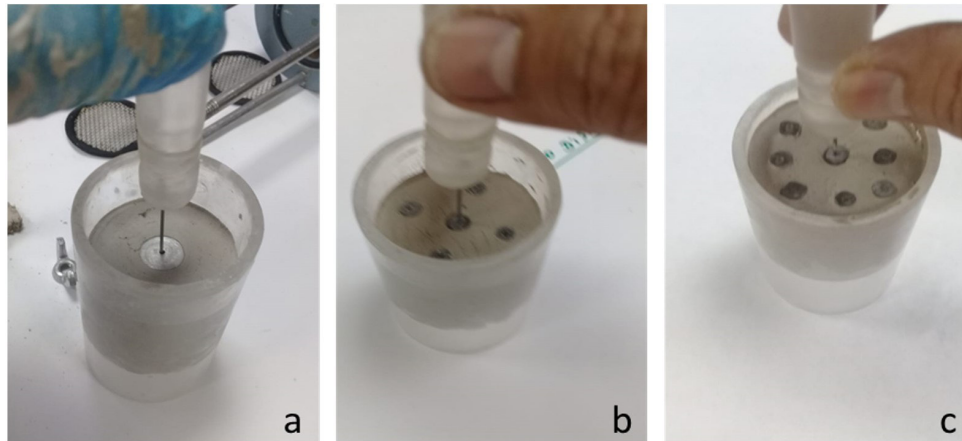


Figure 7. Pinhole test modification: (a) NM, flow direction; (b) Modification I, flow direction; and (c) Modification II, flow direction.

results in soils with index values of dispersivity obtained through the original pinhole test (Figure 7).

3.3 Testing program

Thirty (30) dispersive and non-dispersive soil samples were prepared to provide dispersivity measurements using the new mechanical modifications of the Pinhole test. Since the natural samples obtained are non-dispersive, artificial dispersivity will be imposed in order to reduce the uncertainty of false negatives in the crumb (ASTM, 2013b), pinhole (ASTM, 2013a), and SCS test (ASTM, 2018).

Taking as a reference the SCS test methodology, better known as double hydrometer, where a comparison is made between samples tested with a deflocculating agent and without a deflocculating agent; the methodology proposed by Galvis (2020) and Galvis et al. (2021) is replicated. In this study, the samples are artificially dispersed, trying to simulate the natural dispersive behaviour of the samples. With this, it was possible to advance the research by obtaining first-hand dispersive samples due to the impossibility of obtaining them *in situ*.

Each of the specimens was compacted in three separate layers using a small hammer as a compaction tool in a bottomless mold, with eight blows per layer according to the standard energy calculation required for the volume of the specimens. The optimum moisture content for all layers is close to 28%, with a maximum dry unit weight of 1.36 g/cm³. The layers have a thickness of 13 mm for parallel flow through the synthetic tubing channels. The dispersive samples contain 4% sodium hexametaphosphate added to the compaction water of the specimens. This method is analogous to that used in the double hydrometer test.

For the pinhole test, a 1 mm diameter hole is drilled through the 40 mm long and 35 mm diameter cylindrical soil samples, using the geometric distribution for each modification (Figure 2). Distilled water is percolated under pressures of 50, 180 and 380 mm water column, using a reservoir. For the loading values, the effluent flow rate is

recorded at a controlled time of 60 and 300 seconds to observe the qualitative condition of the water after the pinhole test.

4. Analysis and results

Detecting a dispersive anomaly in soil is fundamental to the lifetime of a geotechnical structure—particularly those with hydraulic stresses, as discussed above. The timing of anticipating the dispersive potential of the material is critical at the geotechnical design stage.

4.1 Pinhole test

Tests based on the unmodified pinhole method, *i.e.* conventional testing, are developed using ASTM Standard Method A. This method is designed for samples suspected of being dispersive. The samples are initially known as non-dispersive. However, the planned set of tests is performed on them. Method A must be tested by imposing all pressure heads and measuring the flow rates encountered.

Table 2 shows the dispersivity results for the originally non-dispersive samples, considering the modifications in the flow channels of the specimen, as shown in the methodology. It is important to note that the modified samples, imposing more piping channels (*a priori* more realistic), present a different classification according to the ASTM D4647-13 standard (ASTM, 2013a). However, regarding dispersivity values, the qualitative mention is the same, *i.e.*, the samples are also classified as non-dispersive. Nevertheless, the standard already identifies a difference between ND1 and ND2. In principle, this is a minimal distinction based only on the change in the effluent rate flow, which begins to reveal the consequences of the modification, even in naturally non-dispersive samples.

The samples conventionally used according to the reference standard reveal a No dispersive - ND1 classification for a single artificial piping channel. However, the modified samples (I-II), although also classified as non-dispersive, have the ND2 symbology added to them. This aspect means they

Table 2. Classification of non-dispersive samples.

Sample	Modification	Max. head reached (mm)	Flow rate (mL/s)	Hole size after test (mm)	Classification Method A	Qualitative effluent turbidity
1	⊙	1020	1.73	1.0	No dispersive - ND1	Clear
2	⊙	1020	1.75	1.0	No dispersive - ND1	Clear
3	⊙	1020	1.78	1.0	No dispersive - ND1	Clear
1	⊗	1020	3.37	1.0	No dispersive - ND2	Clear
2	⊗	1020	3.45	1.0	No dispersive - ND2	Clear
3	⊗	1020	3.42	1.0	No dispersive - ND2	Clear
1	⊗	1020	3.90	1.0	No dispersive - ND2	Clear
2	⊗	1020	3.78	1.0	No dispersive - ND2	Clear
3	⊗	1020	3.95	1.0	No dispersive - ND2	Clear

Legend: ⊙ NM, No Modification; ⊗ I, Modification I; ⊗ II, Modification II.

present a higher flow rate, which is strongly influenced by the proposed modification. In all three cases, the flow turbidity was completely clear. The difference between clays that are referenced as non-dispersive ND1 and ND2, their difference lies in the flow rate, which must be less than equal to and greater than 3.0 mL/s, respectively.

The only differentiated aspect in the proposed modification for the non-dispersive samples, i.e. I and II, is the flow rate increase, which, although it seems obvious, is a point of interest before evaluating the dispersive samples, as analyzed, according to the observation in Figure 5.

The results are interesting concerning the samples with imposed dispersivity (Table 3), in which a traditional deflocculant is used, as in the particle size distribution test for the fine fraction of a material. As expected, the unmodified (NM) sample with induced dispersivity changes its dispersive response. It is shown that at a pressure head of 380 mm, it is already dispersive, and its category reveals that it is classified as Slightly Dispersive - ND3 for method A and Dispersive - D for method B since the turbidity of the source was described as dark and cloudy, respectively.

This aspect already shows a disparity in criteria that generates uncertainty in the analysis of the pinhole test. At this point, the singular process demonstrated that dispersing the soil by adding salt with sodium ions allows the test to reflect a true positive. Of course, this only explains some things. Although the samples are not naturally dispersive, they present a high susceptibility to this pathology when infiltrated by humidity with sodium salts.

This description will be incomplete without describing the response obtained in the dispersive samples with modifications I and II. Consequently, the samples with modifications type I and II (five and nine pinholes) in the first instance exhibit a lower pressure head at which dispersivity was achieved (180 mm) and a logical increase in flow rate, as explained above in the analysis of the non-dispersive samples. The pinhole orifice remains constant between 5 and

6 mm, and the turbidity description remains dark or cloudy, depending on the method approach.

Here, referring to the change shown in the dispersivity classification is necessary according to the reference standard. It is evident that in the specimens without modification (NM), the description for method A resulted in Slightly dispersive - ND3 and for method B, Dispersive D. After subjecting the samples to modification I and II, the classification changes from Slightly dispersive - ND3 to Moderately dispersive - ND3. In other words, the modifications generate a change in the dispersivity of the material. It is important to note that the samples are dispersive for method B in all cases. However, method A, which is considered more accurate due to the ASTM D4647-13 standard (ASTM, 2013a), reveals an apparent change, which is not small, since the change from one step to another in the degree of dispersivity implies a variation in the parameters of pressure head, flow rate, orifice size and effluent turbidity (see Figure 8). A summary of the results obtained is illustrated in Figure 9.

Having done this analysis, it is worth clarifying one more aspect of the sample modification. The proximity of the chamber can influence the border effect on the eccentric pinholes. Figures 5 and 7, show that the tubing channels are naturally generated around the imposed pinhole. In addition, in Figure 7, the centre of the pinhole is distanced from the chamber wall at a distance ten times the pinhole diameter (1 mm). This distance is considered sufficient to avoid a possible boundary effect.

4.2 Double hydrometer and crumb test

For the complementary crumb and double hydrometer tests, three natural samples and three samples with artificial dispersivity were taken to compare with the pinhole tests developed. Table 4 shows the results of this qualitative test (crumb), which reveals low dispersivity degrees for the natural samples. In contrast, the values of the dispersive reaction are indeed high for the samples with imposed dispersivity. The results are valid for the ASTM D6572-13 standard (ASTM, 2013b).

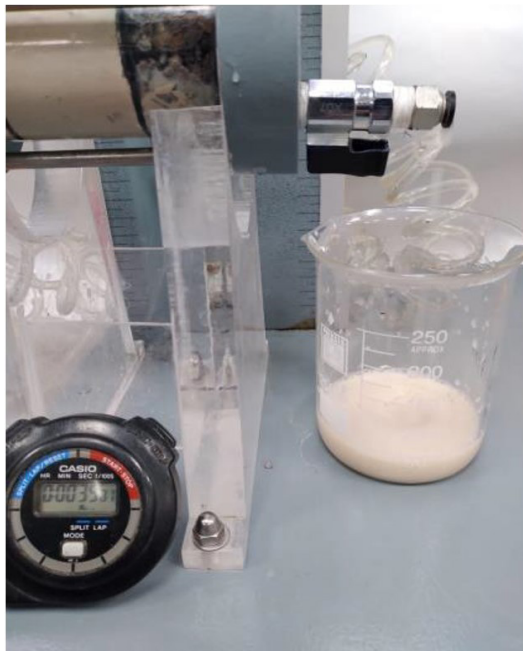
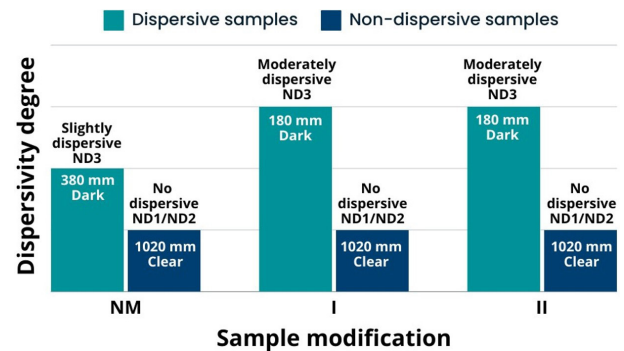
Table 3. Classification of dispersive samples.

Sample	Modification	Max. head reached (mm)	Flow rate (mL/s)	Hole size after test (mm)	Classification method A / B	Qualitative effluent turbidity
1	⊙	380	1.60	6.0	Slightly dispersive - ND3 / Dispersive - D	Dark/Cloudy
2	⊙	380	1.92	6.0	Slightly dispersive - ND3 / Dispersive - D	Dark/Cloudy
3	⊙	380	2.07	6.0	Slightly dispersive - ND3 / Dispersive - D	Dark/Cloudy
1	⊕	180	2.47	5.0	Slightly to moderately dispersive - ND3 / Dispersive - D	Dark/Cloudy
2	⊕	180	2.32	5.0	Slightly to moderately dispersive - ND3 / Dispersive - D	Dark/Cloudy
3	⊕	180	2.37	5.0	Slightly to moderately dispersive - ND3 / Dispersive - D	Dark/Cloudy
1	⊕⊕	180	2.50	5.0	Slightly to moderately dispersive - ND3 / Dispersive - D	Dark/Cloudy
2	⊕⊕	180	2.40	6.0	Slightly to moderately dispersive - ND3 / Dispersive - D	Dark/Cloudy
3	⊕⊕	180	2.42	6.0	Slightly to moderately dispersive - ND3 / Dispersive - D	Dark/Cloudy

Legend: ⊙ NM, No Modification; ⊕ I, Modification I; ⊕⊕ II, Modification II.

Table 4. Degree of dispersivity for the crumb test.

Solution	Crumb Condition	Time (min)	Dispersivity grade	Dispersivity classification
Distilled water	Air dried (from pinhole)	2	1	Non-dispersive
		60	2	Intermediate
		360	2	Intermediate
Sodium hexametaphosphate	Air dried (from pinhole)	2	1	Non-dispersive
		60	4	Highly dispersive
		360	4	Highly dispersive

**Figure 8.** Turbidity classified as *dark* in the effluent for dispersive samples.**Figure 9.** Summary of results of the pinhole tests.

In the double hydrometer test (SCS), conducted under the standard ASTM D4221-18 (ASTM, 2018), the results of the degree of dispersivity according to the test configuration are presented (Figure 10). The *A/B* ratio denotes the percentage of dispersivity, which correlates qualitatively with the degree of dispersivity. According to note 5 of the reference standard, when the dispersion percentage is 100, the material is said to be fully dispersive. If it is zero, the soil is entirely non-dispersive. The official standard categorises a dispersion

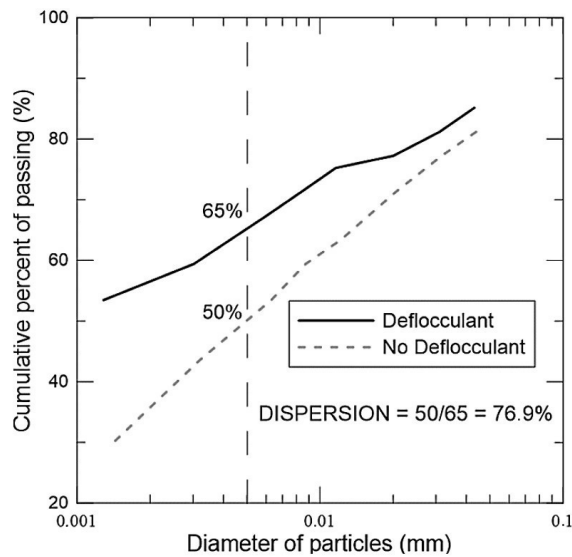


Figure 10. Percentage of dispersivity obtained from the SCS test.

value of 76.9% as a dispersive clay-fraction because the value is higher than 50%.

5. Conclusion

A set of qualitative tests such as pinhole, crumb and double hydrometer is necessary to obtain the reaction to the dispersivity of soil with high reliability. It is recommended to include quantitative tests with a physical-chemical approach (total dissolved salts, pH, exchangeable sodium percentage, and cation exchange capacity). Even analyze the possibility of including less studied tests in the literature, such as the Hole Erosion Test, Jet Erosion test and Inderbitzen test.

Inducing dispersivity in non-dispersive natural samples can be an adequate technique, parallel to the classical procedure, to properly calibrate the pinhole equipment used to evaluate the degree of dispersivity. By imposing artificial dispersivity on the specimen, the whole dispersive panorama that the apparatus is capable of measuring in the standardized test can be obtained.

The proposed variation to the pinhole test based on the ASTM reference standard, which is more realistic through the specimen, reveals that in specimens with induced dispersivity, it is possible to obtain different dispersivity values than those found in the conventional procedure. That is, specimens with more piping channels available have higher dispersivity values. However, further research is required with different degrees of artificial dispersivity of the sample and different sodium salts or other ions that may cause dispersivity in the clay material.

This research demonstrates that the pinhole test can be flawed in accurately assessing soil dispersivity. For highly dispersive samples, evaluated by methods A and B, the difference in typification can be high.

A further corollary is related to modifying the samples by inducing more 1 mm diameter holes in the cross-section, demonstrated in Figure 5, where even new trajectories were spontaneously generated without the need to impose them. This strongly justifies modifying the samples towards a more realistic situation. However, a more extensive test campaign, varying the clay material, is required to obtain more accurate results.

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

Juan Carlos Ruge: conceptualization, data curation, writing – original draft. Henry Giovanni Martínez: conceptualization, methodology, validation. Eliana Martínez Rojas: formal analysis, methodology.

Data availability

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

List of symbols and abbreviations

i	Hydraulic gradient
pH	Potential of hydrogen
ASTM	American Society for Testing Materials
A/B	Dispersion ratio
D	Dispersive
G_s	Specific gravity
I	Modification 1
II	Modification 2
ND1	No dispersive
ND2	No dispersive
ND3	Slightly dispersive (method A)
NM	Unmodified
NP	Non-plastic soils
PaSD	Particle size distribution
SCS	Soil Conservation Service
US	United States of America
XRD	X-Ray diffraction
XRF	X-Ray fluorescence
γ	Specific weight

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Pull-out response of a geogrid buried in recycled sands

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Article

Keywords

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Soil-reinforcement interaction
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Abstract

Recycled construction and demolition waste (RCDW) has demonstrated geotechnical properties that encourage it to be used in reinforced soil structures (RSS) with geosynthetics. However, the interaction between RCDW and reinforcements needs to be better understood, given its importance for design. This paper presents a qualitative study on the interaction between geogrid and recycled sands by means of pull-out tests performed on small equipment. Tests with the same degree of compaction and a geogrid buried in different types of recycled and natural sands (for comparison purposes) were performed. The characterisation of the materials was carried out in the laboratory and the variability of their geotechnical properties was evaluated. In addition, fill material moisture content was investigated as another potential factor influencing soil-geogrid interface shear. The results of the pull-out tests demonstrated the specific influences of the factors investigated. The comparative study showed that recycled sands can be suitable materials to be used as backfill in geosynthetic reinforced soil structures, meeting physical, mechanical and environmental requirements for this kind of work.

1. Introduction

Global development and the consequent demand for services and goods from the construction industry have caused an increase in the volume of so-called 'construction and demolition waste (CDW)'. It is estimated that 10 billion tons of CDW are generated annually in the world and most of this waste has recycling potential (Wu et al., 2019). According to Peng et al. (1997), recycling these materials leads to a new possibility for the input market, and is an environmentally friendly alternative for the inadequate disposal of CDW. Therefore, CDW can be reintroduced into the geotechnical construction works.

Geosynthetics are products whose applications have not only entered the market in various sectors of the construction industry, because of their well-defined properties, but have also exceeded conventional products. The economic and environmental benefits have led to an increasing use of geosynthetics in geotechnical works (Koerner, 1990). Reduced energy consumption, low transportation costs and lower environmental impacts are some ecologically positive factors when comparing the use of these materials with the traditional ones found in retaining structures (Jones, 1994; Stucki et al., 2011; Heerten, 2012; Damians et al., 2016).

In most cases, soil-inclusion interaction can be pointed out as the main factor for the adequate performance of reinforced soil structures (RSS) because these materials show

complementary behaviour. Bearing in mind that the use of recycled construction and demolition waste (RCDW) in RSS seems to be an interesting strategy to promote the concept of sustainability in the construction industry, it is essential to examine the characteristics of this non-conventional material and its interaction, under field and laboratory conditions, with geosynthetics.

1.1 Characteristics of RCDW used in geotechnical works

CDW is made up of different types of materials that are part of the building or infrastructure during construction, reconstruction, extension, alteration, maintenance, and demolition, which is the result of different activities and techniques in the construction industry (Kartam et al., 2004; Esin & Cosgun, 2007; Wang et al., 2010; Vieira & Pereira, 2015; Di Maria et al., 2018; Rosado et al., 2019). It is worth mentioning that currently CDW does not have a consensus about its definition, varying from country to country (Domiciano et al., 2020). However, despite the heterogeneity of CDW, it can be observed that they mainly consist of crushed concrete, bricks, tiles, pieces of wood, glass, plastic, metal, and cardboard (Huang et al., 2002; Lai et al., 2016; Asgari et al., 2017; Di Maria et al., 2018).

The physical properties of RCDW depend mainly on their composition (Ossa et al., 2016), thus the composition of

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CDW is decisive for the properties of the former. Materials derived from the incorporation of ceramic or asphalt materials usually present lower density, resistance to fragmentation and load capacity, given that they have a larger number of pores compared to those derived solely from Portland cement (Nagataki et al., 2004; Brito et al., 2005; Silva et al., 2014). The issues surrounding RCDW are not only related to environmental aspects, but also to the mechanical properties that control their stiffness and resistance to permanent deformation (Niekerk et al., 2002).

Concerning the particle size distribution of the RCDW investigated for geotechnical purposes, it can be observed that there is a wide range of grain sizes. The variability of particle size distribution is usually affected by the techniques adopted at the recycling plant, such as crusher equipment, a set of sieves, and process quantity control (Nagataki et al., 2004; Brito et al., 2005). Some studies revealed that RCDW collected at a recycling plant presented acceptable variabilities for geotechnical purposes, enabling their application in reinforced soil structures (RSS) (Santos & Vilar, 2008; Santos et al., 2010, 2013, 2014; Fleury et al., 2019). In addition, some characteristics encourage the use of RCDW in RSS: the specific gravity seemed similar to those found for soil, and non-plastic behaviour is highly recommended by some international standards (Santos and Vilar, 2008; Fleury et al., 2019; Domiciano et al., 2020).

1.2 RCDW pull-out tests

According to Koerner (1990), reinforcing the soil with geosynthetics has become increasingly attractive and allows safer, more economical and audacious construction works. The strain responses of the structures, as well as the redistribution of stresses, depend on the shear strength properties of the soil, tensile properties of the inclusions, and the mechanism of stress transfer between soil and inclusions (Jewell et al., 1984; Palmeira, 1987; Gilbert et al., 1992; Teixeira et al., 2007).

Great efforts have been made in recent decades to understand the behaviour of RSS, and research has been conducted to investigate several aspects: i) performance against earthquake (Liu et al., 2014); ii) the influence of soil wetting (Chen and Wu, 2012; Balakrishnan & Viswanadham, 2016) and iii) different faces (Rowshanzamir & Aghayarzadeh, 2015). Studies involving the geosynthetic-RCDW interaction mechanism are quite scarce. The study carried out by Santos et al. (2010, 2013, 2014) investigated the use of RCDW as backfill material in two 3.6 m-high RSS. The authors pointed out that the performance and deformations of the wall, the durability of the reinforcement and the forces in the reinforcement were satisfactory and similar to the performance of comparable structures built with conventional granular embankments.

Designs or analysis parameters of RSS can be obtained by pull-out tests, thus determining the geogrid-soil interaction, which is the combination of frictional interaction and passive strength of the cross members happening simultaneously

(Palmeira, 1987, 2004, 2009). Research has been conducted in different conditions to obtain these parameters: laboratory tests and in-field. Investigations in the laboratory are normally carried out with soils and geosynthetics in rigid metal boxes built with rectangular cross-sections and classified according to their size: i) large scale (Palmeira, 1987; Christopher and Berg, 1990; Farrag et al., 1993; Lopes & Ladeira, 1996; Palmeira, 2004, Teixeira et al., 2007; Chen & Wu, 2012; Sadat Taghavi & Mosallanezhad, 2017, for instance); and ii) small scale (Nakamura et al., 2003; Hataf & Sadr, 2014; Portelinha et al., 2018, for instance).

In a small-scale box, the short length of the testing sample and the difficulty of installing accessories, such as sleeves to reduce the effects of stress developed on the wall internal frontal face, can significantly increase the measured pulling force due to the increase in the normal stress on the geogrid surface (Farrag et al., 1993). In addition, experimental results have shown that boundaries above and below the reinforcement can lead to increases in the normal stress in the vicinity of the geogrid surface, especially when the soil thickness is small and the soil dilatancy restrained (Farrag et al., 1993; Palmeira, 2004). Although a large-scale box is preferable due to the fact of low boundary condition effects, a small-scale box may be used to perform initial studies, mainly on a qualitative basis, on the interaction that occurs between the fill material and the geogrid when subjected to pull-out.

Laboratory pull-out tests have already been carried out aiming to analyse several influence factors on the strength parameters: i) system of confinement stress application (Palmeira & Milligan, 1989); ii) condition of the frontal face (Palmeira, 1987, 2004, 2009; Raju, 1995; Sugimoto et al., 2001); iii) soil moisture content (Portelinha et al., 2018); iv) stiffness of the testing box walls (Farrag et al., 1993); v) grain size, geogrid aperture size and element thickness (Jewell et al., 1984; Palmeira 2004, 2009); and others. However, there are few studies on pull-out tests using RCDW as fill material (Santos, 2007; Santos & Vilar, 2008; Vieira et al., 2016; Araújo Neto, 2017, for instance). Therefore, this fact highlights the importance of determining interaction parameters when using such unconventional backfill material in RSS.

2. Materials and methods

2.1 Materials

2.1.1 Granular materials

The RCDW used in the tests was provided by a recycling plant located at Aparecida de Goiânia-GO, Brazil. The RCDW is named by the company as “grey recycled sand” (GRS). This material (GRS) is obtained from the crushing (jaw crusher) and sieving processes of CDW predominantly comprised of concrete blocks.

For the sake of comparison, another fill material was used in the pull-out tests, namely natural sand (NS) – material in accordance with the international standard recommendations for RSS works – which was purchased from a local supplier. In addition, a material produced in the laboratory, from sieving the GRS sample, was investigated. This material was called ‘produced grey recycled sand’ (PGRS). It should be noted that PGRS was produced to present its particle size distribution to be as similar as possible to that of NS.

2.1.2 Geogrid

A polyester geogrid with high tenacity and low creep susceptibility, with a protective polymeric coating, was tested. Geogrid samples of the same length (1,200 mm) and width (200 mm). Figure 1 shows an image of the geogrid. Table 1 shows some properties of the geogrid tested.

2.2 Experimental program

2.2.1 Materials characterisation

To verify possible variability of RCDW, five samples were collected, and the sampling procedure was carried

Table 1. Geogrid properties (provided by the manufacturer).

Property	Value
Polymer	Polyester
Aperture size (mm × mm)	20 × 25
Ultimate tensile strength (kN/m)	35
Strain at rupture (%)	< 10
Secant stiffness at 5% strain (kN/m)	≥ 350

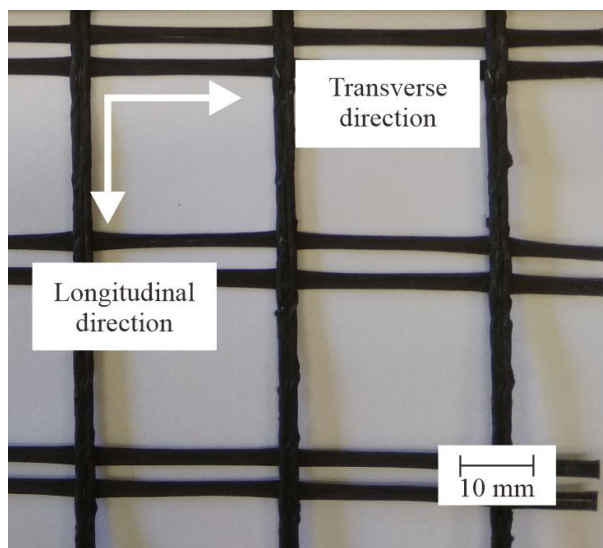


Figure 1. Geogrid.

out in different parts of the waste pile (bottom, middle and top) following the Brazilian Standard ABNT NBR 10007 (ABNT, 2004). An additional collection was made to investigate geotechnical properties and carry out the pull-out tests. The laboratory tests consisted of i) specific gravity, ii) grain-size distribution iii) Atterberg limits, iv) compaction test (standard Proctor), and v) maximum and minimum void ratios.

Direct shear tests were performed for different values of normal stresses: 50 kPa, 100 kPa, 150 kPa, and 200 kPa. The tests were performed using equipment with box dimensions equal to 60.00 mm (length) × 60.10 mm (width) × 19.68 mm (height). Samples were tested at optimum (w_{op}) and hygroscopic (w) moisture contents, and both conditions had a degree of compaction (DC) of 90%.

Scanning electron microscopy (SEM) and energy-dispersive X-ray spectroscopy (EDS) tests were conducted with JSM-7100 F equipment (JEOL Ltd.) on the NS and GRS samples.

2.2.2 Laboratory pull-out test

The equipment (small scale) consists of a test box built with 3 mm-thick steel plates. It is 250 mm long, 300 mm wide and 150 mm high, resulting in a volume of approximately 0.01 m³. The box has a 10 mm-high opening along its entire width on the frontal wall, through which the geosynthetic material is pulled out. The normal confinement pressure is applied by a pressurised rubber bag attached to the lid of the box. Figure 2 shows the equipment dimensions.

The granular materials were air-dried and sieved (aperture mesh of 4.8 mm) before being used. The NS was tested after being air-dried. The GRS and PGRS were tested in two moisture content conditions: i) at hygroscopic moisture; ii) at optimum water content. The compaction was performed in four layers by depositing the materials to obtain the compaction degree of 90% as this was the value obtained by Fleury et al. (2019) when investigating the use of RCDW as backfill material. A total pressure cell (TPC) was installed at the top of the second layer – in the centre of the box and 10 mm below the reinforcement – to monitor the internal stress development during the tests. The pull-out force was applied using a universal test machine with 300 kN capacity. All tests were carried out at the same speed of 4.6 mm/min, consistent with the displacement rate adopted by Teixeira et al. (2007). Figure 3 shows the testing setup.

The variable used for the pull-out test analyses consisted of the: i) filler material (recycled sands or natural sand); and ii) moisture content (hygroscopic moisture and optimum water content). All test scenarios were subjected to the following normal confinement pressure: 12.5 kPa, 25 kPa, 37.5 kPa, 50 kPa, 75 kPa, and 100 kPa.

The authors acknowledge that the size of the equipment (0.01125 m³) is rather small compared to others in the literature (e.g. 0.03675 m³, Hataf & Sadr, 2014; 1.224 m³,

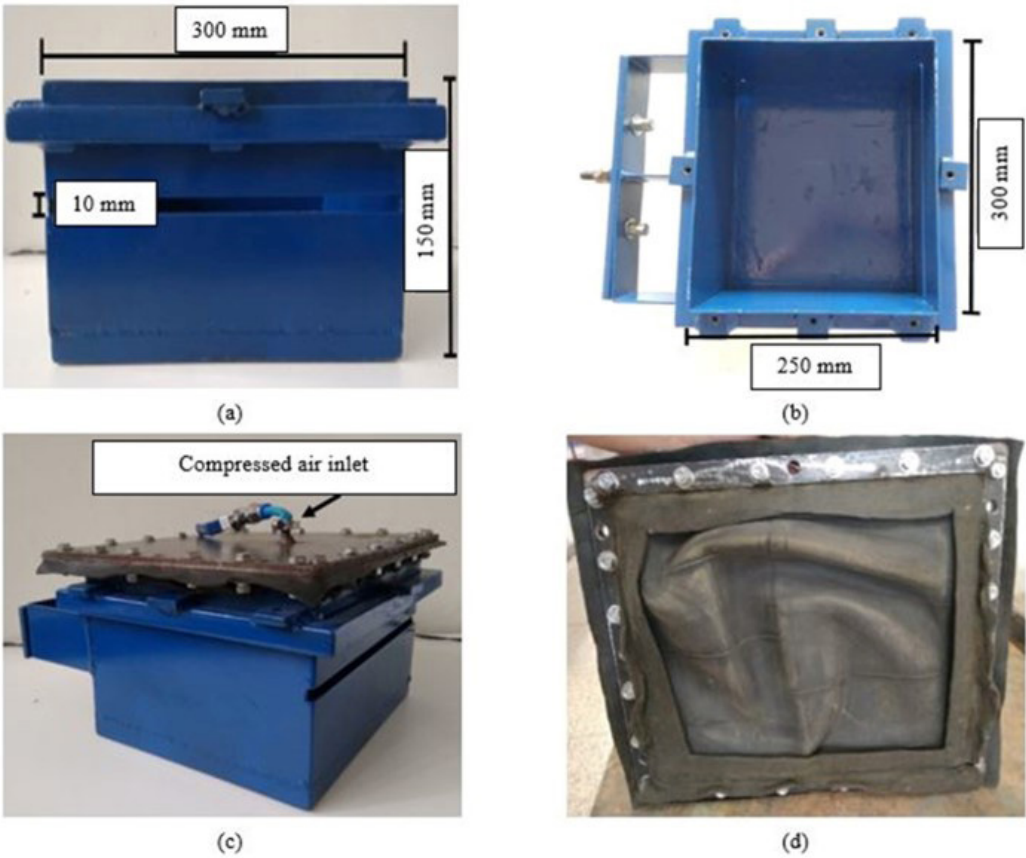
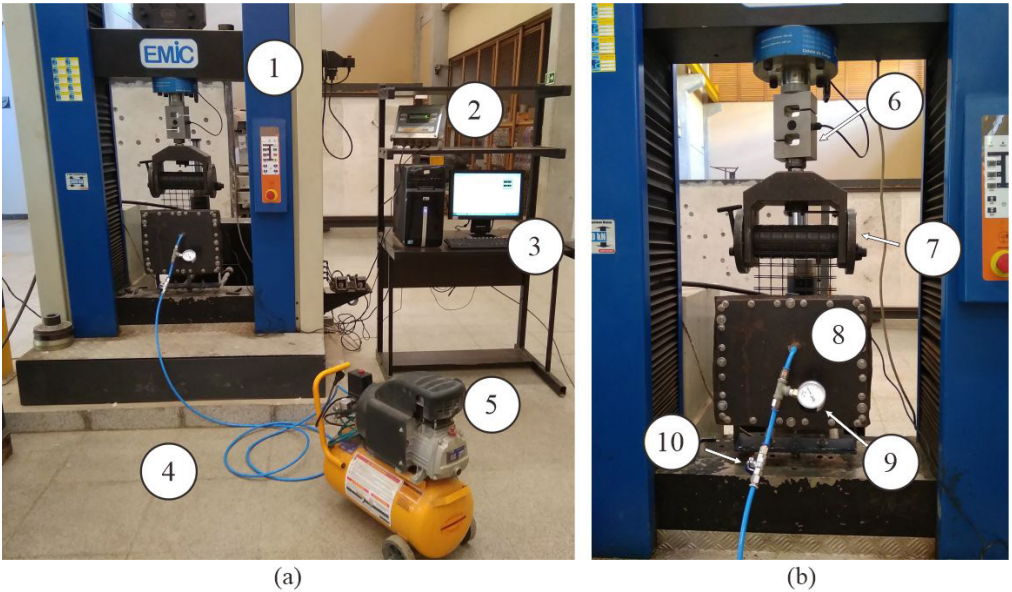


Figure 2. Pull-out box: (a) Front view; (b) Top view; (c) Perspective view of the closed box; and (d) Rubber bag attached to the box cover.



- Legend:
- | | |
|----------------------------|---------------------------------|
| 1 - Universal test machine | 2 - Total pressure cell monitor |
| 3 - Computer | 4 - Compressed air hose |
| 5 - Air compressor | 6 - Load pressure cell |
| 7 - Roller clamp | 8 - Pull-out test box |
| 9 - Manometer | 10 - Compressed air valve |

Figure 3. Pull-out test setup: (a) general view and (b) detailed view.

Lopes & Ladeira, 1996). Thus, some effects of boundaries would be expected. The internal frontal face of the box was covered by plastic layers and grease to minimise friction along that boundary. However, the main aim of the research was to validate the use of RCDW as fill material under similar conditions to other materials recommended and commonly used in practice. Therefore, the results must be seen as index and qualitative values, aiming to investigate the potential for using alternative fill materials as substitutes for conventional ones. In this type of investigation, using small equipment brings versatility and allows the execution of a large number of tests in less time, which is interesting for comparing different fill materials.

3. Analysis and results

3.1 Geotechnical analyses of granular materials

The GRS samples presented an average specific gravity (G_s) of 2.706, with a coefficient of variation (CV) of 0.28%. The NS and PGRS samples showed similar values of G_s equal to 2.713 and 2.710, respectively. Low variability value for G_s of RCDW collected at the same recycling plant was found by Fleury et al. (2019). The original features of CDW, such as material quality and particle sizes, and recycling procedures are factors that affect the G_s of RCDW aggregates (Silva et al., 2014). It is worth mentioning that the GRS presented the average G_s inside the range found for the local soil (tropical soil): from 2.664 (Silva et al., 2019) to 2.740 (Mascarenha et al., 2018).

An analysis of the grain size distribution of the GRS samples showed a low variability. However, sample GRS-06 presented a grain size distribution distinct from the other

ones. This fact can be explained due to changes adopted by the company in the recycling process. In addition, changes in the shape of the curves have been reported due to the variability of materials found in the samples of other recycled aggregates from the same recycling plant (Fleury et al., 2019). The PGRS grain size distribution curve revealed that the attempt to construct it was successful, presenting a curve similar to the one of NS.

An analysis of the grain size distribution of GRS revealed that the GRS-06 sample was the one that best matched the specifications of standard BS 8006 (BSI, 2010), and manuals FHWA (2010) and NCMA (2010) for the selection of backfill material in RSS. Considering FHWA (2010) recommendations, it was observed that some granulometric corrections would be needed for the complete adjustment of the GRS material (Figure 4). The upper part of the GRS grain size distribution curves (defined by the grains greater than 0.3 mm) should be corrected to be within the limits recommended by all the mentioned standards.

It was also observed that the NS and PGRS samples presented curves that were within the range recommended by BS 8006 (BSI, 2010), but some granulometric corrections related to particles greater than 0.4 mm would be needed to attend the limits stated in FHWA (2010) and NCMA (2010) manuals, as can be seen in Figure 4. Table 2 presents the main properties of the fill materials.

In general, the SEM images revealed that both materials (GRS and NS) have grains with angular shapes and rough surfaces (Figure 5). Although the presence of cementitious material is verified on the surface of GRS grains, the effects of such an occurrence are not the focus of this study. Energy-dispersive X-ray spectroscopy (EDS) tests for the same granulometric ranges identified the material composition of the GRS and NS samples (Figure 6). The EDS tests

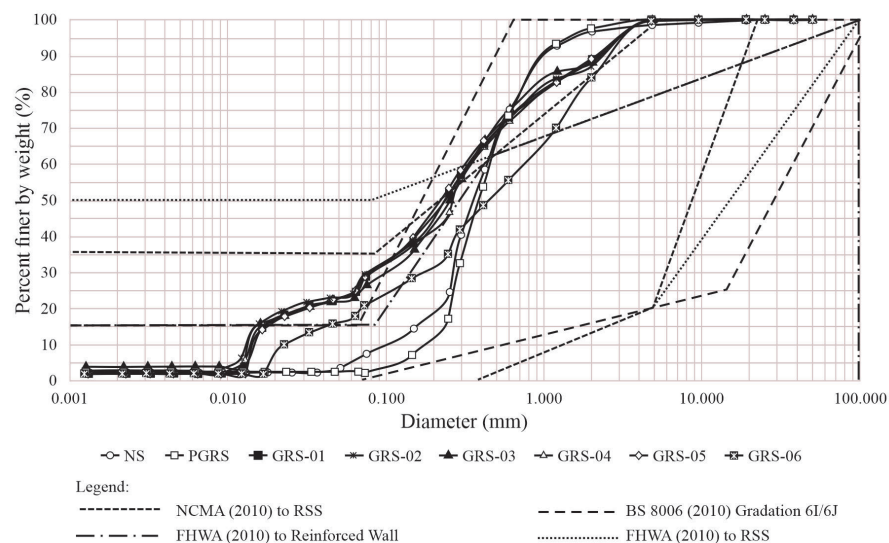
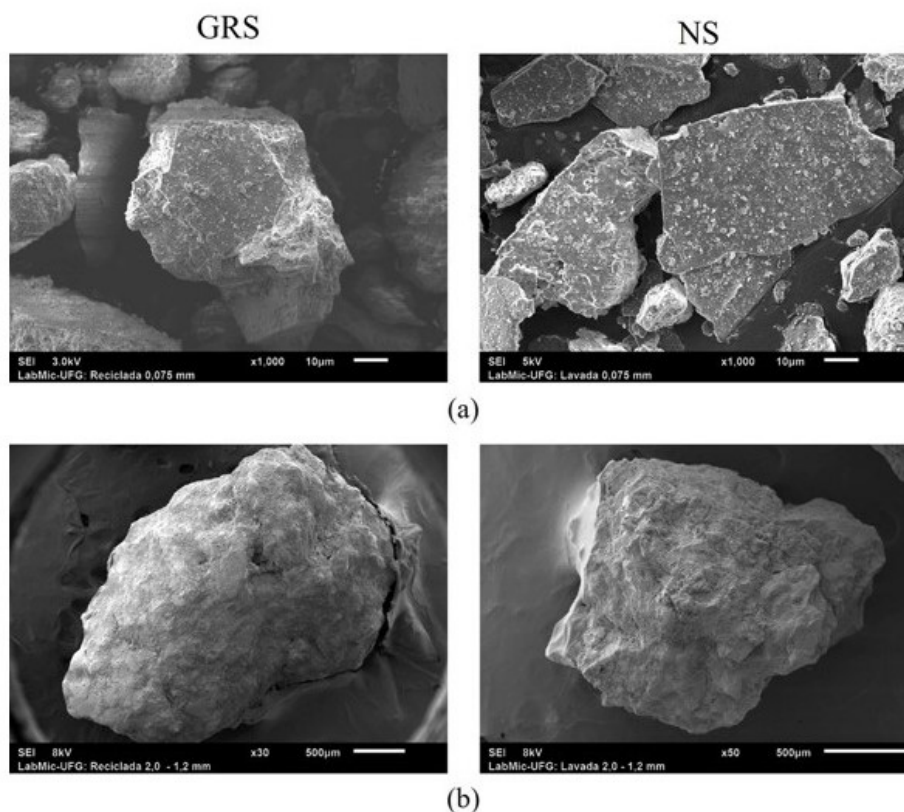


Figure 4. Grain size distribution curves versus standard limits for backfill material in RSS.

Table 2. Geotechnical properties of fill materials.

Parameter	Average Value		
	NS	GRS	PGRS
Specific gravity of the particles, G_s	2.71	2.71	2.71
D_{50} (mm)	0.34	0.28	0.28
D_{85} (mm)	0.84	1.36	0.84
C_c	1.61	1.42	3.81
C_u	4.20	24.07	13.60
e_{max}	0.87	1.06	0.98
e_{min}	0.58	0.69	0.64
Maximum dry unit weight, γ_{dMax} (kN/m ³)	n.a.	16.31	16.15
Optimum water content, w_{op} (%)	n.a.	19.48	20.00
c (kPa) - w_{op}	n.a.	7.30	24.80
ϕ (degrees) - w_{op}	n.a.	37.90	41.00
c (kPa) - hygroscopic moisture	2.6	2.40	0.00
ϕ (degrees) - hygroscopic moisture	39.4	44.10	46.40

Note: NS = natural sand; GRS = grey recycled sand; PGRS = produced grey recycled sand; n.a. = not available.

**Figure 5.** SEM images of GRS and NS samples of different granulometric size ranges (a) $D < 0.075$ mm; (b) $1.2 \text{ mm} < D < 2.0$ mm.

performed at the GRS samples identified the predominant presence of silica (SiO_2 , quartz), calcium oxide (CaO), alumina (Al_2O_3), ferrous oxide (Fe_2O_3), potassium oxide (K_2O) and magnesium oxide (MgO). The fact that GRS consists of Portland concrete and mortar justifies the presence of calcium. Studies on concrete-made aggregates revealed

higher content of SiO_2 , Al_2O_3 , and CaO (Angulo et al., 2009; Medina et al., 2014; Oliveira et al., 2020). The EDS tests on the NS particles revealed a predominance of silicon (Si) and aluminium (Al) components of silica (SiO_2 , quartz), which is consistent with the typical mineralogical composition of local natural sands (Figure 6).

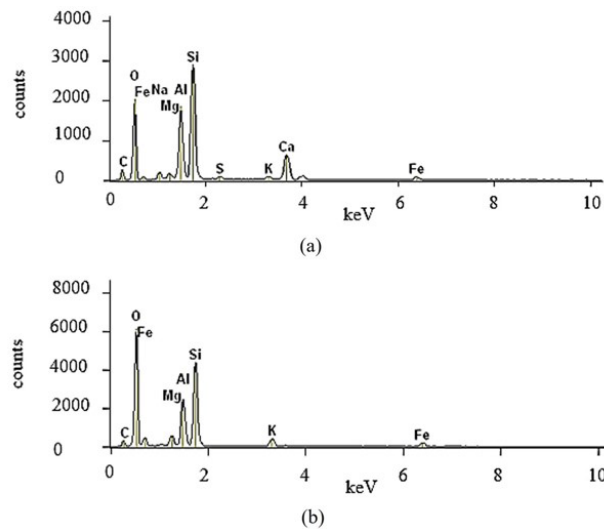


Figure 6. EDS results for fill material particles ($2.00 \text{ mm} < D < 4.80 \text{ mm}$): (a) GRS and (b) NS.

3.1.1 Analysis of pull-out force versus pull-out displacement

Materials tested with different moisture contents (dry or wet) and the same compaction degree (90%) allowed the geogrid pull-out at low values of normal confinement pressure (from 12.5 kPa to 37.5 kPa). There was a rupture of the fill material and greater mobilisation of the transverse elements nearest to the point of load application (box frontal opening), a characteristic behaviour of reinforcements with low tensile stiffness. At these low confinement stresses (from 12.5 kPa to 37.5 kPa), all materials presented curves with typical behaviour, in which the applied force values increased until reaching a peak followed by a decrease in the value of the pull-out force (Figure 7).

Geogrids buried in NS experienced more damage during the pull-out tests than those buried in recycled sands. The exhumation of the geogrid tested with NS (Figure 8a) revealed great mobilisation of longitudinal elements located at the lateral edges of the specimen and some damage to the transverse elements (at some junctions, these elements were completely separated from the longitudinal members).

Results of the tests performed with normal confinement pressure equal to 50 kPa showed steeper curves compared to the curves of the tests with lower normal stress (Figure 9). A small portion of the specimen was mobilised, where the geogrid elements presented deformation – only at the external portion of the box. Thus, rupture occurred by an isolated tensile mechanism, in which the testing box worked as a clamp. Mobilisation of a few points of the buried portion of the specimen was observed in tests with NS, in which most elements remained intact (Figure 10a), which was the opposite behaviour compared to the test with normal

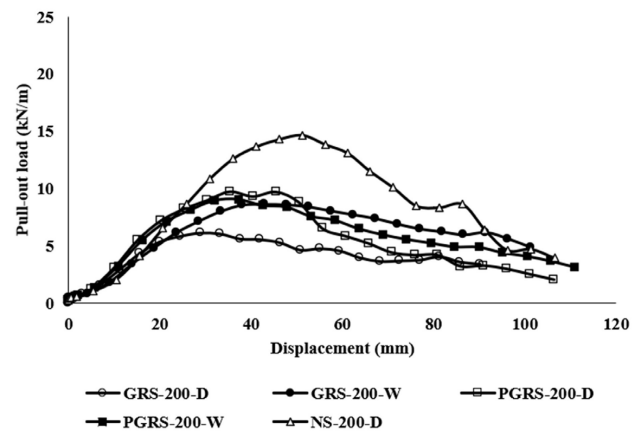


Figure 7. Pull-out test with normal stress of 12.5 kPa. Note: GRS = grey recycled sand; PGRS = produced grey recycled sand; NS = natural sand; 200 = two-hundred-millimetre width of the geogrid sample; W = optimum moisture; D = hygroscopic moisture.

confinement pressure equal to 12.5 kPa). Further studies on the geogrid damage during the pull-out tests are required.

Tests with normal confinement pressure equal to 75 kPa and 100 kPa showed no mobilisation of the specimens in their buried portions. Thus, the geogrid pull-out mechanism was not observed for all fill material types and test configurations. The rupture of the geogrid occurred in which the testing box worked as a clamp.

It was found that increasing the normal stress value and the value of the pull-out force caused an increase in the stiffness of the soil-geogrid system. This fact was previously verified in other studies (Alfaro et al., 1995; Ochiai et al., 1996, for instance).

The geogrid tensile strength values obtained from those tests with high normal confinement pressure (75 kPa and 100 kPa) were lower than the nominal reference value provided by the manufacturer. This may be due to the fact that the geogrid elements experienced some localized damage during such tests, as it was observed that the material confinement worked as a jaw-type clamp. This probably generated a concentration of stresses in some regions of the geogrid, causing early failure of the elements in such regions.

Figure 11 shows the pull-out resistance envelopes for all tests performed using 200 mm-wide geogrids. It was observed that the highest pull-out resistances increased with the applied normal confinement pressure in those scenarios where the geogrid pull-out mechanism occurred. The stabilisation, observed for the highest normal confinement pressures, revealed that the geogrids were submitted to a condition similar to that of a conventional tensile test in isolation. Test results carried out with NS presented a higher value of pull-out resistance, even for the lowest normal confinement pressure. Regarding the performance of all the fill materials, three distinguished patterns were observed: i) pull-out without detachment of elements, ii) partial deformation zone with displacement of

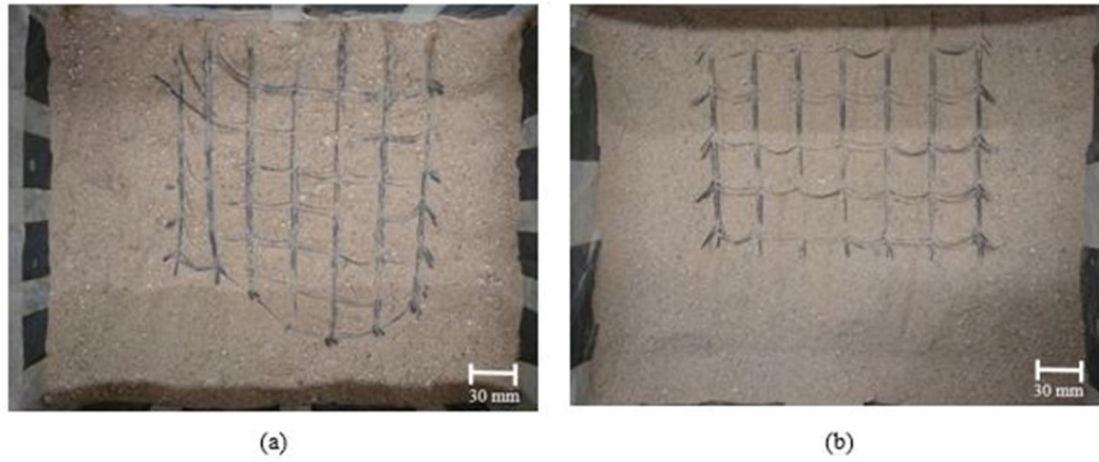


Figure 8. Exhumation of the geogrids used in the pull-out test ($\sigma = 12.5$ kPa): (a) NS and (b) PGRS-200-D.

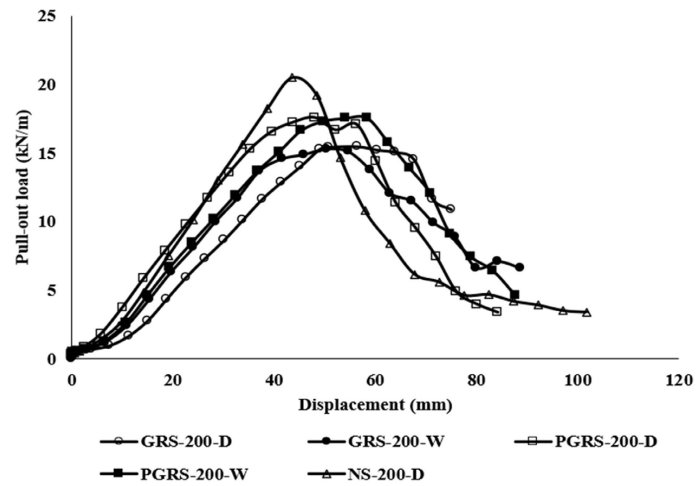


Figure 9. Pull-out test with normal stress equal to 50 kPa. Note: GRS = grey recycled sand; PGRS = produced grey recycled sand; NS = natural sand; 200 = two-hundred-millimetre width of the geogrid sample; W = optimum moisture; D = hygroscopic moisture.



Figure 10. Exhumation of the geogrids used in the pull-out test ($\sigma = 50$ kPa): (a) NS and (b) GRS-200-D.

the geogrids and detachment of their elements and iii) no pull-out, with damage to the geogrid elements near the external frontal face of the box. The investigated materials revealed the pull-out mechanism for different normal confinement pressure ranges, where the recycled sands (limits defined by the black dashed lines) presented a wide range compared to natural sand (limits defined by grey dashed-dotted lines).

3.1.2 Interface strength coefficient analysis

Tests revealed a tendency for the interface resistance coefficient values (f) to stabilise as the normal confinement pressure value increases (Figure 12), particularly under large confining stresses when tensile failure of the reinforcement prevails. This tendency was observed in other studies (Lopes & Ladeira, 1996). When testing the extensible geogrid, higher tangential stresses

are generated along the soil-geogrid interface near the point of application of the pull-out force, and lesser mobilisation of these stresses occurs at the back of the reinforcement.

For normal confinement pressure below 37.5 kPa – at this value, the geogrid elements were detached – different values of f were found for the investigated materials. For higher normal confinement pressures, there was a tendency for the values of f to come closer because of tensile failure of the reinforcement. For normal confinement pressure equal to 100 kPa, they were practically the same regardless of the material and the moisture condition – at this normal confinement pressure, the rupture of the geogrid was observed. When reinforcement pull-out prevailed, higher values of interaction coefficients were obtained in tests with NS compared to those obtained with recycled sands, and this was observed mainly for the lower values of normal confinement pressure (Figure 12).

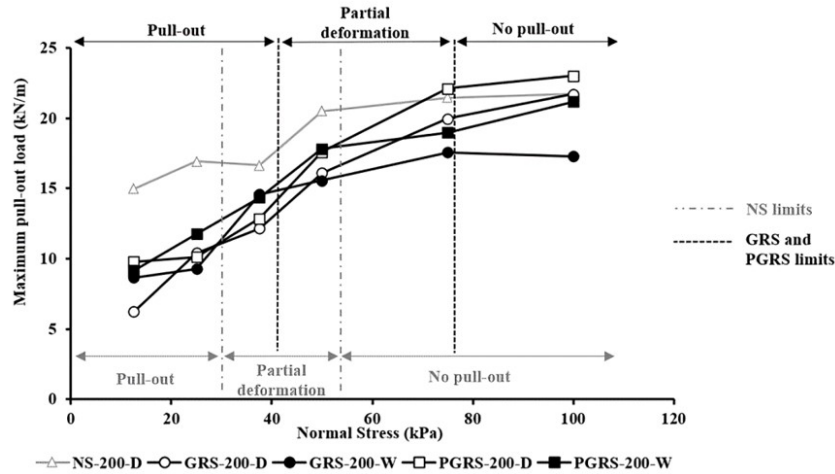


Figure 11. Soil-inclusion interface resistance envelope (pull-out tests performed using 200 mm-wide geogrid samples) and pull-out mechanism limits.

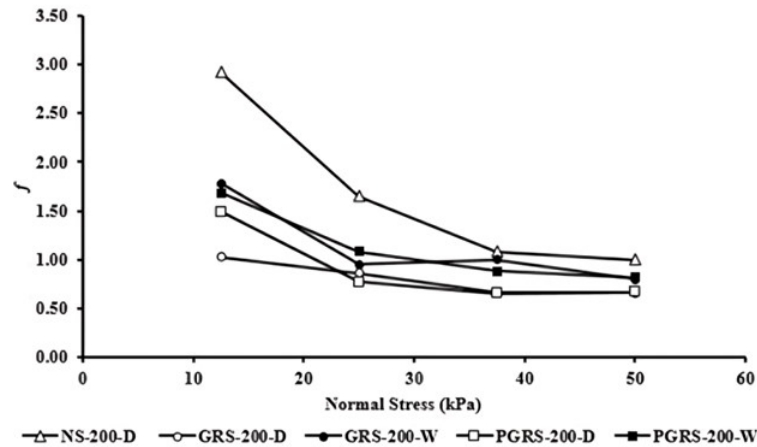
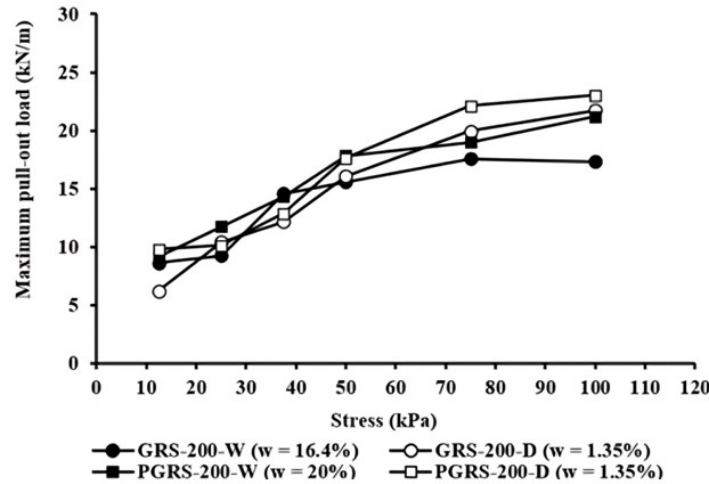


Figure 12. Variation of the interface coefficient (f) with normal confinement pressure. Where: $f = (\tau)/(\sigma_n \cdot \tan \phi)$; τ = tangential stress on the interface; σ_n = normal stress on the interface; ϕ = internal friction angle of the soil.

Table 3. Strength parameters for the tests with waste varying in moisture content.

Material	ϕ (degrees)	c (kPa)	δ (degrees)	a (kPa)
GRS-200-D	44	2.40	45	13.50
GRS-200-W	38	7.32	40	21.97
PGRS-200-D	46	0.00	40	24.22
PGRS-200-W	41	24.81	42	24.48

Note: GRS = grey recycled sand; PGRS = produced grey recycled sand; 200 = two-hundred-millimetre width of the geogrid sample; W = optimum moisture; D = hygroscopic moisture; ϕ = internal friction angle of the soil; c = cohesion; δ = interface friction angle; a = adhesion.

**Figure 13.** Maximum pull-out strength versus confining stress from tests with fill materials at different moisture contents.

PGRS presented slightly higher f values compared to GRS, either at optimum or hygroscopic moisture content. In general, for each recycled sand (GRS and PGRS), the tests performed at optimum moisture content showed slightly higher values than those performed at hygroscopic moisture. Values greater than one are due to the effect reported by Palmeira (1987) in tests with short grids with few bearing members.

3.1.3 Effect of moisture variation on the interface strength

The interface friction angles (δ) presented the same trend as the internal friction angles of the sands (ϕ), as it was observed that under dry and wet states, similar values were obtained. The lubrication between the grains at the interface contact with the geogrid elements was not sufficient to interfere with these values. Regarding adhesion (a), it is important to note that there was a significant increase in the wet conditions (at optimum water content). However, further studies on such aspects are needed. Table 3 shows the parameters obtained from the strength envelopes for the recycled sand-geogrid interfaces under pull-out (normal stresses of 12.5 kPa and 25 kPa) and the shear strength parameters of the recycled sand obtained from direct shear tests.

Figure 13 shows that the maximum pull-out force was higher for GRS and PGRS when they were at the optimum moisture content. When the sands are compared at the same

moisture content, it can be seen that PGRS presented higher values of pull-out strength, although the values for both materials (GRS and PGRS) were very close. In general, it was observed that for the range of values tested moisture content did not present significant impacts on the pull-out force for the investigated systems, especially for those where the geogrids were completely pulled out.

4. Conclusions

This paper investigated the interaction between geogrids and recycled sands by mean of pull-out tests. The tests were carried out on natural sand, grey recycled sand (as provided by the recycling plant) and laboratory-produced recycled grey sand (with the same grain size distribution in the natural sand), for varying moisture conditions. Based on the results obtained, the main conclusions are presented below.

1. There is a variability of the geotechnical characteristics of the recycled sands tested, but this would not prevent the application of this non-conventional material as backfill in GRS structures. After adequate characterisation, its grain size distribution can be adjusted to attend gradation limits recommended by technical standards;
2. The results of geogrid pull-out force versus displacement showed that the pull-out condition

was observed only for lower normal confinement pressures at the sample top. For very high pressure, the pull-out box worked as a clamp, preventing the geogrid from being pulled out and yielding to its tensile failure;

3. Based on the effect of the normal confinement pressures adopted in the tests, behavioural zones were identified for different mechanisms. This conclusion encourages the use of the small box in pull-out tests as a way to obtain preliminary qualitative results, particularly when investigating the potential use of a non-conventional fill material in RSS. However, results from larger equipment should be used for design purposes;
4. The recycled sands tested at optimum moisture showed higher values of pull-out strength (considering the normal stress range in which full grid pull-out occurred).

Therefore, compared to the behaviour of natural sand, the recycled sands showed good strength properties, mechanical behaviour and interaction with a geogrid under a pull-out condition that encourages further studies on the use of such environmentally friendly material in RSS.

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Declaration of interest

The authors have no affiliation with or involvement in any organisation with a direct or indirect financial interest in the subject matter discussed in the manuscript that could bias its results. All co-authors have seen and agree with the content of the manuscript and certify that it has not been submitted to, nor is under review at another journal or other publishing venue.

Authors' contributions

Priscila F. S. Oliveira: methodology, investigation, validation, writing – original draft preparation. Eder C. G. Santos: conceptualisation, methodology, supervision, validation, funding acquisition, writing – reviewing and editing. Ennio M. Palmeira: supervision, validation, writing – reviewing and editing.

Data availability

Research data available upon request.

List of symbols and abbreviations

a	Adhesion
c	Cohesion
e_{max}	Maximum voids index
e_{min}	Minimum voids index
f	Interface resistance coefficient
w	Water content
w_{op}	Optimum water content
Al	Aluminium
Al_2O_3	Alumina
Ca	Calcium
CaO	Calcium oxide
C_c	Soil curvature coefficient ($C_c = D_{30}^2 / D_{10} D_{60}$)
CDW	Construction and demolition waste
C_u	Soil coefficient of uniformity ($C_u = D_{60} / D_{10}$)
CV	Coefficient of variation
D	Diameter of the particle (mm)
D	Hygroscopic moisture
DC	Degree of compaction (%)
D_{50}	Diameter of the particle for which 50% of soil in mass is smaller (than that diameter)
D_{85}	Diameter of the particle for which 85% of soil in mass is smaller (than that diameter)
EDS	Energy-dispersive X-ray spectroscopy
Fe_2O_3	Ferrous oxide
GRS	Grey recycled sand
GRS-01	Grey recycled sand (sample 01)
GRS-02	Grey recycled sand (sample 02)
GRS-03	Grey recycled sand (sample 03)
GRS-04	Grey recycled sand (sample 04)
GRS-05	Grey recycled sand (sample 05)
GRS-06	Grey recycled sand (sample 06)
GRS-06-D	Dry grey recycled sand (sample 06)
GRS-06-W	Wet grey recycled sand (sample 06)
GRS-200-D	Dry grey recycled sand sample with two-hundred-millimetre width of the geogrid sample
GRS-200-W	Sample wet grey recycled sand with two-hundred-millimetre width of the geogrid sample
G_s	Specific gravity
K_2O	Potassium oxide
MgO	Magnesium oxide
NCMA	National Concrete Masonry Association
NS	Natural sand
NS-D	Dry natural sand sample
NS-200-D	Dry natural sand sample with two-hundred-millimetre width of the geogrid sample

PGRS	Produced grey recycled sand
PGRS-06-D	Dry produced grey recycled sand (sample 06)
PGRS-06-W	Wet produced grey recycled sand (sample 06)
PGRS-200-D	Dry produced grey recycled sand sample with two-hundred-millimetre width of the geogrid sample
PGRS-200-W	Sample wet produced grey recycled sand with two-hundred-millimetre width of the geogrid sample
RCDW	Recycled construction and demolition waste
RSS	Reinforced soil structures
SEM	Scanning electron microscopy
Si	Silicon
SiO ₂	Silica
SP	Poorly graded sand
SP-SC	Poorly graded sand with clay
TCP	Total pressure cell
TSC	Total stress cell
USCS	Unified Soil Classification System
γ_{dMax}	Maximum dry unit weight (kN/m ³)
δ	Interface friction angle
σ	Normal confinement pressure
σ_n	Normal stress on the interface
τ	Tangential stress on the interface
ϕ	Internal friction angle of the soil

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Homogenization theory and nonlinearities in Darcy's law

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Article

Keywords

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Abstract

The theory of homogenization of differential equations has become an open field of research in several areas of the exact sciences and has proved to be a powerful tool for understanding the global behavior of heterogeneous materials. Despite knowing that the deduction of Darcy's law through the Navier-Stokes equations has been debated for decades, many questions remain open, mainly regarding more complex boundary conditions, cases involving multiphase flows and the numerical homogenization techniques. It is known today that Darcy's law is presented in the form of a linear relationship only for a range of hydraulic gradient and that this range overlaps the range of laminar flow through soil voids. Therefore, it is proposed in this work to understand the loss of linearity in Darcy's law, based on the theory of homogenization, modifying and exploring the limit results obtained by Allaire in 1991.

1. Introduction

After Darcy, mainly at the beginning of the 20th century, researchers such as Forchheimer and Kochina brought to light the emergence of a non-linear relationship between the hydraulic gradient and the flow velocity, a concept to be formally defined in this work. What should be emphasized here is that Darcy's law loses its linear character from a certain hydraulic gradient, a quantity that is directly proportional to the pressure gradient of the fluid (Bear, 1985). By admitting that the non-linearity is relative to a critical hydraulic gradient and that the pressure gradient is dependent on the domain where it is being evaluated, and knowing that the flow does not occur through the solid particles, but through the pores of the material, the hypothesis of a study on the pore scale is valid. Therefore, a process of homogenization of the differential equations will be presented so that the mathematical objects living in the pore scale are captured on a larger scale, in real geotechnical problems.

For the arguments just presented, a mathematical analysis of the possible mechanisms and triggers of this non-linearity is suggested, which will later be associated with a loss of hydrodynamic stability and a possible transition to the turbulent regime. To mathematically capture these phenomena, a rigorous study of the differential equations that govern the flow problem will be carried out throughout this work. Not only differential equations will be studied, but also the construction of numerical methods capable of exploring the behavior of the fluid in a tortuous and locally discontinuous domain such as the soil. This work will use arguments from functional analysis, general topology, dynamical systems

and the modern theory of differential equations to support the simulations carried out by the Finite Element Method.

2. Proof of the convergence of the Stokes problem for unit cells with non-Dirichlet boundary conditions

Let the Stokes problem be studied by Allaire (1991a), being this:

Find $(u_\varepsilon, p_\varepsilon) \in [H_0^1(\Omega_\varepsilon)]^N \times L^2(\Omega_\varepsilon)$ solutions of the problem (P1)

$$\nabla p_\varepsilon - \Delta u_\varepsilon = f, \text{ at } \Omega_\varepsilon$$

$$\nabla \cdot u_\varepsilon = 0, \text{ at } \Omega_\varepsilon \quad (P1)$$

With $\varepsilon \rightarrow 0$, p_ε , u_ε and f being the pressure, velocity and the driving forces acting on the fluid. For the problem with non-Dirichlet conditions (slip condition) on part of the boundary and with Dirichlet conditions on the rest, the

functional space $H_0^1(\Omega_\varepsilon)$ is no longer adequate. Solutions must be sought in $H^1(\Omega_\varepsilon) \times L^2(\Omega_\varepsilon)$. Let be the variational formulation of the same problem (P1):

$$\begin{aligned} \int_{\Omega_\varepsilon} \nabla p_\varepsilon \cdot v \, dV - \int_{\Omega_\varepsilon} \Delta u_\varepsilon \cdot v \, dV = \\ \int_{\Omega_\varepsilon} f \cdot v \, dV \quad \forall v \in H^1(\Omega_\varepsilon)^N, \text{ at } \Omega_\varepsilon \\ \int_{\Omega_\varepsilon} q (\nabla \cdot u_\varepsilon) \, dV = 0 \quad \forall q \in L^2(\Omega_\varepsilon), \text{ at } \Omega_\varepsilon \end{aligned} \quad (P2)$$

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Take $v = \phi w_\varepsilon^k$ and $q = \phi q_\varepsilon^k$ where $\phi \in D(\Omega)$. The functions $w_\varepsilon^k \in H^1(\Omega_\varepsilon)^N$ and $q \in L^2(\Omega_\varepsilon)$ are sequences whose convergence will be proved from some hypotheses. Allaire (1991a) assumed the problem in $\mathbb{R}^N$, with no boundary conditions except on the surface of grains (holes) periodically distributed throughout the domain. In this work, we will assume that the problem is restricted to an open set Ω such that:

$$\Omega = \Omega_\varepsilon + \bigcup_{i=1}^{N(\varepsilon)} T_i^\varepsilon \quad (\text{Property 1})$$

$$\partial\Omega = \bigcup_{j=1}^n \Gamma_j, \Gamma_i \cap \Gamma_j = \emptyset \text{ for } i \neq j \quad (\text{Property 2})$$

$$\partial\Omega \cap \partial T_i^\varepsilon = \emptyset \quad (\text{Property 3})$$

$$\bar{\Omega} = \Omega + \partial\Omega \quad (\text{Property 4})$$

Integrating by parts (P2):

$$\begin{aligned} & - \int_{\Omega_\varepsilon} p_\varepsilon (\nabla \cdot v) dV + \int_{\partial\Omega_\varepsilon} (p_\varepsilon v) \cdot n dS + \int_{\Omega_\varepsilon} \nabla u_\varepsilon \cdot \nabla v dV \\ & = \int_{\Omega_\varepsilon} f \cdot v dV \quad \forall v \in H^1(\Omega_\varepsilon)^N, \text{ at } \Omega_\varepsilon \\ & \int_{\Omega_\varepsilon} q (\nabla \cdot u_\varepsilon) dV = 0 \quad \forall q \in L^2(\Omega_\varepsilon), \text{ at } \Omega_\varepsilon \quad (P2) \end{aligned}$$

Note that the boundary condition was omitted in (P2) because the expression refers only to the functional inside the domain and not on its boundary. Replacing v and q by their previously given definitions and expanding (P2):

$$\begin{aligned} & - \int_{\Omega_\varepsilon} p_\varepsilon (\phi \nabla \cdot w_\varepsilon^k + w_\varepsilon^k \cdot \nabla \phi) dV + \\ & \int_{\Omega_\varepsilon} \nabla u_\varepsilon \cdot (\phi \nabla w_\varepsilon^k + w_\varepsilon^k \nabla \phi) dV \\ & = \int_{\Omega_\varepsilon} f \cdot \phi w_\varepsilon^k dV \quad \forall v \in H^1(\Omega_\varepsilon)^N, \text{ at } \Omega_\varepsilon \\ & \int_{\Omega_\varepsilon} \phi q_\varepsilon^k (\nabla \cdot u_\varepsilon) dV = 0 \quad \forall q \in L^2(\Omega_\varepsilon) \setminus R, \text{ at } \Omega_\varepsilon \quad (P2) \end{aligned}$$

Consider the following six hypotheses regarding the functions w_ε^k and q_ε^k , based on the work of Allaire (1991a):

$$w_\varepsilon^k \in H^1(\Omega)^N, q_\varepsilon^k \in L^2(\Omega) \quad (\text{Hypothesis 1})$$

$$\nabla \cdot w_\varepsilon^k = 0 \text{ at } \Omega \text{ and } w_\varepsilon^k \text{ inside } T_i^\varepsilon \quad (\text{Hypothesis 2})$$

$$w_\varepsilon^k \rightharpoonup e_k \text{ weakly in } H^1(\Omega) \text{ and } q_\varepsilon^k \rightharpoonup 0 \text{ weakly in } L^2(\Omega) \quad (\text{Hypothesis 3})$$

$$\exists \mu_k \in W^{-1,\infty}(\Omega)^N \quad (\text{Hypothesis 4})$$

For every sequence v_ε and for every v such that:

$$v_\varepsilon \rightharpoonup v \text{ weakly in } H^1(\Omega)^N, \quad v_\varepsilon = 0 \text{ at } T_i^\varepsilon \quad (\text{Hypothesis 5.a})$$

And for each $\phi \in D(\Omega)$, holds:

$$\begin{aligned} & (\nabla q_\varepsilon^k - \Delta w_\varepsilon^k, \phi v_\varepsilon)_{H^{-1}, H^1(\Omega)^N} \rightharpoonup \\ & (\mu_k, \phi v)_{H^{-1}, H^1(\Omega)^N} \quad (\text{Hypothesis 5.b}) \end{aligned}$$

There exists an extension linear operator R_ε such that:

$$R_\varepsilon \in L(H^1(\Omega)^N; H^1(\Omega_\varepsilon)^N) \quad (\text{Hypothesis 6.a})$$

$$u \in H^1(\Omega_\varepsilon)^N \Rightarrow R_\varepsilon \tilde{u} = u, \text{ at } \Omega_\varepsilon \quad (\text{Hypothesis 6.b})$$

$$\nabla \cdot u = 0 \text{ at } \Omega \Rightarrow \nabla \cdot (R_\varepsilon u) = 0 \text{ at } \Omega_\varepsilon \quad (\text{Hypothesis 6.c})$$

$$R_\varepsilon u_{H^1(\Omega_\varepsilon)} \leq C u_{H^1(\Omega)}, \quad C \text{ independent of } \varepsilon \quad (\text{Hypothesis 6.d})$$

The functions w_ε^k and $q_\varepsilon^k, 1 \leq k \leq N$ are test functions for the homogenization process by the energy method. In Allaire's work (1991a) all elements of the Hilbert spaces cited in the hypotheses are functions with compact support in Ω . The aim of this section is to prove that convergence also occurs for H^1 spaces with functions that guarantee specific boundary conditions that are not of the Dirichlet type. Rewriting (P2) with respect to the six hypotheses presented, we have:

$$\begin{aligned} & - \int_{\Omega_\varepsilon} p_\varepsilon w_\varepsilon^k \cdot \nabla \phi dV + \int_{\Omega_\varepsilon} \nabla u_\varepsilon \cdot w_\varepsilon^k \nabla \phi dV + \\ & \int_{\Omega_\varepsilon} \phi \nabla u_\varepsilon \cdot \nabla w_\varepsilon^k dV \\ & = \int_{\Omega_\varepsilon} f \cdot \phi w_\varepsilon^k dV, \quad \forall v \in H^1(\Omega_\varepsilon)^N, \text{ at } \Omega_\varepsilon \end{aligned}$$

$$\int_{\Omega_\varepsilon} \phi q_\varepsilon^k (\nabla \cdot u_\varepsilon) dV = 0, \quad \forall q \in L^2(\Omega_\varepsilon), \text{ at } \Omega_\varepsilon \quad (P2)$$

The case studied here has four boundaries where two of them receive Dirichlet conditions (for pressure) and the other two receive slip conditions (free shear stress). Figures 1 to 3 illustrate the problem to be passed to the limit.

Let be the following boundary conditions (BC) for Ω_ε :

$$\Gamma_1 \Rightarrow p_\varepsilon = p_\varepsilon^{in} \quad (BC \ 1)$$

$$\Gamma_2 \Rightarrow -n \cdot \left(-p_\varepsilon I + \mu (\nabla u_\varepsilon + \nabla u_\varepsilon^T) \right) \cdot t = 0 \quad (BC \ 2)$$

$$\Gamma_3 \Rightarrow p_\varepsilon = p_\varepsilon^{out} \quad (BC \ 3)$$

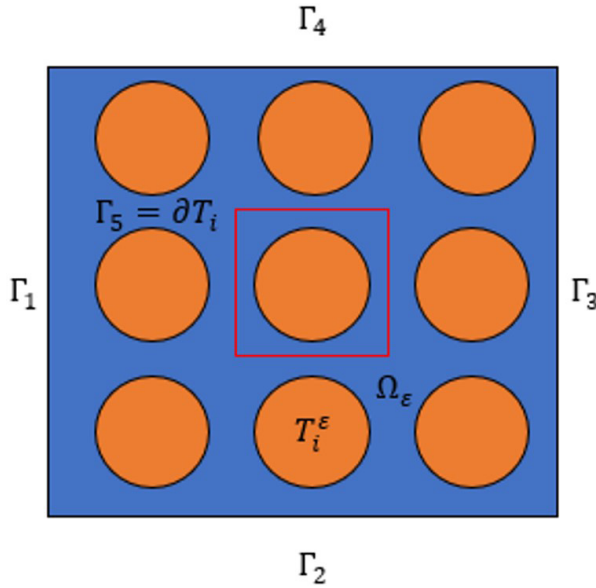


Figure 1. Periodically perforated domain.

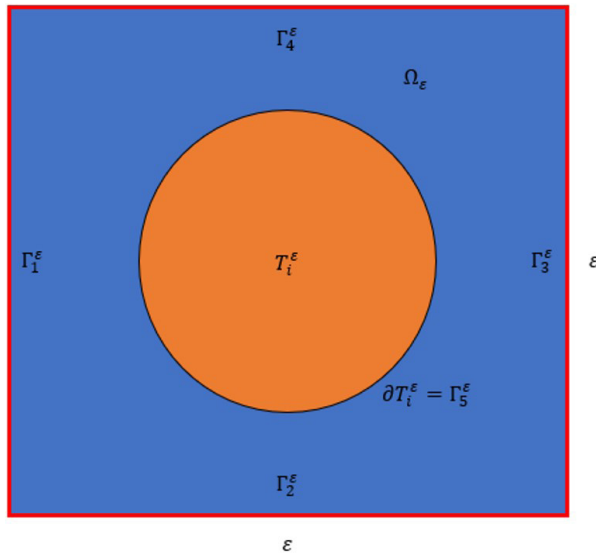


Figure 2. Unit cell.

$$\Gamma_4 = \Gamma_2 \quad (BC \ 4)$$

$$\Gamma_5 \Rightarrow u_\varepsilon = 0 \quad (BC \ 5)$$

How then does the functional behave for each boundary? Intuitively, different functionals are expected to act on each type of boundary. It remains to be seen whether they are compatible.

On the boundary Γ_1 one can write the inlet pressure from the Bernoulli relation such that:

$$p_{\varepsilon 0} = p_\varepsilon + \frac{1}{2} \rho |u_\varepsilon|^2 \quad (1)$$

Where p_ε is the stagnation pressure imposed on the boundary and p is the static pressure on the boundary. It is important to point out at this point that there are no boundary conditions for pressures in the Stokes and Navier-Stokes equations; for this, the Bernoulli's relation is assumed. Assuming $\rho = 1$, the pressure boundary condition becomes a Dirichlet condition for velocities, such that the imposed velocity is:

$$|u_\varepsilon|_{\Gamma_1} = \sqrt{2(p_{\varepsilon 0} - p_\varepsilon)} \quad (2. \ a)$$

Assuming that this input velocity is purely horizontal, it becomes possible to construct the velocity vector from its magnitude such that:

$$u_{\varepsilon \Gamma_1} = \left[\sqrt{2(p_{\varepsilon 0} - p_\varepsilon)} \right] e_k, \quad e_k = \begin{pmatrix} 1 \\ 0 \end{pmatrix} \quad (2. \ b)$$

Note that for the equality in (2.b) to be true, the right-hand side must also be in H^1 . It is known that $p_\varepsilon \in L^2(\Omega_\varepsilon)$ and that, being $p_{\varepsilon 0} \in L^2(\Omega_\varepsilon)$ then, by the linearity of space, $(p_{\varepsilon 0} - p_\varepsilon) = \widehat{p_\varepsilon} \in L^2(\Omega_\varepsilon)$, therefore, every sequence or subsequence $\widehat{p_\varepsilon}$ is continuous in Ω_ε .

On the boundaries Γ_2 and Γ_4 , the free sliding condition operates, where the shear stress is considered zero. This condition comes from Navier's hypothesis about the proportionality between sliding velocity and shear stress. One has:

$$n^T \cdot (2\mu \mathbb{D}(u) - pI) \cdot t - \beta u \cdot t = 0 \quad (3. \ a)$$

where

$$D(u) = \mu (\nabla u + \nabla u^T) \quad (3. \ b)$$

Where β is the coefficient of friction between the fluid and the boundary, so $\beta = 0$ indicates free sliding. It is important to point out that the imposition of a slip boundary condition affects the very formulation of the variational problem, as the condition acts on the stress tensor itself, thus necessitating another variational formulation for this specific boundary. Allaire (1991b), in another work on homogenization with slip conditions at the boundary

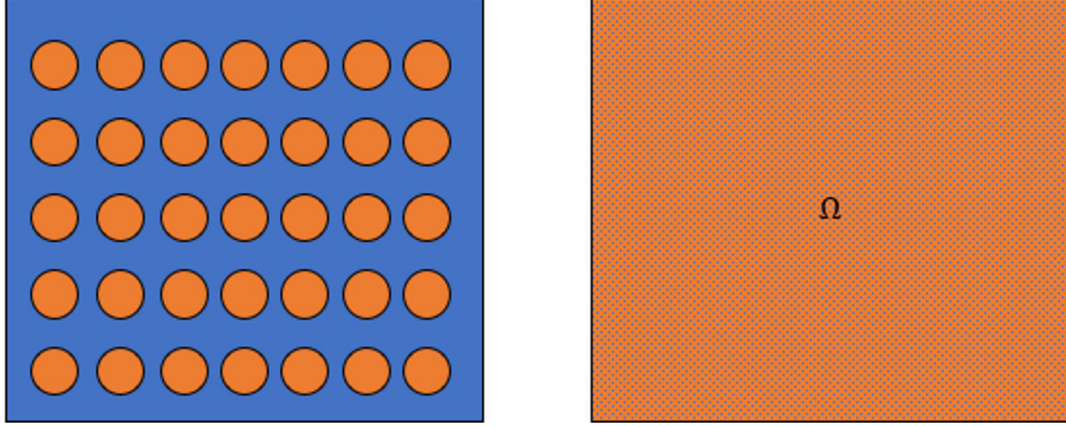


Figure 3. Homogenized domain.

of the obstacle (hole), proved the coercivity and convergence of the Stokes system to a Brinkman-type law. Here, the free-sliding condition is imposed on the domain boundary and not on the grain surface (which is maintained with Dirichlet), but the result applies equally. It can be proposed that, by $\nabla \cdot u_\varepsilon = 0$, we have:

$$\Delta u_\varepsilon = 2\nabla \cdot \mathbb{D}(u) \quad (4)$$

The term corresponding to the Laplacian in the variational becomes:

$$\int_{\Omega_\varepsilon} 2\nabla \cdot \mathbb{D}(u) \cdot v \, dV \quad (5)$$

whose integration by parts results in:

$$-2 \int_{\Omega_\varepsilon} \mathbb{D}(u) \cdot \nabla v \, dV + 2 \int_{\Gamma_{2,4}} \mathbb{D}(u) \cdot v \cdot n \, dS \quad (6)$$

The first term, by symmetry, can be rewritten as:

$$\begin{aligned} \int_{\Omega_\varepsilon} \mathbb{D}(u) \cdot \nabla v \, dV &= \int_{\Omega_\varepsilon} \mathbb{D}(u) \cdot \frac{\nabla v}{2} \, dV + \\ \int_{\Omega_\varepsilon} \mathbb{D}(u) \cdot \frac{\nabla v^T}{2} \, dV &= \int_{\Omega_\varepsilon} \mathbb{D}(u) \cdot \mathbb{D}(v) \, dV \end{aligned} \quad (7)$$

Because, by symmetry, $\mathbb{D}(u)^T = \mathbb{D}(u)$. Knowing that the Stokes equation can be rewritten with the explicit tensor:

$$\nabla \cdot (pI - \mu \mathbb{D}(u)) = f \quad (8)$$

The variational formulation can then be rewritten as:

$$- \int_{\Omega_\varepsilon} \nabla \cdot (-pI + \mu \mathbb{D}(u)) \cdot v \, dV = \int_{\Omega_\varepsilon} f \cdot v \, dV \quad (9. a)$$

$$- \int_{\Omega_\varepsilon} (-\nabla p_\varepsilon + \mu \nabla \cdot \mathbb{D}(u)) \cdot v \, dV = \int_{\Omega_\varepsilon} f \cdot v \, dV \quad (9. b)$$

$$\int_{\Omega_\varepsilon} \nabla p_\varepsilon \cdot v \, dV - \mu \int_{\Omega_\varepsilon} \nabla \cdot \mathbb{D}(u) \cdot v \, dV = \int_{\Omega_\varepsilon} f \cdot v \, dV \quad (9. c)$$

$$\begin{aligned} - \int_{\Omega_\varepsilon} p_\varepsilon \nabla \cdot v \, dV + \int_{\Gamma_{2,4}} p_\varepsilon v \cdot n \, dS + 2\mu \int_{\Omega_\varepsilon} \mathbb{D}(u_\varepsilon) \cdot \\ \mathbb{D}(v) \, dV + 2\mu \int_{\Gamma_{2,4}} \mathbb{D}(u_\varepsilon) \cdot v \cdot n \, dS \end{aligned} \quad (9. d)$$

$$= \int_{\Omega_\varepsilon} f \cdot v \, dV, \text{ at } \Omega_\varepsilon \cup \Gamma_2 \cup \Gamma_4 \, \forall v \in H^1(\Omega_\varepsilon)$$

The above expression can be further condensed by joining the terms on the boundary, such as:

$$\begin{aligned} - \int_{\Omega_\varepsilon} p_\varepsilon \nabla \cdot v \, dV + 2\mu \int_{\Omega_\varepsilon} \mathbb{D}(u_\varepsilon) \cdot \mathbb{D}(v) \, dV - \\ \int_{\Gamma_{2,4}} (-p_\varepsilon I + 2\mu \mathbb{D}(u_\varepsilon)) \cdot v \, dS = \int_{\Omega_\varepsilon} f \cdot v \, dV \end{aligned} \quad (9. e)$$

By decomposing $v \in H^1(\Omega_\varepsilon)$ at the boundary into its normal and tangential components:

$$v = v \cdot n + \sum_{i=1}^{N-1} v \cdot t_i, \, v \in H^1(\Omega_\varepsilon)^N \quad (10)$$

The variational formulation becomes:

$$\begin{aligned} - \int_{\Omega_\varepsilon} p_\varepsilon \nabla \cdot v \, dV + 2\mu \int_{\Omega_\varepsilon} \mathbb{D}(u_\varepsilon) \cdot \mathbb{D}(v) \, dV - \\ \int_{\Gamma_{2,4}} n^T (-p_\varepsilon I + 2\mu \mathbb{D}(u_\varepsilon)) n v \cdot n \, dS \\ + \int_{\Gamma_{2,4}} n^T (-p_\varepsilon I + 2\mu \mathbb{D}(u_\varepsilon)) t v \cdot t \, dS = \int_{\Omega_\varepsilon} f \cdot v \, dV \end{aligned} \quad (11)$$

Where by the impenetrability of the boundary, the variational is reduced to:

$$\begin{aligned} & - \int_{\Omega_\varepsilon} p_\varepsilon \nabla \cdot v dV + 2\mu \int_{\Omega_\varepsilon} \mathbb{D}(u_\varepsilon) : \mathbb{D}(v) dV + \\ & \int_{\Gamma_{2,4}} n^T (-p_\varepsilon I + 2\mu \mathbb{D}(u_\varepsilon)) tv \cdot t dS \\ & = \int_{\Omega_\varepsilon} f \cdot v dV \end{aligned} \quad (12)$$

Being, by condition imposed on the border, $\beta = 0$

$$\int_{\Gamma_{2,4}} n^T (-p_\varepsilon I + 2\mu \mathbb{D}(u_\varepsilon)) tv \cdot t dS = 0 \quad (13)$$

Thus, at $\Gamma_2 \cup \Gamma_4 + \Omega_\varepsilon$:

$$- \int_{\Omega_\varepsilon} p_\varepsilon \nabla \cdot v dV + 2\mu \int_{\Omega_\varepsilon} \mathbb{D}(u_\varepsilon) : \mathbb{D}(v) dV = \int_{\Omega_\varepsilon} f \cdot v dV \quad (14)$$

With that, one has the (apparent) result that such conditions on the boundary of Ω simulate (within a few possible adjustments) the condition of Allaire (1991a) on $\mathbb{R}^N$ since the Dirichlet conditions are just inputs of “velocity at the infinity” and the slip conditions recover the Stokes partial differential equation itself. Soon, the problem becomes just a Stokes flow around the cylinder, from where we can henceforth analyze the grain size problem and the emergence of “strange terms”.

With the variational defined in Ω_ε , Ω (by extension to be used later) and its boundaries, it is assumed that $v = w_\varepsilon^k \phi$ and $q = q_\varepsilon^k \phi$ with hypotheses similar to those of Allaire (1991a). It is verified that the variational generated above converge to a Brinkman-type equation, regardless of whether it is on the boundary or inside the domain.

For the variational referring to Ω_ε :

$$\begin{aligned} & - \int_{\Omega_\varepsilon} p_\varepsilon w_\varepsilon^k \cdot \nabla \phi dV + \int_{\Omega_\varepsilon} \nabla u_\varepsilon \cdot \\ & w_\varepsilon^k \nabla \phi dV + \int_{\Omega_\varepsilon} \phi \nabla u_\varepsilon \cdot \nabla w_\varepsilon^k dV \\ & = \int_{\Omega_\varepsilon} f \cdot \phi w_\varepsilon^k dV, \quad \forall v \in H^1(\Omega_\varepsilon)^N, \text{ at } \Omega_\varepsilon \\ & = \int_{\Omega_\varepsilon} f \cdot \phi w_\varepsilon^k dV, \quad \forall v \in H^1(\Omega_\varepsilon)^N, \text{ at } \Omega_\varepsilon \end{aligned} \quad (P2)$$

Note that the third term to the left of the equality has the following identity:

$$\begin{aligned} & \int_{\Omega_\varepsilon} \phi \nabla u_\varepsilon \cdot \nabla w_\varepsilon^k dV + \int_{\Omega_\varepsilon} u_\varepsilon \nabla \phi \cdot \nabla w_\varepsilon^k dV = \\ & - \int_{\Omega_\varepsilon} \phi u_\varepsilon \cdot \Delta w_\varepsilon^k dV \end{aligned} \quad (15)$$

Up to one term on the boundary that disappears because it is a functional on Ω_ε . Therefore, the same term can be replaced by:

$$\begin{aligned} & \int_{\Omega_\varepsilon} \phi \nabla u_\varepsilon \cdot \nabla w_\varepsilon^k dV = - \int_{\Omega_\varepsilon} u_\varepsilon \nabla \phi \cdot \nabla w_\varepsilon^k dV - \\ & \int_{\Omega_\varepsilon} \phi u_\varepsilon \cdot \Delta w_\varepsilon^k dV \end{aligned} \quad (16)$$

Integrating by parts the continuity equation (div-free condition):

$$\int_{\Omega_\varepsilon} \nabla \cdot (\phi q_\varepsilon^k) \cdot u_\varepsilon dV = 0, \quad \forall q \in L^2(\Omega_\varepsilon), \text{ at } \Omega_\varepsilon \quad (17)$$

Expanding it as:

$$\begin{aligned} & \int_{\Omega_\varepsilon} \phi \nabla q_\varepsilon^k \cdot u_\varepsilon dV + \int_{\Omega_\varepsilon} q_\varepsilon^k \nabla \phi \cdot u_\varepsilon = \\ & 0, \quad \forall q \in L^2(\Omega_\varepsilon) \setminus R, \text{ at } \Omega_\varepsilon \end{aligned} \quad (18)$$

Because we are in a strictly real Hilbert space, the order of ϕ in the first integral does not change the result, so:

$$\begin{aligned} & \int_{\Omega_\varepsilon} \nabla q_\varepsilon^k \cdot \phi u_\varepsilon dV + \int_{\Omega_\varepsilon} q_\varepsilon^k \nabla \phi \cdot u_\varepsilon = 0, \\ & \forall q \in L^2(\Omega_\varepsilon), \text{ at } \Omega_\varepsilon \end{aligned} \quad (19)$$

The continuity equation is part of the PDE system and therefore, by linearity, the Stokes equation can be added, thus generating the PDE:

$$\begin{aligned} & - \int_{\Omega_\varepsilon} p_\varepsilon w_\varepsilon^k \cdot \nabla \phi dV + \int_{\Omega_\varepsilon} \nabla u_\varepsilon \cdot w_\varepsilon^k \nabla \phi dV \\ & - \int_{\Omega_\varepsilon} u_\varepsilon \nabla \phi \cdot \nabla w_\varepsilon^k dV - \int_{\Omega_\varepsilon} \phi u_\varepsilon \cdot \Delta w_\varepsilon^k dV \\ & + \int_{\Omega_\varepsilon} \nabla q_\varepsilon^k \cdot \phi u_\varepsilon dV + \int_{\Omega_\varepsilon} q_\varepsilon^k \nabla \phi \cdot u_\varepsilon dV \\ & = \int_{\Omega_\varepsilon} f \cdot \phi w_\varepsilon^k dV, \quad \forall v \in H^1(\Omega_\varepsilon)^N, \text{ at } \Omega_\varepsilon \end{aligned} \quad (20)$$

The PDE (20) is only in Ω_ε . The decision is then made to use the linear extension operator to evaluate the behavior of

the PDE in Ω . By the sixth hypothesis, when applying the operator in all terms of EDP (6.20), one recovers:

$$\begin{aligned} & -\int_{\Omega} P_{\varepsilon}(p_{\varepsilon}) w_{\varepsilon}^k \cdot \nabla \phi dV + \int_{\Omega} \widetilde{\nabla u_{\varepsilon}} \cdot w_{\varepsilon}^k \nabla \phi dV - \\ & \int_{\Omega} \widetilde{u_{\varepsilon}} \nabla \phi \cdot \nabla w_{\varepsilon}^k dV - \int_{\Omega} \phi \widetilde{u_{\varepsilon}} \cdot \Delta w_{\varepsilon}^k dV \\ & + \int_{\Omega} \nabla q_{\varepsilon}^k \cdot \phi \widetilde{u_{\varepsilon}} dV + \int_{\Omega} q_{\varepsilon}^k \nabla \phi \cdot \widetilde{u_{\varepsilon}} dV \\ & = \int_{\Omega} f \cdot \phi w_{\varepsilon}^k dV, \forall v \in H^1(\Omega_{\varepsilon})^N, \text{ at } \Omega \end{aligned} \quad (21)$$

Regrouping some terms, specifically the fourth and fifth term to the left of the equality:

$$\begin{aligned} & -\int_{\Omega} P_{\varepsilon}(p_{\varepsilon}) w_{\varepsilon}^k \cdot \nabla \phi dV + \int_{\Omega} \widetilde{\nabla u_{\varepsilon}} \cdot w_{\varepsilon}^k \nabla \phi dV \\ & - \int_{\Omega} \widetilde{u_{\varepsilon}} \nabla \phi \cdot \nabla w_{\varepsilon}^k dV \\ & + \int_{\Omega} (\nabla q_{\varepsilon}^k - \Delta w_{\varepsilon}^k) \cdot \phi \widetilde{u_{\varepsilon}} dV + \int_{\Omega} q_{\varepsilon}^k \nabla \phi \cdot \widetilde{u_{\varepsilon}} dV \\ & = \int_{\Omega} f \cdot \phi w_{\varepsilon}^k dV, \forall v \in H^1(\Omega)^N, \text{ at } \Omega \end{aligned} \quad (22)$$

Where one immediately recognizes the pair of dualities:

$$\begin{aligned} & \int_{\Omega} (\nabla q_{\varepsilon}^k - \Delta w_{\varepsilon}^k) \cdot \phi \widetilde{u_{\varepsilon}} dV = \\ & (\nabla q_{\varepsilon}^k - \Delta w_{\varepsilon}^k, \phi \widetilde{u_{\varepsilon}})_{H^{-1}(\Omega), H^1(\Omega)} \end{aligned} \quad (23)$$

Let the following convergences (C) be:

$$\widetilde{u_{\varepsilon}} \rightharpoonup u \in H^1(\Omega)^N \text{ weakly} \quad (C1)$$

$$w_{\varepsilon}^k \rightharpoonup e_k \in H^1(\Omega)^N \text{ weakly} \quad (C2)$$

$$q_{\varepsilon}^k \rightharpoonup 0 \in L^2(\Omega) \text{ weakly} \quad (C3)$$

$$P_{\varepsilon}(p_{\varepsilon}) \rightharpoonup p \in L^2(\Omega) \text{ weakly} \quad (C4)$$

Finally, the functional on Ω_{ε} converges to a functional on Ω in the form:

$$\begin{aligned} & -\int_{\Omega} p e_k \cdot \nabla \phi dV + \int_{\Omega} \nabla u \cdot e_k \nabla \phi dV \\ & + \mu_k, \phi u_{H^{-1}(\Omega), H^1(\Omega)} \end{aligned}$$

$$= \int_{\Omega} f \cdot \phi e_k dV, \forall v \in H^1(\Omega)^N, \text{ at } \Omega \quad (24)$$

Since $\nabla e_k = 0$. Integrating by parts and passing to the limit in Ω :

$$\begin{aligned} & \int_{\Omega} \nabla p \cdot \phi dV - \int_{\Omega} \Delta u \cdot \phi dV \\ & + (\phi \mu_k, u)_{H^{-1}(\Omega), H^1(\Omega)} \\ & = \int_{\Omega} f \cdot \phi e_k dV, \forall \phi \in D(\Omega), \text{ at } \Omega \end{aligned} \quad (25)$$

Thus:

$$\nabla p - \Delta u + M u = f, \text{ at } \Omega \quad (\text{PDE. a})$$

$$\nabla \cdot u = 0, \text{ at } \Omega \quad (\text{PDE. b})$$

Where M is independent of u .

Now, let's focus on the free-sliding boundaries. Let be the variational formulation on the boundary:

$$\begin{aligned} & -\int_{\Omega_{\varepsilon}} p_{\varepsilon} \nabla \cdot v dV + 2\mu \int_{\Omega_{\varepsilon}} \mathbb{D}(u_{\varepsilon}) \cdot \mathbb{D}(v) dV = \\ & \int_{\Omega_{\varepsilon}} f \cdot v dV \end{aligned} \quad (26)$$

Replacing v with its previously established definition:

$$\begin{aligned} & -\int_{\Omega_{\varepsilon}} p_{\varepsilon} \nabla \cdot (\phi w_{\varepsilon}^k) dV + 2\mu \int_{\Omega_{\varepsilon}} \mathbb{D}(u_{\varepsilon}) \cdot \mathbb{D}(\phi w_{\varepsilon}^k) dV = \\ & \int_{\Omega_{\varepsilon}} f \cdot \phi w_{\varepsilon}^k dV \end{aligned} \quad (27)$$

The first term to the left of the equality has already been discussed previously for the functional on Ω_{ε} . We must now analyze the second term to the left of the equality, referring to the stress tensor, specifically its deviatoric part. At this point, it seems difficult to extract any property from the functional above that might resemble the results obtained so far. However, it is enough to go back a few steps in the development of the variational formulation so that one can take advantage of the incompressibility of the test function field and the velocity field itself.

It is known that:

$$2D(u_{\varepsilon}) \cdot D(\phi w_{\varepsilon}^k) = 2D(u_{\varepsilon}) \cdot D(v) = D(u_{\varepsilon}) \cdot \nabla v \quad (28)$$

From this, the following integration by parts follows naturally:

$$\int_{\Omega_\varepsilon} \mathbb{D}(u_\varepsilon) \cdot \nabla v \, dV = -\frac{1}{2} \int_{\Omega_\varepsilon} \nabla \cdot (\nabla u_\varepsilon + \nabla u_\varepsilon^T) \cdot v \, dV + \frac{1}{2} \int_{\partial\Omega_\varepsilon = \tilde{A}_{2,4}} (\nabla u_\varepsilon + \nabla u_\varepsilon^T) \cdot v \cdot n \, dS \quad (29)$$

Where the last term to the right of the equality vanishes because $v \cdot n = 0$ in $\Gamma_{2,4}$. Regarding the first term to the right of the equality, it is known that:

$$\nabla \cdot \nabla u_\varepsilon = \Delta u_\varepsilon \quad (30. a)$$

$$\nabla \cdot \nabla u_\varepsilon^T = \nabla (\nabla \cdot u_\varepsilon) = 0 \quad (30. b)$$

Then:

$$\int_{\Omega_\varepsilon} \mathbb{D}(u_\varepsilon) \cdot \nabla v \, dV = - \int_{\Omega_\varepsilon} \Delta u_\varepsilon \cdot v \, dV = - \int_{\Omega_\varepsilon} \Delta u_\varepsilon \cdot \phi w_\varepsilon^k \, dV \quad (31)$$

Then, the variational formulation is rewritten such that:

$$- \int_{\Omega_\varepsilon} p_\varepsilon \nabla \cdot (\phi w_\varepsilon^k) \, dV - \mu \int_{\Omega_\varepsilon} \Delta u_\varepsilon \cdot \phi w_\varepsilon^k \, dV = \int_{\Omega_\varepsilon} f \cdot \phi w_\varepsilon^k \, dV \quad (32)$$

Where Stokes is recovered for every test function $v = \phi w_\varepsilon^k$. Once you have the Stokes system in hand, for such test functions, ones arrive at the same homogenized equation:

$$\nabla p - \Delta u + Mu = f, \mu = 1 \quad (33)$$

This time, developed on the boundary and with the same matrix M . This result is the core of the question about the compatibility of variationals on the boundaries and in the domain, for non-Dirichlet boundary conditions.

The results obtained refer to the open Ω problem composed of Ω_ε and T_ε^i . According to the procedure established by Allaire (1991a), the cell problem must be a microscale representation of the global problem, that is, the Stokes problem must be solved in a domain whose boundary conditions reflect (or simulate) the conditions in the macroscopic problem. For a bounded open domain, how should the cell problem be formulated? This issue will be addressed in the next section and, with it, results on the limits of M in relation to grain size.

2.1. The cell problem

Paraphrasing Allaire (1991a), “the functions $w_\varepsilon^k \in H^1(\Omega_\varepsilon)$ and $q_\varepsilon^k \in L^2(\Omega_\varepsilon)$ seem mysterious at first glance. But these have physical meaning.” In fact, these need to make physical sense and this condition will be dealt with in this session. Unlike a generic variational problem, the test function in an energy homogenization problem is a specific function, carefully chosen to satisfy both mathematical (regularity) and physical (coherence with the model) criteria.

Despite the numerical nature of this work, the analysis of the convergence of the Stokes problem for one of its three possible limits – Stokes, Brinkman or Darcy – must be done from the analytical solution. This subsection will then be dedicated to presenting the hypotheses that will allow the resolution of the Stokes system in the unit cell, inspired by the hypotheses presented by Allaire (1991a) and Cionarescu & Murat (1982). Let the following differential problem be:

Find $(w_\varepsilon^k, q_\varepsilon^k)$ solutions of the system:

$$\nabla q_\varepsilon^k - \Delta w_\varepsilon^k = 0, \text{ at } \Omega_\varepsilon^{unit} \quad (\text{Cell - problem. a})$$

$$\nabla \cdot w_\varepsilon^k, \text{ at } \Omega_\varepsilon^{unit} \quad (\text{Cell - problem. b})$$

With the following conditions (Cond.):

$$\nabla q_\varepsilon^k - \Delta w_\varepsilon^k = 0, \text{ at } B_\varepsilon^k \quad (\text{Cond. 1})$$

$$w_\varepsilon^k = 0, \text{ at } T_\varepsilon^k \quad (\text{Cond. 2})$$

$$w_\varepsilon^k = U_\infty, \text{ at } D_\varepsilon^k \quad (\text{Cond. 3})$$

$$w_\varepsilon^k = U_\infty, \text{ at } \partial\Omega_\varepsilon^{unit} \quad (\text{Cond. 4})$$

But here, consider the problem in polar (radial) coordinates and that the pressure field has a previously known “behavior”. A periodic pressure field will be adopted such that:

$$q_\varepsilon^k = \sum_{k=1}^{\infty} q_\varepsilon e^{\frac{ir_k}{\varepsilon}} = f(r) \quad (34)$$

Symmetric (does not depend on φ or θ) and k is the spatial frequency of the solution. Hence, the problem to be solved becomes a Poisson problem on the ring, such that the solution satisfies:

$$\Delta w_\varepsilon^k = f(r), \text{ at } B_\varepsilon^k \quad (\text{Cond. 5})$$

$$w_\varepsilon^k = 0, \text{ at } T_\varepsilon^k \quad (\text{Cond. 6})$$

$$w_\varepsilon^k = U_\infty, \text{ at } D_\varepsilon^k \quad (\text{Cond. 7})$$

$$w_\varepsilon^k = U_\infty, \text{ at } \partial\Omega_\varepsilon^{unit} \quad (\text{Cond. 8})$$

The problem can be further rewritten as:

$$\frac{\partial^2 w_\varepsilon^k}{\partial r^2} + \frac{1}{r} \frac{\partial w_\varepsilon^k}{\partial r} = f(r), \text{ at } B_\varepsilon^k \quad (35. a)$$

Alternatively

$$\frac{\partial^2 w_\varepsilon^k}{\partial r^2} + \frac{1}{r} \frac{\partial w_\varepsilon^k}{\partial r} = \sum_{k=1}^{\infty} q_\varepsilon e^{\frac{ir}{\varepsilon} k}, \text{ at } B_\varepsilon^k \quad (35. b)$$

With same conditions. The right-hand side of the PDE can be rewritten according to the Voss-Weyl formula such that:

$$\frac{1}{r} \frac{\partial}{\partial r} \left(r \frac{\partial w_\varepsilon^k}{\partial r} \right) = \sum_{k=1}^{\infty} q_\varepsilon e^{\frac{ir}{\varepsilon} k}, \text{ at } B_\varepsilon^k \quad (36)$$

The general solution becomes easier to find from this approach, as follows:

$$\begin{aligned} w_\varepsilon^k(r, \theta) &= -\frac{\varepsilon i}{k} E_i \left(\frac{ikr}{\varepsilon} \right) + \frac{\varepsilon^2}{k^2} \\ &E_i \left(\frac{ikr}{\varepsilon} \right) + C_1 \log(r) + C_2 \end{aligned} \quad (37. a)$$

Where $E_i(r)$ is the exponential-integral function that can be expressed by the convergent Ramanujan series (for small r):

$$\begin{aligned} E_i(r) &= \gamma + \ln(r) + e^{\frac{r}{2}} \sum_{n=1}^{\infty} \frac{(-1)^{n-1} r^n}{n! 2^{n-1}} \\ &\sum_{k=0}^{\frac{n-1}{2}} \frac{1}{2k+1} \end{aligned} \quad (37. b)$$

with γ being the Euler-Mascheroni constant.

Note that applying the boundary conditions on $r = a_\varepsilon$ and $R = \varepsilon$, considering that:

$$e^{\frac{r}{2}} \sum_{n=1}^{\infty} \frac{(-1)^{n-1} r^n}{n! 2^{n-1}} + \sum_{k=0}^{\frac{n-1}{2}} \frac{1}{2k+1} \rightarrow \alpha r^p e^{\frac{r}{2}} \quad (38)$$

For large n, k , a result of the form is expected:

$$\begin{aligned} &-\frac{\varepsilon i}{k} \left(\gamma + \ln \left(\frac{ikr}{\varepsilon} \right) + \alpha ikr^p e^{\frac{ikr}{2\varepsilon}} \right) \\ w_\varepsilon^k(r, \theta) &= \sum_{k=1}^{\infty} \frac{\varepsilon^2}{k^2} \left(\gamma + \ln \left(\frac{ikr}{\varepsilon} \right) + \alpha ikr^p e^{\frac{ikr}{2\varepsilon}} \right) \\ &+ C_1 \log(r) + C_2 \end{aligned} \quad (39)$$

Simplifying:

$$\begin{aligned} w_\varepsilon^k(r, \theta) &= \sum_{k=1}^{\infty} \left(-\frac{\varepsilon i}{k} + \frac{\varepsilon^2}{k^2} \right) \\ &\left(\gamma + \ln \left(\frac{ikr}{\varepsilon} \right) + \alpha ikr^p e^{\frac{ikr}{2\varepsilon}} \right) + C_1 \log(r) + C_2 \end{aligned} \quad (40)$$

generates:

$$\begin{aligned} w_\varepsilon^k(r, \theta) &= \sum_{k=1}^{\infty} \left(-\frac{\varepsilon i}{k} + \frac{\varepsilon^2}{k^2} \right) \left(\gamma + \ln \left(\frac{ik\beta a_\varepsilon}{\varepsilon^2} \right) + \right. \\ &\left. \alpha i k \left(\frac{\beta a_\varepsilon}{\varepsilon} \right)^p e^{\frac{ik\beta a_\varepsilon}{2\varepsilon^2}} \right) \\ &+ C_1 \log \left(\frac{\beta a_\varepsilon}{\varepsilon} \right) + C_2 \end{aligned} \quad (41)$$

For $\beta \in \mathbb{R}$. Where it is noted that the bounding of the solution will explicitly depend on the relationship between the radii (grain and unit cell) and that this ratio will be controlled by the product of r by the logarithm of the ratio. This result is very similar to that found by Allaire (1990, 1991a). Here, precisely, one can estimate the convergence for the limits $0, +\infty$ or $0 < C_0 < +\infty$:

$$\varepsilon \left| \ln \left(\frac{a_\varepsilon}{\varepsilon} \right) \right|^{\frac{1}{p}}, p \in \mathbb{R}^+ \quad (42)$$

3. The particle size distribution function and the grain size decay

In geotechnical engineering, it is common to classify soils based on their granulometric distribution, that is, to analyze the composition of the soil according to the size (diameter) of the grains that compose it. Figure 4 is an example of this type of test.

Some parameters can be extracted from this type of analysis, such as the coefficient of uniformity (C_u) and the coefficient of curvature (C_c), calculated as:

$$C_u = \frac{D_{60}}{D_{10}} \quad (43. a)$$

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} \quad (43. b)$$

Where D_{60} , D_{10} and D_{30} indicate that 60%, 10% and 30% of the grains have a diameter smaller than X mm. Both parameters C_u and C_c characterize how uniform a soil is, that is, how well distributed its granulometry is. The ABNT-NBR-6502/22 (ABNT, 2022) proposes that a soil is considered uniform if

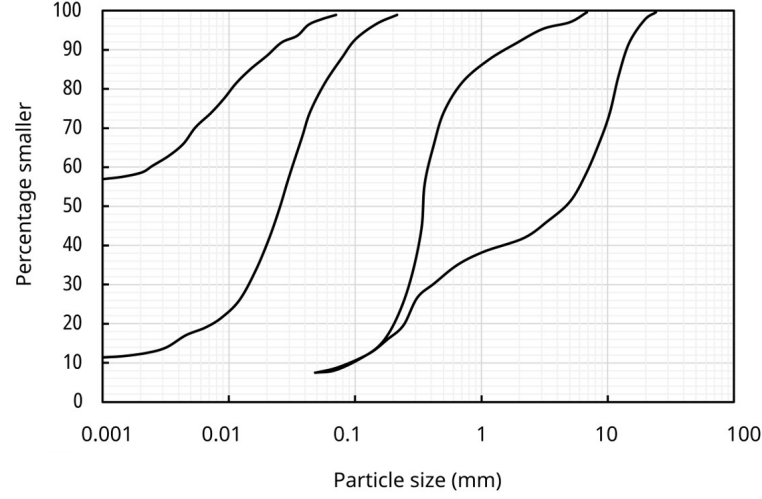


Figure 4. Particle size distribution of soils

$C_u < 5$. ASTM-D2487-17 (ASTM, 2017) proposes that a soil is uniform if $C_c > 3$ or $C_c < 1$. With these definitions known, we propose the following *ansatz* that will be justified later:

Ansatz (grain size decay): the function $a(\varepsilon)$ used in homogenization theory can be represented as:

$$a(\varepsilon) = e^{-\frac{C_u}{\varepsilon^{C_c}}} \quad (44)$$

Where ε must be understood as the size of the representative elementary volume.

Applying Equation 44 directly to the critical grain size Equation 42, using $p = 2$ as suggested by Allaire (1991a), taking $\varepsilon \rightarrow 0$, we have:

$$\sigma^2 \rightarrow 0, C_c < 2 \text{ (Darcy)} \quad (45. a)$$

$$\sigma^2 \rightarrow +\infty, C_c > 2 \text{ (Navier-Stokes)} \quad (45. b)$$

$$\sigma^2 \rightarrow 0 < C_0 < +\infty, C_c = 2 \text{ (Brinkman)} \quad (45. c)$$

It is now up to prove that these limits make some physical sense in this new proposed approach. Before proving, the following geotechnical analogue can be made:

- Situation 1: $C_c < 2$ implies a well-graded soil, making it possible to observe grains for any sample size. Therefore, the voids of the larger grains are occupied by smaller grains, increasing the total specific surface, consequently increasing the friction of the fluid in the solids, and preventing the development of nonlinearities at high speeds. Darcy's law is then formed.
- Situation 2: $C_c > 2$ implies a uniform soil, poorly graded, meaning that from a certain size of ε small enough, no more grains are observed, but free fluid. The pores become larger, and the total specific surface

does not reach high values. The fluid percolates almost freely, similar to flow governed directly by Navier-Stokes. The term "similar" here becomes necessary as it will later be proved that the presence of grains will generate an apparent viscosity for the flow.

- Situation 3: $C_c = 2$ implies a medium uniform soil. It is the critical distribution for it to be neither a free flow nor a diffusion flow as envisaged by Darcy. There is a flow of the Navier-Stokes type, but with the presence of a drag term, multiplying the velocity, as a form of damping generated by the presence of some grains. However, nonlinear terms remain relevant.

It is relevant to comment that, physically, a sample with size $\varepsilon \rightarrow 0$ does not exist. This problem will also appear when the problem is treated numerically, since there is a computational limit that does not allow taking the grain size to zero.

So let Equation 42 be rewritten as:

$$\sigma(\varepsilon) = \varepsilon \left| \ln \left(\frac{a_\varepsilon}{\varepsilon} \right) \right|^{\frac{1}{2}} \quad (46)$$

The different limits of (48) are represented in Figure 5.

4. Numerical convergence of the homogenization problem of Navier Stokes equations in two dimensions

It is proposed in this brief introduction that, for reasons of numerical limitation, Allaire's theory of homogenization (1991a) had to be modified in fundamental aspects, affecting

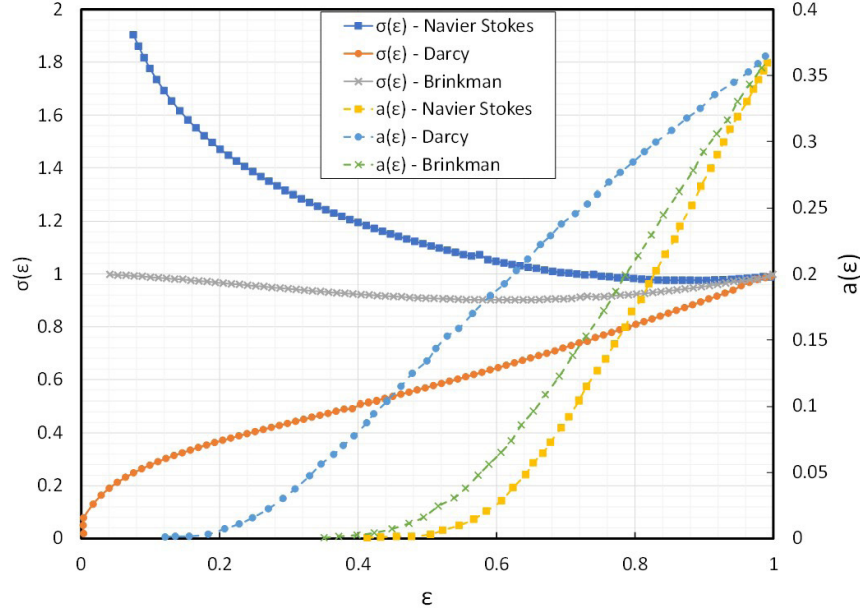


Figure 5. Limits for $a(\varepsilon)$ and $\sigma(\varepsilon)$.

the hypotheses about the test functions that will no longer be able to adopt certain limits.

4.1 The first steps and first struggles

Let the following problem be numerically homogenized by the conventional finite element method:

Find $(u_\varepsilon, p_\varepsilon) \in [H_0^1(\Omega_\varepsilon)]^N \times L^2(\Omega_\varepsilon)$ solutions of the problem (P1-Num)

$$\rho u_\varepsilon (u_\varepsilon \cdot \nabla) + \nabla p_\varepsilon - \Delta u_\varepsilon = f, \text{ at } \Omega_\varepsilon \quad (P1 - \text{Num. } 1)$$

$$\nabla \cdot u_\varepsilon = 0, \text{ at } \Omega_\varepsilon \quad (P1 - \text{Num. } 2)$$

$$p_\varepsilon = p_{in}, \text{ at } \Gamma_1 \quad (P1 - \text{Num. } 3)$$

$$p_\varepsilon = p_{out}, \text{ at } \Gamma_3 \quad (P1 - \text{Num. } 4)$$

$$u_\varepsilon = 0, \text{ at } \Gamma_2 \text{ e } \Gamma_4 \quad (P1 - \text{Num. } 5)$$

So that when $\varepsilon \rightarrow 0$, $(u_\varepsilon, p_\varepsilon) \rightharpoonup (u, p)$ weakly, in some sense. In the theory of analysis in abstract spaces, it was possible to set the size of the grain, or the hole, to zero. However, when dealing with the numerical problem, the grain size cannot be taken to zero.

It is known that a fluid loses energy when percolating through a porous medium by friction with the solid walls. Therefore, it is known that the greater the specific surface of the grain, the greater the friction with the fluid and the greater the dissipation of energy during percolation. A greater loss of energy, induces in geotechnical theory, a lower permeability

coefficient, if Darcy's law is assumed as true. The numerical consequence is relevant: since it is not possible to disappear with the grains in the limit $\varepsilon \rightarrow 0$, each simulation with smaller grains increases considerably the specific surface no matter how much the porosity is maintained.

In Allaire's theory (1991a), a cell problem with a single obstacle is proposed, in order to represent in a unitary cell, the same porosity of the total domain, since this is just an infinite and periodic repetition of these cells. In the numerical treatment carried out here, it was necessary to find a way to introduce the specific surface of the grains and the porosity in the unit cell. How to do?

4.2 The multigrain cell problem

With the mathematical impasse raised in the previous paragraph, it became necessary to propose a new way of representing the unit cell, contemplating the porosity and the specific surface of the grains. The proposal is to use more than one grain in the unit cell periodically and symmetrically distributed. To do this, simply solve the following nonlinear system of two algebraic equations:

$$1 - n\pi r^2 = \phi \quad (47. a)$$

$$\frac{2n}{r} = M_s^{unit} \quad (47. b)$$

Where (49.a) is the porosity for a square domain of unitary sides and (47.b) represents the unit specific surface of the grain. This second equation needs a more detailed explanation.

Suppose you want to generate a unit cell with only one grain that meets the requirements of porosity and specific surface. Also suppose that this problem has a solution for a certain radius r of the grain. Therefore, the specific surface of this grain is calculated, in two dimensions, as:

$$M_s^{unit} = \frac{A_s}{V_s} = \frac{2\pi r}{\pi r^2} = \frac{2}{r} \quad (48)$$

But it was said earlier that it is unlikely, unless a coincidence, that a radius r fulfills the requirements of porosity and specific surface. Therefore, it becomes natural to question whether it is possible to divide this grain into grains of smaller radius so that a greater number of grains fulfills the two aforementioned requirements.

A cell with more than one grain is generated, as represented by Figure 6.

In the unit cell, the following problem is solved:

Find (w, q) solutions of the problem (P1-Num)

$$\rho w(w \cdot \nabla) + \nabla q - \Delta w = f, \text{ at } \Omega_{unit} \quad (P_{Cel} - Num. 1)$$

$$\nabla \cdot w = 0, \text{ at } \Omega_{unit} \quad (P_{Cel} - Num. 2)$$

$$w = w_0, \text{ at } \Gamma_1 \quad (P_{Cel} - Num. 3)$$

$$w = w_0, \text{ at } \Gamma_3 \quad (P_{Cel} - Num. 4)$$

$$n^T \cdot (2\mu \mathbb{D}(w) - qI) \cdot t - \beta w \cdot t = 0, \text{ at } \Gamma_2 \cup \Gamma_4 \quad (P_{Cel} - Num. 5)$$

It is notable that (w, q) are analogous to the velocity and pressure fields mentioned on the global scale Ω_ε . The domain

in which one wants to solve the problem $(P_{Cel} - Num)$ is graphically represented in Figure 7.

4.3 Homogenization via direct numerical simulation (DNS)

The objective of this subsection is to investigate the convergence, in whatever sense, of a homogenization process by direct numerical simulation, that is, gradually decreasing the size of the grains in relation to the domain, increasing their quantity to maintain the porosity and respecting a specific grain size decay function.

Let the problem (P1-Num) be the one we want to solve using the conventional finite element method. The momentum conservation equations can be written in a variational form such that:

$$\begin{aligned} \int_{\Omega_\varepsilon} \rho u_\varepsilon (u_\varepsilon \cdot \nabla) \cdot \phi \, d\Omega_\varepsilon + \int_{\Omega_\varepsilon} \nabla p_\varepsilon \cdot \phi \, d\Omega_\varepsilon - \\ \int_{\Omega_\varepsilon} \mu \Delta u_\varepsilon \cdot \phi \, d\Omega_\varepsilon = \int_{\Omega_\varepsilon} f \cdot \phi \, d\Omega_\varepsilon \end{aligned} \quad (49. a)$$

where, integrating by parts:

$$\begin{aligned} \int_{\Omega_\varepsilon} \rho u_\varepsilon (u_\varepsilon \cdot \nabla) \cdot \phi \, d\Omega_\varepsilon - \int_{\Omega_\varepsilon} \nabla p_\varepsilon \cdot \phi \, d\Omega_\varepsilon \\ + \int_{\Omega_\varepsilon} \mu \nabla u_\varepsilon \cdot \nabla \phi \, d\Omega_\varepsilon \\ + \int_{\partial\Omega_\varepsilon} p_\varepsilon \phi \cdot n \, dS + \int_{\partial\Omega_\varepsilon} \mu \nabla u_\varepsilon \cdot \phi \cdot n \, dS = \int_{\Omega_\varepsilon} f \cdot \phi \, d\Omega_\varepsilon \end{aligned} \quad (49. b)$$

One then has $u_\varepsilon \in H^1(\Omega_\varepsilon)$ and $p_\varepsilon \in L^2(\Omega_\varepsilon)$, at least. Let $V \subset H^1(\Omega_\varepsilon)$ be a linear subspace. Having V finite dimension,

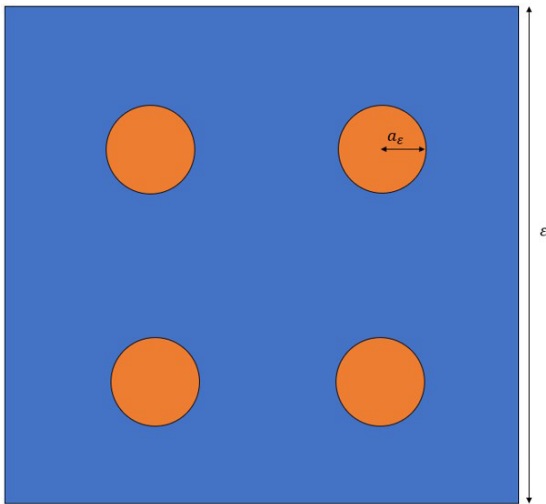


Figure 6. Representation of the unit cell with more than one grain.

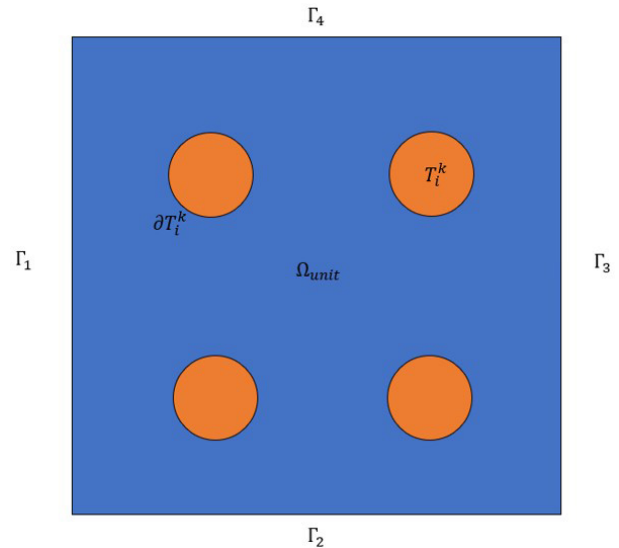


Figure 7. $P_{Cel} - Num$ domain.

taking advantage of the properties of orthogonal projection in Hilbert spaces, an element of V is proposed as:

$$u'_\varepsilon = \sum_{n=0}^N U_n \Phi_n, \quad N < +\infty \quad (50)$$

With $U_n \in \mathbb{R}^N$ and $\Phi_n \in V \subset H^1$. Similarly, there is an element of V :

$$p'_\varepsilon = \sum_{n=0}^M P_n \Phi_n, \quad M < N \quad (51)$$

With $P_n \in \mathbb{R}^M$. By the orthogonality of V in relation to $H^1(\Omega_\varepsilon)$, it is known that:

$$u'_\varepsilon - u_{\varepsilon 2} + p'_\varepsilon - p_\varepsilon \leq z - u_{\varepsilon 2} + w - p_\varepsilon \quad (52)$$

$$\forall z, w \in V$$

That is, u'_ε and p'_ε minimize the solution approximation error.

The continuity equation can be formulated in its variational form as:

$$\int_{\Omega_\varepsilon} (\nabla \cdot u_\varepsilon) \Psi \, d\Omega_\varepsilon = 0 \quad (53)$$

Where $\Psi \in E$, being $E \subset L^2(\Omega_\varepsilon)$.

For the direct numerical simulation, a first-order polynomial function (Lagrange polynomial) was used for Φ_n and a constant, merely continuous function for Ψ_n . In terms of finite element theory, triangular elements of type T2 were chosen, that is, triangular elements with three nodes per side through which a first-order polynomial for the velocity and a zeroth order polynomial for pressure pass.

The formulation assumed that the characteristic size of the elements was 1% of the domain size, yielding about 10^4 elements in a square domain of unit sides. Close to the holes, the mesh was refined to 1% of the hole perimeter (Figure 8). The non-linearity of the convective term was treated with the Newton-Raphson iterative method. In other words, the residual of each component of the variational formulation was iterated with changes in the nodal value until a minimum was reached. The element used in the approach is shown in Figure 9.

4.4 Error estimation and the influence of convective nonlinearity: a compact perturbation

To estimate the error due to approximation of the solution, a theorem is needed (Brenner & Scott, 2008):

Theorem (error estimate): Suppose the relation in (8.6) is true. Let $u \in H^k(\Omega)$ and also $u \in H^s(\Omega)$. Also let $m < s < k$. Then $\|u - u_h\|_{H^m(\Omega)} \leq Ch^{s-m} \|u\|_{H^s(\Omega)}$:

$$u - u_h \in H^m(\Omega) \leq Ch^{s-m} \|u\|_{H^s(\Omega)} \quad (\text{Theorem})$$

Where $u_h \in M \subset H^m(\Omega)$.

The proof of this theorem can be found in (Brenner & Scott, 2008). The theorem also applies to Sobolev spaces and allows one to obtain:

$$u - u_h \in W_p^1(\Omega) = o(h^{s-1}) \quad (54)$$

Applying it to the problem to be solved in this work, we obtain:

$$u_\varepsilon - u'_\varepsilon \in H^1(\Omega_\varepsilon) = o(h^{2-1}) \quad (55. a)$$

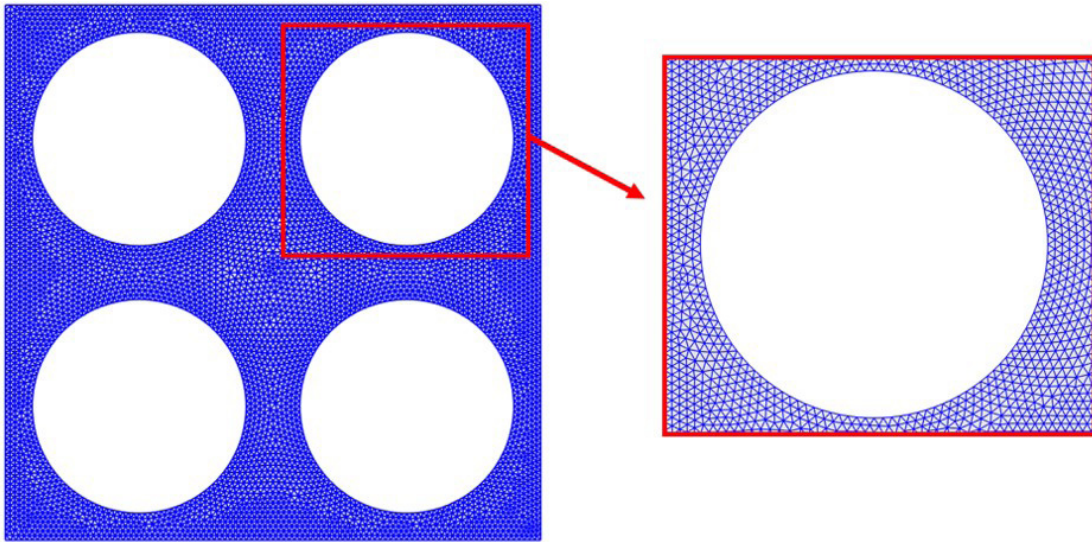


Figure 8. Discretization of the finite element space.

Since $u_\varepsilon \in H^2(\Omega_\varepsilon)$. Assuming a square domain of unit sides and a mesh with a characteristic size of 1% of the domain size:

$$u_\varepsilon - u'_\varepsilon \in H^1(\Omega_\varepsilon) = o(0, 01^1) \quad (55. b)$$

That is, the calculated velocities have an error bounded above by 1% of the actual value.

The convective term is classified as a compact perturbation. In other words, the operator:

$$Tu'_\varepsilon = \int_{\Omega_\varepsilon} u'_\varepsilon (u'_\varepsilon \cdot \nabla) \cdot \Phi \, d\Omega_\varepsilon \quad (56)$$

It is said to be compact because it belongs to the space of linear operators of $V \subset H_1(\Omega_\varepsilon)$. The space V is compact and therefore every linear operator in its dual is also compact.

4.5 Simulation script

First, a square domain of unit dimensions of 1m x 1m is constructed. Next, the grain size decay function is chosen. Adopt, for example, the equation given by *ansatz* (44), such that (42) tends to zero for $\varepsilon \rightarrow 0$, with $p = 2$. Take an initial value of $\varepsilon < 1$ (less than the whole domain) and calculate the grain size. Then, choose a porosity ϕ and using relation (47.a), determine how many grains will be needed to fulfill the desired porosity.

A mesh is generated with a characteristic size of 1% of the domain size, as well as the hole perimeter is discretized with 1% of its size to refine the mesh in the vicinity of the hole. The parameters $\rho = 1000 \text{ kg/m}^3$ and $\mu = 0,001 \text{ Pa}\cdot\text{s}$ are determined. Using Figure 1 as a reference, the following boundary conditions are imposed:

$$u_\varepsilon = 0, \text{ at } \Gamma_2 \text{ and } \Gamma_4 \quad (BC 6)$$

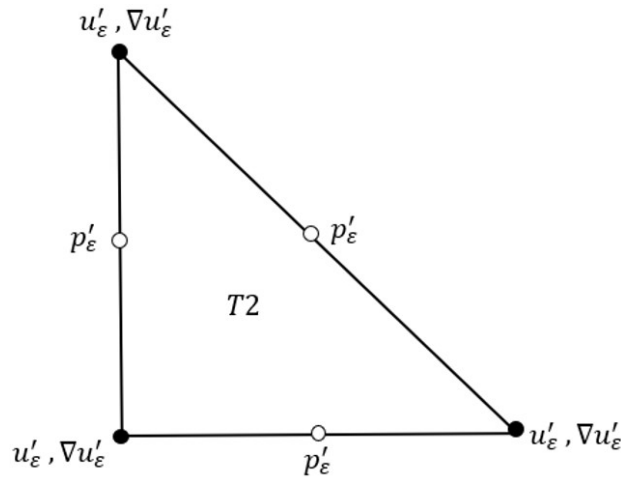


Figure 9. T2-type element.

$$p_\varepsilon = 0 \text{ Pa}, \text{ at } \Gamma_3 \quad (BC 7)$$

$$p_\varepsilon = p_{0\varepsilon}, \text{ at } \Gamma_1 \quad (BC 8)$$

Where the value of $p_{0\varepsilon}$ is gradually increased to investigate the evolution of the velocity field with the variation of the hydraulic gradient. As the speed varies pointwise, it was determined that the average velocity in the domain would be used as a reference, that is:

$$\overline{u_\varepsilon} = \frac{1}{|\Omega_\varepsilon|} \int_{\Omega_\varepsilon} u_\varepsilon \, d\Omega_\varepsilon \quad (57)$$

Assuming a flow fundamentally horizontal (gravitational forces are neglected), the hydraulic gradient is written in simplified form as:

$$i = \nabla p_\varepsilon \cong \frac{1}{L} p_\varepsilon \cong \frac{p_{0\varepsilon}}{1} = p_{0\varepsilon} \quad (58)$$

So, one is looking for something like:

$$\overline{u_\varepsilon} = Ki \quad (59)$$

The simulation is performed, (57) and (58) are calculated, then the value of K is determined from (59). The value of $p_{0\varepsilon}$ is increased and the process is repeated until a curve of the relationship between the average velocity and the hydraulic gradient is constructed. The results are presented in the figures Figure 10 and Figure 11. Note that the behavior of the curve corresponding to a porosity of 0.7 differs from the rest. It is believed to be the beginning of an oscillatory behavior that could not be captured by the numerical scheme used. Note also that in Figure 10, only one of the tests starts with ε close to 1. This is due to the required computational cost for such situation. For such porosity values (0.3), the amount

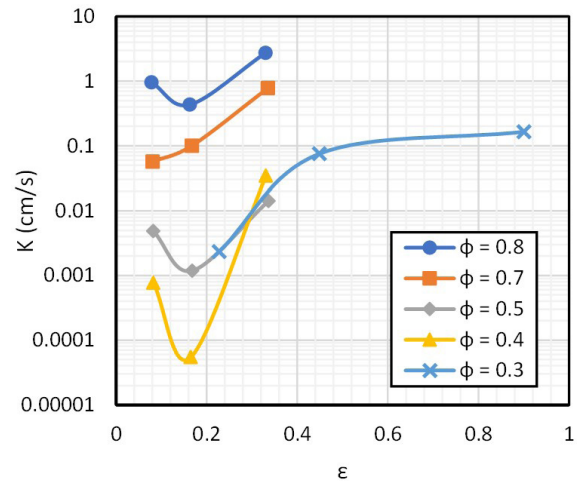


Figure 10. DNS convergence for Darcy.

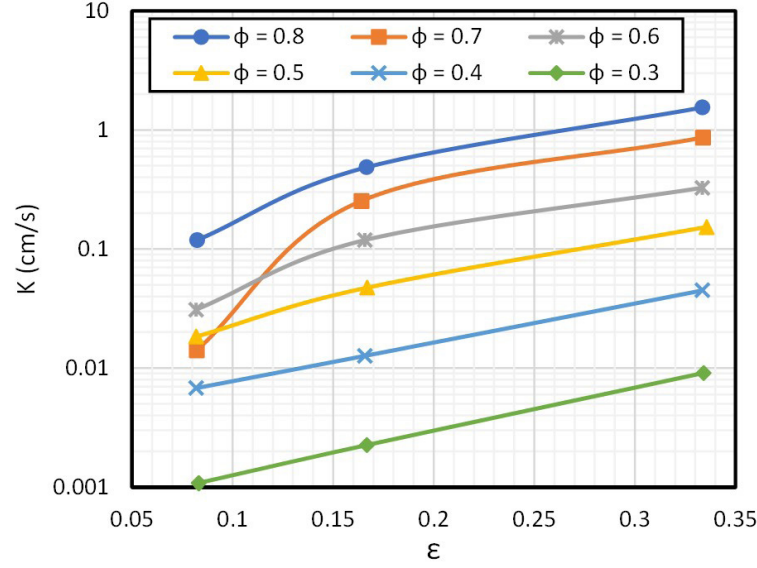


Figure 11. DNS convergence for Navier-Stokes.

of elements needed for a low enough epsilon exceeds the limits of the machine used for the simulations. Therefore, it was preferred to evaluate the convergence trend for higher epsilon values.

4.6 Homogenization using the multigrain cell problem: the Darcy case

Assume a grain size decay function $a(\varepsilon)$ that makes $\sigma(\varepsilon) \rightarrow 0$ as $\varepsilon \rightarrow 0$. For the sake of simplicity, assume the case $\phi = 0.6$ and $\sigma(\varepsilon) \rightarrow 0$.

First, the number of grains and their radius must be calculated so that the multigrain unit cell is created. Assuming that the direct numerical simulations were able to go up to $\varepsilon = 0.0833m$ and that the function that defines

the grain size is $a(\varepsilon) = 0.2\sqrt{\varepsilon}$ and $\sigma(\varepsilon) = \varepsilon \left| \ln \left(\frac{a(\varepsilon)}{\varepsilon} \right) \right|^{\frac{1}{2}}$, one has $a(\varepsilon) = a(0.0833) = 0.0577m$.

Knowing that $\phi = 0.6$, then the nonlinear system to be solved becomes:

$$1 - n\pi r^2 = 0.6 \quad (60. a)$$

$$\frac{2n}{r} = \frac{2}{0.0577} \quad (60. b)$$

Resulting in $n = 3.369$ grains and $r = 0.1944m$. As it is not possible to obtain a non-integer number of grains, the value of the nearest integer is assumed, therefore, $n = 3$. A unitary cell is then built with three grains of radius $0.1944m$, where the problem specified by the equations of $(P_{Cel} - Num)$ and

its due boundary conditions are solved. Once the velocity field solution is in hand, M_0 is calculated, given by:

$$M_0 = \int_{\Omega_{unit}} \nabla w : \nabla w \, d\Omega_{unit} \quad (61)$$

Where the tensor product of the integrand, in two dimensions, is written in extended form as:

$$M_0 = \int_{\Omega_{unit}} (\partial_x u)^2 + (\partial_y u)^2 + (\partial_x v)^2 + (\partial_y v)^2 \, d\Omega_{unit} \quad (62)$$

Where $M_0 \in \mathbb{R}$. Remember here that the tensor product $A:B$ for $\dim(A) = m$ and $\dim(B) = n$ has dimension $\dim(A : B) = m + n - 4$.

The value found was $M_0 = 46.40$. It is known that:

$$K = \left(\frac{M_0}{2} \right)^{-1} \quad (63)$$

Therefore, $\mu M_0 = 0.001 \times 46.40 = 0.04640$ is already known. It is then enough to find the value of $\sigma(\varepsilon)$ taken directly from the curve:

$$\sigma(\varepsilon) = \sigma(0.0833) = 0.0833 \left| \ln \left(\frac{a(0.0833)}{0.0833} \right) \right|^{\frac{1}{2}} = 0.05045 \quad (64)$$

thus

$$\sigma^2 = 0.05045^2 = 0.00254 \quad (65)$$

Then:

$$K = \left(\frac{0.04640}{0.00254} \right)^{-1} = 0.0549 \quad (66)$$

The mean value found by the direct numerical simulation was 0.0604. Figure 12 shows the convergence of the DNS to the predicted limit.

5. Homogenization for different Darcy limits: Navier-Stokes apparent viscosity and the Brinkman problem

It has been mentioned that the limits found by Allaire (1991a) cannot be obtained numerically due to limitations of the machines used. For equivalent limits to be reached, modifications in the theory will be proposed here. These modifications act at the core of Allaire's deduction, making terms that once disappeared in his work, gain non-negligible importance in the solution of the partial differential equations presented here.

Let the coefficient $M = \mu M_0$ where M_0 is:

$$M_0 = \int_{\Omega_{unit}} \nabla w : \nabla w \, d\Omega_{unit} \quad (67)$$

Where $w \in H^1(\Omega_{unit})$ and it is known that $v = w\phi \in H^1(\Omega_\varepsilon) \forall \phi \in D(\Omega)$. By the Riesz representation theory, $\exists! z \in H^1(\Omega_\varepsilon)$ such that:

$$\int_{\Omega_\varepsilon} \beta z \cdot u_\varepsilon \, d\Omega_\varepsilon = \Delta u_\varepsilon \quad (68)$$

For $\beta \in \mathbb{R}$. The above result is also verified for the extension in Ω by the linearity of Hilbert spaces. Realize now that:

$$\frac{\mu}{\sigma^2} \int_{\Omega_{unit}} \nabla w : \nabla w \, d\Omega_{unit} = z \in H^1(\Omega_{unit}) \quad (69)$$

Ω_{unit} being a homeomorphism of Ω_ε , and the PDE in the unit cell being identical to the PDE in the perforated domain, except for the viscosity and density constants which are obviously limited. With that, it is deduced that:

$$\frac{\mu M_0}{\sigma^2} u = \beta \Delta u \quad (70)$$

Making the general homogenized PDE (PDE):

$$\rho u \cdot (u \cdot \nabla) + \nabla p - (\mu + \beta) \Delta u = f \quad (71)$$

Note that for the limit in Navier Stokes found by Allaire (1991a), $\beta \rightarrow 0$ for $\sigma^2 \rightarrow +\infty$. Equation (71) can be interpreted as a Navier-Stokes system with an apparent viscosity, which is greater than the actual viscosity of the fluid:

$$\rho u \cdot (u \cdot \nabla) + \nabla p - \mu^* \Delta u = f \quad (72)$$

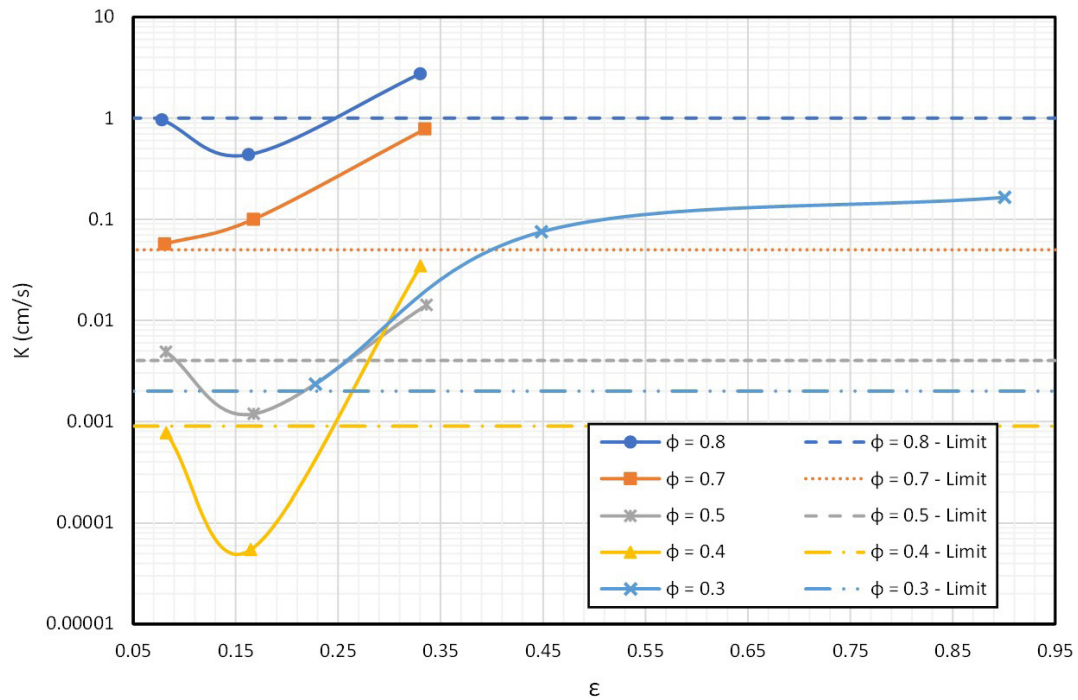


Figure 12. DNS convergence for Darcy with calculated limit.

This apparent viscosity only exists in the numerical problem because the grains do not reach the limit of zero and therefore M_s starts to have a drag and friction effect large enough in the solution to be neglected.

To exemplify the procedure and make it less abstract, consider the case of a soil with porosity $\phi = 0.5$ and $a(\varepsilon) = e^{\frac{0.8}{\varepsilon^{2.1}}}$, that is, with a limit at Navier-Stokes. The direct numerical simulation managed to reach the value of $\varepsilon = 0.5$. Thus, the value of $a(\varepsilon) = a(0.5) = 0.033$.

The unit cell must be constructed for $\phi = 0.5$ and $M_s^{unit} = \frac{2}{0.033} = 60.60$. The solution of the system of non-linear algebraic equations to define the unit cell configuration resulted in $n = 5.267$ and $r = 0.174$. The value of M_0 calculated in the unit cell was 11931.06. Furthermore, $\sigma = 0.99$ was found, so:

$$\mu^* = \frac{\mu M_0}{\sigma^2} = \frac{0,001 \times 11931.06}{0.99^2} = 11.93 \quad (73)$$

Using 11.93 as apparent viscosity and calculating the average velocity within the domain, with a hydraulic gradient ranging from $1 \times 10^{-15} \text{ Pa/m}^2$ to $1 \times 10^{-2} \text{ Pa/m}^2$, a value of average permeability coefficient of 0.007892. Direct numerical simulations (DNS) showed a permeability coefficient tending

to a limit at 0.01. Convergence is graphically represented in Figure 13.

5.1 Brinkman's limit: a gray area between Darcy and Navier Stokes

So far only numerical convergence for Darcy and Navier-Stokes has been presented. These two limits are the two extremes to which the Navier Stokes equations can converge when treated by the homogenization process by the well-known energy method. However, as initially raised by Allaire (1991a) and studied more closely by Cionarescu & Murat (1982), there is a limit for Brinkman when $\sigma \rightarrow 0 < C_0 < +\infty$. How then to identify this limit in the numerical approach proposed here?

Sticking directly to Allaire's theory (1991a), the only way to achieve convergence for Brinkman would be with a function of the kind:

$$a(\varepsilon) = e^{\frac{\alpha}{\varepsilon^2}} \quad (74)$$

Where α can be any positive real, but the exponent of ε must necessarily be 2. This is due to the fact that $\sigma(\varepsilon)$ is written in the form of a natural logarithm raised to the 1/2 power. When trying to bring this concept to a more applied perspective,

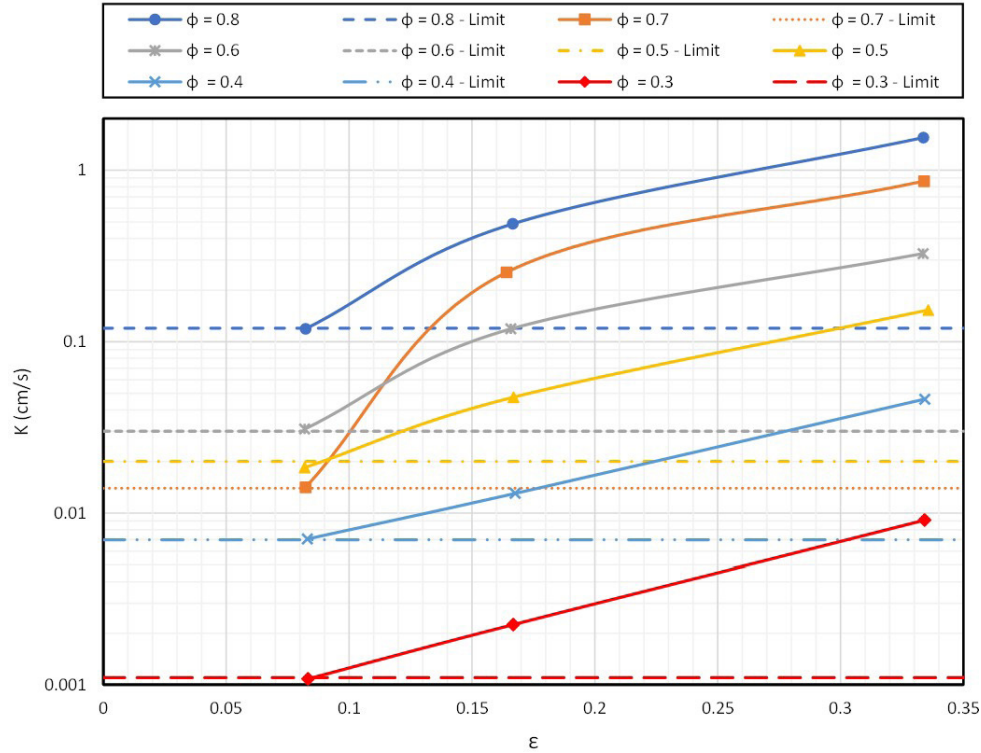


Figure 13. DNS convergence for Navier-Stokes with calculated limit.

only soils whose grain size variation varied exactly with expression (74) would be treated with Brinkman's formulation.

The hypothesis that arises is that for some soils, depending on the size of the representative volume, a function $a(\varepsilon)$ equal to (74) or with exponents close to 2, would generate approximately the same value of σ^2 . This would suggest that soils with theoretical limits in Navier Stokes could be represented by Brinkman. What is suggested is that the Brinkman limit, in the numerical approach, is just a transitional regime between Navier Stokes and Darcy, acting as an overlap between these two limits.

6. Comparison with laboratory results

It should not be forgotten that this work is a paper in geotechnical engineering. Theoretical results are good, but results that match reality are even better. In this sense, this section will be devoted to the construction of a simple mathematical model for predicting hydraulic conductivity, based on the results of Allaire's theory of homogenization, using parameters known from geotechnical engineering.

6.1 A hydraulic conductivity model based on D_{50} , C_u and C_c

Let the definitions of C_c , C_u and $\sigma(\varepsilon)$ be given in (45.a), (45.b) and (46). The model that is proposed from here on will depend only on these parameters and on a characteristic size of the soil particles, represented by the D_{50} .

It is intended to predict the hydraulic conductivity of a homogeneous and isotropic soil from the following sequence of steps:

- From the granulometric curve of the soil in question, identify the values of D_{10} , D_{30} , D_{50} and D_{60} so that the values of C_u and C_c can be calculated.
- The typical grain size decay curve as a function of the size of the elemental representative element will be given by (46), that is:

$$a(\varepsilon) = e^{-\frac{C_u}{\varepsilon^{C_c}}} \quad (75)$$

- The value of ε that represents the chosen sample will be arbitrated as six times the value of D_{50} , that is, $\varepsilon_{ref} = 6D_{50}$. This approach is not ad-hoc. The work of Hattamleh et. al (2009) points out that this relationship is feasible for granular soils.
- The value of $\sigma(\varepsilon)$ will be adjusted by a function A that depends only on the ratio of C_u / C_c . Otherwise, we have:

$$\sigma(\varepsilon)^* = A\left(\frac{C_u}{C_c}\right)\sigma(\varepsilon) \quad (76)$$

- Knowing the porosity of the soil and calculating the unitary specific surface as:

$$M_s^{unit} = \frac{2\pi r}{\pi r^2} = \frac{2}{r} = \frac{4}{D_{50}} \quad (77)$$

- The nonlinear system of equations proposed in (60.a) and (60.b) is solved to find the radius and the number of grains used in the multigrain unit cell.
- In the multigrain unit cell, the problem ($P_{Cel} - Num$) is solved with $w_0 = 1$ and the value of M_0 given by (67) is calculated.
- The value of the average hydraulic conductivity of the soil, directly in cm/s, is given by:

$$K = \frac{\sigma(\varepsilon)^*}{\mu M_0} \quad (78)$$

Being μ the viscosity of the fluid that saturates the soil, in Pa.s. Remember here that the relationship between permeability and hydraulic conductivity is given by $K = k\rho g / \mu$ where K , k , μ , ρ and g are respectively the hydraulic conductivity, intrinsic permeability, fluid viscosity, fluid density and the gravitational constant.

Note that this procedure provides an average hydraulic conductivity value, i.e. independent of the limiting equation. If one wants to know the point of non-linearity, one must simulate the problem (PDE) assuming $\mu^* = \frac{\mu M_0}{\sigma^{*2}}$ as increased viscosity for Navier Stokes or $M = \frac{\mu M_0}{\sigma^{*2}}$ for the drag (damping) term in the Brinkman limit.

The equation $A\left(\frac{C_u}{C_c}\right)$ was adjusted for 36 different permeability tests (as shown in Table 1), with soils of different porosities and granulometric curves. The adjustment was made by checking for which value of A , the value of K corresponded to that found in the laboratory. Then, graphs of A were drawn in relation to the specific surface of the grains, porosity and average grain size. None of these relationships offered a fit result with R^2 close enough to 1 to establish a correlation. However, when comparing A with C_u / C_c , a power law curve with R^2 above 0.46 was obtained, even with tests whose values were drastically different from those expected for the material tested. The function found for A was:

$$A\left(\frac{C_u}{C_c}\right) = 0.0428\left(\frac{C_u}{C_c}\right)^{-1.754} \quad (79)$$

The relationship between parameter A and the value of the ratio between C_u and C_c is graphically represented in Figure 14. Note that the value of A tends to a vertical asymptote for $C_u/C_c \rightarrow 3/2$. For the C_u/C_c range between 2 and 4 with

Table 1. Laboratory tests used for calibration of the model.

Reference	e	ϕ	D_{50} (mm)	C_c	C_u	A	$K\text{-Lab}$ (cm/s)
Yang et al. (2016)	0.525	0.34426	0.5	0.75	3	0.0055	1.84e-3
	0.525	0.34426	1	1.03	5.6	0.002	3.83e-3
	0.525	0.34426	2	1.45	11	0.0015	3.68e-3
	0.559	0.35856	2	0.75	5.6	0.0017	1.23e-3
	0.559	0.35856	0.5	1.03	11	0.001	3.35e-3
	0.559	0.35856	1	1.45	3	0.0011	8.99e-3
	0.572	0.36387	1	0.75	11	0.002	1.36e-3
	0.572	0.36387	2	1.03	3	0.0006	3.18e-2
Akbulut et al. (2014)	0.572	0.36387	0.5	1.45	5.6	0.0002	9.44e-4
	0.664	0.39904	1.27	2.14	7.41	0.001	1.89e-1
	0.796	0.44321	1.63	0.89	1.35	0.075	1.26
	0.892	0.47146	0.84	0.93	1.41	0.05	4.18e-1
	0.641	0.39062	1.27	2.14	7.41	0.007	1.73e-1
	0.788	0.44072	1.63	0.89	1.35	0.073	1.23
	0.843	0.45741	0.84	0.93	1.41	0.05	3.75e-1
	0.495	0.3311	1.27	2.14	7.41	0.001	1.39e-1
	0.645	0.3921	1.63	0.89	1.35	0.07	1.07
	0.707	0.41418	0.84	0.93	1.41	0.05	3.96e-1
	0.966	0.49135	1.27	2.14	7.41	0.001	2.69e-1
	1.059	0.51433	1.63	0.89	1.35	0.09	1.46
	1.153	0.53553	0.84	0.93	1.41	0.07	5.97e-1
Calabar & Akbulut (2016)	0.86	0.46237	3.1	0.93	1.5	0.03	1.61
	0.82	0.45055	1.63	0.89	1.35	0.036	0.35
	0.77	0.43503	0.84	0.93	1.41	0.034	0.13
	0.72	0.4186	0.5	1	1.19	0.02	0.05
	0.6	0.375	0.35	0.98	1.16	0.018	0.03
	0.87	0.46524	1	0.73	4.39	0.011	0.03
	0.82	0.45055	0.69	1.01	3.63	0.006	0.02
	0.79	0.44134	0.5	1.44	3.47	0.011	0.02
	0.7	0.41176	0.39	1.74	3.27	0.0004	0.02
	0.63	0.3865	0.32	1.29	3.09	0.0015	0.01
	0.56	0.35897	0.16	0.92	2.07	0.012	0.01
	0.74	0.42529	2.2	0.98	2.6	0.015	0.26
	0.67	0.4012	1.18	0.88	1.93	0.022	0.12
	0.52	0.34211	2	1.22	3.28	0.0083	0.14
	0.7	0.41176	0.99	0.8	2.41	0.025	0.06
	0.55	0.35484	0.7	0.93	1.71	0.016	0.04

permeability values lower than expected, the tested soils had a fines content that may have affected water transport. Soils with high fines content do not have a predictable hydraulic behavior by the model.

Figure 15 shows two different data sets. The blue dots are the hydraulic conductivity values measured in the laboratory for 36 different tests. These same data were used to feed the model. The orange dots represent the prediction of these permeabilities by the model itself. The power law curve fit for both cases (prediction and measured) highlights the similar behavior between the laboratory data and those predicted by the proposed model. Note also that for high values of C_u/C_c we have the lowest model errors. This means that for soils with a tendency to converge to Darcy, the model is more accurate.

Figure 16 shows the results of direct numerical simulations of the Navier-Stokes equations with varying viscosity, thus simulating the various scenarios of increased viscosity. It is noticed that non-linearities appear quickly for low viscosity values and that, for soils with increased viscosity above 100, the relationship between the hydraulic gradient and the flow velocity is perfectly linear for gradients up to 1000 Pa/m.

It was identified in Figure 17 for which gradient values the non-linearity begins to be observed. Relating these values to their respective C_u/C_c values identifies a power law behavior with $R^2 = 0.9816$. The same relationship occurs between the critical gradient and the hydraulic conductivity of the soil. This relationship is represented in Figure 18.

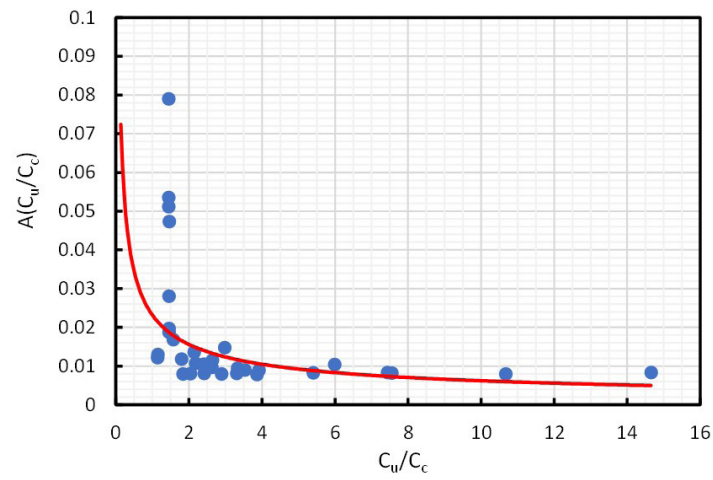


Figure 14. Relation between A and C_u/C_c .

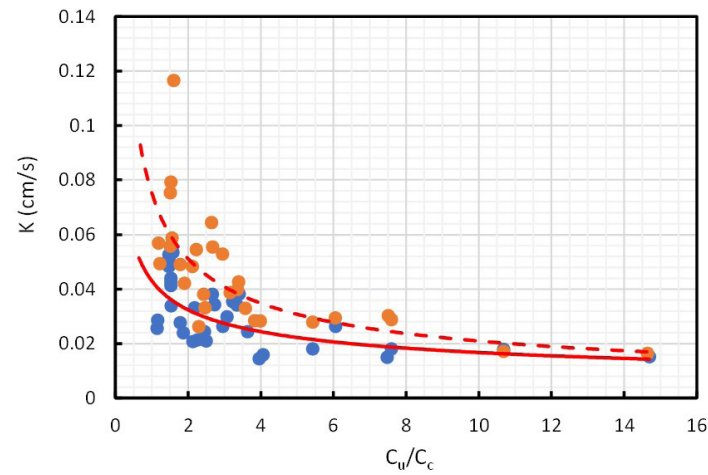


Figure 15. Comparison of laboratory results and model prediction for K .

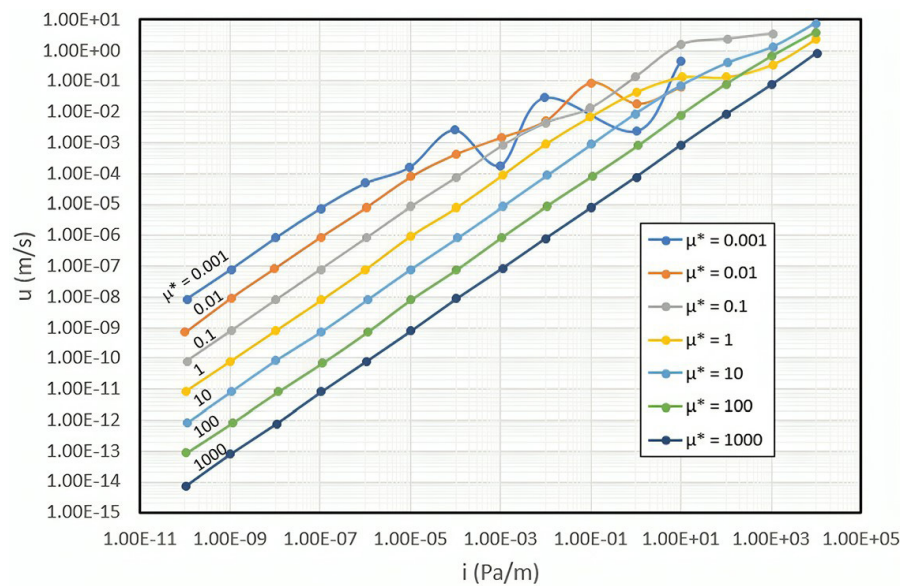


Figure 16. Loss of linearity in function of the hydraulic gradient for different augmented viscosities.

Using the formulation presented in Bear (1985) for the Reynolds number in porous media, it was calculated for which Reynolds values the linearity of Darcy's law ceases to apply. Take:

$$R_e = \frac{uD_{50}}{\frac{\mu}{\rho}} = \frac{uD_{50}}{\nu} \quad (80)$$

Reynolds values $1 \leq R_e \leq 10$ indicate transition from linear to non-linear regime, i.e., Darcy's law would no longer apply. Comparing this limit interval range with those found in the numerical simulations and listed in Tables 2 and 3, it can be seen that these Reynolds values are those calculated for the last limit in Navier-Stokes and the limit in Brinkman. In the numerical experiment, soils that lost the linearity of the

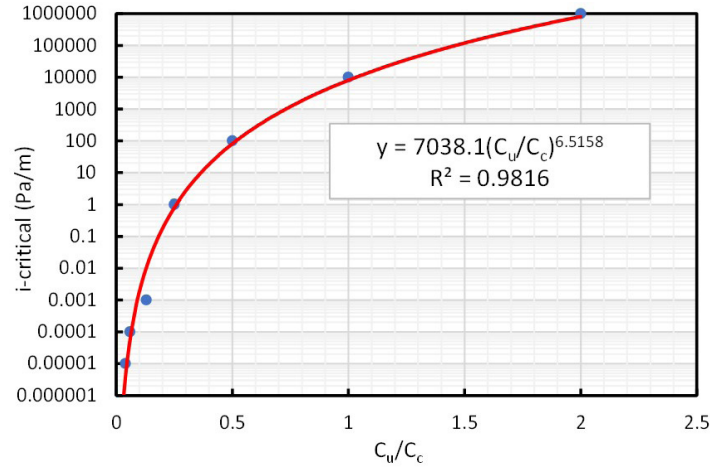


Figure 17. Relation between the ratio C_u/C_c and the critical hydraulic gradient.

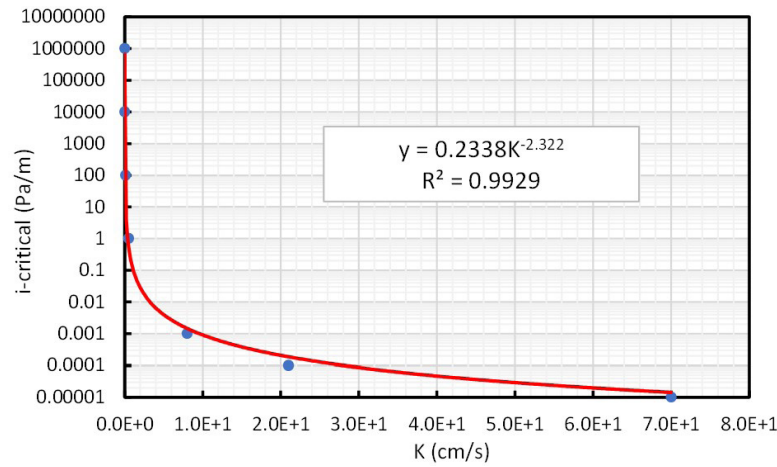


Figure 18. Relation between the estimated hydraulic conductivity and the critical hydraulic gradient.

Table 2. Relation between augmented viscosity and the PDE limit.

u^*	M_0	C_c	$a(\varepsilon)$ (mm)	ε (mm)	A	σ^2	σ^2 - fit	Limit model
0.001	12848.4	27.3	1	6	14.14	912.12	908.6	NSI
0.01	12848.4	14.2	1	6	4.49	289.82	285.95	NSI
0.1	12848.4	7.3	0.99	6	1.39	90.21	91.86	NSI
1	12848.4	3.8	0.99	6	0.44	28.72	28.87	NSI
10	12848.4	1.95	0.97	6	0.14	9.05	9.3	Brinkman
100	12848.4	1	0.84	6	0.04	3.01	3	Darcy
1000	12848.4	0.5	0.66	6	0.012	1	1.01	Darcy
10000	12848.4	0.25	0.53	6	0.003	0.33	0.3415	Darcy

Table 3. Critical Reynolds number in function of the ratio C_u/C_c .

C_c	C_u	C_u/C_c	i -critical	μ^*	K (cm/s)	u -critical (m/s)	R_e	Limit model
27.3	1	0.03663	1.00E-05	0.001	70.71	7.00E-07	7.07E-04	Navier-Stokes
14.2	1	0.07042	1.00E-04	0.01	22.25	2.23E-06	2.23E-03	Navier-Stokes
7.3	1	0.13699	1.00E-03	0.1	7.149	7.15E-06	7.15E-03	Navier-Stokes
3.8	1	0.26316	1	1	2.247	2.25E-03	2.23E+00	Navier-Stokes
1.95	1	0.51282	1.00E+02	10	0.724	7.24E-02	7.24E+01	Brinkman
1	1	1	1.00E+04	100	0.233	2.34E+00	2.34E+03	Darcy
0.5	1	2	1.00E+06	1000	0.078	7.88E+01	7.88E+04	Darcy

relationship between hydraulic gradient and flow velocity had a relationship $0.26 < C_u/C_c < 0.52$.

7. Conclusion

A numerical homogenization process was proposed, based on the finite element method, in its standard formulation (Bubnov-Galerkin). Homogenization in a limited domain had its existence driven by the need to impose conditions on the boundaries of a finite element space. This process consists of the direct numerical simulation (DNS) of a multigrain unit cell, which takes into account two main variables that affect soil permeability: porosity and the specific surface of the grains. From the resolution of a system of algebraic equations, it is determined how many grains must be placed symmetrically in the unit cell and what is the radius of these grains. In the unit cell, a horizontal flow is simulated with specific boundary conditions justified throughout the thesis. With the calculated velocity field, the tensor M is determined.

In Allaire's theory, the M tensor is divided by a factor σ^2 which is a way of measuring the ratio between grain size and domain size. The value of σ depends on the grain decay curve and can have its limit going to $+\infty$, 0 or $0 < C_\theta < +\infty$. In the first case, the PDE would converge to Navier-Stokes, in the second to Darcy and in the third to Brinkman. Since the grains can become very small, but not zero, the specific surface in numerical simulations assumes large, non-negligible values. What was proposed in this work was:

- Modify the measure $\sigma(\varepsilon)$ by a pre-factor $A\left(\frac{C_u}{C_c}\right)$.
- Define a size for the representative elementary volume from the average grain size, more specifically $\varepsilon = 6 \times D_{50}$.
- Calculate $\sigma(\varepsilon)$ and divide the value of M found by the value of σ^2 .

Depending on the value of C_c , define whether the value of M / σ^2 will be used as increased viscosity for Navier-Stokes, Brinkman drag coefficient or the inverse of the hydraulic conductivity in Darcy.

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

Karl Igor Martins Guerra: conceptualization, data curation, visualization, writing – original draft. Celso Romanel: supervision, validation, writing – original draft.

Data availability

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

List of symbols

$a(\varepsilon)$	Grain size decay function
i	Hydraulic gradient
k	Intrinsic permeability
r	Grain radius
w_ε^k	Test function depending on ε
A	Correction function depending on the grain size distribution
C_c	Curvature coefficient
C_u	Uniformity coefficient
$D(\Omega)$	Space of infinite differentiable functions with compact support
H_0^k	Hilbert space of functions k -times differentiable and compact support
H^k	Hilbert space of functions k -times differentiable
K	Hydraulic conductivity
L^p	Lebesgue space with p Hölder coefficient
M_0	Measure of the tortuosity based on the velocity gradient
M_s^{unit}	Unitary specific surface area
R_e	Reynold's number
$\mathcal{W}^{p,q}$	Sobolev space with conjugated p - q Hölder coefficients

μ	Fluid viscosity
μ^*	Augmented viscosity
Δ	Laplacian operator
σ	Measure of the ratio between the size of the grains and the size of the domain
ϕ	Test function or porosity
Ω	Domain
Ω_ε	ε -dependent domain
$\partial\Omega$	Boundary of the domain
$\mathbb{R}^N$	N-dimensional space of the reals
$\ \cdot\ _X$	Norm of a X-space
∇	Nabla operator
ε	Size of the representative element volume.

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Parameters controlling the expansive behavior of bentonite-kaolin mixtures stabilized with alkali-activated waste

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Nilo Cesar Consoli¹ 

Article

Keywords

Sugarcane bagasse ash
Hydrated eggshell lime
Alkaline cement
Expansive soils
Swelling

Abstract

Expansive soils can cause large-scale damage to the infrastructure. Soil stabilization with Portland cement and lime has been widely utilized as a solution to this problem. However, these stabilizers are non-renewable and energy-intensive. Alkali-activated binders are alternatives with lower carbon dioxide emissions. This research evaluated an expansive soil stabilization with an alkali-activated binder produced from sugarcane bagasse ash (SCBA), hydrated eggshell lime (HEL) and sodium hydroxide (NaOH). Free-swelling tests alongside a statistical analysis evaluated the influence of dry unit weight (12.5 and 14.5 kN/m³), binder (4 and 10%) and moisture content (19.7 and 24.7%) and curing time (0 and 7 days) on the stabilized mixtures. A four factors factorial design with duplicates and central points was outlined. To better understand the NaOH and SCBA influence over the soil expansion additional tests were performed. In general, an increase on the studied factors reduced swelling, especially binder content. However, the alkali-activated cement presented no clear correlation between higher density and higher expansion. Swell reduced from 13.8% (12.5 kN/m³ and 19.7% moisture) and 8.8% (12.5 kN/m³ and 24.7% moisture) to 2.5% and 0%, respectively, after 7 days and 10% binder addition for the alkaline cement. For Portland cement, swell reduced from 13.8% (10.2 kN/m³ and 22.5% moisture) and 12.5% (10.2 kN/m³ and 27.5% moisture) to 1.8% and 1%, respectively, after 7 days and 4% binder addition. Samples containing NaOH expanded less than samples molded with only water. Finally, the alternative binder might be a viable option to replace Portland cement for expansion control.

1. Introduction

Expansive soils suffer considerable volume changes upon the variation of their water content (Consoli et al., 2021; Ferreira et al., 2013; Silvani et al., 2023; Tonini de Araújo et al., 2021). The magnitude of volume change depends on several factors, e.g. mineralogy, type of clay, clay content, and exchangeable ions (Yang et al., 2020), and cause large-scale damage to infrastructure (e.g. roads, buildings, foundations) resulting in economic and safety issues (Gaspar et al., 2022; Yaghoubi et al., 2021). Common solutions to these issues are soil replacement and soil stabilization; due to lower costs associated to the latter, soil stabilization is often preferred in engineering practice (Buhler & Cerato, 2007).

The stabilization of expansive soils with cementing agents such as ordinary Portland cement (OPC) (Abdelkrim & Mohamed, 2013; Ahmadi Chenarboni et al., 2021; Mahedi et al., 2018; Sahoo & Prasad Singh, 2022) and lime (Boobalan & Sivakami Devi, 2022; Dang et al., 2016;

Indiramma et al., 2020; James, 2020; James et al., 2022; Khadka et al., 2020) has been widely explored. However, these conventional stabilizers are energy-intensive materials, generate significant amounts of CO₂, and possess high costs associated with their production processes (Burris et al., 2015; Gartner & MacPhee, 2011; Zhang et al., 2020). Thus, alternative materials and/or techniques for expansive soils stabilization are needed.

Alkali-activated materials are alternatives to conventional cementing agents. These binders present lower carbon dioxide emissions compared to lime and cement (Zhang et al., 2020) and incorporate several industrial and agro-industrial wastes, such as: sugarcane bagasse ash (Bruschi et al., 2021b, 2022); carbide lime (Carvalho Queiróz et al., 2022; Pereira dos Santos et al., 2022; Queiróz et al., 2022); ground glass waste (Secco et al., 2021), fly ash (Miranda et al., 2020; Goldoni et al., 2023), eggshell residue (Ferrazzo et al., 2023a, b; Levandoski et al., 2023) and, rice husk ash (Pelissaro et al., 2023). Alkali-activation is the chemical reaction between an

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amorphous/semi-crystalline aluminosilicate source and an alkaline activator, generating gel similar to the ones formed in OPC hydration (Duxson et al., 2007).

Past research has been conducted on the use of alkali-activation for the stabilization of problematic soils (Parhi et al., 2018; Phougat, 2015; Samuel et al., 2020; Syed et al., 2020, 2022; Žurinskas et al., 2020). Nevertheless, a research gap exists regarding the application of alkali-activated waste materials to control expansive soils swelling. Thus, this research analyzed the stabilization of an expansive soil by an alkali-activated binder composed by sugarcane bagasse ash and eggshell lime. Free-swelling tests were performed on soil-alkali activated binder samples and a statistical analysis evaluated the influence of studied factors (e.g. curing period) on swelling results.

2. Materials and methods

2.1 Materials and characterization

The materials used in this research were (i) bentonite, (ii) kaolin, (iii) sugarcane bagasse ash (SCBA), (iv) hydrated eggshell lime (HEL), and (v) sodium hydroxide (NaOH). The sodium bentonite is classified as CH (inorganic clay of high plasticity) and kaolin as CL (inorganic clay of low plasticity) in accordance with USCS (ASTM, 2020). The SCBA was obtained from a local supplier in southern Brazil. HEL was produced from the calcination of eggshells obtained in bakeries. Specifications of the lime production process can be found in Tonini de Araújo et al. (2021). The alkali-activator was NaOH (micro pearls, 98% purity). The materials physical characterization is presented in Table 1.

Bentonite consists of 64.74% SiO₂, 13.25% Al₂O₃, smectite, and quartz minerals. Kaolin consists of 52.50% SiO₂, 31.75% Al₂O₃, and kaolinite mineral (Consoli et al., 2021). SCBA consists of 60.65% SiO₂, 13.87% Fe₂O₃, 5.76% Al₂O₃, quartz, hematite, and magnetite minerals (Bruschi et al., 2021a). HEL consists of CaO (72.90%), portlandite, calcite, and magnesium minerals (Consoli et al., 2020).

Previously developed and characterized by Tonini de Araújo et al. (2023a), the alkali-activated binder was composed by 80% SCBA and 20% HEL, 1M NaOH solution (2.61% Na₂O), and water/binder rate equal to 0.8.

Modified Proctor tests (ASTM D1557 (ASTM, 2012)) were performed for bentonite-kaolin-SCBA-HEL samples to define the dry unit weight and moisture content of the mixtures (Figure 1). Bentonite-kaolin proportion of 10/90 was established to simulate an expansive soil. The compaction curves resulted in the following maximum dry unit weights and optimum moisture contents: 14.50 kN/m³ and 27.60% (soil), 12.50 kN/m³ and 24.70% (soil-10% binder). The moulding points represent the dry unit weight and respective moisture content applied to the tested samples as described in item 2.2.

2.2 Experimental design

A four factors factorial design with duplicates (24) and central points (i.e. 36 tests) was outlined for this study. Analyzed factors were dry unit weight (*A*), binder content (*B*), moisture content (*C*), and curing time (*D*). The experimental runs are shown in Table 2.

To better understand the sodium hydroxide and SCBA influence over the soil expansion additional tests were performed, as described in Table 3. These tests entailed samples with soil and isolated materials.

Table 1. Physical properties of the materials.

Physical properties	Standard/Equipment	Materials			
		Bentonite (BT)	Kaolin (K)	SCBA	HEL
Unit weight of grains (g.cm ⁻³)	ASTM D854 (ASTM, 2014a); NBR 16605 (ABNT, 2017)	2.75	2.61	2.08	2.24
Liquid limit, <i>LL</i> (%)	ASTM D4318 (ASTM, 2017)	501	43	-	-
Plastic limit, <i>PL</i> (%)	ASTM D4318 (ASTM, 2017)	66	13	-	-
Plasticity index, <i>PI</i> (%)	ASTM D4318 (ASTM, 2017)	436	30	-	-
% of coarse sand	ASTM D7928 (ASTM, 2021)	0	0	-	-
% of medium sand	ASTM D7928 (ASTM, 2021)	0	0	-	-
% of fine sand	ASTM D7928 (ASTM, 2021)	6	4	-	-
% of silt	ASTM D7928 (ASTM, 2021)	30	67	-	-
% of clay	ASTM D7928 (ASTM, 2021)	64	29	-	-
Main particle diameter, <i>D</i> ₅₀ (μm)	Laser diffraction particle size analyzer (CILAS, model 1064)	-	-	31.01	7.43
Specific surface area (m ² .g ⁻¹)	QuantaChrome equipment (model NOVA 1200e)	-	-	125.15	4.18
Pozzolanic activity index (mg Ca(OH) ₂ /g of pozzolan)	NBR 15895 (ABNT, 2010)	-	-	817.6	-

2.3 Molding and curing of soil-binder samples

From Proctor results (Figure 1) the maximum γ_d adopted was 14.5 kN/m³ (maximum γ_d of the soil-10% binder curve) and the minimum 12.5 kN/m³. For moisture content, the optimum content of 24.7% was the maximum adopted value, and 19.7% the minimum. The binder contents of 10% and 4% were chosen following previous researches (Consoli et al., 2021; Zhang et al., 2013).

The dry unit weight determined the dry mass of materials; after materials weighing, bentonite, kaolin, SCBA, and HEL were mixed, followed by NaOH solution

and distilled water addition. The added NaOH solution was defined from an optimum 2.61% alkali content value obtained from SCBA-HEL-NaOH pastes. Additional water aimed to reach the moisture content of the mixture. Next, each sample was statically compacted in a single layer inside a ring (71.33 mm in diameter, 25 mm high). Finished the molding process, each specimen was weighed and specimens suitable for testing met the following tolerances: γ_d within $\pm 1\%$ of target value, and moisture content within $\pm 0.5\%$ of target value. Then, samples were stored in a plastic bag to avoid humid loss and cured in a humid chamber ($23 \pm 2^\circ\text{C}$ and 95% moisture).

2.4 One-dimensional swell test

The one-dimensional swelling test followed standard ASTM D4546 (ASTM, 2014b), Method A. Each specimen was allocated in a modified odometer cell, with top and bottom saturated porous stones and filter papers, and preloaded by a 1kPa top cap. Next, specimens were inundated using distilled water ($20 \pm 2^\circ\text{C}$). A LVDT device coupled to a data acquisition system measured swell at time intervals of 30 s, 1 min, 2 min, 4 min, 8 min, 15 min, 30 min, 1 h, 2 h, 4 h, 8 h, 24 h, and 48 h following the aforementioned standard. The swell percentage is then measured (Equation 1):

$$\varepsilon_s = \frac{\Delta H}{H} \times 100 \quad (1)$$

Where ΔH is the specimen height change (mm) and H_0 is the initial specimen height (i.e. the ring height of 25mm).

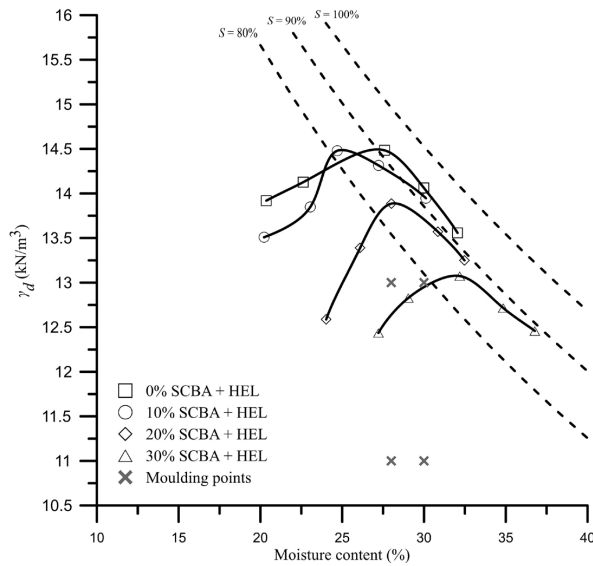


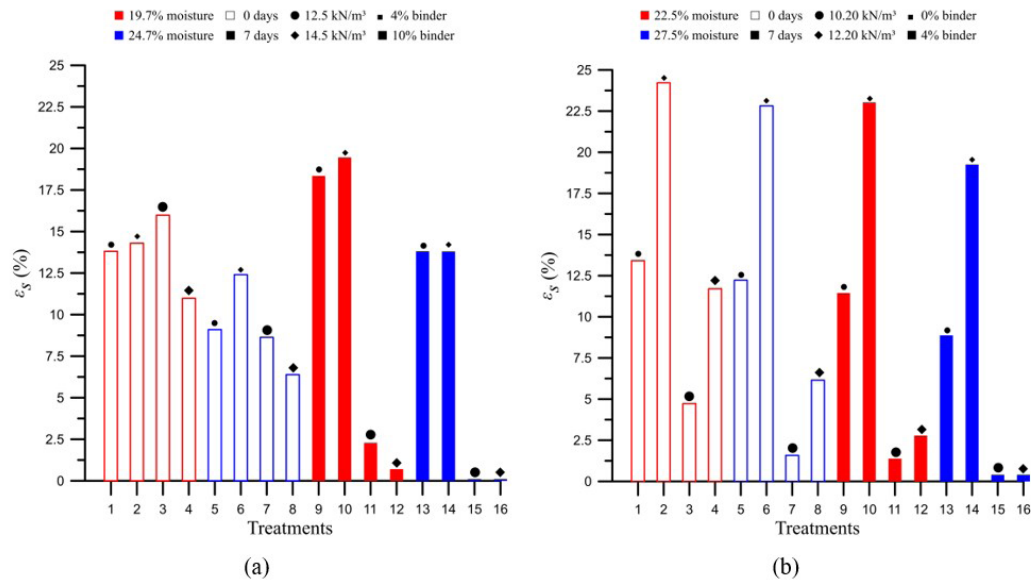
Figure 1. Compaction curves.

Table 2. Experimental design.

Treatment	γ_d (kN/m ³)	Binder content (%)	Moisture content (%)	Curing time (days)
1; 17	12.5	4% (3.2% SCBA + 0.8% HEL)	19.7	0
2; 18	14.5	4% (3.2% SCBA + 0.8% HEL)	19.7	0
3; 19	12.5	10% (8% SCBA + 2% HEL)	19.7	0
4; 20	14.5	10% (8% SCBA + 2% HEL)	19.7	0
5; 21	12.5	4% (3.2% SCBA + 0.8% HEL)	24.7	0
6; 22	14.5	4% (3.2% SCBA + 0.8% HEL)	24.7	0
7; 23	12.5	10% (8% SCBA + 2% HEL)	24.7	0
8; 24	14.5	10% (8% SCBA + 2% HEL)	24.7	0
9; 25	12.5	4% (3.2% SCBA + 0.8% HEL)	19.7	0
10; 26	14.5	4% (3.2% SCBA + 0.8% HEL)	19.7	7
11; 27	12.5	10% (8% SCBA + 2% HEL)	19.7	7
12; 28	14.5	10% (8% SCBA + 2% HEL)	19.7	7
13; 29	12.5	4% (3.2% SCBA + 0.8% HEL)	24.7	7
14; 30	14.5	4% (3.2% SCBA + 0.8% HEL)	24.7	7
15; 31	12.5	10% (8% SCBA + 2% HEL)	24.7	7
16; 32	14.5	10% (8% SCBA + 2% HEL)	24.7	7
33; 34; 35; 36	13.5	7+ (5.6% SCBA + 1.4% HEL)	22.2	3.5

Table 3. Additional tests.

Test	γ_d (kN/m ³)	Binder content (%)	Moisture content (%)	Curing time (days)
Soil + H ₂ O	12.5	-	19.7	0
Soil + H ₂ O	14.5	-	19.7	0
Soil + H ₂ O	12.5	-	24.7	0
Soil + H ₂ O	14.5	-	24.7	0
Soil + NaOH	12.5	-	19.7	0
Soil + NaOH	14.5	-	19.7	0
Soil + NaOH	12.5	-	24.7	0
Soil + NaOH	14.5	-	24.7	0
SCBA + Soil + H ₂ O	12.5	4% (3.2% SCBA + 0.8% HEL)	19.7	0
SCBA + Soil + H ₂ O	14.5	4% (3.2% SCBA + 0.8% HEL)	19.7	0
SCBA + Soil + H ₂ O	12.5	10% (8% SCBA + 2% HEL)	19.7	0
SCBA + Soil + H ₂ O	14.5	10% (8% SCBA + 2% HEL)	19.7	0
SCBA + H ₂ O	-	-	19.7	-
SCBA + H ₂ O	-	-	24.7	-

**Figure 2.** Free swelling: (a) soil-alkali-activated binder, (b) soil-Portland cement (Consoli et al., 2021).

2.5 pH tests

The *pH* of soil-NaOH and soil-water samples was measured in accordance with ASTM D4972 (ASTM, 2013).

3. Results and discussions

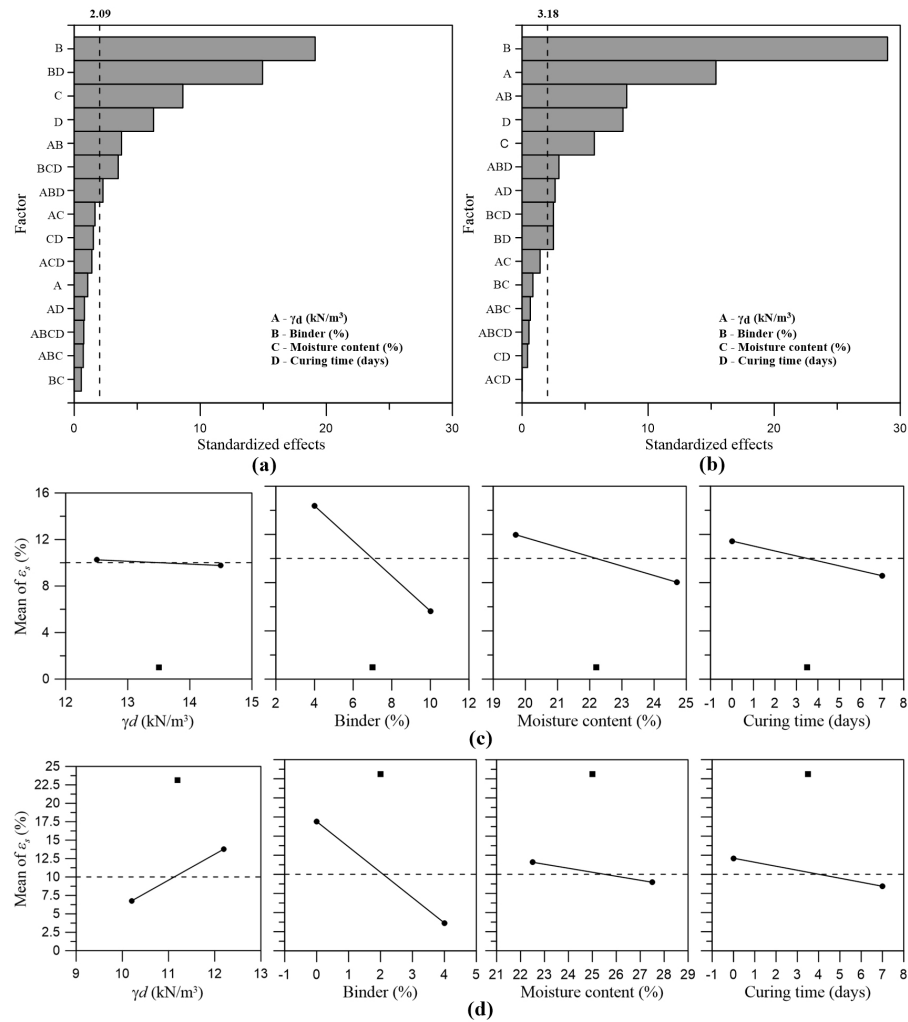
Figure 2 shows the free swelling results. Figure 2a presents the mean of the duplicates for each treatment. For comparison with a traditional binder, swelling results for the same soil stabilized by high initial strength Portland cement (type III) (Consoli et al., 2021) are shown in Figure 2b. Although in the later factors values are different from the

ones in this study (Table 4), factors influence over swelling results are compared for both binders. Figure 3 presents the Pareto chart and the main effects plot and Figure 4 shows free swelling results over time.

According to Figure 2, a higher moisture content decreased swell. This is due a decrease in suction effects with higher soil saturation, and, consequently, a reduction in soil affinity for water adsorption, as acknowledge by Fattah et al. (2017). In soil-Portland cement samples (0% and 4% cement addition) an increase in density led to higher expansion, as samples that are more compact have lower porosity and less available volume for particles to rearrange after swelling. Also, as stated by Silvani et al. (2020) higher densities result

Table 4. Experimental run for the swelling tests of soil-Portland cement samples.

Treatment	γ_d (kN/m ³)	Binder content (%)	Moisture content (%)	Curing time (days)
1	10.2	0	22.5	0
2	12.2	0	22.5	0
3	10.2	4	22.5	0
4	12.2	4	22.5	0
5	10.2	0	27.5	0
6	12.2	0	27.5	0
7	10.2	4	27.5	0
8	12.2	4	27.5	0
9	10.2	0	22.5	7
10	12.2	0	22.5	7
11	10.2	4	22.5	7
12	12.2	4	22.5	7
13	10.2	0	27.5	7
14	12.2	0	27.5	7
15	10.2	4	27.5	7
16	12.2	4	27.5	7
17	11.2	2	25.0	3.5

**Figure 3.** Free swelling statistical results: (a) Pareto chart for the soil-alkali-activated binder, (b) Pareto chart for the soil-Portland cement, (c) mean effects plot for the soil-alkali-activated binder, (d) mean effects plot for the soil-Portland cement.

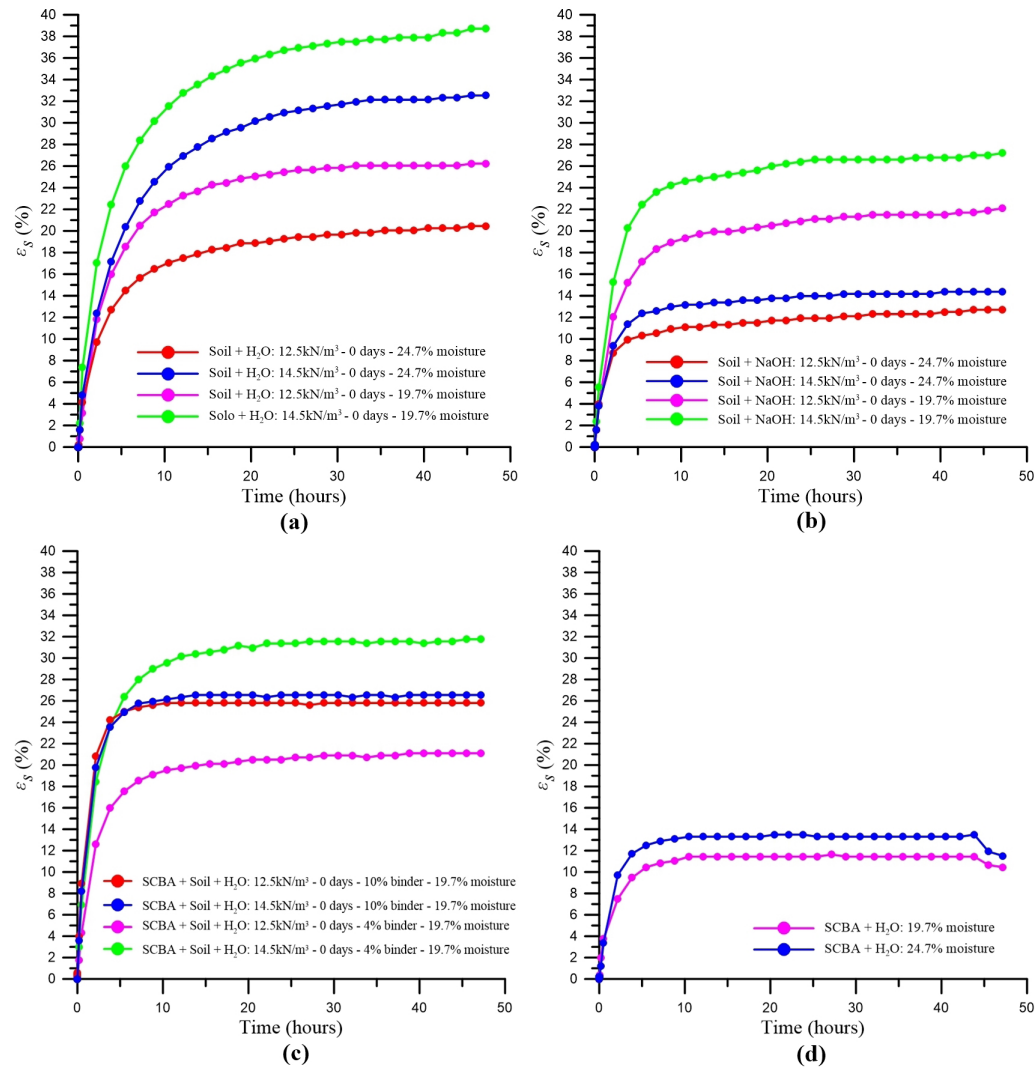


Figure 4. Free swelling results over time for samples containing (a) soil and water; (b) soil and NaOH; (c) SCBA, soil and water; and (d) only SCBA.

in higher expansion due to expansive soil particles higher concentration. For soil-alkali activated binder samples, there is not a clear correlation between higher density and higher expansion. However, alkali-activated samples with higher binder content showed a reduction in swell at higher density values. This is because the significant presence of NaOH in high-density low-binder content samples controlled swelling due to decrease in the bentonite diffuse double layer in the electrolyte nature (Reddy & Sivapullaiah, 2010).

In general, for both binders a higher curing time and binder content resulted in lower swelling, the exception being 4% binder content-7 days curing time alkali-activated samples. In the later, the precipitation of cementitious compounds (e.g. C-A-S-H and C-(N)-A-S-H gels) was not sufficient to counterbalance the bentonite (Figure 4a) and SCBA (Figure 4d) swell during the curing period (i.e. SCBA and bentonite swell disturbed the development of reaction products). However, in

10% binder content-7 days curing alkali-activated samples the precipitation of gels was higher. Thus, these products solidified, preventing bentonite and SCBA expansion during the curing period and after sample's submersion (e.g. treatments 15 and 16, Figure 2a, had almost no swell).

Swell reduced from 13.8% (12.5 kN/m³ and 19.7% moisture) and 8.8% (12.5 kN/m³ and 24.7% moisture) to 2.5% (82% reduction) and 0% (100% reduction), respectively, after 7 curing days and 10% binder addition for the alkali-activated binder. Syed et al. (2020) found similar results for an expansive clayey soil stabilized with fly ash activated by sodium hydroxide and sodium silicate: the swell percentage decreased 62% with 10% binder addition. For Portland cement, swell reduced from 13.8% (10.2 kN/m³ and 22.5% moisture) and 12.5% (10.2 kN/m³ and 27.5% moisture) to 1.8% (86% reduction) and 1% (92% reduction), respectively, after 7 curing days and 4% binder addition (Consoli et al., 2021).

In the Pareto chart, horizontal bars represent the effect magnitudes of each factor and bars exceeding the vertical line represent factors with significant influence over the response variable. The Pareto chart of alkali-activated samples (Figure 3a) shows that the binder content (*B*), second order interaction *BD* (binder content and curing time), and moisture content (*C*) possess, in this order, higher influence over swelling. For Portland cement (Figure 3b), the binder content (*B*), dry unit weight (*A*), and second order interaction *AB* possess, in this order, more influence over swelling. For both binders, the binder content (*B*) has more influence over swelling. This is mainly due to the precipitation of reaction products. Also, after the addition of those binders in soil, the *pH* of the environment increases leading to Ca^{2+} release and cationic exchange between Na^+ and Ca^{2+} . The cation exchange followed by flocculation-agglomeration and pozzolanic reactions result in the formation of a coarse fabric which prevents swelling (Mitchell & Soga, 2005; Soltani et al., 2017).

For the main effects plot, the dotted line corresponds to the mean response of the tests, and line steepness is the effect intensity. Figure 3c,d shows that an increase on mean factors binder content, moisture content, and curing time reduced swelling. The higher line steepness of binder content denotes the higher influence of this factor. The middle points attest a non-linear behavior of the mean factors. In addition, for studied variables both binders presented a similar mean swell result (10%).

Figure 4 entails additional tests and corroborates previous conclusions; Figure 4b shows that samples containing NaOH expanded less than samples molded with only water (Figure 4a). This is explained by an increase in the *pH* in soil-NaOH samples (*pH* = 14) compared to soil-water samples (*pH* = 8.57). The *pH* increase might change the bentonite edge-face charge sign reducing soil fabric dispersion due to particle aggregation (Fan et al., 2013). In addition, a higher amount of cation in the environment, i.e. Na^+ from NaOH, causes shrinkage of the mineral's double-diffusion layer and flocculation of soil particle reducing swelling (Mitchell & Soga, 2005). Figure 4c shows that 4% SCBA-12.5 kN/m³-19.7% moisture content (pink line) and 10% SCBA-14.5 kN/m³-19.7% moisture content (blue line) soil samples, had a similar swell rate to soil-NaOH samples with same moisture content and density (pink and green lines of Figure 4b, respectively). However, other samples in Figure 4c swelled more than soil-NaOH samples with same moisture content and density. This is explained by Figure 4d: the SCBA swells over time.

4. Conclusion

This paper analyzed the swelling behavior of an expansive soil stabilized by an alkali-activated binder. Soil-Portland cement swell behavior was compared to the aforementioned binder. Free swelling tests and statistical analyses were performed. For both binders, the binder content

had more influence over swelling. In addition, an increase on the main factors (binder content, moisture content, and curing time) reduced swelling. However, contrary to Portland cement specimens, soil-alkali activated binder samples no presented a clear correlation between higher density and higher expansion.

The significant presence of NaOH in high-density low-binder content alkali-activated samples controlled swelling; and soil-NaOH samples expanded less than samples molded with only distilled water. This is because *pH* increase might change the bentonite edge-face charge sign reducing soil fabric dispersion due to particle aggregation. For Portland cement, cementation of soil particles due to cement hydration reactions was the main responsible for swell reduction. For studied samples, swell reduced 100% and 91% with 10% alkali-activated binder and 4% Portland cement additions, respectively. Thus, aiming at swell reduction, Portland cement can be replaced by the alkaline binder, considering conditions similar to this study.

Although extensive research has been reported on the use of alkali-activation for soft soils (e.g. Abdullah et al., 2020; Cristelo et al., 2012; Liu et al., 2016; Phetchuay et al., 2016; Tonini de Araújo et al., 2023b; Zhang et al., 2013), few studies explored the use of these binders for swelling control. This study contributes to the literature by analyzing the use of alkali-activated wastes binder (SCBA and recycled eggshell) for a clayey soil stabilization. The utilized precursor expanded over time, and the activator controlled swelling. Thus, the influence of materials composing the alkali-activation binder on swelling should be carefully analyzed before any decision-making regarding soil stabilization.

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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

Mariana Tonini de Araújo: conceptualization, data curation, visualization, writing – original draft. Suellen Tonatto Ferrazzo: conceptualization, data curation, methodology, validation, writing – original draft. Giovanni Jordi Bruschi: conceptualization, data curation, methodology, validation, writing – original draft. Nilo Cesar Consoli: supervision, validation, writing – review & editing. The authors above kindly granted the permission of using parts of their publications in this template.

Data availability

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

List of symbols and abbreviations

pH	pH of soil-NaOH and soil-water samples
A	Dry unit weight
AB	Dry unit weight and binder content
B	Binder content
BD	Binder content and curing time
BT	Bentonite
C	Moisture content
CH	Inorganic clay of high plasticity
CL	Inorganic clay of low plasticity
$CNPq$	Conselho Nacional de Desenvolvimento Científico e Tecnológico
D	Curing time
D_{50}	Main particle diameter
H_0	Initial specimen height
HEL	Hydrated eggshell lime
K	Kaolin
LL	Liquid limit
OPC	Ordinary Portland cement
PL	Plastic limit
PI	Plasticity index
S	Degree of saturation
$SCBA$	Sugarcane bagasse ash
$USCS$	United soil classification system
ΔH	Specimen height change
ε_s	Swell percentage
γ_d	Density

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Design, construction, and validation of an experimental inclinometer probe

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Article

Keywords

Low cost
Geotechnical instrument
Inclinometer
Monitoring
Experimental probe
Horizontal displacement

Abstract

The inclinometer probe is used to measure angle variations along a soil profile. These variations can be transformed into horizontal displacements along the profile depth, enabling the understanding of the deformation of geotechnical structures (e.g., tailings dams, embankment dams, rockfill dams, and excavations) and natural slopes. Such knowledge can guide preventive and corrective actions and mitigation measures to reduce the severity of socioeconomic and environmental impacts. In this study, a low-cost, reduced-dimension inclinometer probe was developed and validated. Accessible technologies (MEMS inclinometer sensor, microprocessor, logic level converters, communication module, and other electronic components) were used for the probe. The sensor reading programming was developed using Python language and interpreted using an Excel spreadsheet. The probe was calibrated on a three-dimensional measurement table with the aid of a mechanical device to stabilize its inclination and validated by comparison of its output with those of a commercial probe. The probe had a lower cost than commercial probes and generated similar readings, although the reading time was longer due to the increased number of measurements required.

1. Introduction

The inclinometer, also known as the inclinometric probe, is a well-established instrument for measuring soil behavior in the field of geotechnics. This instrument is designed to monitor the rates of horizontal displacement of geotechnical structures (slopes, embankments, dams, etc.) over time and soil depth (Silveira, 2006; Bo & Barrett, 2023). The inclinometer was developed based on a device built in 1952 by S.D. Wilson of Harvard University and was made commercially available by the end of the 1950s by the Slope Indicator Company (Green & Mikkelsen, 1988).

Penz (2013) indicated that the term “inclinometer” refers to “every instrument that due to the constructive characteristics propitiates the measurement of the local inclination or to verify the existence of angular variations, being the process done in a differential or absolute manner.”. Most aspects of these instruments are described in the geotechnical literature (e.g., Dunnicliff, 1988; Green & Mikkelsen, 1988; Mikkelsen, 2002, 2003; Silveira, 2006; Stark & Choi, 2008; Fernandes, 2011). Coimbra et al. (2023) conducted a review of the state of the art and practice regarding the operation of inclinometers for slope monitoring in geotechnical works.

Dunnicliff (1988) and Stark & Choi (2008) reported that inclinometers are used to determine the magnitude, rate, direction, depth, and type of soil movement. Generally, this information is vital for the understanding of landslide causes, behavior, and correction requirements, although this objective cannot be achieved without continuous monitoring. Hunt (2005) noted that the continuous measurement of lateral deflection from the surface with an inclinometer is very useful in the monitoring of embankments and pile foundations.

Over time, the inclinometer has been refined and has become more accurate (e.g., Wilson, 1962; Hanna, 1985; Silveira, 2006; Machan & Bennett, 2008). In the literature, there are experimentally developed inclinometric probes (e.g., Formoso, 1999; Penz, 2013; Covassi et al., 2018). From a commercial point of view, the technological development of these devices occurred with the advent of the 21st century and the introduction of MEMS accelerometers and wireless (i.e., via Bluetooth and Wi-Fi) data transfer. Freddi et al. (2023) introduced an automated in-place inclinometer system using MEM sensors. Manufacturers such as the Slope Indicator Company, Geokon, Geodac, and Encardiorite have developed more versatile, efficient, and accurate probes. The in-place ShapeArray inclinometer developed by Measurand is used

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in the instrumentation of dams, embankments, slopes, and excavations.

Despite its importance, the monitoring of small and medium geotechnical constructions and natural slopes on highways and in urban areas is incipient because the significant costs of guide tube installation, probe purchase or rental, technical support, monitoring, and maintenance make it financially unfeasible. In this work, we developed and validated a reduced-proportion, low-cost inclinometer probe to aid the dissemination of inclinometer use at geotechnical sites and on natural slopes.

2. Background

2.1 Reading procedures and data interpretation

Stark & Choi (2008) and Dunnicliff (1988) explained that inclinometers generate a zero reading or initial profile, established with at least two readings, that serves as a reference. Dunnicliff (1988) noted that in case of doubt, the absolute transverse deformation of the top of the guide tube should be monitored regularly using topographic survey methods. Slope measurements should be acquired starting from the inclinometer's stable base (Dunnicliff, 1988; Stark & Choi, 2008). In the reading procedure in conventional inclinometers, the probe is lowered slowly to the desired depth using the intervals in the cable (0.5–1 m). Before the zero readings are taken, the probe must be held in position for at least 10 min to adjust to the temperature in the hole. Dunnicliff (1988) listed the subsequent procedures:

- the taking of readings at different depths;

- the removal of the probe, its rotation through 180° , and its lowering again to the same point;
- the taking of readings at different depths after temperature stabilization; and
- verification of the validity of the checksums.

The 180° rotation of the probe is necessary to distribute the reading errors (e.g., sensor position and wheellock play). Stark & Choi (2008) further explained that the probe is lowered to the bottom of the guide tube with the wheels in the $A0^\circ$ groove for the taking of the first measurements after the zero readings. When the probe reaches the bottom of the inclinometer, the cable is attached to the pulley grips at the foot marker corresponding to the lowest depth reached for the initial readings.

The probe should then be lifted to the surface in the intended increments, with readings taken from the $A0^\circ$ and $B0^\circ$ directions at each interval (Figure 1). A measuring range equal to the probe wheelbase is commonly used to achieve maximum accuracy.

The $B0^\circ$ direction is 90° clockwise from $A0^\circ$, and the tilt on the B axis is measured by the second sensor on the probe. After all readings have been taken with the wheels in the $A0^\circ$ slot and the probe hits the surface, the probe is carefully removed and rotated 180° so that the lower wheels are inserted into the $A0^\circ$ slot and another set of readings is taken in the $A180^\circ$ and $B180^\circ$ directions.

Souza et al. (2016) pointed out that the guide tube must have a minimum length of 5 m from its lower section fixed below the greatest depth at which some movement is expected. Thus, the end of this tube can be considered as a fixed reference. Bartholomew et al. (1987) explained that the inclinometer probe measures angles of inclination from

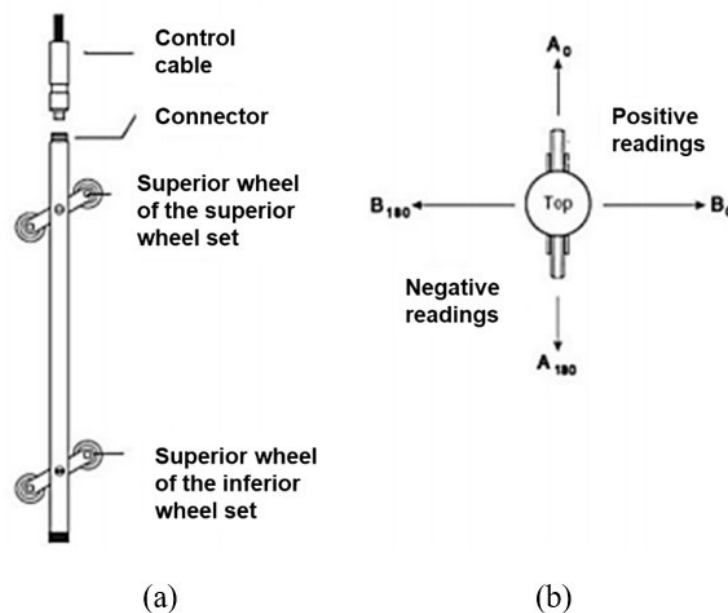


Figure 1. Schematic drawings of the inclinometer probe: (a) side view; (b) top view.

vertical in two planes oriented at 90° (orthogonally) to each other, which can be uniaxial or biaxial (Figure 2). With uniaxial probes, two additional series of measurements are taken when deformation data from both vertical planes are needed by aligning the probe wheels in the second pair of grooves (i.e., 90° relative to the first series; Dunnicliff, 1988).

Stark & Choi (2008) emphasized the importance of using the same probe (to reduce possible errors) and measuring displacements at the same depth intervals as the initial reading. The deviation from vertical displacement, that is, the horizontal displacement (Figure 2), is determined by a sinusoidal function using Equation 1:

$$\delta_{DH} = L \cdot \sin\theta \quad (1)$$

Where δ_{DH} is the calculated horizontal displacement, L is the distance in pairs of probe wheels and θ is the rotation angle of the probe. The vertical deviation in each measurement range reflects the lateral position of the guide tube to the fixed end. The total horizontal (cumulative) displacement is obtained by summing the individual lateral deviations between the fixed end and the top of the guide tube using Equation 2:

$$\delta_{DHT} = \sum \delta_{DH} \quad (2)$$

2.2 Sources of error

Green & Mikkelsen (1988) and Stark & Choi (2008) stated that the accuracy and reliability of measured positions and displacement profiles depend on the quality of an inclinometer's housing, probe, cable, readings, and selected accessories. The sensor quality (resolution, sensitivity, offset, accuracy, and precision) is of great relevance for the faithful representation of displacements. In addition to the quality of the equipment, the accuracy, precision, and reliability of inclinometer monitoring data depend on factors such as the installation and measurement methods, the functionality and performance of the installed instrument (Bo & Barrett, 2023).

Dunnicliff (1988) and Souza et al. (2016) noted that inclinometers' sensor wheels commonly have a certain clearance to the orthogonal direction, which results in the generation of more errors along the B axis. This clearance originates from imperfections in the grooves of the tube or from sensor wheel wear.

In the current context, the term "error" refers to random and systematic errors and is important for landslides that have not moved significantly. If the movement measured on a tilt inclinometer does not exceed the system's field accuracy, the consultant or expert should view it carefully and assess whether it is within the probe's expected error (Stark & Choi,

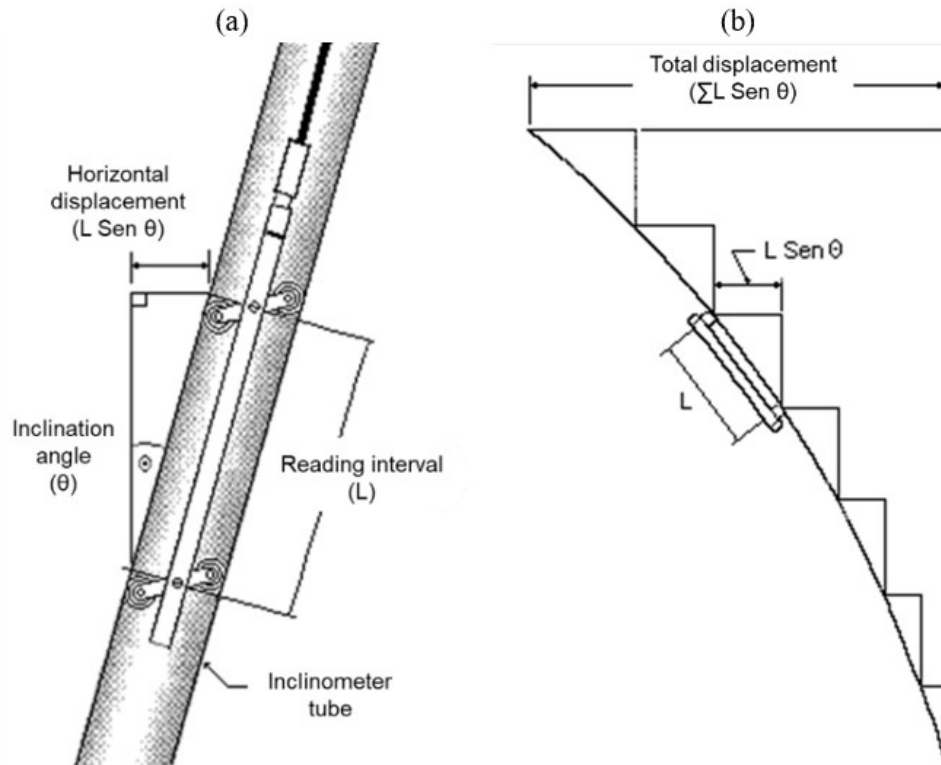


Figure 2. Schematic drawings of the reading procedure: (a) detail of the probe inserted into the tube; (b) reading along the entire length of the tube (adapted from Dunnicliff, 1988).

2008). Mikkelsen (2003) listed four systematic errors that affect inclinometer measurements:

- tilt (bias) shift;
- sensitivity deviation;
- rotation; and
- depth position errors.

3. Experimental program

The experimental program was carried out in five stages. First, the electronic components used for the construction of the internal measurement and communication circuit were selected. Next, the geometric design of the small-proportion probe was developed. Then, the probe assembly, reading, and operation procedures were described. The probe was constructed, assembled, and calibrated, and, finally, the equipment was validated by comparison with a commercially available inclinometer probe.

4. Materials

The components of the probe, including its electronic circuit, are listed in Table 1.

4.1 Electronic components and circuits

Among the MEMS accelerometers available on the electronics market, the SCA 103T D04-1 (Murata, 2019; Figure 3a) was selected for the inclinometer probe. Its parameters are listed in Table 2. This accelerometer has

high calibration accuracy, high resolution, and a low noise level. It is not sensitive to vibration because its sensory elements are dampened, and it can withstand mechanical shocks of 20,000 g. This accelerometer can be used for platform leveling and stabilization, laser level rotation, instrument leveling, and level building.

The electrical connection of the accelerometer for tilt measurements is performed using pins 5 (OUT_2) and 11 (OUT_1; Aa). Ab shows typical outputs (channels 1 and 2 and differential output OUT_1–OUT_2). Rotation angles are calculated as a function of the output's electrical response (Figure 4).

Voltage measurements (analog outputs) can be converted to the accelerometer's inclination angle (α ; Murata, 2019), which represents the probe body's angle of rotation, using Equation 3:

$$A = \arcsin\left(\frac{V_{D,out} - Offset}{sensitivity}\right) \quad (3)$$

Inclination angles close to 0° can be estimated accurately with straight-line conversion, but the use of arcsine conversion is preferred for the greatest accuracy (Murata, 2019). The sensor sensitivity (in V/g) is calculated using Equation 4, considering the sensor position and electrical output voltage:

$$Sensitivity = \left(\frac{V_{out_{tilt1}} - V_{out_{tilt2}}}{\sin(tilt1) + \sin(tilt2)}\right). \quad (4)$$

Table 1. Experimental inclinometer probe components.

Item	Unit	Quantity
Inclinometer's probe body - stainless steel 304	Unit	1
Reading unit housing - stainless steel 316	Unit	1
Base (3D printing) - ABS	Unit	1
MEMS Accelerometer SCA 103T D04-1	Unit	2
Analog/Digital Converter Module - ADS 1115 (4 channels)	Unit	1
RS-485 module	Unit	2
Plastic case (100 mm × 60 mm × 25 mm)	Unit	1
TTL-USB Converter	Unit	1
Logic level converter module	Unit	1
Mike connector 5 pins PCI F/M 025-70	Unit	1
Key NA Push Button DS-323 without Thermal Lock 023-9	Unit	1
Raspberry Pi 0 W Anatel board	Unit	1
1/8" stainless steel cable clips	Unit	3
3/32" 2.4 mm 6×7 AA stainless steel cable	m	21
Shielded network cable - 4 pairs	m	20
Plastic spool	Unit	1
Centering CAP (75 mm / 86 mm)	Unit	1
Nylon cable ties 100 mm × 2.5 mm	Pkt	1
Cable gland	Unit	1
O-ring	Unit	1

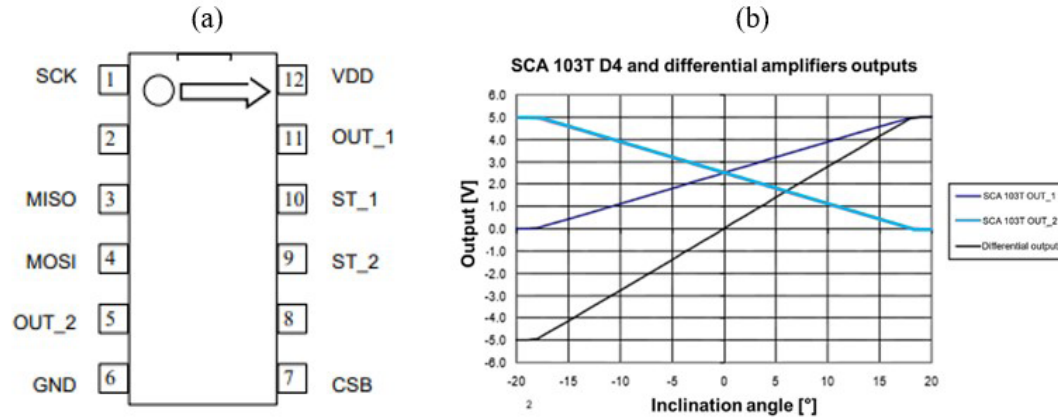


Figure 3. Electrical connection of the SCA 103T D04-1 MEMS accelerometer: (a) circuit diagram; (b) output behavior (adapted from Murata, 2019).

Table 2. SCA 103T D04-1 MEMS accelerometer parameters (adapted from Murata, 2019).

Parameter	Unit	Values
Measurement range	°	± 15
Force	g	± 0.26
Gravitational analog sensitivity	V/g	16
Angular analog sensitivity	mV/°	280
Digital resolution	°/LSB	0.009
Analog resolution	°	0.0013
Operating temperature	°C	-40 – 125
Typical non-linearity	°	± 0.057
Typical consumption current	mA	4 (5*)
Supply voltage Vdd	V	4.75 – 5.25 (5**)

*The maximum consumption current is 5 mA. **The typical supply voltage is 5 V.

A Raspberry Pi Zero single-board computer ($65 \times 30 \times 5$ mm), made of low-cost components (Adafruit, 2019), was used for data acquisition (Figure 5a). The body of the computer's board is made up of a 1 GHz single-core ARM processor (CPU; similar to the B+ and A+ Pi models) and 512 MB RAM memory unit. The board has a mini-HDMI port (with PAL or NTSC output through two 0.1" pads), micro-USB OTG port, micro-USB power port, and HAT-compatible 40-pin header. Figure 5b shows an illustration of the MEMs sensor, model SCA 103T D04-1 and manufactured by Murata. Technical data is given in Table 2.

A low-cost module based on MAX485ESA chip (Maxim), capable of establishing differential communication according to the RS-485 standard was used (Figure 5c). The operating voltage used was 3.3 V, the same as applied to all other devices. This module's power consumption is 300 μ A (Maxim, 2014) and the long-distance communication was established through a shielded network cable according to the Cat5e standard (Figure 5g).

The conductors are four pairs of intertwined copper wires with individual thermoplastic protection. A USB-TTL

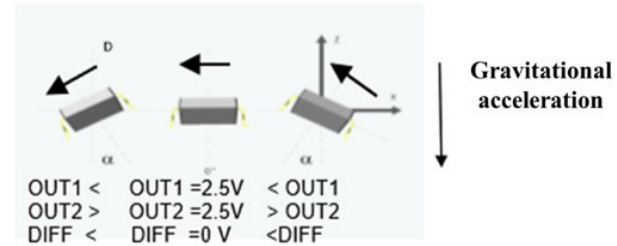


Figure 4. Behavior of the SCA 103T D04-1 analog output (adapted from Murata, 2019).

module (Figure 5d) was used to convert the communication standards used for the microcontrollers (UART or TTL) to USB ports communication pattern (RS-232). This module uses a PL-2303 chip (Prolific), which requires a 24 mA current for operation. To adjust the voltage supplied by the USB port to the input level requested by the devices (5 V), the XL6009E1 (XLSEMI) voltage regulator was used (Figure 5e).

This type of regulator was selected because it has more operational efficiency than other devices. Its operation requires a 5-mA current. In an adjustment of the initial plan, the ADS-1115 (Texas Instruments) analog/digital converter (Figure 5f), which has a 16-bit resolution, was applied to the analog sensor output.

The total cost for the electronic components and machining of the metal parts for the inclinometer probe was 10–20% of that of a commercially available inclinometer probe.

The internal circuit of the inclinometer can be visualized through Figure 6a. The system is powered and the data are viewed via the computer's USB port. To overcome voltage drop along the cable and ensure that a voltage of 5V is maintained, a step-up voltage regulator (Figure 6b) was used. A logic level converter was applied to the communication between the sensor and the Raspberry Pi Zero, as their operating voltages are 5 and 3.3 V, respectively. Two RS-485 modules were used to create a standard signal emission that enables long-

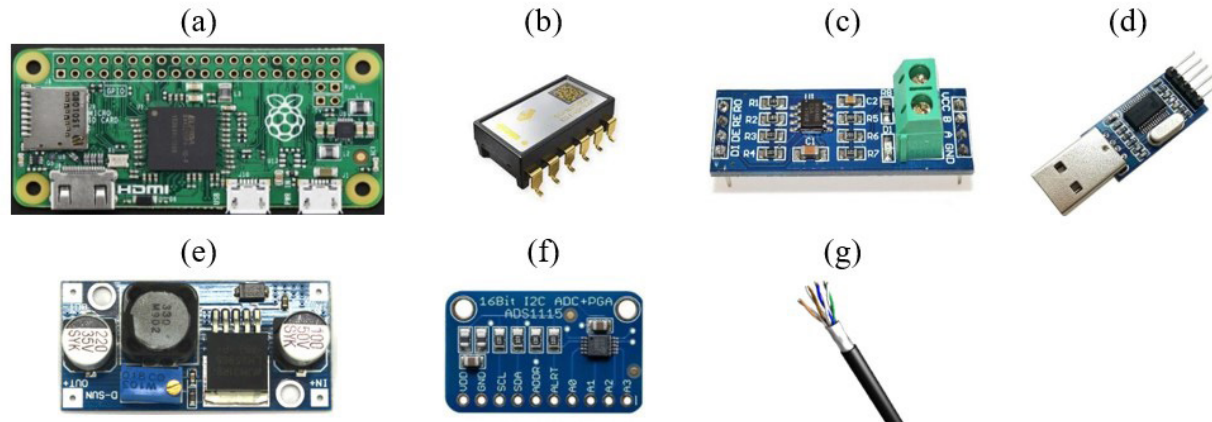


Figure 5. Inclinometer electronic components: (a) Raspberry Pi Zero W board; (b) MEMS SCA 103T D04-1 sensor; (c) RS-485 module; (d) USB-TTL module; (e) voltage regulator; (f) ADS-1115 module; (g) communication cable.

distance communication between the probe and the notebook through a TTL-USB converter (Figure 6c). The USB input enables communication between the inclinometer and the notebook and is the power source for the probe's electronic components.

4.2 Inclinometer probe body

The probe design, including conceptual details of the longitudinal sections, is presented as Figure 6. The distance between wheel axes was 15 cm, based on Penz (2013). This reduced distance provides for a more precise description of localized displacements and ease of removal of the equipment inside the guide tube. The design specifications and geometric dimensions of the inclinometer, in cut sections, are shown in Figure 7 and Figure 8, respectively.

The larger section in the upper region of the case accommodates the 35-mm-wide Raspberry Pi Zero W board. The probe body is made of grades 304 and 316 (in the chamber) stainless steel. Stainless steel is alloyed with iron, carbon, and a minimum of 10.50% chromium (Cr), which guarantees corrosion resistance (Carbó, 2008). Grade 304 stainless steel contains nickel (Ni) as an alloying element, which changes the ferritic structure to austenitic. More specifically, this steel has 18% Cr and 8% Ni contents and is the most popular grade due to its excellent corrosion resistance, ductility (a marked difference in mechanical properties from ferritic steel), and weldability. However, it can be corroded by the chloride anion (Cl^-). Depending on the concentration of chlorides in the exposure medium, and the temperature and pH, three forms of corrosion can occur: pitting, crevice development, and/or stress.

According to ASTM A479 (ASTM, 2007), 316 stainless steel is composed of an alloy that includes chromium, nickel, and molybdenum. The introduction of molybdenum (the 304 stainless is free) gives the material greater chemical resistance in highly corrosive environments, such as salt water

or chlorides. In addition, it has a better resistance to creep at higher temperatures and a higher mechanical resistance compared to stainless 304.

Finally, grades 304 and 316 stainless steel are austenitic and thus not magnetic, and thus are suitable for electronic equipment. Figure 9 shows the machined and welded probe body with the introduction of the cable gland for the data cable output.

5. Probe function

The electronic circuit initially works via the MEM sensors, which measure the inclination continuously based on gravitational acceleration and transform this information into output data (Figure 4). These data are converted to digital readings and sent via SPI communication to the Raspberry Pi Zero board, and then via the serial port to the RS-485 module. Another RS-485 module and a TTL-USB converter receive the data sent via cable and provide them to the computer. All these tasks were programmed using the Python language.

To perform measurements and obtain a displacement graph, the following sequence of procedures must be followed.

- 1) Open the inclinometer.py routine file;
- 2) Inform the COM port activated by the USB port through the reading cable on the inclinometer probe and press the “enter” key (Figure 10);
- 3) Inform the serial value in the program (9600) and press the “enter” key (Figure 10);
- 4) Position the inclinometer at the reading depth;
- 5) Press the “y” button in response to the question “Do you want to collect a reading? [y,n]”; once the probe has stabilized for the last 50 readings, the digital reading value is reported (Figure 10);
- 6) Reposition the equipment to the next measurement depth and repeat the entire procedure;

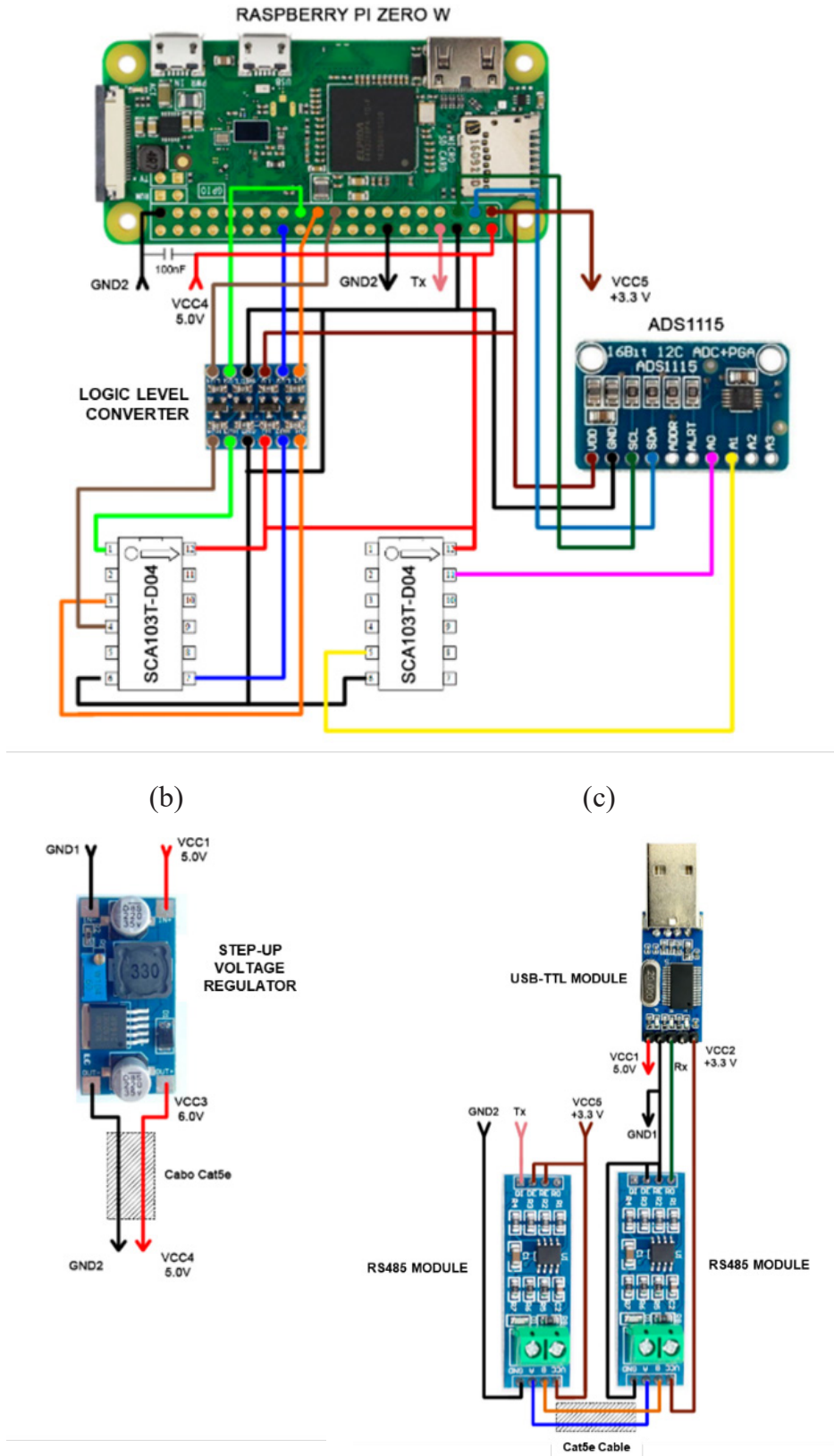


Figure 6. Electrical circuits of the inclinometer: (a) main circuit; (b) step-up voltage regulator; (c) secondary circuit to connect the main circuit to a notebook.

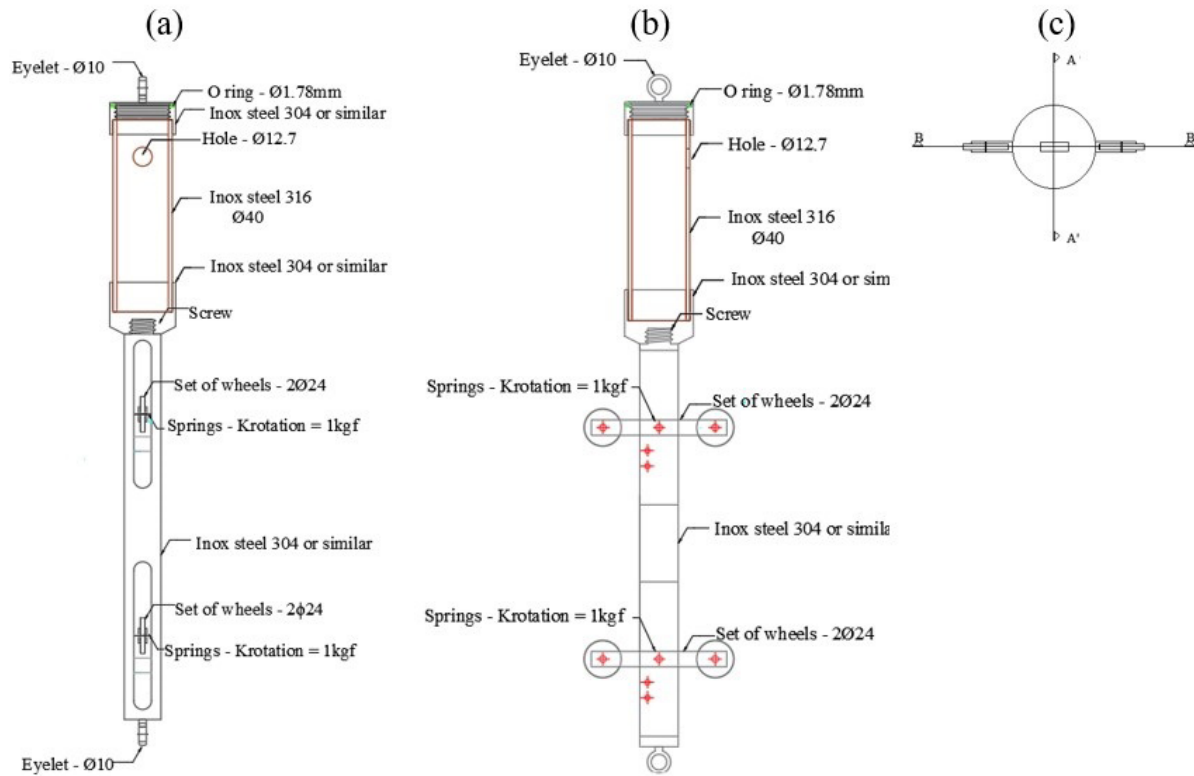


Figure 7. Conceptual drawings of the inclinometer probe: (a) A–A' section; (b) B–B' section; (c) top view.

- 7) After performing all the readings in relation to the x or y axis, rotate the sensor 180° and repeat the entire procedure;
- 8) Repeat all procedures in the position of the orthogonal groove of the guide tube to obtain displacement curves along the x and y axes;
- 9) Type “n” to end the program.

The digital readout response is converted to a tilt angle (α) via the sensor calibration equations (Equations 3 and 4), which consider the sensor offset parameters and sensitivity for the calculation of the offset value for the specific quota based on Equation 1.

During the cyclical routine of the Python program and with the help of the Pi Zero board, the data are transferred to the computer via cable. When all measurements for a given axis have been taken, the average probe displacement is calculated in relation to depth.

The accumulated average displacement is calculated from the sum of the average displacements at each elevation measured based on Equation 2. All accumulated readings are listed using an Excel spreadsheet and graphs are built per series of readings.

6. Methods

The probe was calibrated using a three-dimensional measuring table (coordinate measuring machine; SCIROCCO 100907 model TWPH10M; DEA) at the Department of

Mechanical Engineering of PUC-PR (Pontifical Catholic University of Paraná), Brazil (Figure 11). This machine is used for the three-dimensional measurement of points with a linear optical transducer coupled to a mechanical arm that moves through pneumatic skates in three axes and is handled with a joystick. It has a measurement field of $x = 1400$ mm, $y = 860$ mm, and $z = 660$ mm, an approximate speed of 52 m/min (up to 65 m/min in fly function), and a measurement resolution of 2 μ m.

As this equipment is not practically used for this purpose, an alternative calibration method was created. A calibration apparatus (Figure 11a) was built and fixed to the upper end of the probe, permitting rotation in one direction as a screw was rotated, generating a rotation angle. The rotation response was captured by the three-dimensional coordinates measured by the linear optical transducer for the generation of virtual sections. These sections were created from three points on the probe surface, which were used to generate the center of a partial circumference (Figure 11b). The generation of two sections resulted in the construction of a virtual cylinder whose generatrix inclination enabled the measurement of rotation angles in two directions (main and residual – in which slight equipment rotation generates a small error). These angles were compared with the digital output readings from the MEMS sensors (Figure 11c).

For validation, the experimental inclinometer probe was submitted to displacement reading tests performed through

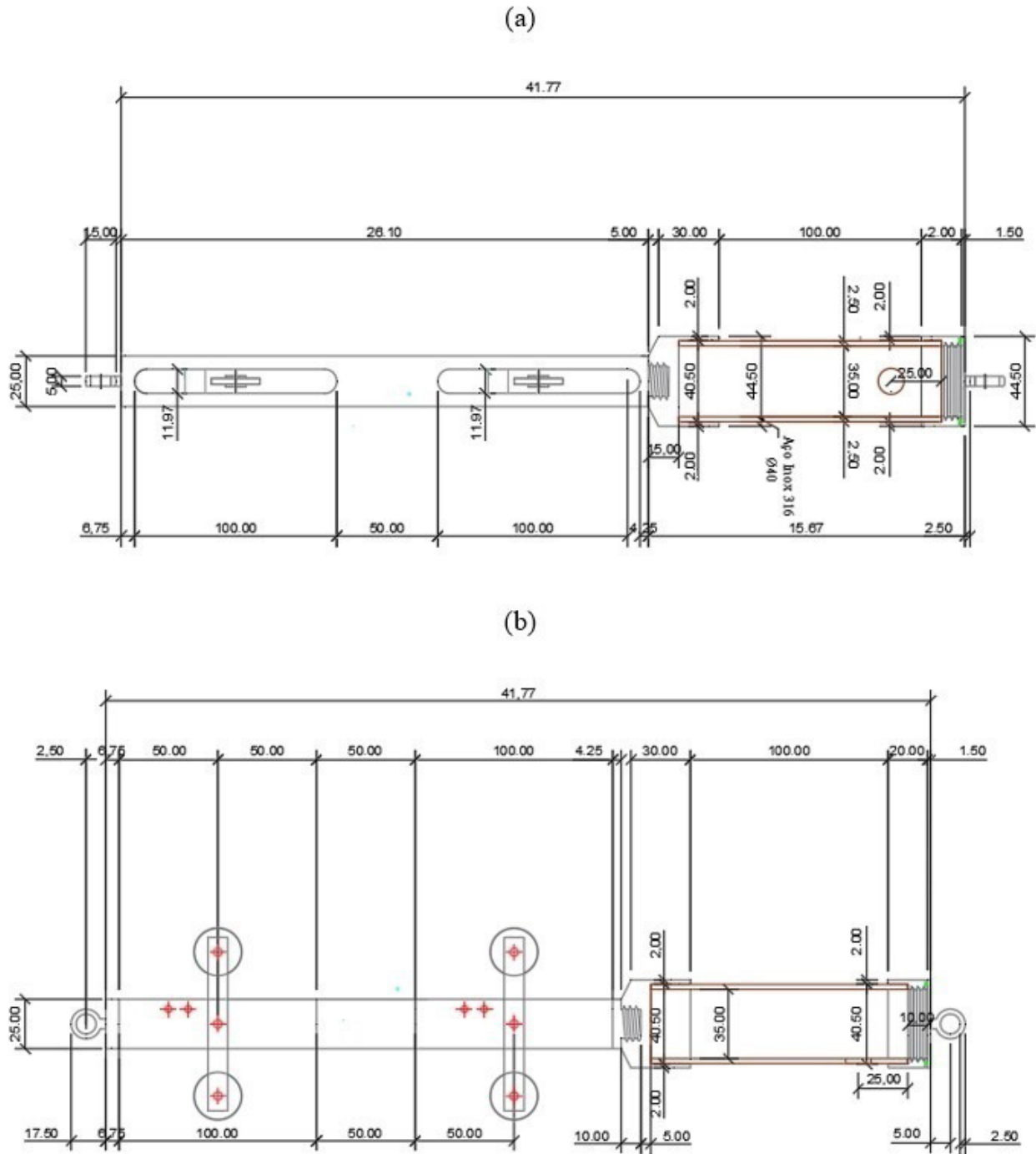


Figure 8. Geometric dimensions of the inclinometer probe (in millimeters): (a) A–A' section; (b) B–B' section.

an aluminum guide tube ($\phi_{\text{ext}} = 85$ mm, length = 3 m) fixed to a metallic structure (Figure 12). To assess the sensitivity of the equipment, fictitious distortions were created at specific points using clamps (Figure 12a and 12b).

The probe used for comparison was the Digitilt Datamate II (DGSII; Figure 13a), a classic type of slope indicator with which readings are obtained via the digital responses of the

sensor, converted from the axis distance using the sensor's calibration coefficient. Figure 13b shows the experimental measurement system with the inclinometer probe coupled to a notebook and the program file open.

The tests were performed between -0.5 and -2.5 m due to the restrictions of the commercial probe with regard to the 3 m-long guide tube; the probe has a length of approximately

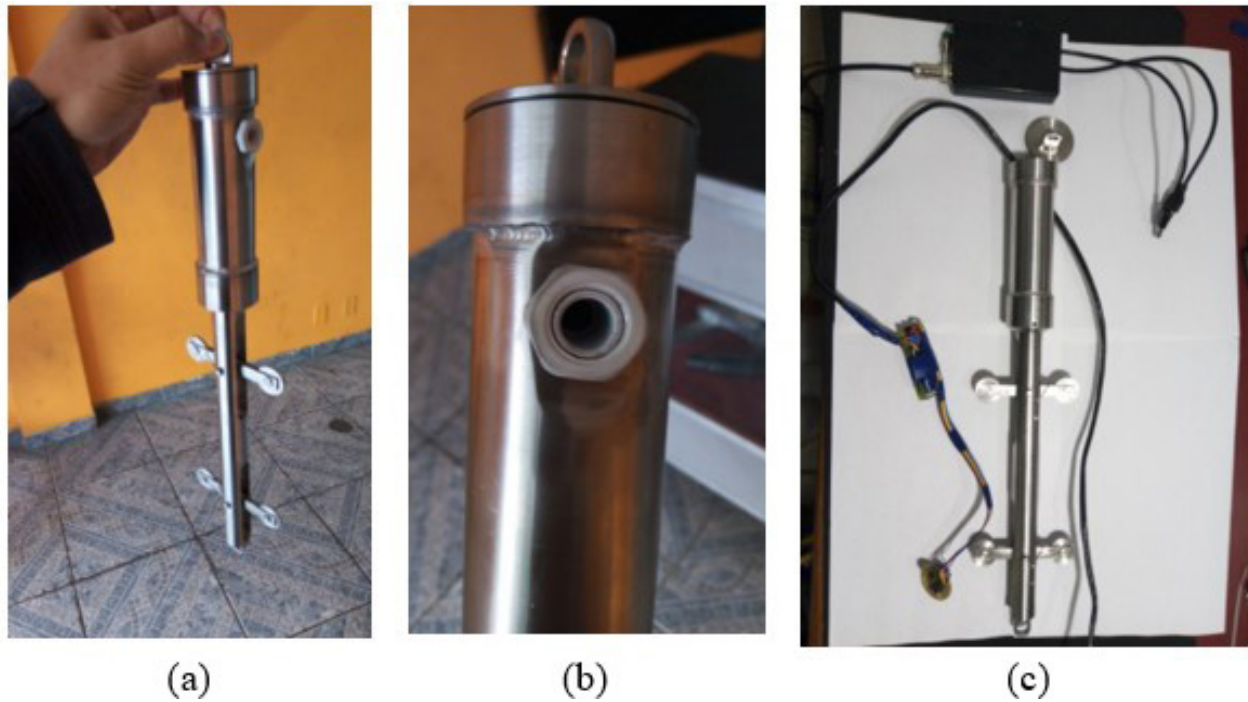


Figure 9. Inclinometer probe body: (a) welded machined parts; (b) detail of the cable gland for the passage of the power and reading cable; (c) the probe with the electronic circuit and USB adapter for notebook access.

```

CAUsers\tenni\Desktop\Inclinometro\inclinometer.exe
Inform serial port (ex. COM10)
COM3
Inform baudrate (ex. 9600)
9600
Serial port open (COM3) at baudrate 9600
Do you want to collect a reading? [y,n]
y
Sensor 1 (300.18)
Sensor 2 (-19941.0)
Do you want to collect another reading?

```

Figure 10. Python program commands to obtain readings.

60 cm and thus could not descend to the bottom of the tube for measurement, and it was impeded at the top due to the absence of an adequate centralizing CAP. The test followed the standard procedure for inclinometer-based measurement: horizontal displacement was measured with each probe in the 0° and 180° positions along the x and y axes (total = 4 measurements). The measurement procedure was not repeated because the magnitude of the horizontal displacement in both directions was known.

7. Results and discussion

Figure 14 shows the assembled electronic circuit for data reading and transfer. Figure 14a shows how the sensor was attached to the body of the inclinometer and Figure 14b shows the complete circuit with the MEMS sensor connection on the Raspberry Pi Zero[®] W board and its modules.

Figure 15 shows the inclinometer probe components in the final phase. Figure 15a shows the communication black box for the protection and Figure 15b shows the positioning of the MEMS sensors on the probe by means of labels. The stainless-steel reinforced signal-sending cable is marked every 15 cm with nylon clamps and accumulated length identifications to facilitate the depth readings (Figure 15c).

7.1 Inclinometer probe body

Probe calibration revealed an excellent linear relationship between the digital readings from sensor S1 and the angle (Figure 16). The offset value was -57.4231 and the sensitivity value was $6500.6001 (R^2 = 1)$.

For sensor S2, a non-linear response was observed at reading values close to 10° and the magnitude of the

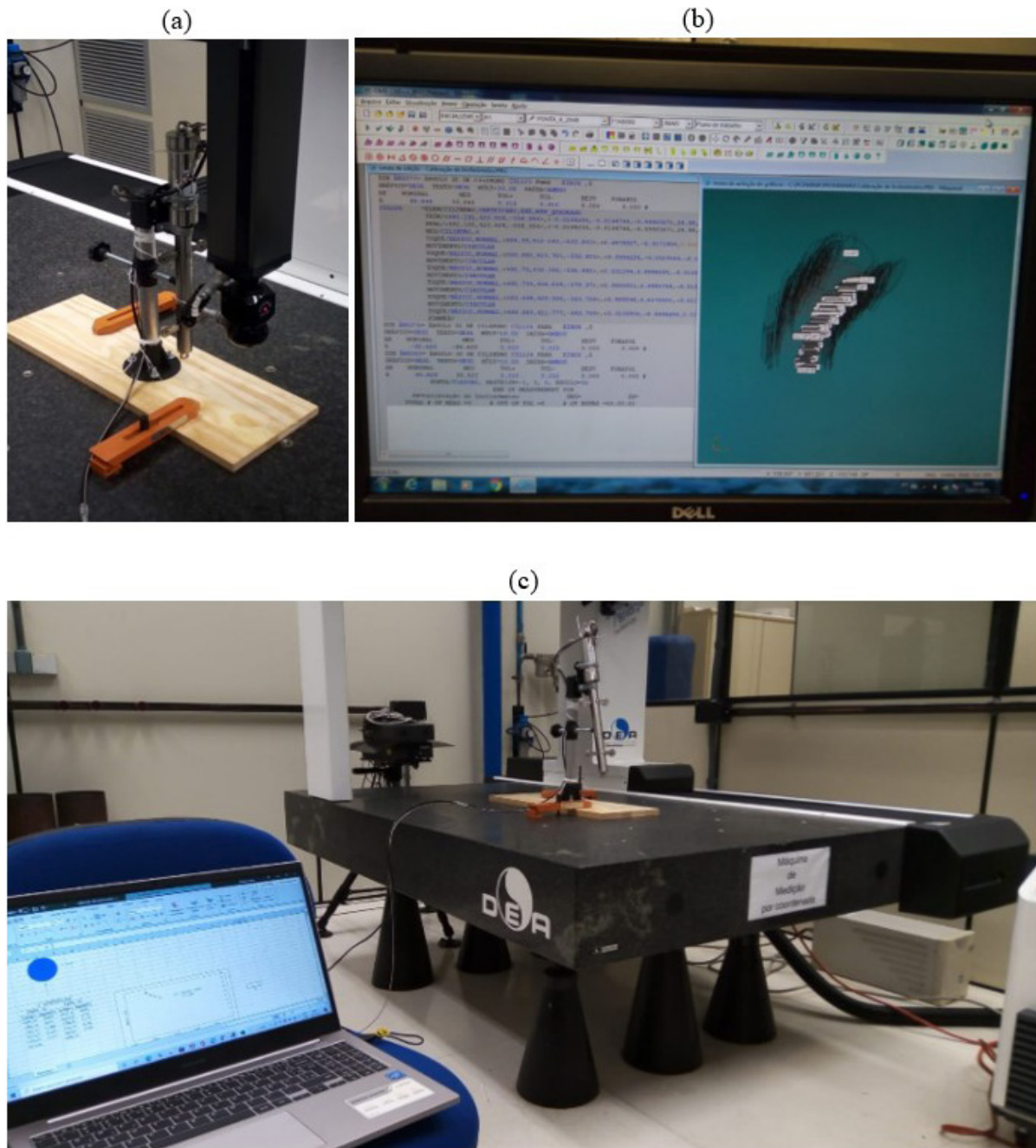


Figure 11. Inclinator probe calibration: (a) measurement; (b) virtual cylinder development; (c) comparison of measurements with digital readings.

digital readings was greater than that for sensor S1 due to analog/digital output conversion. In the main reading direction, the measured angles and digital readings correlated excellently ($R^2 = 0.985$; Figure 17a). However, noise was observed in digital readings ranging from -7000 to 8600 (Figure 17b) when calibrating the S2 sensor. These digital values from the main direction represent significant rotation angles and consequently satisfactory horizontal displacement.

7.2 Comparison of commercial and experimental inclinometer probe performance

Figure 18 shows the measured horizontal displacements as functions of the measured depth for both probes. The displacement curves for the x axis are similar (Figure 18a), reflecting efficient and accurate performance of the experimental probe with <0.5 mm variation. In contrast, significant discrepancies were observed along the y axis (Figure 18b).

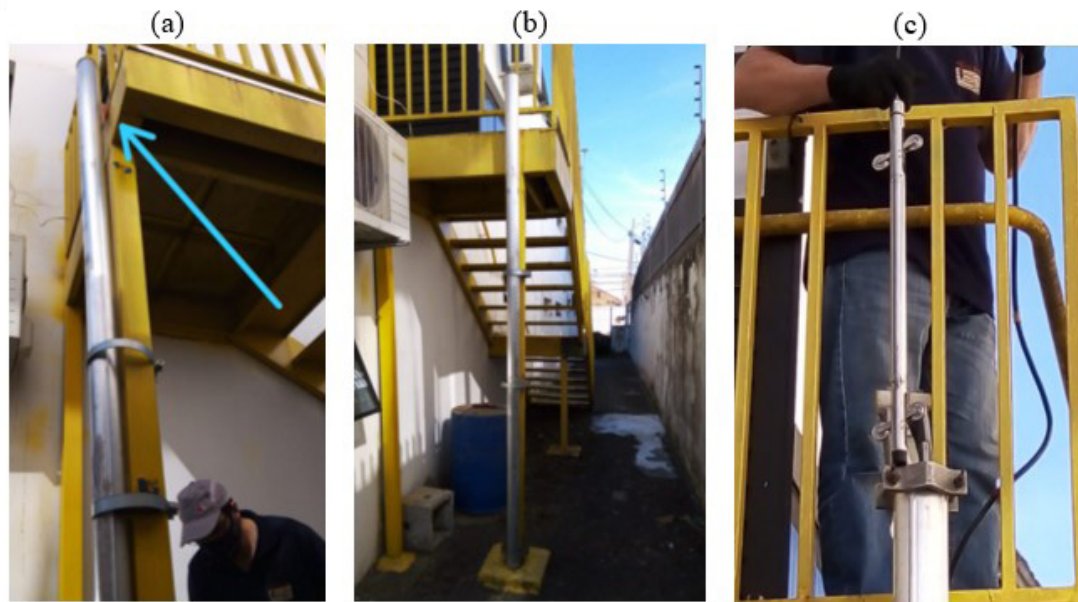


Figure 12. Guide tube used to validate the experimental probe: (a) distortion along the y axis; (b) distortion along the x axis; (c) introduction of the commercial probe into the guide tube.



Figure 13. Inclinometer probes used: (a) Digitilt Datamate II; (b) experimental probe coupled to a notebook.

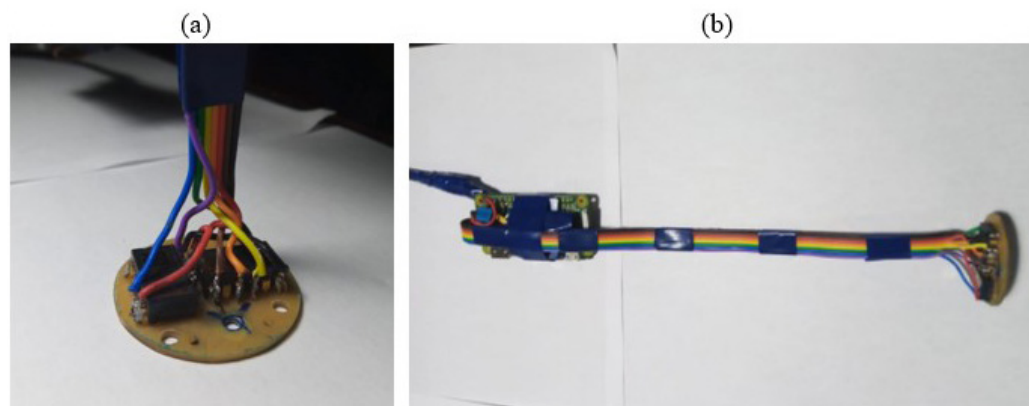


Figure 14. Construction of the inclinometer's electronic circuit: (a) the inclinometer installed on the base; (b) the reading electronic circuit.

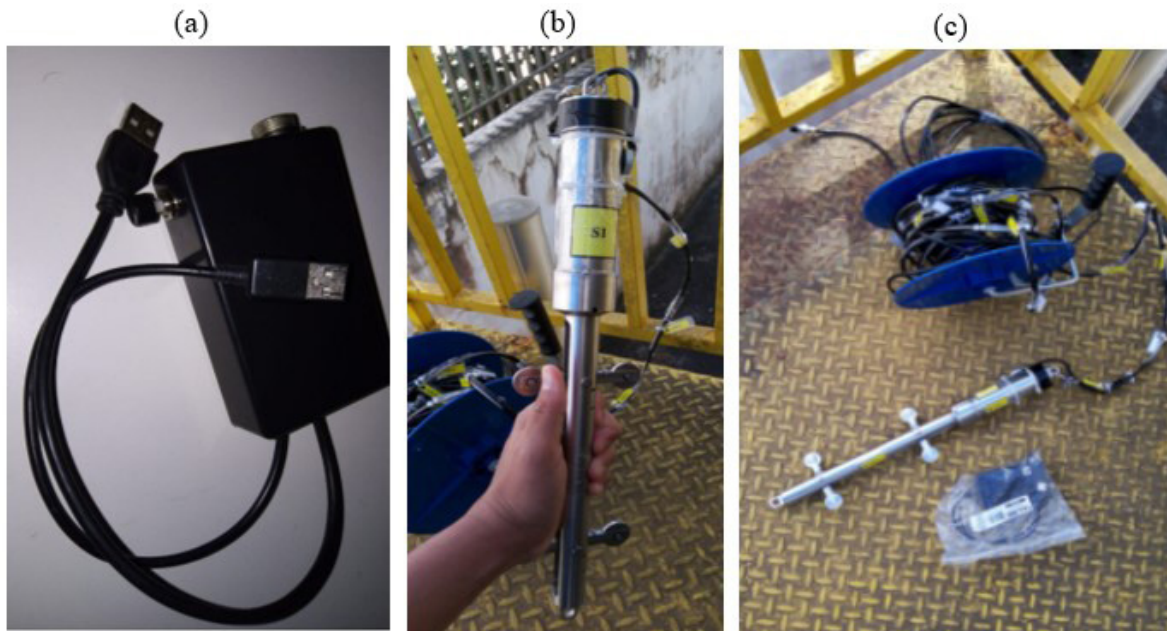


Figure 15. UFRGS inclinometer: (a) black box for communication; (b) inclinometer probe; (c) reading system components.

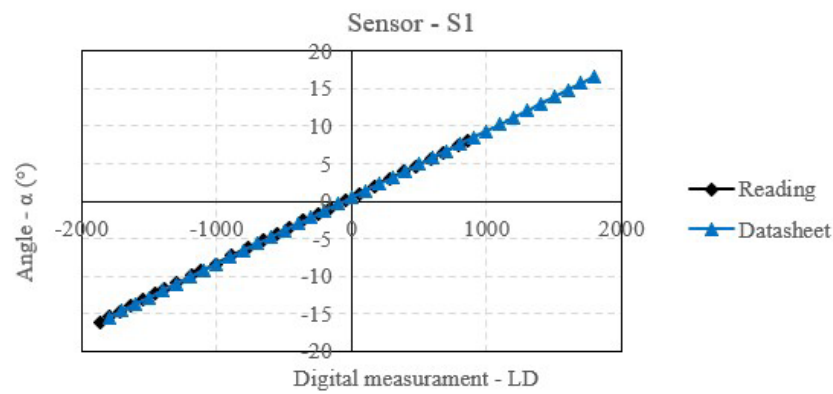


Figure 16. MEMS sensor S1 calibration.

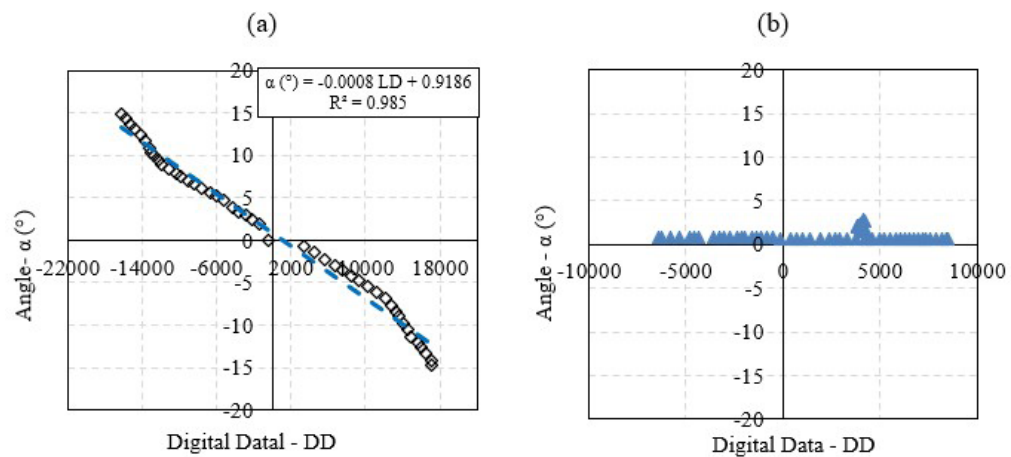


Figure 17. MEMS sensor S2 calibration: (a) principal direction; (b) direction perpendicular to the principal.

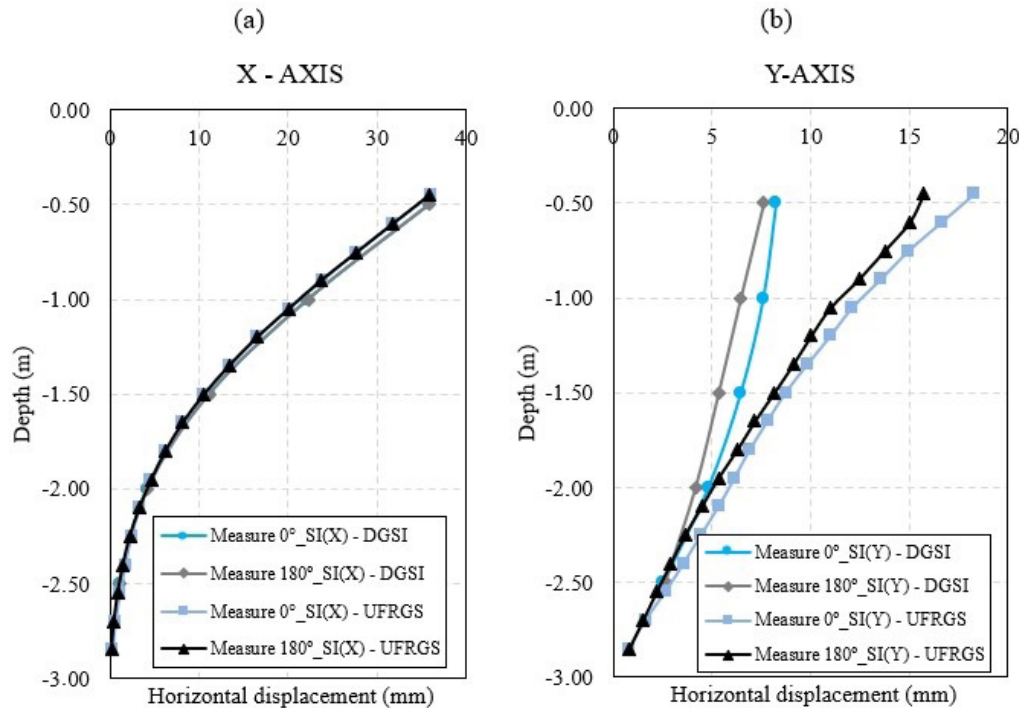


Figure 18. Comparison of results from the Digitilt Datamate II commercial probe and the UFRGS experimental inclinometer: (a) x axis; (b) y axis.

The greatest displacement measured by the commercial probe was approximately 8 mm, whereas that measured by the experimental probe was 17 mm. Considering that sensor S1 was used to obtain measurements along both axes, the low value generated by the commercial probe was considered to be inaccurate.

8. Conclusions

This work demonstrated that a low-cost probe for the measurement of horizontal displacement comparable to commercial probes can be developed with simple and accessible technology, such as precise MEMS sensors, microprocessor boards, and Python programming. This low-cost probe can feasibly be used for the monitoring of geotechnical structures (e.g., dams, reservoirs, slopes, mountains, and landfills) when management entities (e.g., mining companies, commodity exploration companies, and public institutions) do not have sufficient financial resources to implement extensive and safe site monitoring programs. Based on the results obtained, the following conclusions can be made about the inclinometer probe:

- The probe's measurement of displacement along the x axis was consistent with that of a commercial probe. This is an indication that the probe has been properly calibrated and valid;
- The three-dimensional probe calibration method used in this study was an applicable and accurate

alternative. Through the comparison study between the experimental probe and the commercial probe it was observed that there were few differences in the horizontal displacement graph along the depth.

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Declaration of interest

The authors have no conflict of interest, financial or otherwise, to declare. All authors have read and approved the contents of the paper.

Authors' contributions

Tennison Freire de Souza Junior: Conceptualization, Data curation, Methodology, Visualization, Validation, Writing – original draft (in Portuguese). José Lucas Silva Borges: Conceptualization, Data curation, Software (python algorithm), Methodology, Investigation. Cezar Falavigna Silva: Formal Analysis, Writing – original draft (in Portuguese), Translation to English – original draft. Writing – review &

editing. Karla Salvagni Heineck: Formal Analysis, Funding acquisition, Supervision, Resources, Writing – review & editing.

The authors above kindly granted permission to use parts of their publications in this paper.

Data availability

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

List of symbols and abbreviations

$tilt_n$	the amount of tilt angle in degrees
ARM	Acorn Risc Machine
COM	Communication Port
CPU	Central Processing Unit
HAT	Hardware Attached on Top
HDMI	High-Definition Multimedia Interface
K_r	pile stiffness, as a structural piece
L	distance between inclinometer wheels
MEMS	Micro-Electro-Mechanical Systems
NTSC	National Television System(s) Committee
Q_{ffset}	device output in the 0° tilt position
OTG	On the go
PAL	Phase Alternating Line
PUC-PR	Pontifical Catholic University of Paraná
RAM	Random Access Memory
UART	Universal Asynchronous Receiver-Transmitter
USB	Universal Serial Bus
$V_{D,out}$	output voltage of differential amplifier
V_{out}	differential amplifier output
$V_{out_{tilt1}}$	measured output in tilt 1 position
$V_{out_{tilt2}}$	measured output in tilt 2 position
δ_{DH}	displacement generated by the guide tube at a given depth
δ_{DHT}	total horizontal displacement (from top to fixed end)
ϕ_{ext}	external diameter

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Evaluation of limestone aggregates for railway ballast: particle characteristics and shear strength analysis

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Article

Keywords

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Abstract

The West-East Integration Railway (Ferrovia de Integração Oeste-Leste -FIOL) is an infrastructure project designed to integrate and promote the socio-economic development of the Northeast region of Brazil. In countries with heavy-haul railway operations, there has been increased demand for ballast materials that can withstand cyclic loading of up to 40 tons per axle. In this context, in order to achieve greatest operational efficiency, it is essential to evaluate the characteristics of the limestone aggregate to be used in the ballast layer, as well as the level of cyclic stress that the ballast can withstand. The laboratory investigation consisted of characterization tests, single particle crushing tests, monotonic triaxial tests, and evaluation of the morphological properties of the particles using the Aggregate Image Measurement System. This paper introduces the equipment, describes the experimental procedures, outlines the specimen preparation, and presents the experimental results. The purpose of the study is to estimate the long-term behavior of ballast based on monotonic triaxial tests, the shakedown phenomenon, and the cyclic stress ratio. The results suggest that the cyclic stresses commonly observed in heavy-haul railways may exceed the cyclic stress ratio compatible with the shakedown regime, resulting in a progressive plastic creep regime. Furthermore, the quantity of non-cubic particles exceeds the limits set forth in international ballast standards necessitating a rigorous assessment of the particle shape prior to use in the field.

1. Introduction

The West-East Integration Railway (FIOL, in Portuguese) is a cross-country infrastructure project currently under construction in the states of Bahia and Tocantins, Brazil. With an approximate length of 1,527 kilometers, the railway will connect the municipality of Figueirópolis (junction with the North-South Railway), in the state of Tocantins, to Porto Sul, located in the municipality of Ilhéus, in Bahia. The FIOL project is designed to facilitate integration and socio-economic development between the Northeast, North and Central-West regions of Brazil. The railway will enable the transport of agricultural, mineral, and industrial products via an efficient export route. Due to the importance of this project for Brazil, numerous researchers have studied the behavior of the materials that comprise the infrastructure layers of the FIOL (Santos et al., 2022; Gomes et al., 2023b).

The ballast layer of the FIOL will consist of a limestone aggregate. According to Selig & Waters (1994) and Raymond & Diyaljee (1994), the primary lithologies used in a railway ballast layer include limestone, gneiss, basalt, quartzite,

granite, rhyolite, and dolomite. Recently, recycled materials such as steel slag have been the subject of extensive research as an alternative to crushed rock aggregates for use in ballast layers (Delgado et al., 2019; Esmaeili et al., 2019; Esmaeili et al., 2020; Chamling et al., 2020; Jing et al., 2020; Guimarães et al., 2021; Indraratna et al., 2022; Gomes et al., 2023a). Despite this, the technical standards established by international institutions to guide the use of materials in the ballast layer prohibit the use of limestone (sedimentary rock) and alternative materials (REFER, 2015). The most recent Brazilian standard for railway ballast (ABNT, 2021) permits the use of various types of limestone and other lithologies. However, there is considerable concern regarding the efficiency of the limestone ballast to be used in the FIOL project.

The use of crushed rock in the ballast layer is one of the most demanding applications for these materials (Paixão et al., 2016). The stresses imposed by the passage of trains present the greatest challenge to the understanding and application of the materials that make up the ballast layer. Railway deterioration is primarily attributable to the permanent deformation of the ballast layer resulting from the

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passage of trains and track maintenance operations (Guo et al., 2018). A precise understanding of this matter helps to prevent accelerated deterioration and premature failures that can lead to interruptions in railway operations (Indraratna & Ngo, 2018). According to Delgado et al. (2022), there has been a noticeable increase in demand for ballast materials that can withstand cyclic loading of up to 40 tons per axle. This is mainly due to the growing interest in trains with heavier axle loads and longer train compositions, particularly in countries with heavy-haul railway operations. In order to achieve greatest efficiency in the FIOL rail operations, it is necessary to assess the level of cyclical stresses that the limestone ballast can withstand. Laboratory methods that can predict the shear strength of the crushed rock aggregate provide a valuable resource for evaluating ballast effectiveness and enabling more cost-effective designs.

The plastic behavior of ballast is influenced by the cyclic stress ratio (n) as defined in Equation 1 (Suiker, 2002). Suiker et al. (2005) found that for low values of the cyclic stress ratio, $n < 0.82$, the rate of permanent deformation at 1,000,000 load cycles becomes negligible. In other words, the cyclic response of the ballast becomes almost elastic, a phenomenon known as “shakedown” (Werkmeister et al., 2001). Therefore, it is essential to determine the shear strength of the ballast in order to characterize the level of cyclic stress to be used in repeated loading tests. This allows for the prediction of whether the response of the material will remain in the shakedown regime.

$$n = \frac{q_{cyc}}{q_f} \quad (1)$$

where: q_{cyc} is the range of cyclic deviatoric stress to be applied in the repeated load triaxial tests and q_f is the deviatoric stress at failure obtained from the monotonic triaxial tests under the same stress path.

A comprehensive understanding of the mechanical behavior, physical properties, and individual characteristics of the particles, including the crushing strength of the grain and the morphological characteristics of the aggregate to be used in the ballast layer, is essential for the successful implementation of this project. Laboratory research was conducted to verify that the material meets the specifications of the Brazilian standard for railway ballast (ABNT, 2021). Moreover, an estimate of strength and characterization of the level of cyclic stress to be imposed by the passage of trains is carried out to ensure that the resulting permanent deformation rate is negligibly small. This research represents a novel approach to estimating the behavior response of limestone ballast from monotonic triaxial tests. The methodology draws upon the shakedown phenomenon (Werkmeister et al., 2001), the stresses frequently observed in heavy-haul railways (Delgado et al., 2021) and the cyclic stress ratio (n) concept proposed by Suiker et al. (2005).

The present study involved a comprehensive laboratory investigation with the objective of evaluating the efficiency of limestone aggregate for use in the ballast layer of the FIOL. The mandatory tests specified in NBR 5564 (ABNT, 2021) were performed, in addition to some optional tests. In addition, the individual characteristics of the particles, including tensile strength and morphological properties (Figure 1), were analyzed using grain crushing tests and an advanced digital image processing technique, respectively. Finally, the shear strength of the material was evaluated using monotonic triaxial tests. The results were used to determine whether the cyclic stresses frequently observed in heavy-haul railways (Delgado et al., 2021) meet the cyclic stress ratio limit for the material to remain in the shakedown regime (Suiker et al., 2005). Finally, the conclusions drawn from the experimental approach are presented, along with a discussion of its limitations.

2. Materials and methods

2.1 Material description

The ballast used in the laboratory investigation is composed of crushed limestone particles. Ballast is produced by crushing rock and sieving it to obtain the desired particle sizes. The grain-size distribution of the ballast material is depicted in Figure 2. The ballast meets the gradation specified in AREMA No. 4 in the Manual for Railway Engineering (AREMA, 2020). The ballast is classified as poorly graded gravel, in accordance with the definitions of the *Unified Soil Classification System* (USCS) and the Brazilian standard NBR 6502 (ABNT, 1995), as evidenced by the values of the coefficient of uniformity (C_u) and the coefficient of curvature (C_c) of the particle size distribution PSD curve, which were

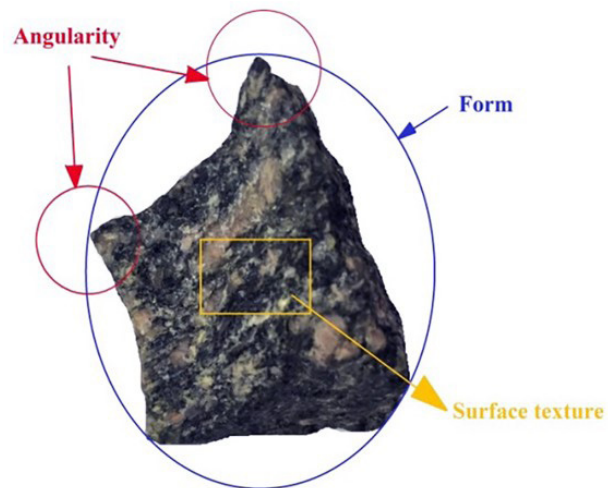


Figure 1. Shape characteristics of ballast particle (reproduced from Guo et al., 2018)

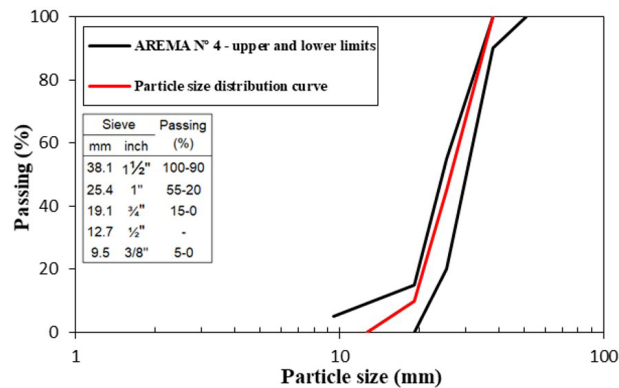


Figure 2. PSD curve of limestone ballast.

1.61 and 0.98, respectively. The primary rationale for using the AREMA N° 4 ballast standard was its use in main railway lines, thus allowing triaxial tests to be conducted without using the parallel gradation technique. This is because the use of scaling-down techniques still has limited acceptance among several authors (Indraratna et al., 1998; Klinecivicius, 2011).

A series of characterization tests were conducted to ascertain the physical properties of the ballast, including particle shape, the number of non-cubic particles, apparent specific mass, apparent porosity, water absorption, weather resistance, the unit mass limit in the loose state, powdery material, clay lumps, the Los Angeles abrasion index and Treton shock resistance index. All tests were conducted in accordance with the recommendations of the Brazilian standard (ABNT, 2021), which establishes specifications and test methods for railway ballast. The results were compared with the limits established by both the Brazilian standard and the AREMA Railway Engineering Manual (AREMA, 2020). Only the quantity of non-cubic particles exceeded the limit allowed by both ballast specifications, as shown in Table 1.

2.2 Specimen preparation

The preparation of the ballast specimen involved the placement and compaction of four layers of equal thickness via vibration within a tripartite mold. Initially, a latex membrane of 2.0 mm thickness was manually placed against the wall of the compaction mold. The thickness of the latex membrane was necessary to prevent puncturing of the membrane by the sharp edges of the ballast particles. The specimen compaction process involved the application of vibration to each layer for a period of one minute, followed by a fifth vibration cycle of two minutes, using a vibrating plate. Once the compaction process was complete, the test specimen was adjusted manually to ensure that the particles were optimally positioned at the top of the specimen.

To regularize the top surface of the specimens, a layer of gypsum plaster was applied, and the surface was smoothed with the use of a glass plate coated with petroleum jelly.



Figure 3. Specimen prepared for monotonic triaxial tests.

Finally, to correct the parameters of the PSD, the excess material was sieved and weighed, with the resulting mass subtracted from the initial mass. After 24 hours, the specimen was placed in the triaxial apparatus. Subsequently, the top cap was attached, and the tripartite mold was removed while maintaining a temporary internal vacuum to support the specimen (Figure 3). A similar procedure for preparing the specimens was carried out by Silva (2018), Rosa et al. (2021), Cescon (2021), and Gomes et al. (2023a).

Specimens prepared following the same experimental procedure exhibited consistent void ratios. The void ratio of the ballast specimens was found to be 0.83, 0.79 and 0.82. The process of compacting the test specimen was considered to be effective, as the resulting values for the initial void ratio (e_0), coefficient of uniformity, and coefficient of curvature were comparable to those reported by Indraratna & Salim (2003), Indraratna et al. (2007) and Anderson & Fair (2008) for ballast materials with uniform particle size distributions.

The tripartite mold used to prepare the ballast specimens has a nominal height of $H = 300$ mm and a nominal diameter of $D = 150$ mm, yielding an aspect ratio of $H/D = 2.00$, a recommended ratio to minimize the effects of friction at the ends of the sample (Bishop & Green, 1965). The maximum particle diameter (d_{max}) in the uniformly graded ballast material

Table 1. Ballast properties in comparison with international standard limits.

Property	Studied Limestone	NBR 5564 (ABNT, 2021)	AREMA (2020)
Average particle shape	Cubic	Cubic	Cubic
Non-cubic particles (%)	18	< 15	< 5
Apparent specific mass (kg/m ³)	2,703	> 2,600	> 2,600
Apparent porosity (%)	0.17	< 2	-
Water absorption (%)	0.06	< 2	< 2
Powdery material (%)	0	< 1	-
Clay lumps (%)	0	< 0.5	< 0.5
Unit mass limit in the loose state (kg/m ³)	1,385	> 1,250	-
Los Angeles abrasion index (%)	22	< 30	< 30
Treton shock resistance index (%)	12.3	< 25	-
Weather resistance (%)	4.75 (63.5 to 38 mm) 1.90 (38 to 19 mm) 3.47 (19 to 12.5 mm)	< 10	< 5

is 25 mm, as illustrated in Figure 3. This corresponds to a ratio between the specimen diameter and the maximum particle diameter of $D/d_{max} = 6$. Skoglund et al. (2000) recommended that a representative sample size should have a D/d_{max} ratio in the range of 5-7, and Indraratna et al. (1993) proposed a value of 6 at which the scale effects become negligible.

2.3 Experimental setup of monotonic triaxial tests

The shear strength of ballast under monotonic loading was investigated using a large-scale triaxial apparatus. Constant confining air pressure was applied to the acrylic chamber through an air compressor. A series of three isotropically consolidated drained triaxial tests (CID) were conducted, with constant confining pressures (σ'_c) of 40, 55 and 70 kPa. The confining pressures employed were maintained within the realistic range of 10-70 kPa, following the recommendations of Indraratna et al. (2013a). The strain rate used in the monotonic triaxial tests was 0.83 mm/min, which is consistent with the strain rates employed by Anderson & Fair (2008) and Aursudkij et al. (2009) in their respective tests.

In this type of test, failure is not easily defined due to the constant fluctuation in the deviator stress resulting from the “stick-slip” processes between individual ballast particles. In other words, the ballast particles that are in contact slide over each other, resulting in an irregular movement, characterized by brief accelerations with interruptions. These local instabilities are observed at the level of the specimen due to the large ratio between the average size of the ballast particles and the size of the specimen. Similar behavior was reported by Suiker et al. (2005), Anderson & Fair (2008), and Chamling et al. (2020).

The instrumentation consisted of a potentiometric displacement transducer installed on the top of the triaxial cell, a 2-ton load cell, a pressure gauge, a data acquisition system, and a notebook computer. Pressure regulation was provided through an external pressure control system.

The potentiometric transducer allowed displacements of up to 100 mm, which proved sufficient for the large deformations required. The experimental data was collected automatically every second, ensuring precise stress-strain curve definition during the tests. Unfortunately, it was not possible to measure the volumetric change during the tests, since the specimens were not saturated, and no instrumentation was used to measure the radial displacements.

All vertical and lateral stress measurements were corrected for the membrane effect in accordance with the methodology outlined in the standard procedure ISO 17892-9 (ISO, 2018).

2.4 Particle morphology analysis

Particle morphology has a direct impact on the performance and deformation of granular material layers, such as railway ballast (Saint-Cyr et al., 2009). However, the traditional methods employed for particle morphology evaluation are inadequate, resulting in inconclusive test results and controversial conclusions (Tafesse et al., 2012; Ionescu, 2004). Consequently, advanced digital image processing techniques have been developed (Tutumluer et al., 2009; Greer & Heitzman, 2017). These methods are more efficient than traditional methods and provide more accurate particle morphology evaluation and corresponding morphological properties, such as particle size distribution, volume, surface area, form (flat or elongated ratio, sphericity), angularity index and surface texture index (Mvelase et al., 2012). The advantages and disadvantages of different apparatuses used to assess morphological properties are presented by Guo et al. (2019).

In this study, the Aggregate Image Measurement System (AIMS) was used (Figure 4). The AIMS provides a means of quantifying morphological properties, including sphericity, angularity, and surface texture of the particles, through digital evaluation of a set of particles using bidirectional images correlated with the field of view depth (Al-Rousan, 2004).



Figure 4. Aggregate Image Measurement System (AIMS).

The apparatus is composed of one camera and two distinct types of illumination for capturing images with varying resolutions. The particles are initially deposited in a compartment of a rotating circular lighting table. The image capture software then carries out three asynchronous measurement cycles on the perimeter of the particles (Delgado et al. 2022).

AIMS quantifies sphericity through the three dimensions of the particle under analysis. According to Fletcher et al. (2003), the camera and microscope assembly provide projections of the particles, which are used to measure the longest, shortest, and intermediate dimensions of each particle. The Sphericity Index (*SP*) value varies within the range of 0–1, where an *SP* of 1 indicates that a particle has equal dimensions. Angularity is defined as the measure of the presence of sharp corners in particles (Masad, 2003). The AIMS software quantifies angularity through the gradient technique, whereby a higher gradient value indicates a higher angularity. The software estimates the changes in the inclination of the gradient vectors and returns an average Angularity Index (*AI*) value, ranging from 0 to 10,000, where a perfectly rounded particle has an *AI* of 0. Surface texture is defined as the irregularity of the particle surface on a very small scale that is not affected by the general grain shape (Al-Rousan, 2004). The AIMS software quantifies the particle surface texture through the wavelet method, which employs three separate images to provide details of the texture in the horizontal, vertical, and diagonal directions. The Texture Index (*TI*) value varies within the range 0–1,000, with *TI* equal to 0 corresponding to a completely smooth particle surface. Further details on the methods and techniques used by AIMS to determine

the morphological properties of particles can be found in Fletcher et al. (2003), Al-Rousan (2004), and Delgado et al. (2022).

The AIMS was compared against other test methods that measure aggregate morphology properties and the results were found to be satisfactory. The comparison was conducted based on statistical analysis in terms of accuracy, repeatability, reproducibility, costs, and operational characteristics (ease of use and interpretation of results). The analysis incorporated aggregates from different geographic locations, rock types and shape characteristics (Al-Rousan, 2004). In recent years, the AIMS has been employed by numerous Brazilian researchers engaged in the investigation of the morphological properties of their respective railway ballast (Rosa et al., 2021; Cescon et al., 2021; Delgado et al., 2022).

The morphological properties of the limestone aggregate were quantified through three sets of 50 particles with different nominal diameters (12.7, 19.0 and 25.4 mm) that comprise the PSD curve of the specimens. The criteria used to define the ranges for morphological properties classification were derived from the Ibiapina et al. (2018) proposal, as it is a classification system developed with aggregates obtained from various regions of Brazil.

2.5 Fracture strength of ballast grains

It is well established that particle fracture (tensile strength) plays a significant role in the behavior of crushable aggregates (McDowell & Bolton, 1998). The tensile strength of rock grains can be indirectly measured by diametral compression between two flat plates (Jaeger, 1967), as

illustrated in Figure 5a. In their respective works, Lee (1992) and McDowell & Bolton (1998) describe the characteristic particle tensile strength (σ_f) as follows:

$$\sigma_f = \frac{F_f}{d^2} \quad (2)$$

where: F_f is the maximum load corresponding to the fracture of the particle and d is the initial particle diameter.

In their 2001 study, Festag & Katzenbach distinguished between two primary mechanisms underlying grain crushing: grain fracture and grain abrasion. Grain fracture, which occurs under conditions of high stress, involves the separation of grains into parts of nearly the same size. In contrast, grain abrasion is an independent process that occurs at any stress level, resulting in detachment of very small particles from the grain surface. When the stress level is low in comparison to the particle strength, grain fracture may be limited. However, grain abrasion persists as a continuous phenomenon (Festag & Katzenbach, 2001).

As stated by Indraratna & Salim (2003), ballast degradation is a complex phenomenon influenced by multiple factors, including amplitude, frequency, number of load cycles, aggregate density, angularity of particles, confining pressure, and degree of saturation. However, the authors emphasize that the fracture resistance of the particles is the most significant factor that controls the breakage of ballast. The tensile strength of the particles has a direct inverse relationship with the degradation of the ballast.

The single particle fracture strength of limestone ballast was measured in the laboratory to evaluate the relationship between tensile strength and particle diameter under comparable loading and boundary conditions. For these tests, a uniaxial loading frame was used, with a strain rate of 1.27 mm/min. Since the ballast met the gradation specification of the AREMA standard No. 4, thirty-five particles with a diameter of 10–50 mm were selected for testing. Each particle was positioned between the top and bottom plates of the compression machine, allowing for the initial particle diameter to be determined

prior to the start of the test (Figure 5b). The maximum load corresponding to the fracture of the particle was recorded, and the tensile strength was estimated using Equation 2, which is consistent with the definition of tensile strength of concrete in the Brazilian test.

A larger number of tests were conducted, but many were subsequently excluded due to the following issues: i) particle breakage along a preferential plane due to the presence of discontinuities; ii) the breaking of grain edges which did not represent crushing rupture.

3. Results and discussion

3.1 Morphology of limestone particles

A qualitative analysis of the morphological properties was initially performed using three surface microscopy images and three 2D images of angularity obtained from the AIMS (Figure 6a and 6b, respectively). The images shown are of particles randomly selected from those analyzed. In terms of surface texture, the microscopic images appear to show different characteristics. Larger diameter particles appear visually smoother than smaller diameter particles. However, in terms of angularity, the different particle diameters do not appear to exhibit distinct characteristics.

After qualitative analysis of the images, the initial assumptions were validated by measuring the morphological properties of the limestone aggregate using the AIMS software. The results of the sphericity, angularity, and texture of the three sets of 50 particles from each of the nominal diameters that comprise the PSD curve of the specimens are shown in Figure 7.

Figure 7a illustrates the relationship between particle diameter and sphericity. It demonstrates that as particle diameter decreases, sphericity also decreases. This finding is consistent with that of Rosa (2019). However, despite this finding, the majority of the particles were classified as having low sphericity, with only a small number of particles

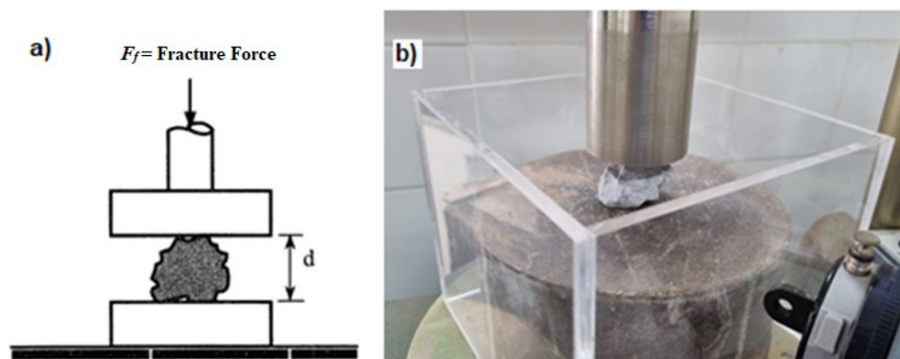


Figure 5. Single particle fracture strength test. (a) Test set-up (Indraratna & Salim, 2003); (b) Limestone ballast particle under test.

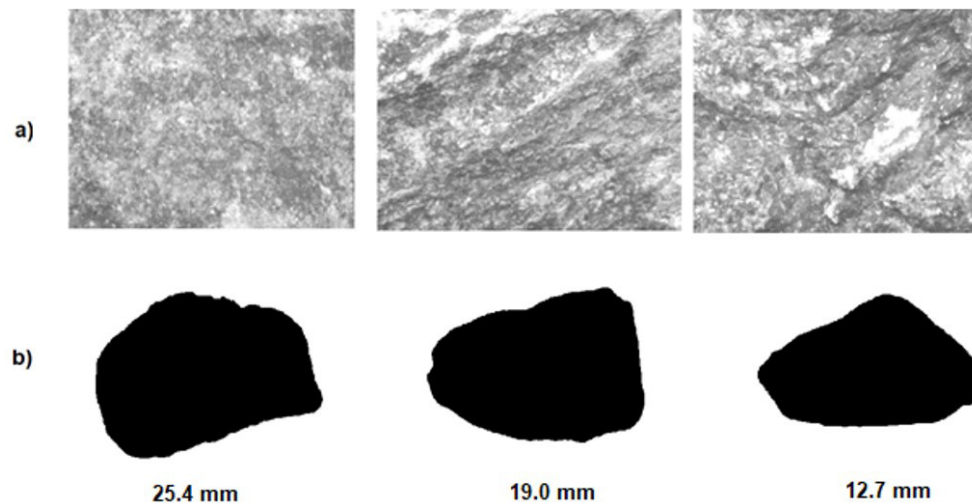


Figure 6. (a) AIMS surface microscopy images; (b) AIMS 2D images.

in the moderate sphericity range. Approximately 30% of the particles with a diameter of 12.7 mm were classified as flattened/stretched.

As noted in Al-Rousan (2004), sphericity values provide an effective indication of the proportions of a particle's dimensions. However, sphericity is insufficient for assessing the lamellarity of particles. Therefore, the F&E correlation, shown in a Zingg (1935) diagram, is necessary. In this correlation, the flatness ratio (F) is calculated as the ratio of the particle's shortest dimension (S) to its intermediate dimension (I), while the elongation ratio (E) is determined as the ratio of the intermediate dimension to the largest dimension (L). The Zingg diagram provides a comprehensive understanding of the lamellarity level of the particles and can serve as a valuable tool for establishing limits on the morphological characteristics of ballast particles (Delgado et al. 2022).

The F&E correlation for the limestone ballast is shown in Figure 8, along with the maximum acceptable F&E limits for ballast particles indicated in the Brazilian standard (ABNT, 2021) and the American standard (AREMA, 2020). Although the limit required by the Brazilian standard is more restrictive (1:2) compared to that indicated by the American standard (1:3), a wider tolerance range is allowed in Brazil (15%) than in the USA (5%), as shown in Table 1.

As illustrated in Figure 8 the majority of the particles analyzed in the AIMS exhibited a markedly variable shape, with a considerable dispersion of values in the Zingg diagram. A significant number of points fell below the recommended normative limits. The shape of the particles is not suitable for use as ballast material, as non-cubic particles tend to align themselves in a preferential direction (vertically or horizontally), forming planes of weakness, as well as being subject to greater wear due to abrasion and particle breakage.

The angularity of the particles was observed to be nearly identical, with the curves practically overlapping

(Figure 7b). This indicates that the angularity of the limestone ballast is not influenced by the different particle diameters, confirming the assumptions made in the qualitative analysis of the 2D images (Figure 6b). The angularity of the particles can be classified as subrounded in almost all cases, with a few particles in the subangular range.

Among the morphological properties measured, it is evident that the surface texture was most significantly influenced by the variation in diameters analyzed. There is a discernible tendency for the surface texture to increase as the particle diameter decreases (Figure 7c). As with the angularity of the particles, the Texture Index measurement conducted by AIMS corroborated the assumptions made through the surface microscopy images (Figure 6a). The majority of the particles analyzed were classified as low or moderate texture.

3.2 Tensile strength of ballast grains

Grain crushing is promoted by angularity, coarseness, uniformity of gradation, low particle strength, stress level and anisotropy. However, according to Indraratna & Salim (2003), the grain resistance to fracture is the most important factor. Figure 9 shows the tensile strength of thirty-five limestone ballast particles, with a diameter of 10-50 mm. There is considerable dispersion in the tensile strength results. It is important to note that tensile strength decreases with increasing grain size. This trend has been similarly documented by McDowell & Bolton (1998), Nakata et al. (2001), Indraratna & Salim (2003) and Delgado et al. (2021), for a variety of materials. This phenomenon can be attributed to the fact that larger particles tend to contain a higher number of flaws or defects, thereby increasing the probability of a defect being present in the particle that will break (Lade et al., 1996).

The lower tensile strength obtained in the particle crushing tests suggests that the larger diameter particles

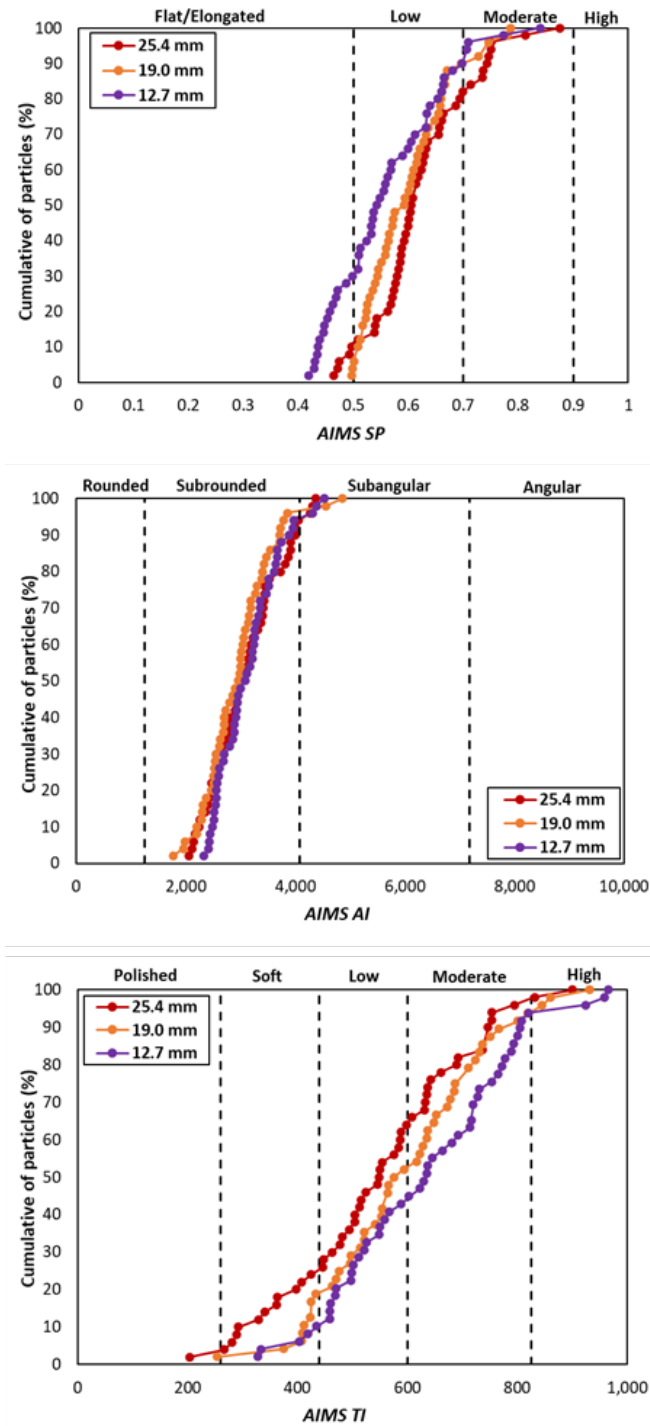


Figure 7. AIMS particle morphology measurement: (a) Sphericity; (b) Angularity; (c) Surface texture.

are subject to more pronounced grain breakage. This can be quantified after the long-term permanent deformation tests. However, this premise cannot be evaluated due to the impossibility of conducting repeated load triaxial tests, which is a limitation of the current study.

3.3 Monotonic triaxial tests

A series of three isotropically consolidated drained triaxial tests were conducted with constant confining pressures of 40, 55, and 70 kPa. Based on the test results, the shear strength

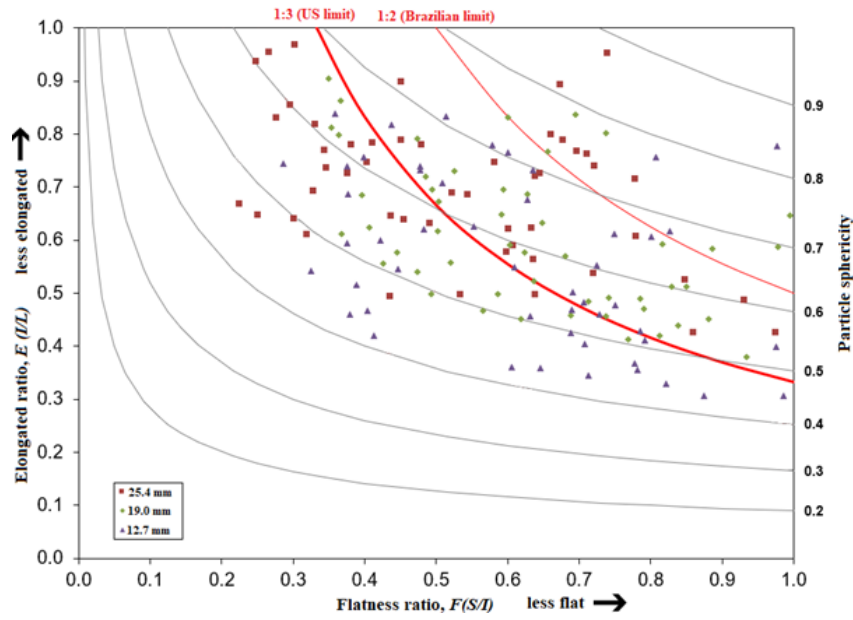


Figure 8. $F&E$ correlation for limestone ballast particles presented in a Zingg diagram.

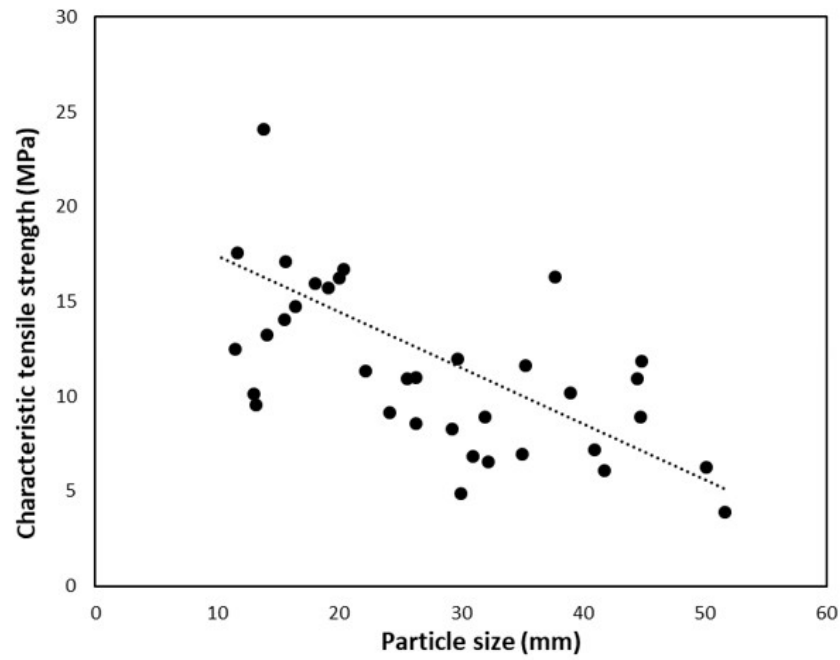


Figure 9. Single particle crushing strength test results.

parameters of the limestone aggregate were determined, and the shear strength of the ballast was compared with the stresses frequently observed on heavy-haul railways. Consequently, the resulting cyclic stress ratios were evaluated to ascertain whether they fell within the limit value of 0.82, as proposed by Suiker et al. (2005), to ensure that the ballast remained in shakedown regime. As stated by Delgado et al. (2022),

the cyclic deviatoric stress (q_{cyc}) values of 280 and 350 kPa are representative of heavy-haul freight trains with 32.5 and 40 t/axle, respectively.

The shape of the grains affects contact tensions. Due to the typical particle size of ballast, grain-to-grain contact is very intense, as there is no matrix of fines to reduce these contacts. The high concentration of non-cubic particles in

the analyzed limestone ballast suggests that the shape of the grain may influence the macroscale behavior. However, this effect has not been fully modeled and will be verified in future research.

The stress-strain curves, measured at each confining pressure, have been plotted in Figure 10a. Initially, it was found that the behavior of the ballast during shear was non-linear. As the confining pressures increased, the deviatoric stress also increased. Additionally, ballast softening was absent during shear, which is attributed to the loose arrangement of ballast particles due to the uniform character of the particle size distribution adopted (AREMA N. 4, see Figure 1). A comparable result was reported by Suiker et al. (2005), who also conducted tests with a PSD curve that met the specifications of AREMA N. 4.

The observed lack of smoothness in the curves may be attributed to the occurrence of “stick-slip” during shearing, breakage of the ballast, or a combination of both. Anderson & Fair (2008) also reported this phenomenon when they conducted monotonic triaxial tests on granite ballast of uniform granulometry with confining pressures varying between 40 kPa and 140 kPa. Chamling et al. (2020) observed the same phenomenon in tests with steel slag ballast specimens

and granite ballast with confining stresses varying between 40 kPa and 80 kPa.

Despite the stress-strain curves exhibiting a lack of smoothness, stabilization of the deviatoric stress was considered for large deformations ($\geq 17\%$), when it was close to a horizontal trend. This stabilization was considered indicative of the material having reached the critical state, given that it was not possible to measure the volume change of the specimens during shearing.

Figure 10b depicts the Effective Stress Path (ESP) and the Critical State Line (CSL) for the three tests, plotted in the p' – q plane. Table 2 provides a summary of the monotonic triaxial test results. The friction angle at the critical state (ϕ'_{cv}), which is an intrinsic parameter of soils and granular materials, and the critical state friction coefficients (M), which were obtained for limestone aggregate, were 50.7° and 2.08, respectively. Aursudkij et al. (2009) also conducted monotonic triaxial tests with limestone ballast, using PSD curves characteristic of uniformly graded granular materials, a similar D/d_{max} ratio, comparable bulk density (Table 2) and similar strain rate. The results presented by those authors align with those obtained in the present research, particularly when the confining stresses were equivalent.

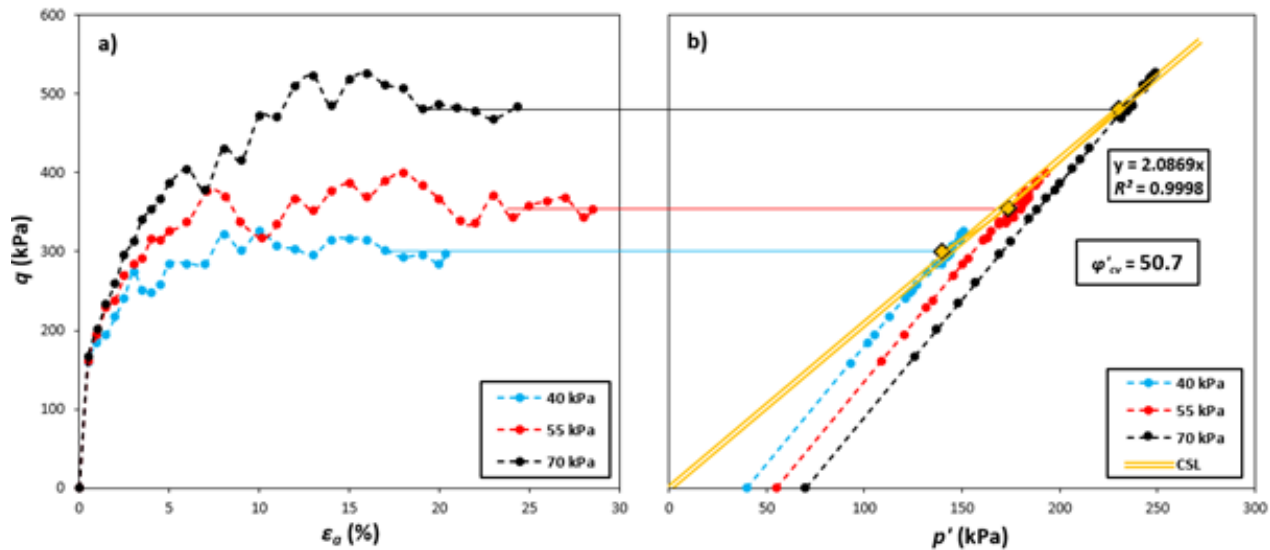


Figure 10. Monotonic triaxial test results of limestone aggregate: (a) stress-strain curves; (b) CSL envelope.

Table 2. Experimental characteristics and monotonic triaxial tests results.

Test	σ'_3 (kPa)	Density (kg/m ³)	e_0	q_f (kPa)	M	ϕ'_{cv} (°)
MT 1	40	1,475	0.83	300		
MT 2	55	1,507	0.79	355	2.08	50.7
MT 3	70	1,486	0.82	481		

Note: MT = Monotonic triaxial; $M = p'/q$ = Critical state friction coefficient; p' = mean effective stress; q = deviatoric stress.

Table 3. Cyclic stress ratio estimated using representative cyclic deviator stress of heavy-haul freight trains with 32.5 and 40 t/axle.

σ'_3 (kPa)	q_f (kPa)	q_{cyc} (kPa)*	n
40	300	280	0.93
55	355	280	0.79
70	481	280	0.58
40	300	350	1.17
55	355	350	0.99
70	481	350	0.73

* According to Delgado et al. (2022).

The cyclic stress ratio (n) was estimated using Equation 1, based on the deviator stress at break obtained in the monotonic triaxial tests and the cyclic deviator stresses representing heavy-haul freight trains with 32.5 and 40 t/axle, as proposed by Delgado et al. (2022). The results are shown in Table 3. As can be seen, for the cyclic deviatoric stress representing the 32.5 t/axle train, both the cyclic stress ratio for the test with 55 and 70 kPa of confining pressure yielded values below the limit of 0.82 proposed by Suiker et al. (2005). In other words, in these two cases, the permanent deformation rates are expected to be extremely small and therefore the material would be in shakedown regime. However, the same behavior was not obtained when considering the cyclic deviatoric stress representing the 40 t/axle train. The cyclic response of the ballast would be considered almost elastic only for the cyclic stress ratio of the test with 70 kPa of confining pressure. This phenomenon illustrates the significance of methodologies designed to increase the horizontal stresses that confine the ballast and, consequently, increase the stability of the track. Such methodologies include the use of geogrids (Indraratna et al., 2013b).

The results indicate that extreme caution should be exercised when the cyclic stress ratio exceeds the 0.82 limit, given that long-term behavior under cyclic loading can lead to a progressive plastic creep regime (Werkmeister et al., 2001). However, the long-term behavior of limestone ballast remains unproven due to the unavailability of repeated load triaxial tests.

4. Conclusion

Laboratory experiments were conducted to evaluate the efficiency of the limestone aggregate for use in the ballast layer of FIOL. The physical properties, tensile strength, morphological properties, and shear strength of the ballast were evaluated. In these experiments, all specimens were prepared in the same way, and the results demonstrated that the methodology used was effective. The specimens were classified as poorly graded gravel, and the PSD curve complied with the specifications of AREMA No. 4. The initial void ratio, coefficient of uniformity, and coefficient of curvature were comparable to those observed in previous studies using

different compaction procedures (Indraratna & Salim, 2003; Indraratna et al., 2007; Anderson & Fair, 2008).

Among the physical properties of the ballast, it was determined that the quantity of non-cubic particles exceeded the established limits set forth by the international standards analyzed. With regard to the morphological properties, the surface texture was found to be most influenced by the variation in diameters analyzed, with smaller particles exhibiting a higher Texture Index. Additionally, it was noted that smaller particles exhibited greater tensile strength. The findings suggested that smaller particles tend to show a reduced incidence of grain breakage after long-term permanent deformation tests.

The monotonic triaxial tests were conducted up to large deformations, allowing for the determination of the shear strength parameters and the critical state line. The cyclic stress ratio (n) was identified as a critical parameter in predicting ballast behavior. Therefore, the increase in axle loads of trains traveling on railways is a phenomenon that must be carefully assessed. The results suggest that the shakedown phenomenon, which is characterized by an almost elastic behavior, occurred in the case representing the 32.5 t/axle train, when the confining pressures were 55 kPa and 70 kPa. Conversely, for the 40 t/axle train, the shakedown regime was reached only at the highest levels of confining pressure.

The first limitation of the research was the impossibility of measuring the volume change of the specimen during the monotonic triaxial tests. Consequently, it was not possible to determine whether the behavior of the ballast was dilatant or contractile during shear. In this research, it was not possible to carry out repeated load triaxial tests to assess the long-term behavior of the limestone ballast and, consequently, the grain size breakage. This represents the main limitation of the present experimental investigation. To achieve greater accuracy, it would be advisable to conduct repeated load triaxial tests with the deviatoric cyclic stress proposed by Delgado et al. (2022) and evaluate the rate of permanent deformation at 1,000,000 load cycles. Furthermore, a logical next step would be to sieve the specimens, following the completion of the tests in order to quantify the extent of grain breakage in terms of the index of particle breakage, B_g (Marsal, 1965) and the ballast breakage index, BBI (Indraratna et al., 2005).

The experimental research yielded crucial insights regarding the efficiency of the limestone ballast under study, as well as providing prediction of the behavior of this material in the face of the cyclic stress frequently observed in railways. To ensure the durability and efficiency of the railway ballast material, it is crucial that the particle shape specifications are met. Nevertheless, further laboratory tests, such as the repeated load triaxial test, are necessary to validate the assumptions made in this study.

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Declaration of interest

The authors have no competing interests to declare. All co-authors have reviewed and approved the content of the paper and have no financial interests to declare.

Authors' contributions

Guilherme Faria Souza Mussi de Andrade: conceptualization, formal analysis, investigation, methodology, data curation, validation, visualization, writing – original draft. Cid Almeida Dieguez: investigation, data curation. Bruno Teixeira Lima: supervision, validation, resources – review & editing. Antonio Carlos Rodrigues Guimarães: supervision, validation, resources – review & editing.

Data availability

The data sets generated and analyzed in the course of the current study are available on request from the corresponding author.

List of symbols and abbreviations

d	Initial particle diameter
d_{max}	Maximum particle diameter
e_0	Initial void ratio
n	Cyclic stress ratio
p'	Mean effective stress
q	Deviatoric stress
q_{cyc}	Cyclic deviatoric stress
q_f	Deviatoric stress at failure
AI	Angularity Index
AIMS	Aggregate Image Measurement System
BBI	Ballast breakage index
Bg	Marsal's index of particle breakage
CAPES	Coordenação de Aperfeiçoamento de Pessoal de Nível Superior
C_c	Coefficient of uniformity
CID	Isotropically consolidated drained
COPPE	Instituto Alberto Luiz Coimbra de Pós-Graduação e Pesquisa em Engenharia
CSL	Critical State Line
C_u	Coefficient of curvature
D	Specimen diameter
E	Elongation ratio
ESP	Effective Stress Path
F	Flatness ratio
FIOL	The West-East Integration Railway
F_f	Maximum fracture load
H	Specimen height

I	Particle's intermediate dimension
ISO	The International Organization for Standardization
L	Particle's longest dimension
M	Critical state friction coefficients
MT	Monotonic triaxial test
PSD	Particle size distribution
R^2	Coefficient of determination
S	Particle's shortest dimension
SP	Sphericity Index
TI	Texture Index
USA	The United States of America
USCS	Unified Soil Classification System
2D	Two dimensional
ε_a	Axial strain
φ'_{cv}	Friction angle at a constant volume (critical state)
σ_f	Characteristic particle tensile strength
σ_3	Confining pressure

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Effect of rock and liner properties, and environmental conditions on thin spray-on liner shear bond

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Article

Keywords

Thin spray-on liners
Shear-bond
Liner thickness
Curing period
Oil

Abstract

Thin spray-on liners (TSLs) are a promising rock support method due to their ease of application and versatility. However, their effectiveness relies on experience and lacks a complete scientific understanding. To bridge some of the knowledge gaps existing in support mechanisms, this study investigated the effect of different parameters on the shear-bond strength of TSLs as a surface support in underground excavations. The parameters included liner thickness, curing period, material type and surface cleanliness. A polymer based cementitious TSL was utilized while the material type included quartz and sandstone rocks and 30 MPa concrete. Liner thicknesses included 16.2 mm, 20.2 mm, 24.2 mm, 28.2 mm, and 32.2 mm while curing periods were 7 days, 14 days, and 28 days. It was observed that the TSL shear-bond strength varied with different materials. Quartz with the lowest porosity had the lowermost shear-bond strength at all liner thicknesses and curing periods. The shear-bond strength improved with liner thickness up to 24.2 mm and then decreased up to 32.2 mm due to the optimum thickness effect. It also increased with curing period. Oil reduced the shear-bond strength for all the material types at all liner thicknesses and curing periods.

1. Introduction

The increasing health and safety measures in the mining industry has added to the worldwide market pressure to improve profits' margins. According to Webber-Youngman & van Wyk (2009), fall of ground (FOG) fatalities represented 39% of total number of fatalities in underground mines in South Africa between August 2006 and March 2009. These statistics are similar to those of other mines worldwide where FOG fatalities account for about 30-40% of all mining fatalities. In 2021, there were an estimated 2,933 FOG fatalities in the mining industry, of which 2,478 were in underground mines. According to the Mine Safety and Health Administration (MSHA), fall of ground (FOG) was the leading cause of fatalities in U.S. coal mines from 2010 to 2020, accounting for 39% of all fatalities (Imam et al., 2023). In China, fall of ground (FOG) fatalities accounted for 31.7% of all mining fatalities in 2021 a significant decrease from 40.5% in 2020 as stated by China Coal Industry Association (Kong et al., 2022).

A rockfall study by Potvin & Nedin (2003) conducted in 26 Australian underground metal mines showed that over 90% of rockfall injuries involved rocks smaller than 1 ton within a few meters of the active face. Therefore, lack of surface

support coverage can be singled out as the most obvious cause of rock falls in mining excavations (Daehnke et al., 1998; Klokow, 1999; Daenke et al., 1999). Rock tendons or props overall provide inadequate support for fragmented rock even in the case of grouting, and therefore blocks of rock from the excavation boundary can easily detach due to gravity, seismic activity, or blasting-induced vibrations. Additionally, support coverage can reduce weathering and unraveling of exposed rock surface (Ortlepp, 1983; Wojno et al., 1986; Applegate, 1987; Zhao et al., 2022).

Rock surface support is widely used to combat rockfalls and the resulting injuries and fatalities within the vicinity of active faces (Adams & Baker, 2002; Potvin, 2002; Lacerda & Rispin, 2002; du Plessis, 2021). The conventional (traditional) surface support systems in underground rock support include passive methods such as mesh and lacing, and active methods such as shotcrete (Kolapo et al., 2021; du Plessis, 2021; Zhao et al., 2022). However, statistical rockfalls associated with these methods over the years seem to suggest a steady increase in rockfall fatalities. They also negatively impact the mining operations with regards to costs, logistics and mine cycle times due to large material volumes (Tannant, 2001; Ozturk & Tannant, 2010; Ferreira, 2012; Roache et al., 2023). Therefore, there is a need for more enhanced systems and

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methods to improve safety standards in the mining industry (Li et al., 2017; Roache et al., 2023).

As a solution, since the 1990s, diverse kinds of Thin Spray-on Liners (TSLs) were developed with the aim of providing alternatives to the conventional methods (Stacey, 2001; Tannant, 2001; Yilmaz, 2011; Jjuuko & Kalumba, 2016; Dondapati et al., 2022). TSLs have various advantages over traditional methods as stated by various researchers including being lightweight, easy to install, flexible, durable, and cost-effective (Adams & Baker, 2002; Tannant, 2001; Hermanus, 2007; Yilmaz, 2011; Kolapo et al., 2021; Wang et al., 2024). TSLs have also been shown to be effective in preventing rockfalls and other ground instability problems. However, their application in the mining industry is still in its early stages. Therefore, their design as surface support systems is still based on experience, assumptions, field observations and cost considerations because the mechanisms by which TSLs act to provide support are not fully understood (Tannant, 2001; Leach, 2002; Saydam et al., 2003; du Plessis & Malan, 2021; Liang et al., 2021).

According to Tannant (2001), Jjuuko and Kalumba (2015) and Ozturk & Tannant (2010), the ability of a liner to resist displacements and fractures, to a greater extent, depends on a combination of shear-bond, tensile-bond and adhesive-bond between the liner and the rock. Though a lot of research has been done on the tensile and adhesive bond of TSLs to the rock substrate (Mason & Stacey, 2008; Ozturk & Tannant, 2010, 2011; Ozturk, 2012a, b; Yilmaz, 2013; Jjuuko & Kalumba, 2014, 2018), limited research has been done on the shear-bond strength of TSLs because early research suggested that shear-bond had negligible consequences on their performance (Espley-Boudreau et al., 1999; Espley-Boudreau, 1999; Liang et al., 2021). However, Stacey & Yu (2004), demonstrated that liner penetration into joints and fractures, where shear-bond is effective in resisting failure, results in a significant support mechanism of block interlock (Stacey, 2001). Therefore, quantification of shear-bond strength is essential in the formulation of design standards and requirements for TSLs.

In this study, the variation of TSL shear-bond strength with liner thickness, substrate type and surface cleanliness, and curing period was investigated. This is in line with previous studies, which have shown that the bond strength of liner materials is affected by several factors, including substrate surface roughness, substrate material, curing

time of liner materials, liner thickness, and environmental conditions (Chen et al., 2020; Shan, 2017). The liner was a polymer based cementitious TSL and substrates were rock and concrete. The aim of the study was to provide valuable insights into the factors that affect the shear-bond strength of TSLs. The information can be used to improve the design and application of TSLs to ensure that they provide effective rock support in underground mines.

2. Materials and methods

2.1 Experimental plan

The laboratory investigation involved three substrates of quartzite and sandstone rocks and concrete, and a cementitious TSL. The TSL thickness and curing period were varied as shown in Table 1. The study also involved dipping the substrates in oil before the preparation of test specimens to examine the effect of oil on TSL shear-bond strength. A total of five specimens were prepared for each testing point. Henceforth, a total of 1125 specimens were prepared for each substrate type. This gave 2250 specimens prepared for the whole study resulting in 2250 experimentation results obtained for shear-bond strength analysis.

2.2 Thin spray-on liner (TSL)

A polymer-based cementitious thin spray-on liner, Capcem KT Fast 2C was obtained. It was based on conventional cement and solvent free water-based polymer, supplied as separate cement and polymer components Table 2 shows further specifications of the TSL.

2.3 TSL mixing

A kitchen food mixer was used to mix the cement and polymer in the measured portions. It was the preferred method since it was easy and found to produce superior quality of TSL mixture. The material was mixed well, to avoid unmixed lobes, for five minutes at medium speed (speed 4/18000rpm). Small portions were consumed since small specimens were prepared. It was impractical to use a whole bag of cement. 1 L of polymer liquid was mixed with 3.5 kg of cement powder as suggested by the manufacturers and no water was added.

Table 1. Experimental plan.

Factors	Level	Code	Description
Substrate Type	3	Q	Quartzite
		S	Sandstone
		C	Concrete
Curing Time	3	T1, T2, T3	7 days, 14 days and 28 days
Liner Thickness	5	L1, L2, L3, L4, L5	16.2 mm, 20.2 mm, 24.2 mm, 28.2 mm and 32.2 mm.
Surface cleanliness	2	O1, O2	Clean and soaked in oil

An electrical weighing scale was used to weigh the TSL cement while a marked plastic tube was used to measure the polymer in the specified ratios. Manual pouring was used to apply the required TSL onto the specimens. Figure 1 presents the materials that were used in the study.

2.4 Rocks

A variety of different rock types that are or equivalent to mining field material were obtained from diverse sources as shown in Figure 2. These included, sandstone from Table Mountain slopes, Figure 2a; quartzite from Table Mountain slopes, Figure 2b, and equivalent to Wits gold rocks; sandstone, Figure 2c, d, f, and shale, Figure 2e, from Exarro Leeuwpan Colliery, Delmas, Mpumalanga. Leeuwpan is located 80 km south-east of Pretoria near the town of Delmas, Mpumalanga province. The Delmas coalfield is situated to the west of Witbank coalfield. The stones weighed around 40 kg and were roughly 400 x 400 mm² in size. The water content, dry density, porosity and UCS of the rocks was determined according to the “The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006”, the results are presented in Table 3.

From the initial tests, it was discovered that weaker rocks could not be utilized in the study. This was since their compressive strength was weaker than their shear-bond

strength to the TSL. Therefore, they crushed before attaining failure between the rock and TSL interface. The rocks in this category included shale and reddish weathered sandstone. Stronger rocks of quartz and sandstone were consequently employed in the study to investigate the effect of rock type on the TSL shear-bond strength.

2.5 Rock core preparation

The cores were prepared by diamond saw cutting using a portable power-driven rock coring machine. Diamond saw cutting process was highly recommended since it was found to result in comparable surface finishes. After cutting, the specimens were cleaned with water and checked for any faults. Those found with faults like cracks were discarded as they would cause deviations in the substrate roughness. The good specimens were air surface-dried and marked, Figure 3a, for cutting into smaller cores for TSL shear-bond testing using a circular diamond blade on a cutting machine. The specimens were cut flat, top and bottom. They were then air-dried for about four days under constant temperature (24-26 °C) and humidity (50-70%). This was to ensure all samples from the same rock type had the same moisture content before bonding with TSL. It was believed that moisture content would influence the bond between the TSL and substrate.

2.6 Concrete core preparation

Occasionally underground excavations are lined with concrete rings as a means of support on which liners are then sprayed. Table 4 gives details of the percentages of the different

Table 2. Specifications for Capcem KT Fast 2C (OMSSA Technical Data Sheet).

Properties (all components at 25 °C)	
Pump Life	40 Minutes
Fresh Wet Density	1777 kg/m ³
Strength Development (25 °C) – Typical	
Tensile Adhesive (28 day)	2.5 MPa
Tensile Adhesive (56 day)	5.4 MPa
Mixing	
Cement Powder (supplied in kit) (kg)	Polymer Liquid (supplied in kit) (litres)
19	5

Table 3. Determined rock properties.

Parameter	Rocks	
	Quartzite	Sandstone
Water content (%)	0.011	0.027
Dry density (kg/m ³)	1829.9	2229.9
Porosity (%)	5.6	14.2
UCS (MPa)	163	72



Figure 1. (a) Packaged cement, (b) Packaged polymer, and (c) Grey-brown cement in a bowl.



Figure 2. (a) Weak sandstone with faults from Table Mountain slopes, (b) Quartz from Table Mountain slopes, (c) Sandstone from Exarro, (d) Weak reddish weathered sandstone from Exarro, (e) Shale from Exarro, and (f) Sandstone from Exarro.

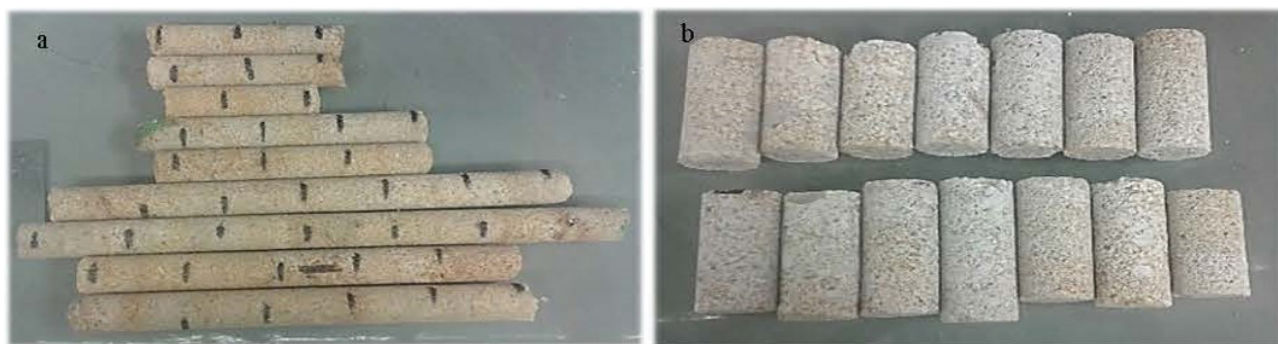


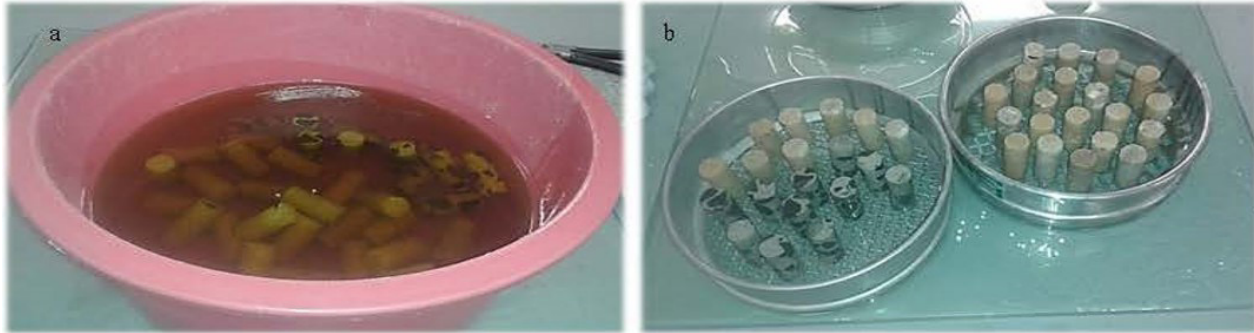
Figure 3. (a) Sandstone cores marked for cutting into specimens and (b) Sandstone core specimens.

Table 4. Details of 30 MPa concrete mix utilized.

W/B	0.7 %	Density (kg/m ³)	Mass (kg)	Batch Volume Volume (m ³)	40 L Batch Mass (kg)
Water	7.7	1000	185.0	0.185	7.40
CEM 1 52.9	11.0	3140	264.0	0.084	10.56
Stone greywacke (19 .0 mm)	43.7	2700	1050.0	0.389	42.00
Dune Sand	37.7	2650	906.0	0.342	36.24

Table 5. Concrete cube strength results.

Date of Cast	Date Tested	Sample Number	Dimensions (mm)	Weight (kg)	Dry Density (kg/m³)	Crushing Load (kN)	Ultimate Compressive Strength (N/mm²)
29/04/2015	21/05/2015	1	150x150x150	7,96	2358	652.5	29.0
		2		7,70	2282	642.0	28.5
		3				675	30.0
		Average Compressive Strength (N/mm²)					29.2

**Figure 4.** (a) Sandstone and concrete substrates submerged in oil for 24 hours, and (b) Air-drying the submerged substrates for 24 hours.

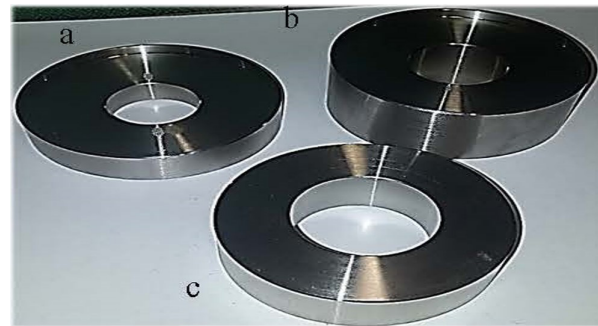
materials used in the mix for a 30 MPa concrete liner. 30 MPa was selected since it is one of the most used concrete strength in underground excavations. After the curing period of 28 days, out of the many prepared cubes, three were randomly selected and tested for the achieved UCS strength. Table 5 summarizes the test results. The remaining concrete cubes were then used in the preparation of cores for TSL shear-bond strength testing. The same procedure used for rock coring was again utilized here. Concrete cores used for TSL shear-bond testing had an average water content of 2.818%, average dry density of 2320 kg/m³ and average porosity of 9.8%.

2.7 Oiled cores

In order to investigate the effect of surface cleanliness on the shear-bond strength of TSLs, some of the rock and concrete cores were submerged in oil for 24 hours, Figure 4a. After 24 hours, they were removed from the oil and air-dried for another period of 24 hours, Figure 4b. They were air-dried standing on top of wire meshes with holes wide enough to allow the dripping of oil away from the bottom of the specimens. After 24 hours of air-drying, the oiled core specimens were used in the preparation of specimens for TSL shear-bond strength testing. The main source of oil contagion in underground rock excavations is from machinery hence hydraulic oil was used in the study. The oil was of CALTEX (Havoline Formula SAE 20W-50).

2.8 Moulds

The study utilized the circumferential shear-bond strength testing method developed by Yilmaz (2011). The moulds

**Figure 5.** (a) Steel rock centering ring, (b) Steel support ring, and (c) One of the steel rings.

included a steel ring, steel rock centering ring and a steel support ring, Figure 5, Figure 6 and Figure 7. Steel was preferred for the moulds due to its high strength, low cost and high corrosion resistance. In case of rusting, there is a possibility of interference with the bonding properties.

2.9 Specimen preparation

The steps followed during specimen preparation are demonstrated in Figure 8. Firstly, the steel centering ring was placed on a flat glass surface, Figure 8a. This was to ensure that the specimen does not move when TSL is poured. The top surface of the centering ring was then slightly oiled to prevent the TSL from bonding to it during setting. The steel ring was then placed on top of the centering ring, Figure 8b. The core was centrally positioned in the inner hole of the centering ring, Figure 8c. This way



Figure 6. (a) Steel ring in steel rock centering ring and (b) steel ring in steel support ring.

it was also centrally located in the steel ring. Afterwards, the TSL components consisting of cement and polymer were mixed according to the manufacturer's specifications of 19 kg to 5 L respectively. The cement was measured using an electronic scale, Figure 8d, while the polymer was measured using a plastic measuring tube, Figure 8e. TSL was then poured into the space between the core and the ring, Figure 8f. A putty knife was used to assist in the settling of the TSL in position and avoid air being trapped within the TSL. Flattening of TSL's exposed top surface to ensure uniform TSL thickness was done by tamping a sheet of plastic against the surface. Masking tape was used to prevent the TSL from coming into contact with the core surfaces where bonding was not wanted.

2.10 Test set-up and execution

The prepared specimen, after curing for the required time, was carefully placed on top of the steel support ring, Figure 9a, and then positioned in the Zwick loading machine (Material prufung – 1474), Figure 10. A spherical seat was used on the top surface of the substrate in order to ensure that the load was uniformly distributed. The machine platen, with springs again for uniform distribution of load, was attached to the spherical seat. The specimen was then made to just make contact with the machine platen. Compression loading was done in displacement control mode. The specimen was initially loaded up to 5 N at 0,001 mm/s for alignment and setting, and then at 0,002 mm/s up to failure. Load and machine displacements were recorded automatically by the Zwick machine. The load at failure was noted for each tested specimen.

After each test, the Zwick machine was used to drive out the rock core by means of a smaller diameter steel rod, Figure 9b. The TSL was also pushed out of the steel ring in the same way by using another steel rod that was a few mm smaller than the inside diameter of the steel ring, Figure 9b. The height of the pushed-out TSL, Figure 9c, was measured evenly at three positions in order to calculate the de-bond area. The average of the measured height was calculated.

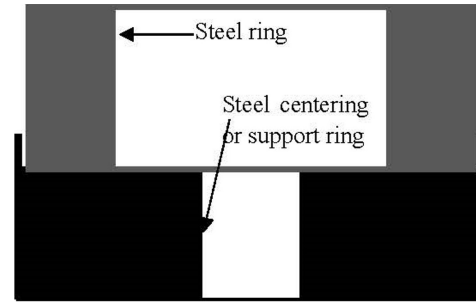


Figure 7. Schematic diagram showing the steel ring seated in either the steel centering or support ring.

2.11 Calculations

The boundary conditions on the specimen assembly are shown in Figure 11. The load is applied on the top surface of the core. The reaction takes place on the fixed support base indicated by red lines. The green lines are stress free surfaces. The bonding of core and TSL, and eventual shearing takes place at the yellow lines. For rocks where the core strength is not reached, full failure takes place at the TSL-core contact surface. Shear movement on the TSL-core boundary develops shear stress (τ_b) calculated as presented in Equation 1, proposed by Yilmaz (2011).

$$\tau_b = \frac{F}{\pi D t} (Pa) \quad (1)$$

where:

F = Maximum applied force;

D = Core diameter;

t = Depth or steel ring height.

3. Analysis and results

3.1 Statistical analysis of results

For statistical validity and comparison of test results, specimen preparation and testing should be repeatable.

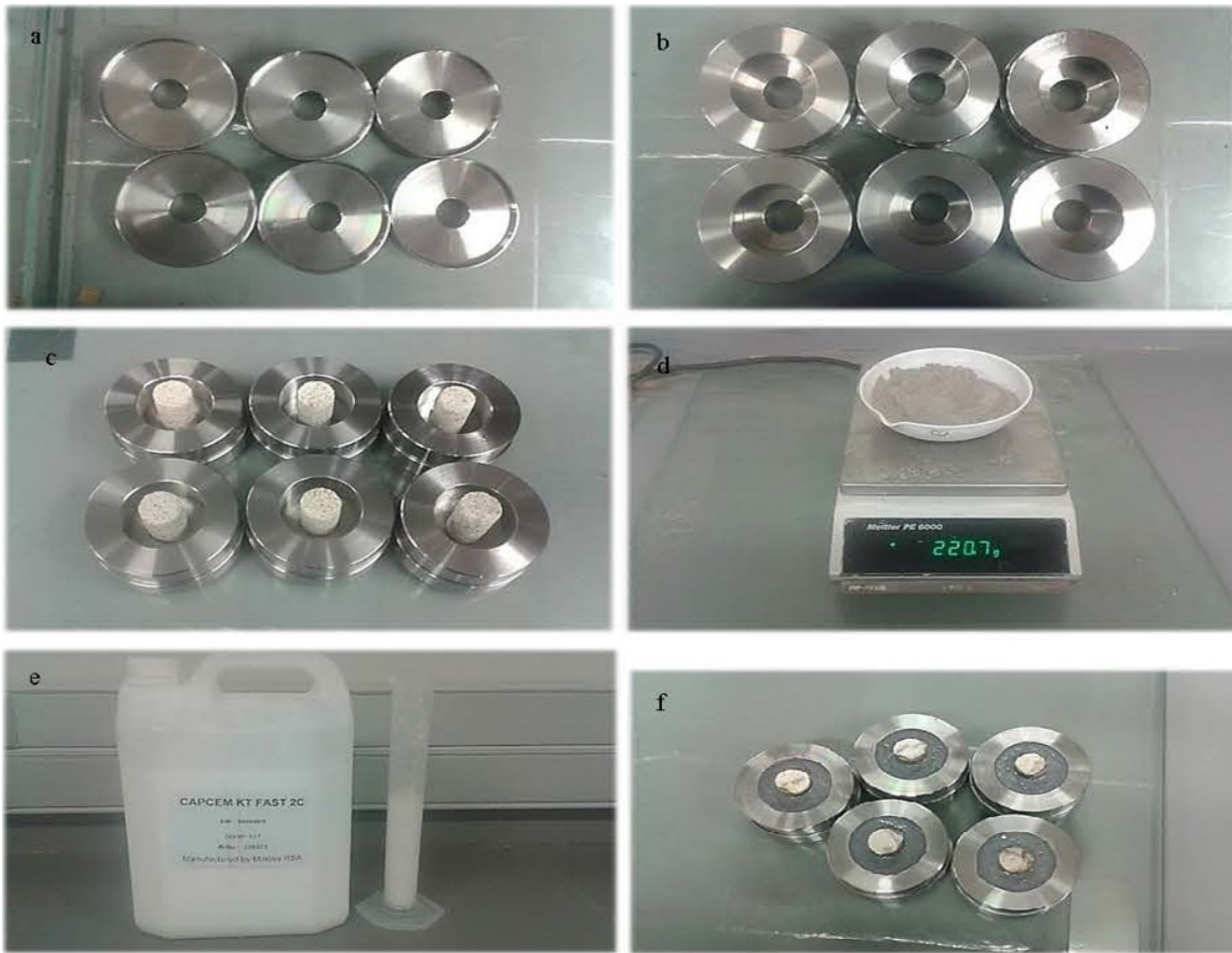


Figure 8. (a) Centering ring placed on flat glass, (b) Steel ring seating on centering ring, (c) Sandstone substrate seated in the steel ring and centering ring, (d) Measuring the required amount of cement, (e) Measuring the required amount of polymer, and (f) Prepared specimens.



Figure 9. (a) Specimen on steel support ring ready for testing, (b) Steel rods for driving out the substrate and liner, and (c) Liner driven out of the steel ring after testing.

The repeatability of the test results was investigated with calculation of repeatability standard deviation and coefficient of variation. The standard deviation (SD) ranged between 0.1 and 2.1. The oiled substrates had all their SD values either equal or under 0.6, hence demonstrating high consistency. The Coefficient of Variation (CV) varied between 3.0% and 25.4%. It should be noted that CV is a better parameter to quantify the dispersion of shear-bond strength values. There was no definite trend in the spread of SD and CV over liner thickness, curing period and substrate. A maximum CV of 25.4% is a good sign of uniformity of the test results obtained. The smaller the calculated values of SD and CV, the more repeatable the testing methodology. The statistical analysis of the laboratory results showed that they were uniform and consistent. Hence, they could be employed in the analysis and conclusion of the variation of shear-bond strength with investigated parameters.

3.2 Variation of shear-bond with liner thickness

Liner thicknesses affect the interpretation of TSL test results. Shear bond strength is the ability of a thin spray-on

liner (TSL) to resist shearing forces when it is bonded to a rock substrate. Liner thickness is the distance between the top and bottom surfaces of the TSL. There is a relationship between shear bond strength and liner thickness. Previous studies have shown that, generally, shear bond strength increases with increasing liner thickness. This is because thicker liners have more surface area to bond to the rock substrate. Several studies have been conducted to investigate the bond strength of TSLs (Ozturk & Tannant, 2010; Chen et al., 2020; Yilmaz, 2011). The results of these studies show that the bond strength of TSLs can vary widely. Yilmaz (2011), demonstrated that the shear bond strength increased from 1.5 MPa to 4.5 MPa as the liner thickness increased from 2 mm to 5 mm. Chen et al. (2020) reported adhesive strength from pull-off tests varying between 0.97 MPa and 2.86 MPa for liner thicknesses of 8 mm and 1 mm respectively.

In this study, liner thicknesses of 16.2 mm, 20.2 mm, 24.2 mm, 28.2 mm, and 32.2 mm were utilized to investigate the effect of varying liner thickness on TSL shear-bond strength. Figure 12 illustrates the variation of shear-bond strength with liner thickness for the substrate of Quartzite, Sandstone and Concrete respectively. Five tests were carried out for each liner thickness at a specific curing period. The averages of the respective shear-bond strengths were used to come up with the curves of variation as shown on the graphs.

It was believed that another set of liner thicknesses may show the same variation, as obtained in the graphs, with a peak point at a different liner thickness depending on the contact shear area at the start of the test. To overcome this constraint, an aspect ratio was calculated as presented in Equation 2.

$$\text{Aspect ratio} = \frac{\text{Liner thickness (m)}}{\text{Shear area (m}^2\text{)}} (m^{-1}) \quad (2)$$

Aspect ratios of 13.0, 16.2, 19.4, 22.7, and 25.9 were utilized in this study. Graphs of shear-bond strength against



Figure 10. Zwick machine.

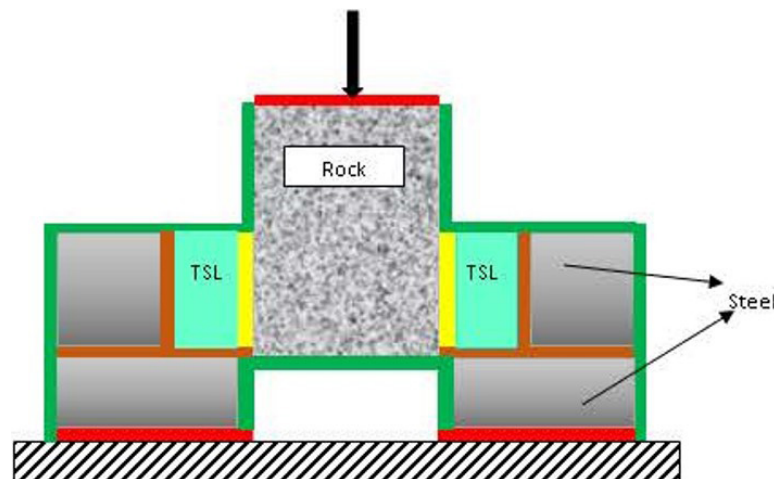


Figure 11. Boundary conditions in shear-bond strength testing.

aspect ratio were then plotted. The shear-bond strength generally increased with increase in aspect ratio up to 19.4 m^{-1} and then decreased. Initially, there was a small increment in shear-bond strength with the curve concave upwards followed by a sharp increment of a concave downward curve up to the peak value. After the highest value, there was a sharp decrement in shear-bond strength and then a slight decrement.

The variation of shear-bond strength with liner thickness is due to the optimum thickness effect. The bond strength between the liner and the substrate is affected by the surface area of the interface (Zhang et al., 2022). When the liner thickness is very thin, the surface area of the interface is small, which limits the bond strength. As the liner thickness increases, the surface area of the interface increases, which leads to an increase in the bond strength. However, when the liner thickness becomes too thick, the bond strength starts to decrease again. This is because the thicker liner becomes more difficult to apply evenly, which can lead to voids and other defects in the bond (Chen et al., 2020). In this study, the optimum thickness was 24.2 mm.

3.3 Variation of shear-bond strength with substrate

In laboratory TSL shear-bond testing, rocks associated with higher UCS values are desired. Their surfaces are competent, and their structure is more resistant to any type of loading. Weaker rocks have the risk of rock failure before shear failure between the substrate and TSL. This is in addition to rocks with cracks or any other structural weakness. The selection of substrates utilized in the investigation represented the main rock types encountered in the mining industry; quartzite for gold mines, sandstone for platinum mines and concrete cubes representing concrete ring covers. This was to contribute to the knowledge of TSLs in mining applications as one of the aims of this investigation.

The variation of shear-bond strength with substrate type at the respective curing days was plotted against the aspect ratio in Figure 13. It was noted that Quartzite had the lowest shear-bond strength at all liner thicknesses and curing periods while sandstone had the highest values.

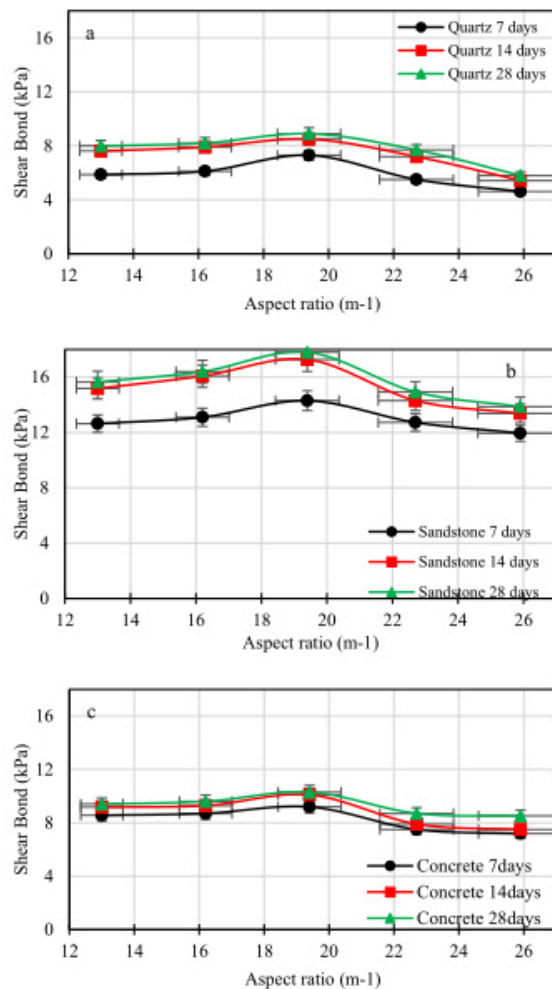


Figure 12. Variation of shear-bond strength with aspect ratio for (a) Quartz, (b) Sandstone and (c) Concrete.

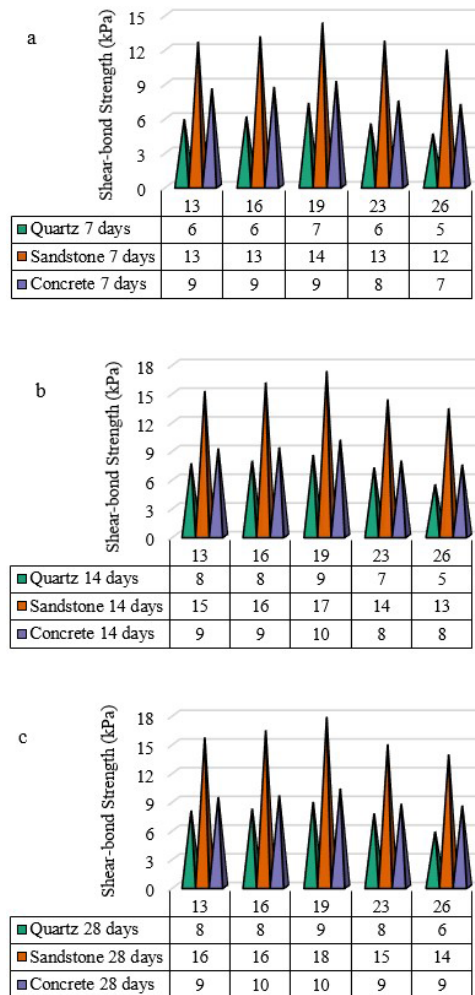


Figure 13. Variation of shear-bond strength with substrate type at (a) 7, (b) 14 and (c) 28 days of curing.

The shear-bond strength values of concrete substrate were close to those of quartzite substrate. At all liner thicknesses, the shear-bond strength of sandstone was more than 50% higher than that of quartz for all the curing periods. Concrete shear-bond strength values were higher than those of quartz by values ranging between 36.1% and 11.5% for all liner thickness and curing times.

Previous studies have shown that the shear-bond strength of TSLs is higher with concrete substrates than with rock substrates (Saydam et al., 2011; Chen et al., 2020). This is because concrete is more uniform and has a smoother surface than rock, which makes it easier to apply the liner evenly. Additionally, the surface roughness of the substrate can also affect the shear-bond strength. A rough surface provides more mechanical interlocking between the liner and the substrate, which can improve the bond strength. This is because the rough surface creates a larger surface area for the bond to form, and it also helps to prevent voids from forming between the liner and the substrate.

A study by Saydam et al. (2011) investigated the shear bond strength of TSLs with different types of rock substrates. The study found that the shear bond strength was higher for sandstone than for limestone. The study also found that the shear bond strength was higher for unweathered rock than for weathered rock. Chen et al. (2020) investigated the effects of substrate materials and liner thickness on the adhesive strength of a novel thin spray-on liner (TSL). The TSL was made from a water-based cementitious material and was applied to three different substrate materials: concrete, sandstone, and granite. The liner thickness varied from 2 to 32 mm. The results showed that the adhesive strength of the TSL was affected by both the substrate material and the liner thickness. The TSL had the strongest bond with concrete, followed by sandstone and then granite. Li et al. (2017) reported that the adhesion strength of TSLs increases with rock strength. This is because stronger rocks are more resistant to shear forces, which can help to prevent the TSL from delaminating from the rock surface. In the same study, the adhesion strength of TSLs also increased with surface roughness. This is because a rough surface provides more mechanical interlocking between the TSL and the rock surface, which can improve the bond strength. The optimum surface roughness for TSLs is typically in the range of 20–30 μm . This is because a surface that is too smooth will not provide enough mechanical interlocking, while a surface that is too rough can create voids and other defects that can weaken the bond.

3.4 Variation of shear-bond strength with curing time

Summary graphs of shear-bond strength against curing periods for each substrate and related aspect ratio are shown in Figure 14. The averages of the respective shear-bond strengths are indicated on the graphs as markers. The best fit curve was found to be logarithmic, and it was applied for

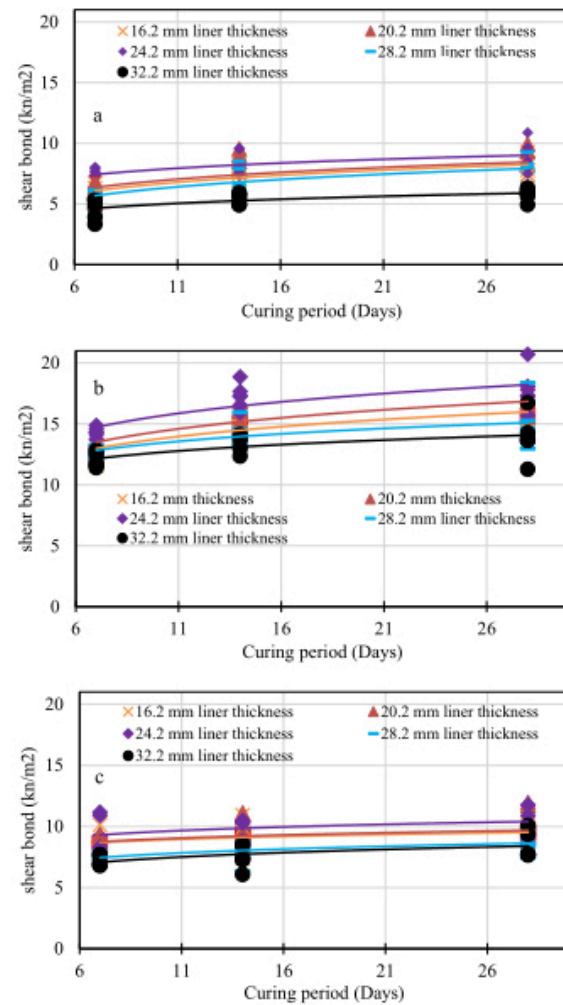


Figure 14. Variation of shear-bond strength with curing period for substrate of (a) quartz, (b) sandstone and (c) concrete.

all the data points on a graph. Additionally, equations and correlation coefficients (R^2) of the best-fit curves are shown. The logarithmic function was chosen because it gave higher coefficients.

Generally, the shear-bond strength increased with increase in curing period for all the substrates. The increase in shear-bond strength from up to 14 days was very steep. However, the increase from 14 days to 28 days was generally moderate. The chemical reaction between the polymer and the cementitious TSL is continuous and this leads to the eventual setting and hardening. The rate of reaction and hence the rate of hardening is faster in the first 14 days thus the steep increase in shear-bond strength. However, the reaction rate reduces after the 14 days, therefore the moderate increase in shear-bond strength.

The shear-bond strength equations and correlation coefficients (R^2) are listed in Table 6 for all the substrates and aspect ratios. The Strength equations and R^2 were obtained by setting the trend lines to best fit the relative test results in an excel spread sheet program. Generally, there was a good

Table 6. Shear-bond strength equation and correlation coefficients (R^2).

Parameters		Substrate					
		Quartzite		Sandstone		Concrete	
		Strength Equation (MPa)	R^2	Strength Equation (MPa)	R^2	Strength Equation (MPa)	R^2
Aspect ratio (m^{-1})	13.0	$1.15Inx + 5.19$	0.92	$1.38Inx + 9.43$	0.92	$0.61Inx + 7.45$	0.94
	16.2	$0.85Inx + 3.02$	0.96	$2.17Inx + 8.76$	0.86	$0.65Inx + 7.48$	0.97
	19.4	$1.59Inx + 2.61$	0.91	$2.38Inx + 8.91$	0.82	$0.79Inx + 7.77$	0.88
	22.7	$1.51Inx + 3.42$	0.86	$2.52Inx + 9.79$	0.86	$0.87Inx + 5.75$	0.96
	25.8	$1.54Inx + 3.10$	0.88	$1.59Inx + 9.79$	0.94	$0.96Inx + 5.22$	0.91

number of high R^2 values. Therefore, the test results could be relied on. Concrete substrate had the highest R^2 values while Sandstone had the lowest values. Concrete, being a material produced under a controlled system, was expected to have the highest values. Quartzite and sandstone are natural materials. They are therefore expected to vary a lot from specimen to specimen.

The shear-bond strength of thin spray-on liners (TSLs) typically increases with curing time. This is because the curing time allows the TSL material to harden and develop its full strength. The longer the curing time, the stronger the bond will be. The optimum curing time for TSLs will vary depending on the specific material and application. Li et al. (2017) investigated the effects of curing time on the shear-bond strength of TSLs. The TSL was made from a cementitious material and was cured for different periods of time. The curing times were 24 hours, 7 days, and 28 days. The results showed that the shear-bond strength of the TSL increased with curing time. The TSL had the strongest bond after 28 days of curing. Chen et al. (2020) investigated the effects of curing time on the shear-bond strength of TSLs. The TSL was made from a water-based cementitious material and was cured for different periods of time. The curing times were 7 days, 14 days, and 21 days. The results showed that the shear-bond strength of the TSL increased with curing time. The TSL had the strongest bond after 21 days of curing. The adhesive strength increased with curing time up to 28 days, after which it plateaued. This suggests that a curing time of 28 days is sufficient to achieve the maximum adhesive strength of the TSL.

3.4.1 Effect of surface cleanliness on shear-bond

Various environmental factors have negative effects on the shear-bond strength of TSLs (Yilmaz, 2011; du Plessis, 2021; Ozturk & Tannant, 2011; Qiao et al., 2015). Among these factors is the contamination of rock surfaces. Rock surfaces may be unclean due to airborne mineral dust particles and salts and minerals present in moisture or free water. The most damaging chemical conditions are those with high contents of sulphides and salts (Ozturk & Tannant, 2011). Bonding and composition of TSLs is affected by the interaction of such materials with TSL. The main surface contaminants are oil (grease) and dust. In this study, only the effect of oil contamination on rock surfaces was investigated.

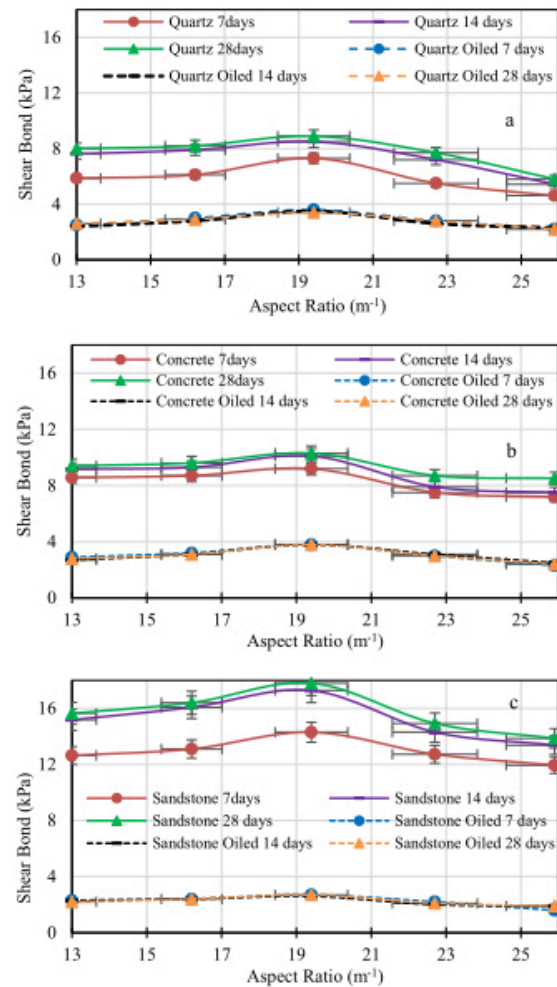
**Figure 15.** Shear-bond for substrates of: (a) Quartz and oiled Quartz, (b) Concrete and oiled Concrete and Sandstone and oiled Sandstone.

Figure 15 shows the effect of oiling the substrates on the shear-bond strength values. Each data point on the graph represents the average of respective five shear-bond strengths. Generally, oiling the substrates greatly reduced the achieved shear-bond. The greatest reduction was observed with the oiled sandstone substrates. This was attributed to the fact that oil filled the pores which would have been filled by the TSL and act as a grip against shear failure. The contribution of

grip effect to shear-bond resistance was more with sandstone substrates with higher porosity. No significant variation of shear-bond strength values with curing period was observed for all the oiled substrates. At 28 days of curing and liner thickness of 24.2 mm, the shear-bond values reduced by 61.8%, 63.1% and 84.8% for Quartz, Concrete and Sandstone substrates.

A clean and dry surface provides a better mechanical bond between the TSL and the substrate. This is because the TSL can interlock with the surface features of the substrate, which can help to prevent the TSL from delaminating from the substrate. du Plessis (2021) showed that the surface cleanliness of the rock face has a significant impact on the bond strength between the TSL and the rock. TSLs applied to clean rock faces had significantly higher bond strengths than those applied to dirty rock faces. This is because the dirt and debris on the rock face can interfere with the adhesion of the TSL. Qiao et al. (2015) investigated the effect of surface cleanliness on the shear bond strength of TSLs. The results showed that the shear bond strength was significantly higher for specimens with clean surfaces than for specimens with dirty surfaces. This is because a clean surface provides a better mechanical interlock between the TSL and the rock substrate.

4. Conclusion

TSLs are increasingly being used for structural support of rock excavations as surface support materials though, to-date, no acceptable standard tests or testing regimes have been defined. There is a need to develop tests and testing regimes, and reliable sets of data. A total of 2,250 tests were conducted to investigate the variation of TSL shear-bond strength with liner thickness, substrate type and surface cleanliness, and curing period. The liner was a polymer based cementitious TSL and substrates were quartzite and sandstone rocks, and concrete.

The material types in this study included quartz and sandstone rocks and 30 MPa concrete. Liner thicknesses included 16.2 mm, 20.2 mm, 24.2 mm, 28.2 mm, and 32.2 mm while curing periods were 7 days, 14 days, and 28 days. It was observed that the TSL shear-bond strength varied with different materials. Quartz with the lowest porosity had the lowermost shear-bond strength at all liner thicknesses and curing periods.

The shear-bond strength improved with liner thickness up to 24.2 mm and then decreased up to 32.2 mm. Therefore, a liner thickness of 24.2 mm is recommended for applications in underground excavations although smaller thicknesses may be utilized for smaller loads.

It also increased with curing period. The increase in shear-bond strength up to 14 days was very rapid while from 14 days to 28 days it was generally moderate. The rate of reaction and hence the rate of hardening is faster in the first 14 days thus the steep increase in shear-bond strength.

The reaction rate reduces after the 14 days, which explains the moderate increase in shear-bond strength.

Oil reduced the shear-bond strength for all the material types at all liner thicknesses and curing periods. The contribution of grip effect to shear-bond resistance was more with sandstone substrates with higher porosity. No significant variation of shear-bond strength values with curing period was observed for all the oiled substrates. Oil is one of the main surface contaminants in underground excavations of the mining industry through leaks from machines utilized in the excavations.

Further research is warranted to investigate the shear-bond strength of various TSL products, such as water-based and polyurethane TSLs, with different substrates and liner thicknesses. Additionally, it is essential to develop or modify testing methods for weaker rocks, such as shale, that cannot be tested with existing methods. These efforts should be complemented by field trials to validate the results of laboratory testing.

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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

Samuel Jjuuko: conceptualization, investigation, methodology, project administration, resources, software, data curation, visualization, writing - original draft preparation. Denis Kalumba: conceptualization, supervision, validation, writing - reviewing and editing, funding acquisition.

Data availability

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

List of symbols and abbreviations

t	Depth or steel ring height
CV	Coefficient of variation
D	Core diameter

<i>F</i>	Maximum applied force
<i>SD</i>	Standard deviation
TSL	Thin Spray-on Liner
<i>UCS</i>	Unconfined compressive strength

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Floating stone column-supported embankments on soft soils: numerical study incorporating stability analysis and basal reinforcement with geosynthetic

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Article

Keywords

Floating stone columns
Embankment
Soft soil
Geosynthetic
Overall stability
Mechanical-hydraulic coupled analysis

Abstract

Although fully penetrating stone columns have widely been studied in the literature, the number of published studies on floating stone columns – which also have been efficiently used to improve vertically lengthy soft-clay deposits – is very limited. Basal reinforcement with geosynthetic is another technique that increases overall stability of embankments on soft soils and can be combined with stone columns. In the paper, mechanical-hydraulic coupled finite element analyses are performed to study a floating stone column-supported embankment on a thick soft soil deposit. A parametric analysis is conducted so that the influence of two governing factors is evaluated: length of stone columns; basal reinforcement with geosynthetic. An overall stability analysis method, which uses the results of finite element analyses, is also applied. Results of several key variables are analysed: overall safety factor, settlements, horizontal displacements, vertical stress at the embankment base, stress concentration ratio, excess pore pressures and geosynthetic tensile force.

1. Introduction

Stone columns have widely been studied in the literature due to their advantages in efficiently improving soft soils during the construction of embankments of railway or road infrastructures. This technique is one of the most adequate when the main purpose is, simultaneously, to increase overall stability and reduce and accelerate settlements.

Although most experimental and numerical studies published in the literature concern fully penetrating stone columns (i.e., end-bearing stone columns, with their tip lying on the lower hard stratum) (e.g. Borges et al., 2009; Chen et al., 2009, 2013; Castro & Sagaseta, 2011; Elsayy, 2013; Almeida et al., 2015; Basack et al., 2017; Castro, 2017; Marques & Borges, 2018a,b; Miranda et al., 2021; Zhou et al., 2021; Zhang et al., 2022; Chen et al., 2022), floating stone columns have also efficiently been used in vertically lengthy soft-clay deposits (Chen et al., 2022) and can reduce cost. However, the number of published studies of floating stone columns is very small.

When floating stone columns are used to reinforce a very thick soft soil deposit, it should be considered that the length of columns determines opposite effects on the cost and structural improvement. Therefore, an adequate length of columns should be defined so that the cost-improvement binomial is as optimized as possible.

Basal reinforcement with geosynthetic is another technique that also increases overall stability of embankments on soft soils (Rowe & Soderman, 1985, 1987; Borges & Cardoso, 2002; Rowe & Li, 2005) and can be combined with stone columns to further increase short-term safety against overall failure.

Although many studies have been published on stone column-supported embankments, the number of studies where basal reinforcement of geosynthetic is combined with stone columns is much smaller. Deb et al. (2011) presented laboratory model tests on unreinforced and geogrid-reinforced stone column-improved soft clay; the results showed that the inclusion of the geogrid increased the load-carrying capacity and decreased the settlement of the soil. Using the concept of the cylindrical unit cell to model the behaviour of a geosynthetic-reinforced and column-supported embankment, Zhao et al. (2019) proposed a closed-form solution applied to columns close to the middle section of the embankment and showed that the inclusion of a geosynthetic reinforcement can increase the load transfer efficiency into the columns.

Zhang et al. (2022), using a finite element model, performed a parametric study of floating geosynthetic-encased stone column-supported embankments with geosynthetic basal reinforcement. The effects of the consistency of substratum soil (soil underlying the column tip), tensile stiffness of basal reinforcement and encasement were analysed. The results

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showed that higher embankment loads were transferred to the surrounding soft soil when the stone columns were constructed on a weaker substratum. Larger settlements at embankment crest, ground surface and the plane of column bottom were also obtained, as well as higher tensile strains of the basal reinforcement and lateral displacements of columns. The increase of encasement stiffness resulted in a decrease in the global settlement and restricted the increase of lateral displacements and tensile strains in the basal reinforcement. When the stiffness of basal reinforcement increased, larger embankment load was transferred to the floating columns and lower lateral displacements was obtained.

Centrifuge tests and numerical modelling were also conducted by Chen et al. (2022) regarding a geosynthetic-reinforced and floating encased stone column-supported embankment. The results illustrated that the inclusion of the basal reinforcement significantly reduced total and uneven settlement at the embankment crest and base, as well as lateral displacement at the top of the column; however, the lateral displacement at the bottom of the floating columns increased.

In this paper, a fully mechanical-hydraulic coupled finite element code is applied to perform a parametric study on the behaviour of a floating stone column-supported embankment on a thick soft soil deposit. Two parameters are studied: length of columns (the effect of its variation is analysed in cases without basal reinforcement); basal reinforcement with geosynthetic (for a particular length of floating columns, different constitutive curves are considered for the geosynthetic). Results of several main variables are analysed, namely: overall safety factor, settlements, horizontal displacements, vertical stress at the embankment base, stress concentration ratio, excess pore pressures and geosynthetic tensile force. The overall safety factor is computed applying a stability analysis method (computer code) which uses the numerical results of the finite element code, as explained below.

2. Computer codes

2.1 Finite element model

The finite element code, developed by Borges (1995), incorporates, among other aspects, the following features applied in this study: (i) fully mechanical-hydraulic coupled analysis; (ii) the p - q - θ critical state model for soil constitutive behaviour simulation (Borges, 1995; Lewis & Schrefler, 1987); (iii) hardening elastoplastic models to simulate constitutive behaviours of the geosynthetic and soil-geosynthetic interfaces (Borges, 1995; Borges & Cardoso, 2001). Subsequently to the initial version of the code (two-dimensional modelling version), several other improvements were also developed and implemented, particularly a three-dimensional modelling version (Borges, 2004).

For two-dimensional (2D) analysis, two types of the six-noded triangular element (nodes at the vertices and middle

of the sides) are used: (i) coupled element, for the soft soil, where consolidation is considered (all the six nodes have displacement degrees of freedom while only the three vertex nodes have excess pore pressure degrees of freedom). (ii) non-coupled element, for stone columns and embankment material (all the six nodes have only displacement degrees of freedom). Bar and interface elements are used to model the geosynthetic and soil-geosynthetic interfaces, respectively.

The p - q - θ model is an extension of the Modified Cam-Clay model into the three-dimensional stress space which uses the Mohr-Coulomb surface as the critical state criterion. This means that the parameter that defines the slope of the critical state line, M , is not constant (as happens in the Modified Cam-Clay model) and depends on the angular stress invariant θ and effective friction angle, ϕ' , as follows:

$$M = \frac{3 \sin \phi'}{\sqrt{3} \cos \theta + \sin \phi' \sin \theta} \quad (1)$$

Therefore, the critical state line (Mohr-Coulomb criterion) is defined when M , given by Equation 1, is introduced in the following equation:

$$q = M \cdot p \quad (2)$$

where q is the deviatoric stress and p is the effective mean stress.

The finite element code has been applied to analyse a large range of geotechnical structures involving fully mechanical-hydraulic coupled modelling (e.g., Borges, 1995, 2022; Domingues, 2006; Costa et al., 2007; Marques, 2008; Borges & Cardoso, 2001, 2002; Borges & Almeida, 2018; Borges & Guerra, 2014; Borges et al., 2009; Borges & Marques, 2011; Borges & Gonçalves, 2016; Borges & Santos, 2020; Marques, 2021). Several case studies were modelled and, comparing field and numerical results, good agreements were obtained, namely in: (i) two embankments on soft soils reinforced with stone columns, one in the Gold Coast Highway of Australia (Marques, 2021), and the other in the northern railway of Portugal (Domingues, 2006) (ii) two trial geosynthetic-reinforced embankments on soft soils (Borges, 1995), one constructed up to failure and the other observed until the end of consolidation; (iii) a braced excavation in very soft ground carried out in the City of San Francisco (Costa et al., 2007; Costa, 2005).

2.2 Stability analysis code

The overall stability analysis code applied in this study was developed by Borges (1995) and uses the results of the finite element model mentioned above. It was also presented and applied in Borges & Cardoso (2002) and was specifically developed for embankments on soft soils

with basal reinforcement with geosynthetics. It can also be efficiently applied when stone columns are used or basal reinforcement is not included.

For any phase of the problem, the stability analysis code computes the overall safety factor, F , with the results of the finite element code (2D coupled analysis). A large number of potential slip circular cylindrical surfaces is analysed so that the critical slip circle is obtained, which corresponds to the smallest value of F calculated.

For a particular potential slip circle, the intersection points of the circle with the sides of the 2D finite elements (six-noded triangular elements) are determined. Thus, the slip circle is divided into small straight-line segments, each of them located inside of only one of the 2D finite elements of the mesh. Intersection of the slip circle with the geosynthetic (or geosynthetics, if there are several reinforcement layers) modelled by bar elements is also determined (Figure 1).

Therefore, for the slip circle divided into small segments, the overall safety factor is calculated with the following equation (Borges, 1995; Borges & Cardoso, 2002):

$$F = \frac{\sum_{i=1}^N q_{fi} l_i + \sum_{j=1}^{N_g} [T_{rj} \cos(c_r \theta_j)]}{\sum_{i=1}^N \tau_i l_i + \sum_{j=1}^{N_g} [T_{aj} \cos(c_r \theta_j)]} \quad (3)$$

where: N – number of 2D elements of the mesh intersected by the circle; l_i – i -segment length; q_{fi} – average value of soil shear strength at i -segment (calculated using formulations of the critical state soil mechanics, as indicated below); τ_i – average value of acting shear stress at i -segment (determined from effective stresses σ'_x , σ'_y and τ_{xy} , known the angle that defines i -segment inclination; xy – plane of the finite element analysis); N_g – number of geosynthetic layers (usually, $N_g=1$); T_{rj} – resisting tensile force of j -geosynthetic at cut point (intersection of the slip circle with the geosynthetic, point I in Figure 1); T_{aj} – acting tensile force of j -geosynthetic at the cut point I (obtained interpolating between the tensile stresses at the two nearest Gauss points of the bar element that contains point I); θ_j – angle between the geosynthetic and the tangent to the slip circle at the cut point; c_r – reduction

coefficient (it ranges between 0 and 1; usually $c_r=1$, the most conservative assumption, which assumes that the geosynthetic remains in its original orientation).

The calculation of resisting tensile force of geosynthetic at cut point, T_r , explained in detail in Borges & Cardoso (2002), is function not only of the geosynthetic strength (T_{rg}) but also of the maximum force that can be mobilised at point I considering the pull-out strength of the soil-geosynthetic interfaces (T_{rp}). T_r is the smaller value of these two strengths, i.e.:

$$T_r = \min(T_{rg}, T_{rp}) \quad (4)$$

Since the p - q - θ model is used in the finite element analysis, shear strength of soils is calculated by the following equation of the critical state soil mechanics (Britto & Gunn, 1987; Borges, 1995):

$$q_{fi} = \frac{1}{2} M \cdot \exp\left(\frac{\Gamma - v_i}{\lambda}\right) \quad (5)$$

where M is obtained by Equation 1 and v_i , the specific volume of soil at i -segment, is computed as follows:

$$v_i = \Gamma - k \ln p_i - (\lambda - k) \ln \alpha_{pi} \quad (6)$$

At the i -segment, $p_i = (\sigma'_{xi} + \sigma'_{yi} + \sigma'_{zi})/3$ is the effective mean stress and α_{pi} is the p -value of the centre of the yield surface in p - q plane (Borges, 1995; Lewis & Schrefler, 1987), extrapolated from α_p -values at Gauss points of finite element; λ , k and Γ , parameters of the p - q - θ model (soil properties), are defined as follows: λ , slope of normal consolidation line and critical state line; k , slope of swelling and recompression line; Γ , specific volume of soil on the critical state line at mean normal stress equal to 1 kPa.

3. Numerical modelling

The problem concerns a 2.5-m-high symmetric embankment, with large longitudinal length, built on a 25-m-thick soft clay lying on a rigid and impermeable stratum (Figure 2). The width of the embankment crest is 12 m. Floating stone columns are installed in the soft soil, as shown in Figure 2. The water level is at the ground surface.

Due to the requirement of large computational time and complexity of the three-dimensional (3D) arrangement of multiple columns, the 3D problem of stone column-supported embankments has been commonly converted into a 2D model considering plane strain analysis. Two basic types of equivalency methods have been used: the equivalent

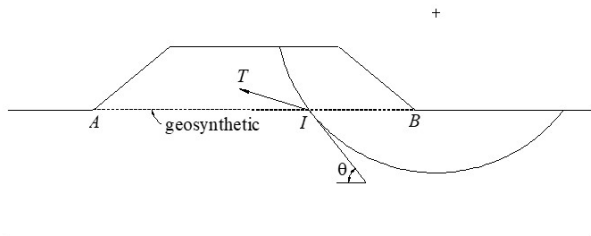


Figure 1. Intersection of the slip circle with the geosynthetic.

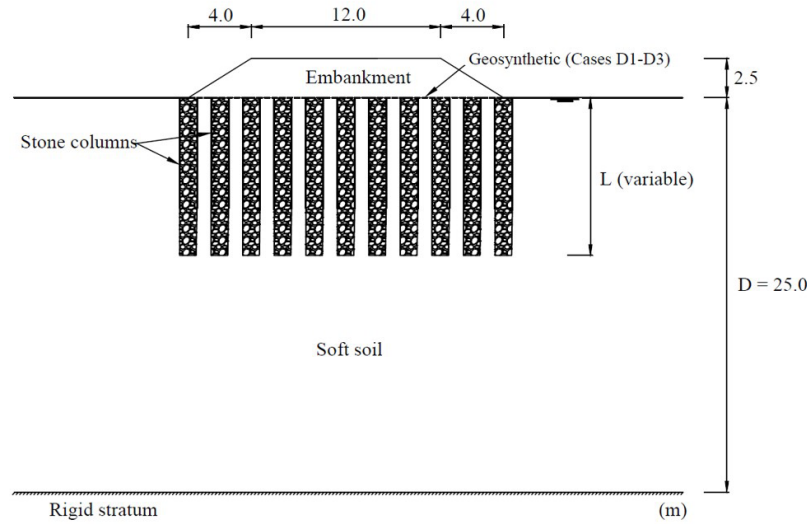


Figure 2. Cross-sectional view of the problem.

area method and the column-wall method (Marques, 2021; Castro, 2017; Marandi et al., 2016; Zhang et al., 2014; Abusharar & Han, 2011; Tan et al., 2008; Cooper & Rose, 1999; Christoulas et al., 1997).

In the equivalent area method, the stone columns and the surrounding soil are considered as a composite ground (also called an improved area), while, in the column-wall method, the columns are transformed into longitudinal stone walls (Figure 3). A column-wall model is adopted in this study with the following geometric parameters for the walls: wall thickness, 0.475 m; midplane-to-midplane spacing between walls, 2.0 m. This corresponds to individual columns with diameter of 1.1 m in a 3D arrangement with the same spacing (i.e. axis-to-axis spacing of 2.0 m) and equal value of the area replacement factor (a_s). The area replacement factor is defined as follows:

$$a_s = \frac{A_c}{A_t} \quad (7)$$

where A_c is the column area and A_t is the tributary area of natural soil for each column ($A_t = s^2$ for stone columns distributed in a square grid; s – axis-to-axis spacing between columns).

Figure 4 depicts the finite element mesh of the problem. Drained analysis is considered in the embankment material and stone walls while fully mechanical-hydraulic coupled analysis is modelled in the clay. The construction of the embankment is modelled by adding layers of elements at a uniform rate and is completed in an overall time of 7 days. The effective thickness of column-walls is taken as constant in time; i.e. possible partial filling of gravel voids with particles of soft soil as a result of water percolation during consolidation is not considered in the calculations.

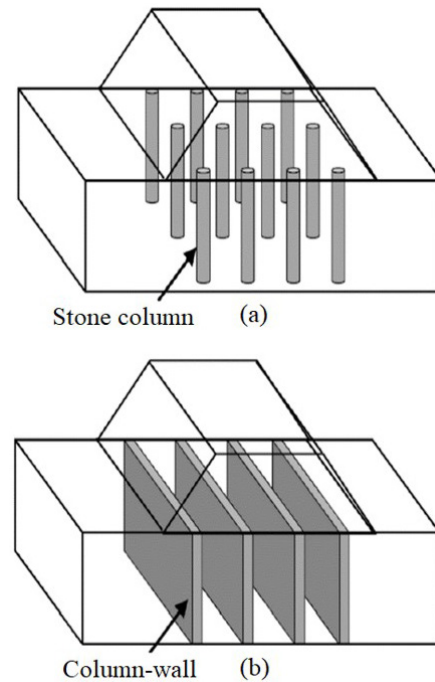


Figure 3. Embankment on soft soil supported by stone columns: (a) individual columns; (b) column-walls.

The following displacement boundary conditions are set: (i) zero-displacement in both x - and y -direction at $y = 0$ line, considering that the soft clay lays on a rigid stratum; (ii) zero-displacement in x -direction at the nodes of the symmetry axis ($x = 0$); (iii) zero-displacement in x -direction at the $x = 41$ m line (lateral boundary), assuming that the horizontal displacement can be set as zero at nodes which are enough distant from the embankment.

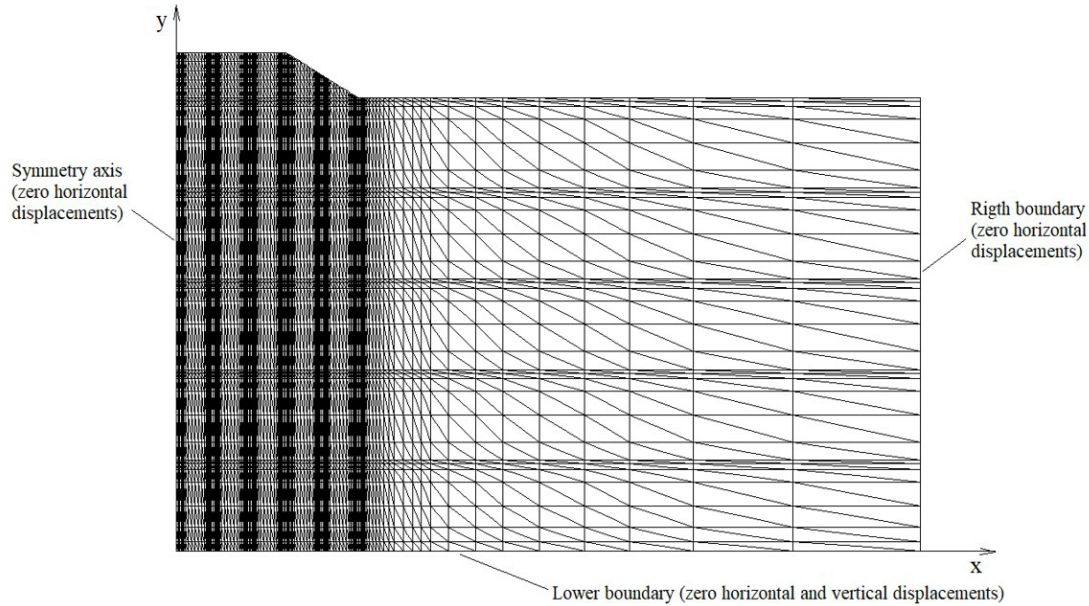


Figure 4. Finite element mesh.

Table 1. Geotechnical properties of the clay, embankment soil and stone column.

	γ (kN/m ³)	ν'	ϕ' (°)	k_h (m/s)	k_v (m/s)	p - q - θ critical state model			
						λ	k	Γ	N
Clay	16	0.25	26	10^{-9}	10^{-9}	0.18	0.025	3.05	3.158
Embankment	20	0.30	35	-	-	0.03	0.005	1.80	1.817
Stone column	17	0.30	40	-	-	0.0038	0.00095	1.914	1.916

Regarding the drainage boundary conditions, excess pore pressure is set as zero on the ground surface ($y = 25$ m plane) and on the lateral surfaces and tip of the column-walls.

The constitutive behaviour of the soils (clay, stone columns and embankment material) is simulated by the p - q - θ critical state model whose parameters are shown in Table 1 (λ , slope of normal consolidation line and critical state line; k , slope of swelling and recompression line; N , specific volume of normally consolidated soil at mean normal effective stress equal to 1 kPa; Γ , specific volume of soil on the critical state line at mean normal effective stress equal to 1 kPa). Other geotechnical properties are also indicated in Table 1: γ , unit weight; ϕ' , angle of friction defined in effective terms; ν' , Poisson's ratio for drained loading; k_h and k_v , coefficients of permeability in horizontal and vertical directions. The parameters of stone columns are based on those of stone columns constructed in soft ground of a railway infrastructure in Portugal (Domingues, 2006). The parameters of the clay are similar to those considered by Finno et al. (1991) in a soft ground in Chicago, USA. The values adopted for the embankment material are the same as those used by Borges & Cardoso (2001) in an embankment on soft ground.

4. Parametric study

4.1 Influence of column length

In order to study the influence of this parameter, five lengths of stone columns are considered for the problem described in section 3, as indicated in Table 2. In cases C1-C4, floating stone columns are considered, i.e. their lengths are smaller than the vertical thickness of soft ground (25 m), while, in case C5, the length of columns is equal to the thickness of soft ground (fully penetrating stone columns). Basal reinforcement with geosynthetic is not considered in all cases C1-C5.

Figure 5 shows the effect of this parameter on settlements of embankment base, at the end of construction and end of consolidation. Figure 6 also depicts maximum settlement on the embankment base, at the end of consolidation, *versus* L/D , ratio of the column length to the vertical thickness of soft soil. As expected, these results show that the longer the stone columns, the smaller the settlements. However, the curve of figure 6 illustrates that, when L/D increases from

0.2 to 0.4, the maximum settlement reduces 13 cm (from 46 cm to 33 cm), while for higher values of L/D the rate of settlement reduction is significantly smaller; maximum settlement reduces 5 cm when L/D increases from 0.4 to 0.6, and only 1 cm when L/D increases from 0.6 to 0.8 and

Table 2. Length of columns for the five analysed cases.

Case	Length of columns (m)	L/D (m)
C1	5	0.2
C2	10	0.4
C3	15	0.6
C4	20	0.8
C5	25	1.0

L – length of columns; D – vertical thickness of soft soil.

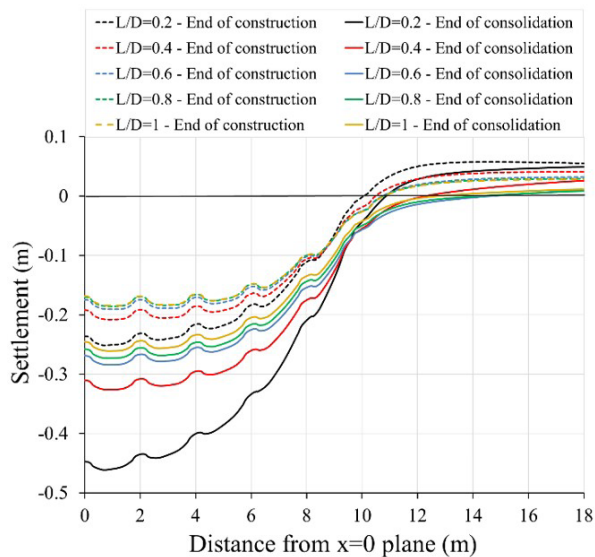


Figure 5. Influence of length of columns on vertical displacements of ground surface (embankment base), at the end of construction (7th day) and end of consolidation (cases C1-C5).

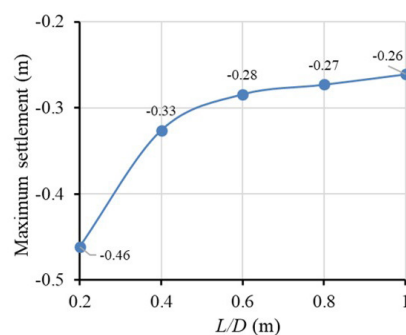


Figure 6. Influence of length of columns on maximum settlement at the embankment base (end of consolidation).

from 0.8 to 1.0. This means that, in terms of settlement improvement – and also considering the cost – the option for floating stone columns, with a length between 10 m and 15 m, would be adequate in practice for this problem.

Numbering the stone columns from the symmetry axis to the embankment toe, their axis x -coordinate are: column 1, $x = 0$; column 2, $x = 2$ m; column 3, $x = 4$ m; column 4, $x = 6$ m; column 5, $x = 8$ m; column 6, $x = 10$ m. The following results are depicted in Figures 7 and 8 for cases C1-C5: horizontal displacements under the embankment toe ($x=10$ m), at the end of construction (7th day) and end of consolidation (Figure 7); settlement *versus* time at midpoint between columns 1 and 2 on the embankment base (coordinates: $x = 1$ m; $y = 25$ m) (Figure 8). These results illustrate that: (i) like for settlements, the longer the stone columns, the smaller the horizontal displacements; (ii) however, the reduction of horizontal displacements is also significantly higher when

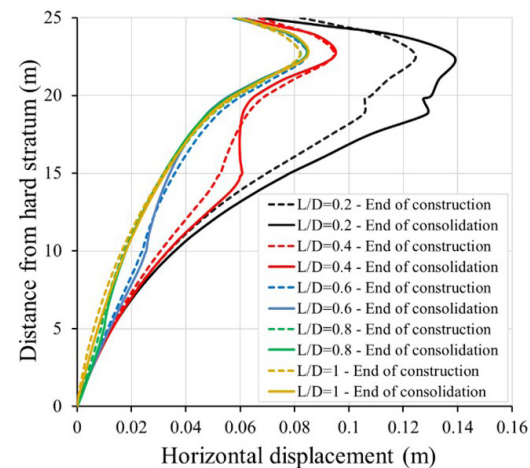


Figure 7. Influence of length of columns on horizontal displacements under the embankment toe ($x = 10$ m), at the end of construction (7th day) and end of consolidation (cases C1-C5).

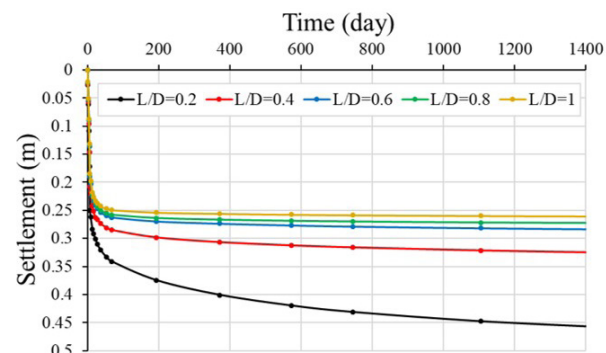


Figure 8. Settlement versus time at midpoint between columns 1 and 2 on the embankment base (coordinates: $x = 1$ m; $y = 25$ m) for cases C1-C5.

L/D increases from 0.2 to 0.4 than when L/D increases from 0.4 to larger values; (iii) due to the draining effect of stone columns, the longer the stone columns, the faster the rate of consolidation, as shown in Figure 8; however, again, the difference of consolidation rate between case C1 ($L/D=0.2$) and case C2 ($L/D=0.4$) is significantly larger than between case C2 and the other cases with longer stone columns.

In stone column-supported embankments on soft ground, since the columns have higher stiffness than the surrounding soft soil, the stone columns carry higher vertical stresses than the surrounding soil (or side soil, if column-walls are modelled, as in this study). This effect (arching effect) is shown in Figure 9 where vertical stress at the embankment base, for cases C1-C5, are depicted, at the end of construction and end of consolidation. Figure 10 also shows the stress concentration ratio on the top of the six columns. The stress concentration ratio, SCR , is a parameter that can also be used to evaluate the degree of stress concentration and is defined as the ratio of the average vertical stress on the column to that on its surrounding soil (or its corresponding side soil, if column-wall modelling is considered).

The results of these figures illustrate that, at the end of construction: (i) the stress concentration ratio (Figure 10a) assumes similar values in all five cases (C1-C5), except for column 6 (column under the embankment toe, which practically does not have embankment fill on its top); therefore, this means that the length of columns does not significantly influence the load transfer to the top of columns during construction; this fact is, somehow, understandable, since arching effect inside the embankment fill is essentially dependent on the vertical stiffness of foundation materials (stone columns and soft soil) and is directly related to respective differential settlements (between each column and its tributary soft soil) on the embankment base (which, at the end of construction,

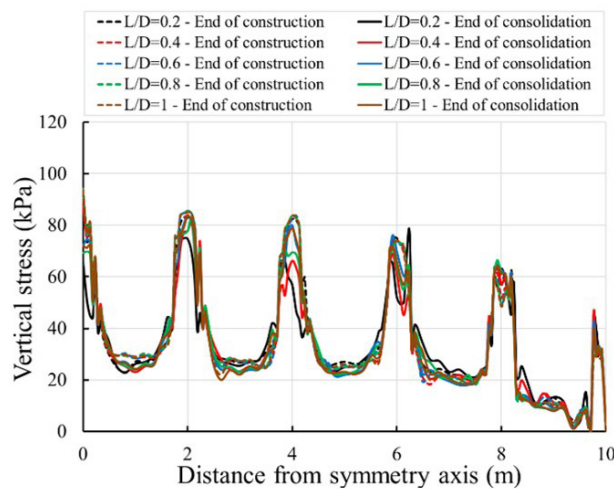


Figure 9. Influence of length of columns on vertical stress at the embankment base for cases C1-C5, at the end of construction (7th day) and end of consolidation.

are similar, as shown in Figure 5); (ii) stress concentration ratio takes similar values in the four central columns (columns 1-4), while, in columns 5 and 6 (columns under the embankment slope), assumes higher values; this means that, although vertical stresses on top of columns 5 and 6 are lower than those on top of columns 1-4 (Figure 9), the magnitude of arching effect (ratio between stresses, on column and on tributary soft soil) is higher in the columns under the embankment slope; this effect is directly related to the lower horizontal confinement of columns 5-6 and their tributary soft soil, due to their position near the embankment toe – where horizontal displacements, shear stresses and stress levels are higher – determining a decrease of stiffness in the soft soil and stone columns, which affects the ratio between their stiffnesses and explains the above-mentioned differences in relation to the other columns. In response to consolidation, stress concentration ratio decreases a little in the central columns (columns 1-4) for all cases.

Figure 11 shows the influence of length of columns on the distributions of excess pore pressure at the end of

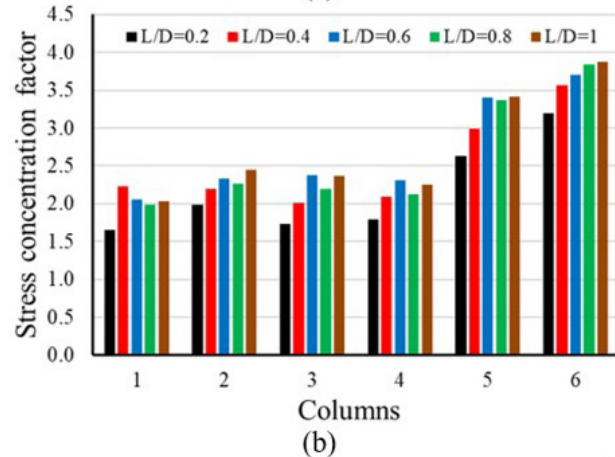
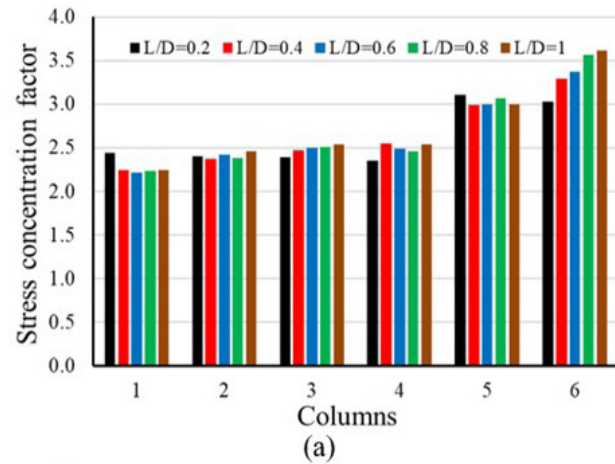


Figure 10. Influence of length of columns on the stress concentration ratio on the top of columns (cases C1-C5): a) end of construction; b) end of consolidation.

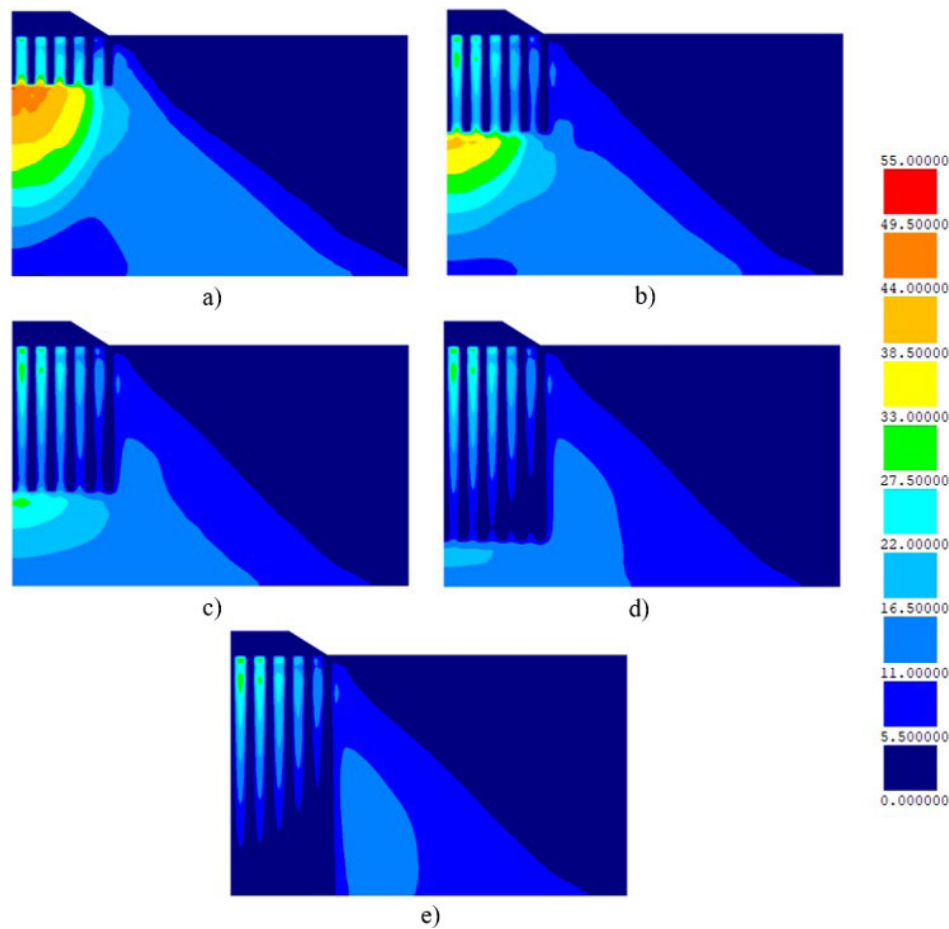


Figure 11. Influence of length of columns on excess pore pressure (u) at end of construction (7th day): a) $L/D = 0.2$ ($u_{max} = 50.453$ kPa); b) $L/D = 0.4$ ($u_{max} = 40.234$ kPa); c) $L/D = 0.6$ ($u_{max} = 30.883$ kPa); d) $L/D = 0.8$ ($u_{max} = 30.991$ kPa); e) $L/D = 1.0$ ($u_{max} = 30.941$ kPa).

construction (cases C1-C5). These results illustrate that, as expected, the shorter the columns, the higher the excess pore pressure at the end of construction, mainly below the end of columns; i.e. the shorter the columns, the higher the load transferred from the column to the soft soil on the column tip. However, like for displacements analysed above, while maximum value of excess pore pressure reduces from 50.5 kPa to 40.2 kPa when L/D increases from 0.2 to 0.4, for values of L/D higher than 0.6 the maximum value of excess pore pressure practically does not change.

The magnitude of excess pore pressures at the end of construction and vertical thickness of soft soil below the column tip are logically related with the magnitude of long-term settlements; i.e., due to consolidation after the end of construction, the higher the former, the larger the latter, as shown in Figures 5 and 11.

To analyse the effect of the length of columns on the overall stability of problem, Figure 12 depicts the critical slip surfaces – and respective overall safety factors – at the end of construction and end of consolidation, calculated from the finite element results with the computer code for stability

analysis described in Section 2.2. Tables 3 and 4 also illustrate the sums of acting and resisting forces at the foundation (soft soil and stone columns) and at the embankment soil, along such critical surfaces, considering the application of Equation 3. It should be mentioned that, in addition to cases C1-C5, the same problem but without stone columns (i.e. $L/D=0$) was also modelled, which showed that, for such case, overall failure would occur (numerical instability and values of F lower than 1 were obtained before the completion of the embankment). This illustrated that some reinforcement or improvement technique would be needed in this problem, as considered in this studied.

Figure 12a and Table 3 show that the influence of the length of columns on the increase of overall stability at the end of construction is only effective for values of L/D lower than 0.4; i.e. F increases from 1.307 to 1.354 when L/D varies from 0.2 to 0.4, while, for values of L/D higher than 0.4, F remains without significant change (the critical slip surface is the same for cases C2-C5). This means that, like for settlements (as said above), in terms of overall stability improvement – and considering the cost – the option for

Table 3. Critical slip surfaces at the end of construction for cases C1-C5: overall safety factor (F) and sums of acting and resisting forces.

Case	X_0 (m)	Y_0 (m)	R (m)	Sum of acting forces (kN/m)		Sum of resisting forces (kN/m)		F (Eq. 3)
				Foundation	Embankment	Foundation	Embankment	
				$\sum_{i=1}^{N_1} \tau_i l_i$	$\sum_{i=1}^{N_2} \tau_i l_i$	$\sum_{i=1}^{N_1} q_{fi} l_i$	$\sum_{i=1}^{N_2} q_{fi} l_i$	
C1	8.5	31.5	12.671	239.49	5.470	305.18	15.027	1.307
C2	9.0	30.0	8.849	154.09	7.645	203.57	15.363	1.354
C3	9.0	30.0	8.849	153.35	7.526	203.28	16.259	1.365
C4	9.0	30.0	8.849	153.69	6.830	203.50	15.026	1.361
C5	9.0	30.0	8.849	153.71	7.171	203.35	15.465	1.360

R – radius; (X_0, Y_0) – coordinates of centre; $N=N_1+N_2$

Table 4. Critical slip surfaces at the end of consolidation for cases C1-C5: overall safety factor (F) and sums of acting and resisting forces.

Case	X_0 (m)	Y_0 (m)	R (m)	Sum of acting forces (kN/m)		Sum of resisting forces (kN/m)		F (Eq. 3)
				Foundation	Embankment	Foundation	Embankment	
				$\sum_{i=1}^{N_1} \tau_i l_i$	$\sum_{i=1}^{N_2} \tau_i l_i$	$\sum_{i=1}^{N_1} q_{fi} l_i$	$\sum_{i=1}^{N_2} q_{fi} l_i$	
C1	9.5	31.5	11.769	210.63	-3.944	315.28	22.515	1.634
C2	9.5	28.0	6.536	124.45	2.680	181.68	15.506	1.551
C3	8.5	28.5	7.558	149.09	6.146	229.39	14.638	1.572
C4	10.0	28.5	7.123	121.81	5.465	183.97	16.673	1.576
C5	10.0	28.5	7.123	123.30	5.641	185.36	15.880	1.561

R – radius; (X_0, Y_0) – coordinates of centre; $N=N_1+N_2$

floating stone columns, with a length between 10 m and 15 m, would also be adequate in practice for this problem.

At the end of consolidation, F takes a little higher value in case C1 ($F=1.634$) than in the other cases with longer stone columns, where F ranges between 1.551 and 1.576 (cases C2-C5). This is explained by the higher effect of consolidation on the increase of soft soil strength (larger reduction of soil volume provoking higher settlements) in case C1 determined by the larger values of excess pore pressure at the end of consolidation below the columns' tips, as shown above.

4.2 Influence of basal reinforcement with geosynthetic

In order to study the influence of this parameter, four cases with 10-m-long floating stone columns are considered: (i) case D0, which is equal to case C2, analysed in the previous section; therefore, basal reinforcement with geosynthetic is not included in this case; (ii) cases D1-D3, which are similar to case D0, but with geosynthetic reinforcement at the embankment base, where different constitutive curves are considered for the geosynthetic. All other parameters of the problem are kept equal in the four cases D0-D3.

Figure 13 illustrates the constitutive curves of the geosynthetic for cases D1-D3, which are simulated by an

elastoplastic model, as explained below. These curves are similar to constitutive curves of three typical commercial geogrids. Considering different geosynthetics, it should be mentioned that their strength and stiffness are usually related to each other, i.e. the higher the geosynthetic stiffness, the higher its strength. This relation is therefore considered in this study. Respectively for cases D1, D2 and D3, the geosynthetic elastic modulus is 500 kN/m, 1000 kN/m and 1500 kN/m, and the geosynthetic strength, 50 kN/m, 100 kN/m and 150 kN/m; the geosynthetic thickness is considered equal to 2 mm in the three cases.

Soil-geosynthetic interfaces are modelled by interface elements. Their constitutive curve varies with the normal stress on the interface and is considered the same in the three cases, D1-D3. Figure 14 depicts the adopted curve for a normal stress of 15 kPa on the interface. The tangential stiffness of soil-geosynthetic interfaces takes the value of 1.6×10^4 kPa/m. Adhesion, a , and frictional angle, δ , for both upper and lower soil-geosynthetic interfaces are set as 0 kPa and 33.7° , respectively.

The elastoplastic models used to simulate the mechanical behaviours of the geosynthetic and soil-geosynthetic interfaces incorporate the following hardening law (Prevost & Hoeg, 1975; Owen & Hinton, 1980; Thomas, 1984; Borges, 1995):

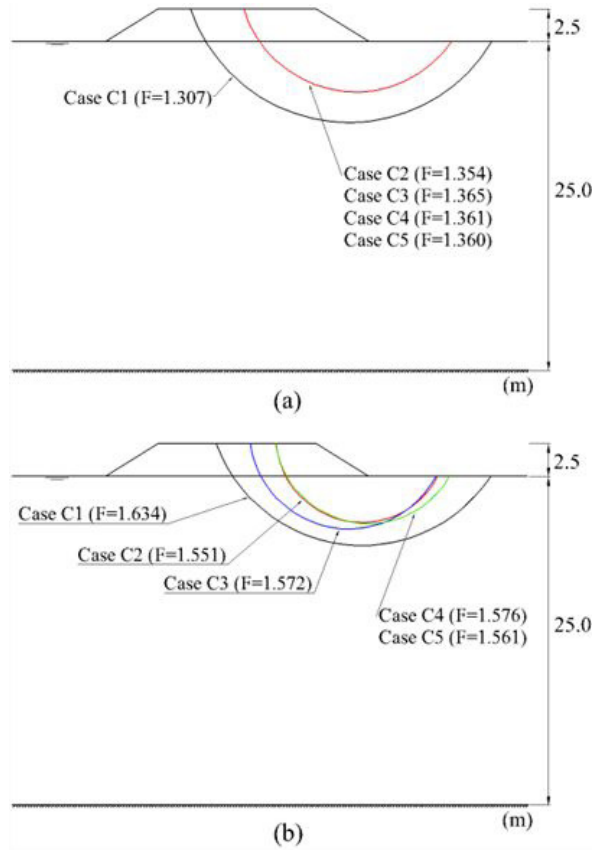


Figure 12. Critical slip surfaces for cases C1-C5: a) end of construction (7th day); b) end of consolidation.

$$Y(h) = c_1 + \frac{c_2 h + c_3 h^2}{1 + c_4 h + c_5 h^2} \quad (8)$$

where h is the hardening parameter and c_1, c_2, c_3, c_4 and c_5 are constants that characterize a particular hardening curve. The following yielding function is used for the geosynthetic (Borges, 1995):

$$f = \sigma - Y(h) \quad (9)$$

where: σ – tensile stress; $Y(h) = \sigma_Y(\varepsilon_p)$ – hardening law; $h = \varepsilon_p$ – hardening parameter; σ_Y – yielding stress; ε_p – plastic tensile strain.

For the soil-geosynthetic interfaces, the yielding function is defined as follows (Borges, 1995):

$$f = |\tau| - \sigma_n Y(h) \quad (10)$$

where: τ – tangential stress; σ_n – normal stress; $Y(h) = \tan \delta (s_p)$ – hardening law; $h = \sum |s_p|$ – hardening parameter; δ – interface friction angle; s_p – plastic tangential relative displacement. More detailed descriptions of the implementation of these models are presented in Borges (1995). Values of c_1, c_2, c_3, c_4 and c_5

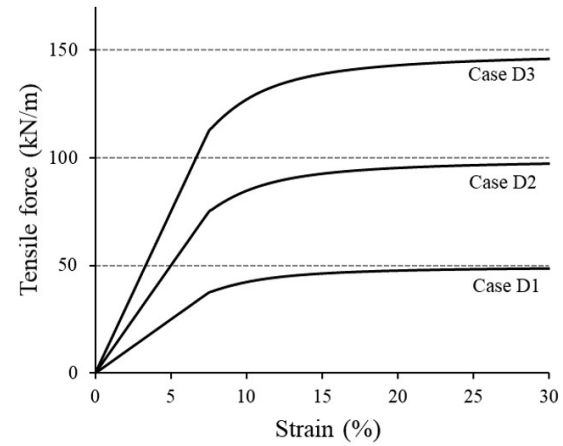


Figure 13. Constitutive curve of the geosynthetic for cases D1-D3.

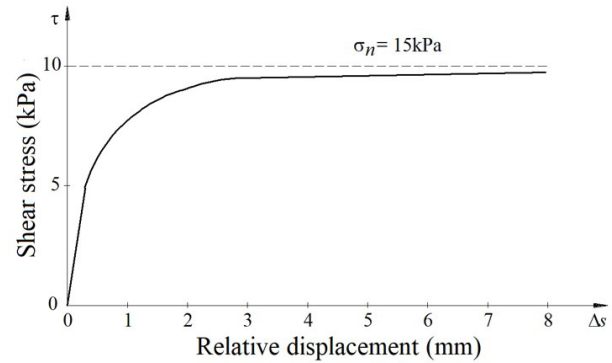


Figure 14. Constitutive curve of the soil-geosynthetic interfaces for normal stress of 15 kPa.

adopted for the geosynthetic and soil-geosynthetic interfaces are illustrated in Table 5, corresponding to the constitutive curves depicted in Figure 13 and 14, as said.

For cases D0-D3, Figure 15 shows the critical slip surfaces – and respective overall safety factors – at the end of construction and end of consolidation, calculated from the finite element results with the computer code for stability analysis described in Section 2.2. Taking into account the application of Equation 3, Tables 6 and 7 depict the sums of acting and resisting forces at the foundation (soft soil and stone columns) and at the embankment soil, along such critical surfaces, as well as the geosynthetic acting force T_a , the geosynthetic strength T_{rg} , the pull-out strength of the soil-geosynthetic interfaces T_{rp} and angle θ at its cut point. The reduction coefficient c_r is taken equal to 1, the most conservative option, as said above.

These results reveal that the safety factor (F) at the end of construction significantly increases when basal reinforcement with geosynthetic is included (cases D1-D3). As expected, the higher the geosynthetic strength, the higher the safety

Table 5. Hardening law constants.

	Case	c_1	c_2	c_3	c_4	c_5
geosynthetic	D1	18750 (kPa)	265625 (kPa)	0 (kPa)	42.5	0
	D2	37500 (kPa)	531250 (kPa)	0 (kPa)	42.5	0
	D3	56250 (kPa)	796875 (kPa)	0 (kPa)	42.5	0
soil-geosynthetic interface	D1-D3	0.333	417.094 (m ⁻¹)	0 (m ⁻²)	1251.408 (m ⁻¹)	0 (m ⁻²)

Table 6. Critical slip surfaces at the end of construction for cases D0-D3: overall safety factor (F) and sums of acting and resisting forces.

Case	X_0 (m)	Y_0 (m)	R (m)	Sum of acting forces (kN/m)				Sum of resisting forces (kN/m)				F (Eq. 3)
				Found.	Embank.	Geosyn. T_a	Geosyn. θ (°)	Found.	Embank.	Geosyn. T_{rg}	Geosyn. T_{rp}	
				$\sum_{i=1}^{N_1} \tau_i l_i$	$\sum_{i=1}^{N_2} \tau_i l_i$			$\sum_{i=1}^{N_1} q_{fi} l_i$	$\sum_{i=1}^{N_2} q_{fi} l_i$			
D0	9.0	30.0	8.849	154.09	7.645	-	-	203.57	15.363	-	-	1.354
D1	8.5	28.5	7.558	150.30	6.362	5.119	62.413	202.28	14.327	50.0	433.39	1.507
D2	8.5	28.5	7.558	149.36	5.625	8.393	62.413	201.65	14.607	100.0	434.26	1.653
D3	9.0	29.0	10.123	212.53	3.791	9.772	66.726	312.60	14.573	150.0	500.03	1.755

R – radius; (X_0, Y_0) – coordinates of centre; $N=N_1+N_2$

Table 7. Critical slip surfaces at the end of consolidation for cases D0-D3: overall safety factor (F) and sums of acting and resisting forces.

Case	X_{θ} (m)	Y_{θ} (m)	R (m)	Sum of acting forces (kN/m)				Sum of resisting forces (kN/m)				F (Eq. 3)
				Found.	Embank.	Geosyn. T_a	Geosyn. θ (°)	Found.	Embank.	Geosyn. T_{rg}	Geosyn. T_{rp}	
				$\sum_{i=1}^{N_1} \tau_i l_i$	$\sum_{i=1}^{N_2} \tau_i l_i$			$\sum_{i=1}^{N_1} q_{fi} l_i$	$\sum_{i=1}^{N_2} q_{fi} l_i$			
D0	9.5	28.0	6.536	124.45	2.680	-	-	181.68	15.506	-	-	1.551
D1	9.5	29.5	9.028	172.44	1.117	3.674	60.101	259.36	16.322	50.0	439.28	1.714
D2	8.5	28.5	7.558	149.70	3.096	8.714	62.413	233.92	15.216	100.0	433.92	1.884
D3	8.5	29.5	11.493	241.80	2.745	12.367	66.950	420.18	23.496	150.0	404.81	2.015

R – radius; (X_0, Y_0) – coordinates of centre; $N=N_1+N_2$

factor. During the post-construction period, in response to consolidation, F increases in all cases, which is due to the increase of the soft soil strength as the excess pore pressure dissipates and effective mean stress increases.

Analysing the resisting forces of the geosynthetic at the cut point (Tables 6 and 7), one can conclude that T_{rg} is significantly smaller than T_{rp} in all cases. This means that the overall failure is determined by the geosynthetic strength and not by the pull-out strength of the soil-geosynthetic interfaces. Another conclusion is that the acting force of the geosynthetic at the cut point, T_a , is also significantly smaller than T_{rg} in all three cases (D1-D3). This reveals that the reserve of geosynthetic resistance remains high in all phases of the construction and post-construction periods, which justifies the larger values of the safety factor in cases D1-D3 than in case without geosynthetic (case D0).

Figure 16 shows the tensile force along the geosynthetic for cases D1-D3, at end of construction and end of consolidation.

As expected, the higher the geosynthetic stiffness, the larger its tensile force (Borges, 1995; Borges & Cardoso, 2001). In all cases, due to the difference of deformability of soft soil and stone columns, tensile force is larger on the columns than on the nearby soft soil, both at the end of construction and end of consolidation. Another conclusion is that, after the end of construction, maximum values of tensile force do not significantly vary in response to consolidation; however, there is a decrease of tensile force in the spaces between columns, which is globally related with horizontal displacements of the geosynthetic during the post-construction period. In order to better understand such decrease, horizontal displacements at two points of the geosynthetic, corresponding to the extremities of the space between columns 2 and 3, for case D3, are indicated below: point A, with x -coordinate equal to 2.237 m; point B, with x -coordinate equal to 3.762 m. The following outward horizontal displacements were obtained at these two points: 1.43×10^{-2} m (end of construction) and

1.29×10^{-2} m (end of consolidation) for point A; 2.36×10^{-2} m (end of construction) and 2.15×10^{-2} m (end of consolidation) for point B. These results show that there is a shortening of the geosynthetic's length between points A and B during

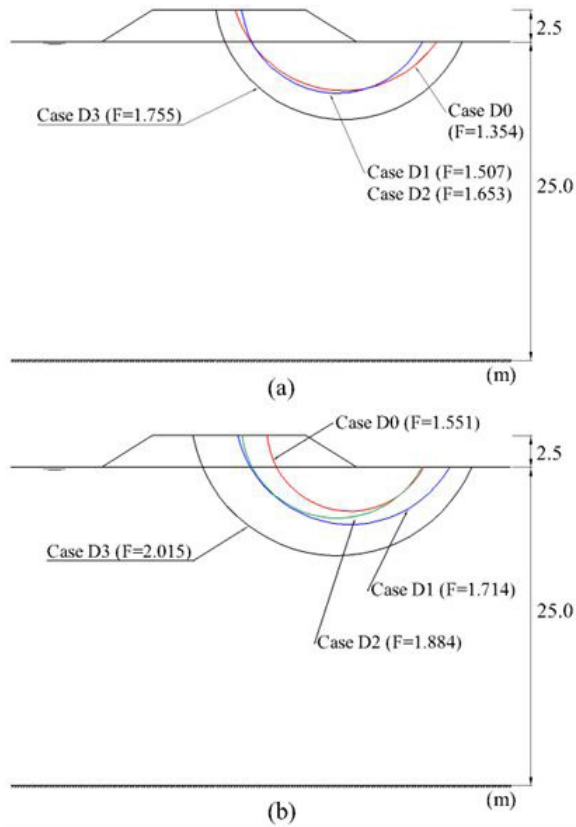


Figure 15. Critical slip surfaces for cases D0-D3: a) end of construction (7th day); b) end of consolidation.

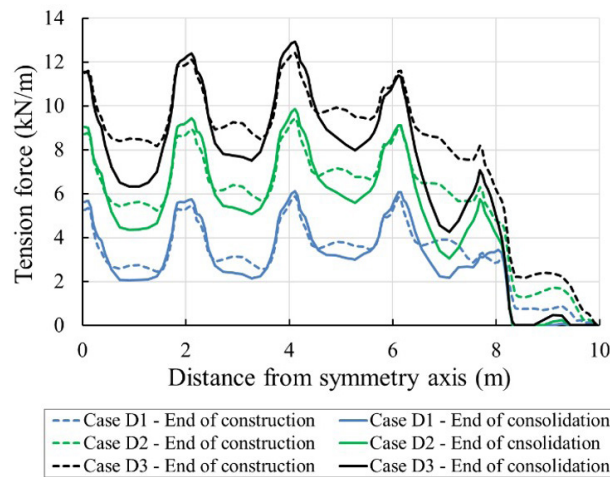


Figure 16. Geosynthetic tensile force for cases D1-D3, at end of construction and end of consolidation.

the post-construction period; i.e., the average tensile strain of geosynthetic between such two points decreases after the end of construction and, therefore, so does the corresponding tensile force, as shown in Figure 16.

The following results are shown in Figures 17 and 18 for cases D0-D3: settlements on the embankment base (Figure 17); horizontal displacements under the embankment toe (Figure 18). These results reveal that the reinforcement with geosynthetic: (i) does not significantly influences the settlements on the embankment base, both at the end

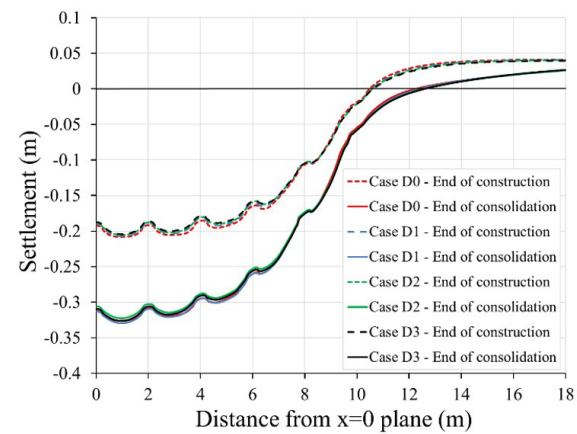


Figure 17. Vertical displacements on the ground surface (embankment base) for cases D0-D3, at the end of construction (7th day) and end of consolidation.

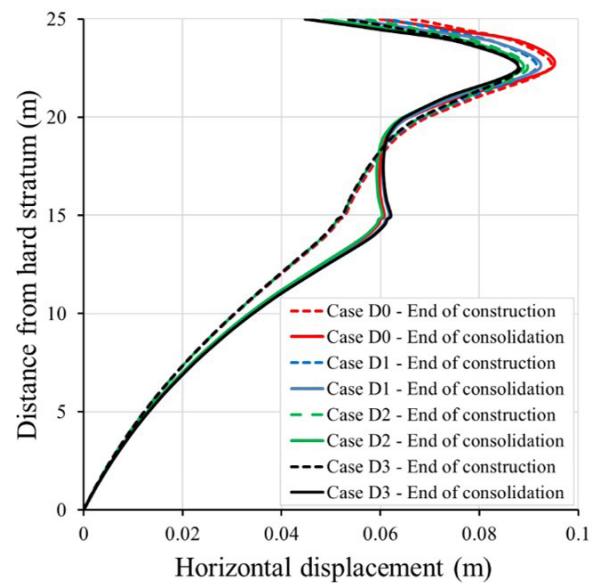


Figure 18. Horizontal displacements under the embankment toe ($x=10$ m) for cases D0-D3, at the end of construction (7th day) and end of consolidation.

of construction and end of consolidation; (ii) reduces a little the horizontal displacements under the embankment toe (on depths up to 3 m, approximately); the higher the geosynthetic stiffness, the larger this reduction, which is precisely explained by the increase of horizontal stiffness due to the inclusion of the geosynthetic. Another conclusion is that the horizontal displacements under the embankment toe do not significantly change after the end of construction in all cases, i.e. they are not significantly influenced by the consolidation process that takes place after the completion of the embankment. This globally explains that maximum values of tensile force in the geosynthetic also do not significantly change after the end of construction, as said above.

5. Conclusions

A parametric study was performed on the behaviour of a floating stone column-supported embankment on a thick soft soil deposit. Two parameters were studied: length of columns; basal reinforcement with geosynthetic (different constitutive curves for the geosynthetic were considered). The following overall conclusions are highlighted:

For values of L/D (ratio of the column length to the vertical thickness of soft soil) smaller than 0.6 approximately, the length of columns significantly influences the behaviour of problem, i.e. the longer the floating stone columns, the lower the settlements, the smaller the horizontal displacements, the higher the overall stability and the smaller the excess pore pressures at the end of construction, mainly below the end of columns. However, for values of L/D larger than 0.6, the study showed that increasing the length of columns does not significantly influence these parameters. This revealed that, also considering the cost, a length of columns between 10 m ($L/D=0.4$) and 15 m ($L/D=0.6$) would be adequate in practice for this problem. Another conclusion is that the length of columns does not significantly influence the load transfer to the top of columns during construction.

Regarding the influence of reinforcement with geosynthetic, the study showed that: (i) the safety factor (F) at the end of construction significantly increases when a geosynthetic is included on the embankment base; (ii) as expected, the higher the geosynthetic strength, the higher the safety factor. On the other hand, settlements on the embankment base are not significantly influenced by the geosynthetic inclusion, both at the end of construction and at the end of consolidation. Another conclusion is that the horizontal displacements under the embankment toe reduce a little with the geosynthetic reinforcement; the higher the geosynthetic stiffness, the larger this reduction. Due to the difference of deformability of soft soil and stone columns, the geosynthetic tensile force is larger on the stone columns than on the nearby soft soil. The higher the geosynthetic stiffness, the larger its tensile force, as expected.

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Declaration of interest

The author has no conflicts of interest to declare.

Data availability

All data of the current study are included in this article.

List of symbols

a_s	area replacement factor
c_r	reduction coefficient
f	yielding function
h	hardening parameter
k	slope of swelling and recompression line
k_h	coefficient of permeability in horizontal direction
k_v	coefficient of permeability in vertical direction
l_i	i -segment length
p	effective mean stress
q	deviatoric stress
q_{fi}	average value of soil shear strength at i -segment
s	axis-to-axis spacing between columns
s_p	plastic tangential relative displacement
u	excess pore pressure
v_i	soil specific volume at i -segment
A_c	column area
A_t	tributary area of natural soil for each column
D	vertical thickness of soft soil
F	overall safety factor
L	length of columns
M	slope of the critical state line
N	specific volume of normally consolidated soil at mean normal stress equal to 1 kPa
N	number of 2D elements of the mesh intersected by the slip circle
N_g	number of geosynthetic layers
T_{aj}	acting tensile force of j -geosynthetic at the cut point
T_{rg}	geosynthetic strength
T_{rp}	pull-out strength of the soil-geosynthetic interface
T_{rj}	resisting tensile force of j -geosynthetic at cut point
Y	hardening law
α_p	p -value of yield surface centre in p - q plane
γ	unit weight
δ	interface friction angle
ε_p	plastic tensile strain
θ	angular stress invariant

θ_j	angle between the j -geosynthetic and the tangent to the slip circle at the cut point
λ	slope of normal consolidation line and critical state line
ν'	Poisson's ratio for drained loading
σ	tensile stress
σ_n	normal stress
σ'_x	effective normal stress in x -direction
σ'_y	effective normal stress in y -direction
σ_y	yielding stress
σ'_z	effective normal stress in z -direction
τ	tangential stress
τ_i	average value of acting shear stress at i -segment
τ_{xy}	shear stresses in xy -plane
ϕ'	angle of friction defined in effective terms
Γ	specific volume of soil on the critical state line at mean normal stress equal to 1 kPa

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Experiments on suffusion of coarse soils from an ageing British dam

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Article

Keywords

Ageing earth dam
Internal erosion
Suffusion
Coarse soils
Laboratory tests

Abstract

This study presents a laboratory investigation on the susceptibility of suffusion in soil samples collected from a very old earth dam. The dam, rebuilt around 1800, is composed of two earth fills and lacks a clay core or filtration components, making it vulnerable to internal erosion. Following an inspection in 2006, concerns were raised about the dam's vulnerability to internal erosion during strong floods. Boreholes and vibro-cores were performed to collect samples, which were then tested for suffusion susceptibility. The study found that four out of six samples were highly susceptible to suffusion erosion, with the erosion process starting at gradients much lower than unity. The opposite was observed for samples with fines content greater than 30% and showing some plasticity. The results were compared to predictions from seven geometric criteria found in the literature, but none of the methods could forecast the behaviour of all tests. The study highlights the need for laboratory tests when there is no agreement between the predictions from available criteria. The findings of this study were used to inform the rehabilitation of the dam, which included installing sand filters designed to capture the smallest particles and relief wells distributed along the entire length and height of the eastern embankment. The study demonstrates the importance of assessing suffusion susceptibility in embankment dam safety control and the need for laboratory tests to validate predictions from geometric criteria.

1. Introduction

The assessment of suffusion is of utmost importance in embankment dam safety control. Suffusion occurs when the finer particles of the soil are carried away by seepage through the constrictions between the coarser particles (ICOLD, 2015). This paper presents a laboratory investigation on the susceptibility of suffusion of soil samples collected from a very old earth dam. It was rebuilt by about 1800 and is composed of two earth fills. The main eastern earth fill is 560 metres long with a maximum height of 12 metres, a crest width of 15 metres and maximum width at its base of 70 metres. The secondary western earth fill is 1300 metres long with a maximum height of 14 metres. The dam fill is heterogeneous with sand, gravel and clay fill, the sands and gravels predominate. The dam has no core or other waterproofing element or filter protection, consequently it is vulnerable to failure by internal erosion. The engineers were concerned about erosion by suffusion, by contact erosion and-or by backward erosion and piping causing unravelling of the downstream slopes (ICOLD, 2017).

Following an inspection in 2006, in which leakages in the Eastern dam were identified, some concerns were raised

about the vulnerability of the dam to internal erosion during strong floods (Thompson, 2011). Boreholes were drilled and trial pits were dug to assess the composition of the dam and its foundations. The findings from these investigations reveal that the dam is best defined as unzoned heterogeneous dam, lacking a clay core or filtration components. Core samples of the dam have also been collected to test their resistance to internal erosion.

Prediction of internal erosion in compacted heterogeneous soils is a recurrent challenge for civil engineers. Gravel-sand-silt-clay mixtures used as fill materials in earthen structures are subjected to seepage flows that can induce suffusion (Benamar et al., 2019). Kezdi (1979) introduced the term suffusion as a phenomenon in which water, seeping through the pores, transports finer particles without disrupting the soil structure. Kovács (1981) defined soil suffusion as the rearrangement of fine grains within a layer, wherein the solid volume remains constant, and only the local permeability undergoes alteration. Soils prone to suffusion are typically recognized based on the configuration of their particle size distribution curve. Soils exhibiting concave upward or gap-graded curves are anticipated to be susceptible to suffusion (Lafleur et al., 1989; Skempton & Brogan, 1994; Fannin & Moffat, 2006; Wan & Fell, 2008).

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The onset of internal instability in soils is determined by a blend of hydromechanical and geometrical factors (Moffat & Fannin, 2006). There is significant apprehension regarding the safety of embankment dams and levees constructed using soils with the susceptibility for internal instability (Ke & Takahashi, 2012; Liang et al., 2017). A significant volumetric deformation may also arise from the washing out of fine particles (Liang et al., 2019). The migration of finer grains toward an exit point can induce substantial settlement, resulting in the loss of freeboard, subsequent overtopping, and eventual dam failure (Fell et al., 2003; Wan & Fell, 2008; Douglas et al., 2019).

There is a need, in professional practice, to better understand the phenomenon and to develop improved methods to evaluate the susceptibility of an embankment soil to suffusion (Correia dos Santos et al., 2015, 2017). In literature focus has been placed on establishing geometric criteria to classify soils in terms of internal stability (Istomina, 1957; Kezdi, 1979; Sherard, 1979; Kenney & Lau, 1985, 1986; Burenkova, 1993; Wan & Fell, 2008).

Kenney & Lau (1985, 1986) emphasized the internal instability and susceptibility to suffusion in soils, asserting that the percentage of the finer fraction (beyond the inflection point on the particle size plot) must be smaller than the available void space. They suggested a threshold of 20% for well-graded soils and 30% for narrowly graded soils. Wan & Fell (2008) expanded the theoretical limit to 40%, revealing this potential in soil samples with fines content ranging from 22% to 33% for broadly graded soils and 29% to 38% for gap-graded soils.

More recently, Deng et al. (2023) carried out large-scale erosion tests on sandy gravels (without fines content) under different hydraulic loading and surcharge pressures, concluding that the seepage velocity is a better measure for the hydraulic loading than hydraulic gradient. Additionally, the findings indicated that the erosion rate exhibits a proportional decrease with the volume fraction of remaining erodible fine sand particles at a given seepage velocity.

Visual observation and turbidity measurements carried out in the laboratory tests provide a comprehensive description

of samples internal instability, enabling the identification of the onset of the suffusion process. The critical hydraulic gradient that initiates suffusion of particles is explored. Criteria proposed in the literature to determine the susceptibility to suffusion erosion is assessed for the tested soils and then compared against the experimental data. In many foundations with heterogeneous soil structures, suffusion and contact erosion can interact simultaneously, especially in double-layered or multilayered soil foundations with a cut-off wall. The retention ratio D_{c35}/D_{85SA} (D_{c35} is the constriction size corresponding to 35% finer in coarser fraction and D_{85SA} is the grain size corresponding to 85% finer by surface area) between soil layers was used in an experimental study (Zhang et al., 2023) and the results reveal that the effect of the retention ratio on suffusion and contact erosion exhibits a consistent trend. When this ratio exceeds the value of 1, simultaneous suffusion and contact erosion can occur.

2. Materials tested

The two embankments are built with materials consisting of a mixture of silty sands with some gravel, glacial till, and residual soils. Boreholes and vibro-cores were performed in the embankment of the eastern dam and samples were extracted to carry out suffusion tests. The samples for suffusion testing were retrieved from the dam by vibro-core (VC) drilling (Figure 1). This drilling method enables samples to be taken less remoulded below the water table and avoids loss of fine material inherent with cable tool boring in such material. Vibro coring uses a 150 mm diameter steel tube vibrated into the ground below water table to recover ‘grading complete’ samples of granular materials, which are retained as a continuous core sealed within a plastic sleeve inside the tube. Samples were selected from the plastic tubes by the authors for specialist testing to assess their vulnerability to suffusion. Table 1 presents the description of the samples from the boring logs, as well as the fines content and the estimated

Table 1. Main features of tested soil specimens from boreholes.

Soils collected	Layer (m)	Fines content* (%)	Dry density (g/cm ³)	Description of soil layer
VC101-1	3 – 3.5	14.3	2.17	Brown to grey silty sand with gravels, disturbed, very soft – silty sand with gravel
VC101-2	3.5 – 4.5	15.2	1.62	Brown silty sand with gravels, disturbed, very soft - silty sand with gravel
VC101A	3 – 3.5	44.4	1.63	Grey to black silty sand with gravels, disturbed, very soft - silty sand with gravel
VC103-1	4.5 – 5	17.0	1.50	Brown silty sand with gravels, disturbed, soft- silty sand with gravel
VC103-2	7 – 7.3	16.9	1.61	Brown silty gravel with sand matrix, disturbed, very soft- silty sand with gravel
VC103-3	7.8 – 8.1	30.3	1.51	Red silty fine sand with gravels, less disturbed – silty, clayey sand with gravel

*Fraction passing no. 200 ASTM sieve. All samples shown low plasticity index ($PI < 6$).

dry density of each sample. Six samples are obtained from three bore holes VC101, VC101A (in the vicinity of VC101) and VC103, at depths between 3 m and 8.1 m.

Samples to be tested against suffusion were carefully collected from each core and placed in the same conditions (density, moisture) as in field in the testing cell. To get less

remoulded sample, the testing cylinder was used to core a subsample from the provided core sample (VC103-3) owing to its fine grading. Previously, a small amount of each sample was collected for sieving and plasticity analysis, and the resulting grain size distribution curves are presented in Figure 2. The sample VC101A is a widely graded soil with 44.4% of fines and 26.2% of gravel. All remaining samples show a gap-graded curve with a deficiency of particles in the 0.08 to 0.30 mm range. According to the literature relating to grading shape, these gap-graded samples are expected to be susceptible to internal instability. The missing middle size fraction in the grading may lead to the finer particles being moved within the pores of coarser matrix. Sample VC103-3 contains a fines content around 30% and has no gravel particles. All samples showed a low plasticity, with plasticity index less than 6, and sample VC101A was the more plastic one.

3. Test setup and procedures

Figure 3 shows the experimental arrangement used for suffusion tests. The samples are placed inside a cylinder mould made of acrylic glass that is then connected to a freshwater tank and subjected to upward flow. Two sizes of mould were used. The larger one is 90 mm in diameter and 150 mm long, and the smaller one is 40 mm in diameter and 120 mm long. The soil sample extracted from the disturbed core was compacted at the same wet density into the larger test mould. This method of sample preparation is applied for all the tests, except for sample VC103-3 (with no gravel content), in which a sub core is sampled to be used in the 40 mm diameter mould. The specimen rests over a 30 mm glass spheres layer used to diffuse uniformly the fluid in the sample. There is a rigid metal grid (opening size of 1.1 mm) covered by a mesh screen (80 μ m opening size) that prevent loss of fines between the soil and the glass beads layer. A mesh with an opening size of 1.2 mm is used on the upper face to allow free washing out of eroded particles. The mould is kept

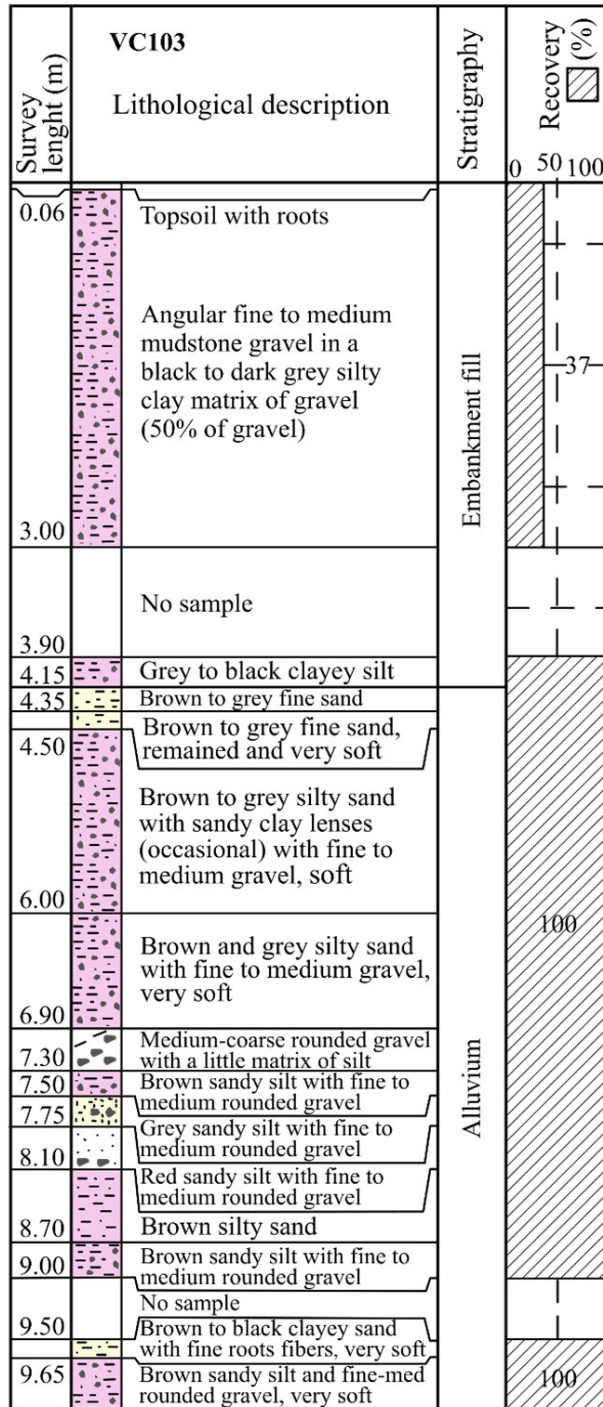


Figure 1. Stratigraphy of one collected core sample VC103 (Dam Safety, UK).

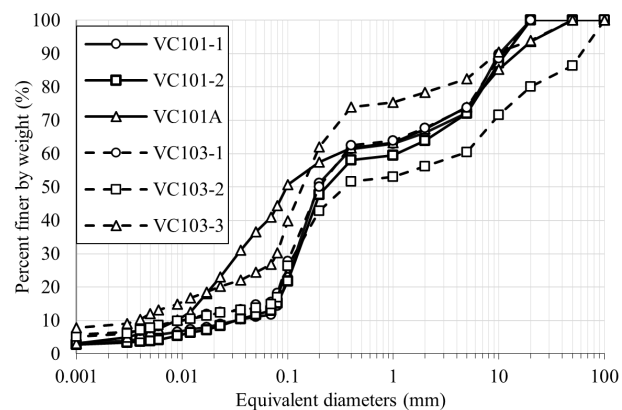


Figure 2. Particle-size distribution curves of tested soils.

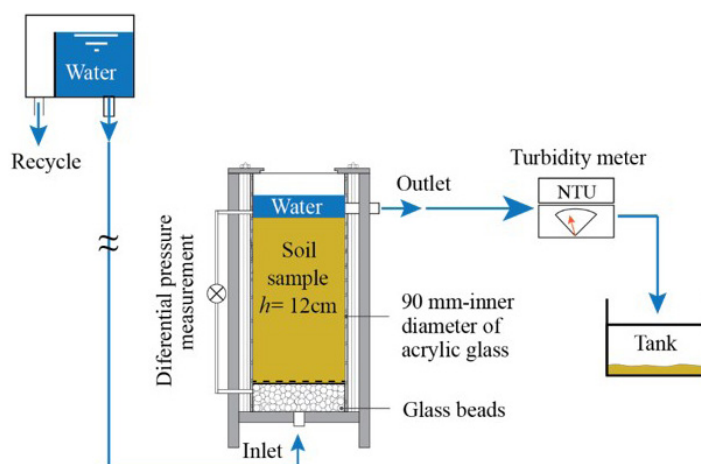


Figure 3. Schema of the typical experimental arrangement.

vertical during the test and is fed by a water supply which can be moved up to increase the hydraulic load applied to the specimen.

To closely match the chemical composition of the reservoir water, salt (NaCl and CaCl_2) is added into the water, aiming for a target pH value near 8.1. The outlet flow is directed towards a turbidity meter (Kobold 5000 NTU), with its readings previously correlated to fines concentrations in the effluent. A differential pressure transducer, positioned between the upstream and downstream extremities of the sample, measures the water head difference, providing the average hydraulic gradient along the sample. The volumetric discharge rate is measured periodically (minute-by-minute) to determine the corresponding hydraulic conductivity. Eroded particles are gathered in a container, and samples from the outlet flow are collected every minute for the analysis of particle size distributions with an automatic particle measuring system (Mastersizer 2000MU from Malvern).

The main objectives of the tests are the investigation of the critical gradient at which suffusion is initiated and the erosion kinetics. For this purpose, the water head in the tank is increased until suffusion is initiated and is then held as long as particles are being washed out from the soil sample. When turbidity falls to no significant value, the water head is increased to the next step and kept constant if suffusion continues. Several hydraulic loads are thus applied successively to the sample. The observation of the suffusion process together with the turbidity measurements allows the identification of the critical hydraulic gradient which initiates the migration of particles.

4. Test results and analysis

4.1. Suffusion results and evolution of soil parameters

In each test, the total hydraulic gradient is increased in steps of 0.1. The gradient of each step is left constant, at least,

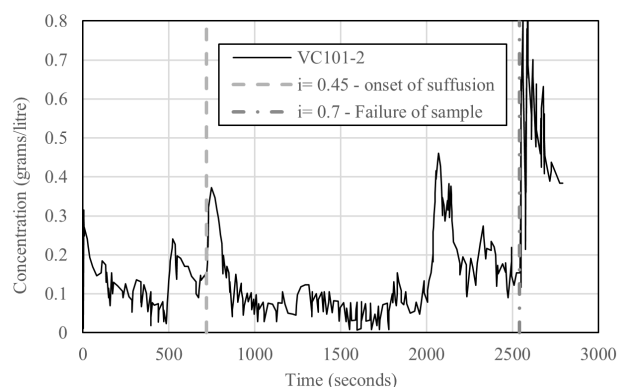


Figure 4. Typical turbidity curve measured during test (sample VC101-2).

until stabilization of the outlet flowrate. The critical gradient occurs for the minimum head at which erosion is clearly first initiated. Erosion of fines typically occurs at the start of the tests at low gradients, leading to some effluent turbidity. This is caused by the boundary conditions of the soil resulting in an initial wash of dirt and dry powder surrounding the soil specimen and is thus disregarded. Figure 4 illustrates a representative record (sample VC101-2) depicting the turbidity of the effluent and its progression with an increasing hydraulic gradient.

Suffusion starts at a hydraulic gradient close 0.45, and prior to this threshold value no significant erosion has been observed. The failure occurs when strong “boiling phenomenon” is visible at the specimen surface, for an average gradient of approximately 0.7, at an elapsed time of $t = 2500$ seconds. The maximum erosion rate is also used as quantitative mass erosion information because the erosion rate in the tests reached its maximum value quickly and did not remain constant during a single hydraulic load increment. In a typical test with occurrence of suffusion (Figure 5),

preferential pathways for erosion are clearly identified along the sample for low hydraulic gradients.

Table 2 summarizes the results obtained from the six suffusion tests, including the critical gradient, the cumulative eroded mass, the percentage of the fines eroded, the initial and final hydraulic conductivity of the samples, and the maximum applied hydraulic gradient. Sample VC103-3 proved to be highly resistant to suffusion erosion for upward flow, with the sample exhibiting uplift. For that reason, the sample VC103-3 was tested using the smaller mould and subjected to horizontal flow. The critical gradient observed in tests on

samples VC101-1, VC102-2, VC102-1 and VC103-2 are considerable smaller than the theoretical critical hydraulic gradient (Terzaghi et al., 1996) which is expected to be around the unity. This is an indication of the high susceptibility of suffusion of those samples. The samples VC101A and VC103-3, which have fines content above 30%, showed critical gradients much higher than the unity. Sample VC103-3 only show signs of erosion for very high gradients (above 7.5). Among the samples with higher fines content, VC103-3 is the only one that does not have gravel, which justifies its high erosion resistance, owing to its higher plasticity (PI close to 6).

4.2. Assessment of internal stability using the available methods

Several studies investigate the geometrical conditions of internal erosion using the particle size distribution. Empirical approaches have been proposed to evaluate the potential for internal instability. They are based on the shape of the gradation curve and, more specifically, on the variation in grain size over a designated portion of the curve. Istomina (1957) proposes three criteria concerning suffusion erosion: absence of erosion if $U < 10$; transitional state for $10 < U < 20$; and occurrence of erosion when $U > 20$. U is the uniformity coefficient of the material, given by Equation 1, being D_{60} and D_{10} the diameters of soil particles for which 60% and 10% of the particles mass are finer, respectively.

$$U = D_{60} / D_{10} \quad (1)$$

The Sherard (1979) criterion defined by the ratio D_{15}/D_{85} equal to 5 involves dividing the gradation at arbitrary points along the curve, to obtain the D_{15} for the coarse fraction and D_{85} for the fine fraction. The results obtained using this criterion show that the assessing of the internal instability of the soil depends on the particle size of the gradation splitting. In addition, Sherard method is formulated and intended to identify soils that do not exhibit self-filtering, a process distinct from internal



Figure 5. Aspect of sample VC103-1 at the end of test.

Table 2. Testing program, soil properties and main findings.

Soil specimens tested	Initial hydraulic conductivity (10^{-4} m/s)	Critical gradient from test, i_{cr}	Maximum gradient in test	Final hydraulic conductivity (10^{-4} m/s)	Mass loss (g)	Relative eroded mass of fines (%)	Visual assessment of soil stability
VC101-1*	3.0	0.35	1.7	3.0	5.0	2.0	Unstable
VC101-2*	12.0	0.45	0.8	19.0	6.7	3.3	Unstable
VC101A*	6.6	1.87	2.2	7.8	6.3	1.0	Stable
VC103-1*	30.0	0.32	0.6	6.1	10.2	4.8	Unstable
VC103-2*	50.0	0.22	0.5	269.4	8.4	3.7	Unstable
VC103-3**	2.0	7.50	10.0	1.9	14.1	24.5	Stable

* Test with upward flow in permeameter with 90 mm diameter and 150 mm length. ** Test with horizontal flow in permeameter with 40 mm diameter and 120 mm length.

instability. Kezdi (1979) suggests a criterion for internal erosion based on the index ratio D_{15}/D_{85} equal to 4. Fannin & Moffat (2006) experimentally demonstrate that gradations near that ratio exhibit stability, while a soil with $D_{15}/D_{85} = 7$ displayed instability behaviour at a relatively low gradient. Similarly, Kenney & Lau (1985, 1986) propose a graphical technique for evaluating a soil's susceptibility to internal erosion by examining the grading curve's shape. F represents a mass fraction smaller than D , where D is the particle size, and H is the mass fraction measured between particle size D and $4D$. The gradation curve is divided to determine the incremental mass fraction passing (H) across a range of particle diameter D to $4D$, which is then compared with the mass passing (F) at the lower limit of the interval D . Thresholds of $H = F$ are recommended for soils with a widely graded coarse fraction when $F < 0.2$ and for soils with a narrowly graded coarse fraction when $F < 0.3$. The area above $H = F$ corresponds to the stable zone. his approach was validated through laboratory permeameter tests (Skempton & Brogan, 1994). Li & Fannin (2008) showed that the Kezdi (1979) method tends to be more conservative for $F < 0.15$, while the Kenney & Lau (1985, 1986) method exhibits greater conservatism for $F > 0.15$. The Burenkova (1993) method relies on the ratios h' (Equation 2) and h'' (Equation 3), where d_{90} represents the sieve size at which 90% of the sample by weight passes.

$$h' = D_{90} / D_{60} \quad (2)$$

$$h'' = D_{90} / D_{15} \quad (3)$$

The h' ratio reflects the steepness of the particle size distribution's coarse section, while the h'' ratio serves as an indicator of the filtration effect between the coarse and finer fractions. Although the Burenkova (1993) method better but not totally accurate predictions of the internal stability of soils tested by Wan & Fell (2008). These authors expand

Burenkova's criteria to encompass clay-silt-sand-gravel soils with limited clay-sized particles and introduce a probabilistic approach for predicting suffusion. Wan & Fell (2008) estimate that P the probability that a soil is internally unstable is given by Equation 4, where z is given by Equation 5. Wan & Fell (2008) also propose an alternative criterion, with alternative boundaries that are defined by D_{90}/D_{60} and D_{20}/D_5 .

$$P = e^z / (1 + e^z) \quad (4)$$

$$z = 2.38 \log(h'') - 3.65h' + 3.7 \quad (5)$$

The six soil samples tested have been assessed for internal instability using methods previously described. Figure 6 plots H versus F curves for each sample, and shows the boundaries defined in Kenney & Lau (1986) and Kezdi (1979) criteria. Figure 7 plots the pairs h' and h'' for each sample, and shows the boundaries defined by Burenkova (1993). Figure 8 plots the results of application of the alternative method proposed by Wan & Fell (2008). Table 3 presents a summary of the classification about internal stability of the soils for all criteria mentioned, as well as the findings from the experimental data.

Relevant findings are that none of the applied criteria met the test results for all samples, and that, for some samples, the different criteria produce conflicting results with each other. Sherard (1979) and Kezdi (1979) criteria are clearly non conservative, suggesting that all samples are stable when selecting a slip gradation at the sizes 0.08 mm and 0.1 mm. For a higher slip size of 0.2 mm the sample VC101A exhibits D_{15}/D_{85} of 12.4. This exceeds the value of 4 proposed by Kezdi (1979) for gap gradations, indicating susceptibility to suffusion (Figure 6). However, contrasting results are obtained from the laboratory suffusion test. Istomina (1957) and Burenkova (1993) criteria suggests

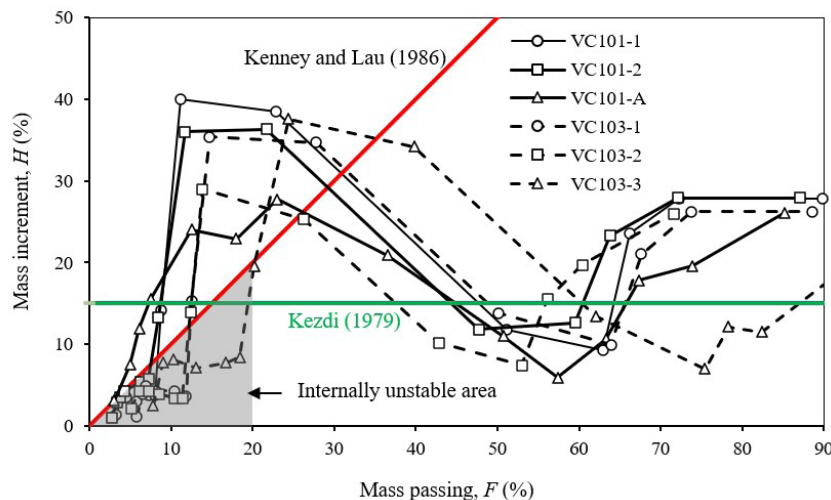


Figure 6. Kenney & Lau (1986) criterion and Kezdi (1979) criterion applied to tested soils.

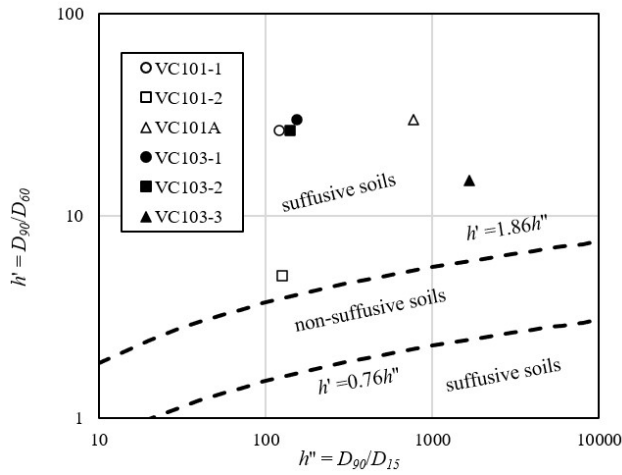


Figure 7. Burenkova (1993) method for assessment of internal stability of tested soils.

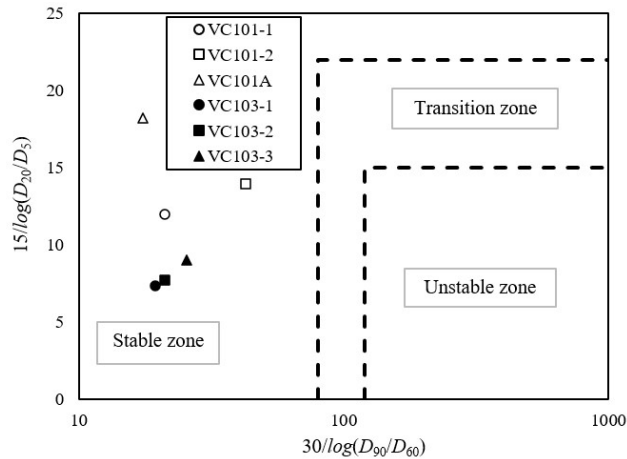


Figure 8. Wan & Fell (2008) alternative criteria for internal instability applied to tested soils.

that almost all tested soils are deemed unstable (Figure 7), indicating the conservative nature of such methods. Kenney & Lau (1985, 1986) method fails at predicting the suffusive behaviour shown in tests on samples VC101-1 and VC101-2, and, in addition, classifies sample VC103-3 as unstable, when laboratory tests show otherwise. One should note however that Kenney & Lau (1985, 1986) criteria is derived from experiments on clean coarse sand-gravel soils (with no fines). Sample VC103-2 that has the higher gravel content and one of the lowest fines content showed suffusive behaviour in experiments. That behaviour is well predicted by Kenney & Lau (1985, 1986), since most of the sample shape curve shows $H < F$ in the region defined by $F < 20\%$ (Figure 6). Both methods (probabilistic and deterministic) criteria defined by Wan & Fell (2008) are over-optimistic at predicting all samples as internally stable. The probabilities of soils being susceptible to suffusion are very low ($P < 10^{-5}$), and the parameter $30/\log(d_{90}/d_{60})$ is in all samples smaller than the threshold of 80 (Figure 8). So, many criteria which were assessed for coarse soils are not applicable for soils containing more than 30% fines (VC101A and VC103-3).

4.3. Eroded particles analysis

The washed-out material deposited at the downstream tank is collected at the end of the tests, and then subjected to particle size analysis. Table 4 shows the maximum gradient applied to the samples, i_{max} , as well as the total mass of dry soil collected, and the corresponding medium size, d_{50} , and maximum size, d_{max} , of the eroded particles. One should state that the mass of eroded soil is not by itself a measure of soil instability and it is not comparable between tests. This is because some of the samples are subjected to high gradients

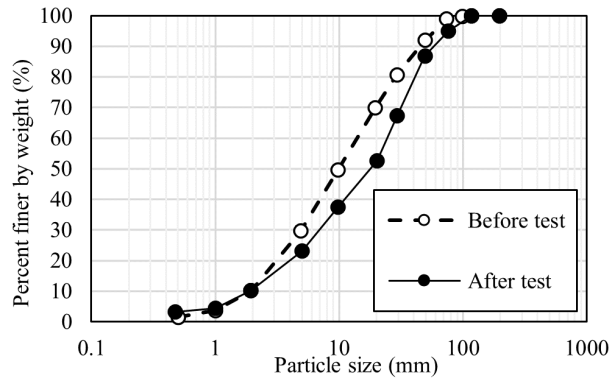
Table 3. Summary of results of upward flow tests and assessment of internal instability.

Criteria	Size split	VC101-1	VC101-2	VC101A	VC103-1	VC103-2	VC103-3
D_{15}/D_{85}	0.08 mm	0.07	0.07	0.06	0.07	0.07	0.06
	0.1 mm	1.56	1.67	3.13	1.67	1.89	1.56
	0.2 mm	2.24	2.00	12.44	1.88	2.50	1.84
$U = D_{60}/D_{10}$		10.86	28.57	42.22	38.00	500.00	45.00
Sherard		Stable	Stable	S/U*	Stable	Stable	Stable
Kezdi		Stable	Stable	S/U*	Stable	Stable	Stable
Istomina		Transition	Unstable	Unstable	Unstable	Unstable	Unstable
Burenkova		Unstable	Unstable	Unstable	Unstable	Unstable	Unstable
Kenney & Lau **		Stable	Stable	Stable	Unstable	Unstable	Unstable
Wan & Fell ***		Stable	Stable	Stable	Stable	Stable	Stable
Experimental results		Unstable	Unstable	Stable	Unstable	Unstable	Stable

* S/U means that sample is stable for size split of 0.08 mm and 0.1 mm and unstable for size split of 0.2 mm. ** Considering the criteria $H/F < 1$. *** Probability of soils being internally unstable, P , is lower than 10^{-5} for all samples.

Table 4. Size of eroded particles and comparison of measured and estimated critical gradient.

Sample	Applied gradient, i_{max}	Cumulative mass of eroded soil (g)	Eroded particle sizes	
			d_{50} (μm)	d_{max} (μm)
VC101-1	1.7	5.0	8	60
VC101-2	0.8	6.7	8	60
VC101A	2.2	6.3	20	200
VC103-1	0.6	10.2	8	80
VC103-2	0.5	8.4	11	260
VC103-3	10	14.1	11	100

**Figure 9.** Particle size distributions of fines (VC103-3) before and after suffusion test.

that are unlikely to occur in dams, and the duration of tests and applied hydraulic gradients are significantly different.

Particle size analysis of the eroded soil collected is compared against the fraction of the original soil that has particles of equal or smaller size than the size of the maximum particle eroded. As example, Figure 9 shows the gradation curve of the material eroded in the suffusion test on sample VC103-3, and the corresponding fraction of the original soil. One can deduce that lost particles are those of size ranging between 2 and 80 μm . Wan & Fell (2008) inferred that approximately half of the finer fraction, determined by the point of inflection in broadly graded soils and the fine limit of the gap in gap-graded soils, undergoes erosion. This statement is verified for sample VC103-3 where the point of inflection on the grading curve is close to 80 μm . No relation is established between the susceptibility to suffusion and the grain size of eroded particles. However, particles washed out are mainly fines ($<80 \mu\text{m}$) with a mean diameter close to 10 μm . Comparison of the mean size (d_{50}) of eroded particles from the different samples indicates that the coarser particles eroded from the soils have a maximum size close to 0.26 mm and sample VC101A has the coarsest size distribution of eroded particles.

5. Conclusion

Visual inspections revealed leakages in the Eastern embankment of a British dam, prompting concerns about

its vulnerability to internal erosion during extreme storms and floods. In response, erosion tests were conducted on samples from various dam sections. Boreholes and vibro-cores were utilized to collect samples from the earth fill. Physical characterization unveiled the gap-graded nature of most samples and their low plasticity, raising doubts about their susceptibility to suffusion erosion.

Subsequently, upward flow suffusion tests were performed on six samples. Experimental data showed that four samples (VC101-1, VC101-2, VC103-1 and VC103-2) are indeed highly susceptible to suffusion erosion, with erosion process phenomenon starting at gradients much lower than the unity (as low as 0.22). The opposite happens with VC101A and VC103-3, whose fines content is greater than 30% and showing some plasticity. Sample VC103-3 proved to be resistant to suffusion for upward flow, with uplift occurring for a high gradient of 7.5, which is likely due to the well graded property of this sample and its plasticity.

The results of the laboratory tests are compared to predictions from seven geometric criteria found in the literature, provided by six different research teams, to assess the potential for suffusion and internal stability of soils. It is found that none of the methods proved to be capable of forecasting the behaviour of all tests. It is also interesting that, in many cases, opposite conclusions are suggested between different methods. Istomina (1957) and Burenkova (1993) criteria are quite conservative, whereas the criteria by Sherard (1979) and Kezdi (1979) and two criteria by Wan & Fell (2008) are very optimistic. Kenney & Lau (1986) criteria appear to predict well the suffusive behaviour of the samples with lower fines content. The apparent contradictions between methods are certainly because the various criteria are based in experiments on different soil types and using distinct test setups and hydraulic loads.

The paramount finding from this study is about the absolute need to carry out suffusion tests when there is no agreement between the predictions from the available criteria. Most of tested samples from the eastern embankment dam exhibits a susceptibility to suffusion.

Following this conclusion, the dam was subjected to rehabilitation, and measures to improve its resistance to erosion have been taken. The suggested remedy includes installing sand filters designed to capture the smallest particles, complemented

by relief wells distributed along the entire length and height of the eastern embankment (Thompson 2011). This solution entailed removing the topsoil and vegetation from the surface, installing a 200 mm layer of graded fine filter material, covering it with a second layer of coarser graded filter material to a depth of approximately 200 mm, and finally, surfacing it with the replaced topsoil. In addition, the gradient of the downstream face was reduced from a typical 1H:3V slope to 1H:2.5V.

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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

Ahmed Benamar: conceptualization, test conduction, writing, supervision, validation – original draft. Ricardo Correia dos Santos: data curation, analysis of test results, methodology, writing.

Data availability

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

List of symbols and abbreviations

d_{50}	Grain medium size of sample
d_{max}	Maximum grain size of sample
i	Hydraulic gradient
i_{cr}	Critical gradient
i_{max}	Maximum gradient applied
h'	Ratio of gradation for estimation of soil instability
h''	Ratio of gradation for estimation of soil instability
pH	Potential of hydrogen
t	Elapsed time
z	Parameter influencing probability of soil being unstable-
ASTM	American Society of Testing Materials
$CaCl_2$	Calcium chloride
D	Particle size
D_{10}	Diameter of soil particles for which 10% of the particles mass are finer
D_{15}	Diameter of soil particles for which 15% of the particles mass are finer
D_{60}	Diameter of soil particles for which 60% of the particles mass are finer

D_{90}	Diameter of soil particles for which 90% of the particles mass are finer
D_{85SA}	Grain size corresponding to 85% finer by surface area
Dc_{35}	Constriction size corresponding to 35% finer
F	Mass fraction
H	Mass fraction measured
H:V	Slope geometric ratio between horizontal and vertical
ICOLD	International Commission on Large Dams
NaCl	Sodium chloride
NTU	Nephelometric Turbidity Unit
P	Probability that a soil is internally unstable
PI	Plasticity index
S	Stable soil
U	Unstable soil
U	Uniformity coefficient
UK	United Kingdom
VC	Vibro-core drilling
VCx	Vibro-core sample with number x

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Performance of geocell reinforced rubber sand mixtures under undrained triaxial test

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Article

Keywords:

Rubber sand mixture
Geocell
Shear strength
Energy absorption capacity

Abstract

Large quantities of scrap tires are produced by the automobile industry each year, which causes disposal challenges and impacts the environment. However, scrap tires exhibit various properties, including tensile strength, abrasion resistance, durability, thermal conductivity, elasticity, and more. Due to their versatile characteristics, these scrap tires can be utilized as construction materials for various civil engineering works to reduce their negative environmental effects and conserve natural resources. This study aims to understand the shear strength behaviours exhibited by geocell-reinforced mixtures of rubber and sand through the unconsolidated undrained triaxial test. Various parameters, including rubber sizes (425 μm to 12 mm), rubber contents (10% to 40% by volume), confining pressures (50 to 300 kPa), and geocell heights (0.2H to 0.8H, where H is the height of triaxial sample), were systematically examined to understand their impact on shear strength characteristics. The experimental findings reveal that deviatoric stress is enhanced with increasing confining pressure and rubber sizes. The maximum benefits of the rubber-sand mixture were observed at 30% rubber content. Geocell-reinforced rubber sand mixture has a higher shear strength with respect to the unreinforced mixture. Furthermore, the energy absorption capacity of the geocell-reinforced rubber sand mixtures was much better as compared to either the clean sand or rubber-sand mixture. The findings of this research demonstrate that geocell-reinforced rubber sand mixtures are suitable for various geotechnical engineering works.

1. Introduction

The growing world population and the globalization of the automobile sector have collectively contributed to a substantial increase in scrap tire generation over the past few decades (Tajabadipour & Marandi, 2017). It has led to a serious environmental problem with their disposal because of their high production volume, durability, and availability of environmentally unfavourable components (Hennebert et al., 2014; Mrad & El Samra, 2020; Tajabadipour & Marandi, 2017). Scrap tires also affect the entire world because they are non-biodegradable, easily ignited, able to produce dangerous vapours when burned, release toxins that contaminate the soil and water, and are able to leach poisonous compounds into the environment (Attom, 2006; Ganjian et al., 2009). Therefore, it would be important to propose some remedies to such a problem. To minimize these problems, waste tires can now be recycled into fibres, strips, chips, granules, crumbed rubber, or shreds that can be utilized as building materials in several civil engineering works (Anvari & Shooshpasha, 2016; Attom, 2006; Das & Bhowmik, 2020; Edinçliler et al., 2010; Ganjian et al., 2009; Lee et al., 1999;

Oikonomou & Mavridou, 2009; Yang et al., 2020; Yoon et al., 2006). Utilizing waste tires mixed with soil is becoming more and more frequent because of the scarcity of natural resources and the high cost of tire waste disposal (Moghaddas Tafreshi et al., 2019; Oikonomou & Mavridou, 2009). Waste tires have certain characteristics that make them desirable as soil-reinforcing materials, such as high flexibility, thermal insulation, durability, tensile strength, elasticity, abrasion resistance, low weight, and more (Abdullah et al., 2022; Akbarimehr et al., 2020; Lee et al., 1999; Madhusudhan et al., 2021; Yang et al., 2020; Zhang et al., 2018).

The use of waste tires mixed with sand is gaining popularity since it can enhance several engineering characteristics of sand, including improving shear strength characteristics, lowering deformation, raising friction angle, enhancing energy absorption capacity, etc. (Al-Neami, 2018; Anbazhagan et al., 2017; El-Sherbiny et al., 2013; Neaz Sheikh et al., 2013; Rouhanifar et al., 2021; Silva et al., 2020). A study conducted by Anbazhagan et al. investigated the influence of rubber contents on the strength characteristics of rubber-sand mixtures. Their findings demonstrated that the rubber-sand mixture achieves its highest shear

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strength when the rubber content is optimally set at 30% by volume (Anbazhagan et al., 2017). Most of the research on rubber sand mixtures suggested that a rubber content of 20–35% provided the greatest advantage (Li et al., 2020; Madhusudhan et al., 2021; Yang et al., 2020). Furthermore, several investigations have reported that rubber sand mixtures can be effectively used as vibration seismic isolation for small structures due to the dampness characteristics of the waste tire (Bandyopadhyay et al., 2015; Chew & Leong, 2019; Senetakis et al., 2012; Shariatmadari et al., 2018; Tsang, 2008). Tsang et al. performed the seismic isolation method by mixing the rubber into the soil and placing it around the building foundation, which provides a function like a cushion, and their results showed that the rubber-sand mixture reduced the ground motion by around 40–60%, making it suitable for use as vibration isolation for low-to medium-rise buildings (Tsang et al., 2012).

In the past few decades, a range of geosynthetic materials, such as geotextile, geocell, geogrid, etc., have found successful applications in soil reinforcement. These materials have proven effective in improving bearing capacity, minimizing settlement, enhancing shear strength, and increasing the frictional angle, showcasing their versatility in geotechnical applications (Adams & Collin, 1997; Ansari & Roy, 2023a; Das et al., 1994; Mittal & Shukla, 2020; Palmeira, 2010; Patra et al., 2005; Sitharam & Sireesh, 2004; Xu et al., 2019). A geocell is a three-dimensional structure designed to provide comprehensive confinement to infilled materials. Normally, materials like sand or gravel, which allow for effective drainage, are used to fill these geocells. An innovative approach involves utilizing a mixture of waste rubber and sand as the infill material, presenting a potential solution to mitigate issues associated with waste tire disposal. Existing literature widely acknowledges that soil reinforced with either geocell or waste rubber can enhance various engineering properties. However, there is limited literature available that highlights the use of waste rubber and geocell together as reinforcement materials for the sand to enhance their properties and performance (Benessalah et al., 2022; Mehrjardi et al., 2015; Tavakoli Mehrjardi et al., 2012). To overcome this limitation, more comprehensive investigations are still needed to understand the performance, behaviours, and impacts of the incorporation of a geocell-reinforced rubber sand mixture.

The aim of this paper is to investigate the potential benefits of incorporating geocell-reinforced rubber sand mixtures in engineering applications, focusing on soil stabilization, foundation design, and other geotechnical practices. The research explores variations in geocell heights, rubber contents, rubber sizes, and confining pressure to understand their influence on shear strength characteristics. Through unconsolidated undrained triaxial tests, the study analyzes stress-strain responses and evaluates the energy absorption capacity of the reinforced mixtures. By elucidating performance and behaviors, this research aims to advance

sustainable and efficient engineering practices, offering valuable insights for practitioners seeking innovative solutions.

2. Materials used

2.1 Sand

In this investigation, the study utilized river sand, Figure 1(a), readily available in the Begusarai district of Bihar, India. Laboratory tests were performed to assess the index properties of the soil, encompassing particle size distribution, specific gravity, coefficient of curvature, coefficient of uniformity, and minimum and maximum dry density. The specific gravity of the sand was determined in adherence to ASTM D854 (ASTM, 2010). As per the unified soil classification system (USCS) outlined in ASTM-D2487 (ASTM, 2017), the soil falls under the category of poorly graded sand (SP). The detailed physical properties of the sand are outlined in Table 1. The specific gravity of sand of 2.7 is notably high, mainly due to its composition rich in silica, a dense mineral. Throughout the experiments, the relative density of the sand was taken as 80%.

2.2 Geocell

The material utilized for the preparation of geocells was purchased from a local market in Begusarai, Bihar, India. The photographic view of the material utilized for the fabrication of the geocell is shown in Figure 1(b). The geocells were fabricated by stitching the material, ensuring that the diameter matched that of the triaxial sample. For finding the optimal height of the geocell, variations in geocell height were introduced. Nylon threads were employed for stitching the geocells consistently throughout the entire experiment. To determine the physical properties of geocell materials, including the properties of the seams, tensile strength tests and seam strength tests were conducted. These tests were performed in accordance with ASTM D 4595 (ASTM, 2009a) and ASTM D 4884 (ASTM, 2009b) standards, respectively. The mass per unit area of geocell was found to be 138.2 g/m², its tensile strength was 32.4 kN/m, and its seam strength was 15.6 kN/m.

Table 1. Physical properties of sand.

Physical Properties	Value
Colour	Reddish-brown
Specific Gravity (G)	2.70
Minimum dry density (γ_{min})	14.47 kN/m ³
Maximum dry density (γ_{max})	17.91 kN/m ³
Effective Size (D_{10})	0.2 mm
Coefficient of uniformity (C_u)	3.32
Coefficient of curvature (C_c)	0.95
Frictional angle (ϕ)	38°



Figure 1. Photographs of material used: (a) sand; (b) geocell material; and (c) rubber.

2.3 Rubber

The waste tire rubbers utilized in the study were obtained from a scrap metal facility situated in Begusarai, Bihar, India. These used scrap rubber tires were cut into different sizes, as shown in Figure 1(c). As per ASTM D6270 (ASTM, 2020) standards, granular rubber refers to sizes extending from 425 μm to 12 mm, while tire chips are defined by sizes from 12 mm to 50 mm. The sieve analysis was performed to segregate these rubbers into different groups as per grain size. The present study utilized five varying sizes of granular rubber, denoted as S_1 (425 μm to 2 mm), S_2 (2 mm to 4.75 mm), S_3 (4.75 mm to 8 mm), S_4 (8 mm to 9.5 mm), and S_5 (9.5 mm to 12 mm). The specific gravity of these granular rubbers was found as per ASTM-D854 (ASTM, 2010), yielding values of 1.13 (S_1), 1.14 (S_2), 1.15 (S_3), 1.16 (S_4), and 1.16 (S_5). Furthermore, the water absorption capacity of each rubber size was evaluated using ASTM-C128 (ASTM, 2015), resulting in respective measurements of 3.35 (S_1), 3.37 (S_2), 3.36 (S_3), 3.38 (S_4), and 3.39 (S_5).

3. Experimental program

3.1 Preparation of rubber sand mixture

The study considered four different granular rubber contents ranging from 10% to 40% by volume (volume of the rubber/total volume of the specimen) and five granular rubber sizes labelled as S_1 to S_5 . The required quantity of sand and granular rubber was estimated for every percentage of

the rubber mixture and for every size of the rubber content. Volumetric measurement was applied to mix the granular rubber with oven-dried sand manually, as volumetric measurement is simpler to use on the site. However, triaxial samples were prepared by weighing the sand and granular rubber instead of using volume in the laboratory. To determine the volume, the weight and specific gravity were taken into account. The dry sand and granular rubber were thoroughly mixed using hand-mixing techniques to achieve homogeneity for various compositions.

3.2 Preparation of rubber sand mixture with geocell

Conventional UU triaxial tests were employed to assess the geocell-reinforced rubber sand mixture, adopting the same testing protocols applied to the unreinforced mixture. The required amounts of sand and granular rubber were calculated for each percentage of rubber content and size variation. Through manual mixing, the dry sand and granular rubber were mixed thoroughly to ensure uniformity across different compositions. The geocell was positioned at the center of the triaxial sample, and the height of the geocell (H_g) was varied as shown in Figure 2. The height of the geocell was kept smaller than the height of the sample (H) because of the stability of the sample. Following the placement of the triaxial membrane inside the split mould, the geocell was securely fixed within the membrane. The rubber sand mixture was then layered into the geocell in five layers and compacted to achieve the desired relative density of 80%. Porous stones were then positioned both top and bottom of the sample before removing the split mould.

3.3 Triaxial test setup

Unconsolidated undrained (UU) tests were conducted using a triaxial apparatus, as depicted in Figure 3. A split cylindrical mould, measuring 200 mm in height and 100 mm in diameter, was utilized to prepare the triaxial test sample. A latex membrane was inserted into the split cylindrical mould, and the required volume of rubber-sand mixture was poured into it in five layers. Employing a rammer for static compaction, the mixture underwent a gentle compaction process to achieve the intended height and density, preventing any potential segregation. After partially compacting the final layer, the top cap was introduced, and further compaction ensued to reach the specified density.

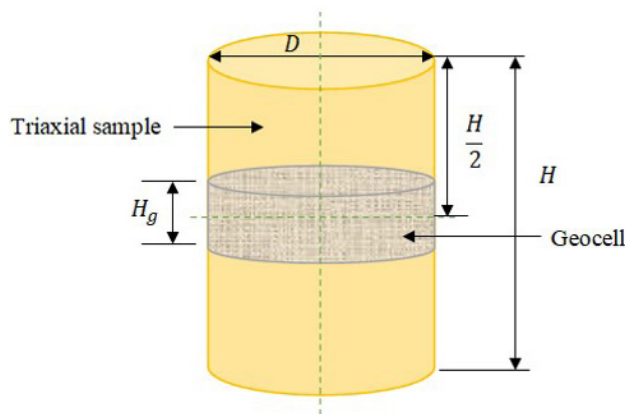


Figure 2. The location of the geocell in the triaxial sample.

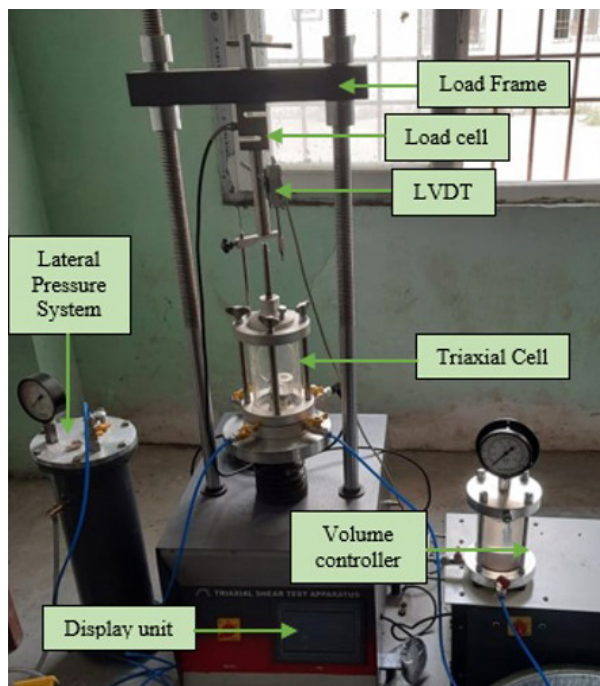


Figure 3. Photographic view of triaxial test setup.

Following this, the specimen was positioned vertically between two rigid ends within the triaxial pressure cell. The recording of both the load and deformation of the test sample during the triaxial test was facilitated through the utilization of a load cell with a capacity of 50 kN and an LVDT with a capacity of 50 mm. A constant strain rate of 1.2 mm/min was consistently applied throughout the entirety of the experimental procedures. To ensure the stabilization of the test sample before the initiation of shearing, distinct cell pressures of 50 kPa, 100 kPa, 200 kPa, and 300 kPa were successively applied for durations ranging from 10 to 15 minutes.

4. Results and discussion

Unconsolidated undrained triaxial tests were performed for homogeneous sand as well as the sand-rubber mixture with and without geocell reinforcement. The effects of rubber sizes, rubber contents, confining pressures as well as the heights of the geocell on the shear strength were investigated by varying these parameters.

4.1 Effects of the rubber content

Figure 4 illustrates the axial strain versus deviatoric stress behavior in unconsolidated undrained triaxial compression tests for rubber size S_3 , covering rubber contents from 0% to 40% by volume under a confining pressure of 300 kPa. The graph demonstrates a clear influence of rubber content on the stress-strain dynamics of the rubber-sand mixtures. Notably, at 10% rubber content, the deviatoric stress is marginally lower compared to the clean sand (0% rubber content). This decrease is attributed to a stronger interaction between sand particles, surpassing interactions between sand and rubber or among rubber particles. Consequently, the rubber fails to serve its reinforcing function in the sand-rubber mixture. Similar findings were reported by previous researchers, indicating a decrease in the strength of the sand rubber mixture at 10% rubber content (Li et al., 2020).

Moreover, an increase in deviatoric stress is observed with an increasing rubber content from 10% to 30%. However, a subsequent decline in deviatoric stress occurs with a further rise in granulated rubber content from 30% to 40%, as depicted in Figure 4. This decrease is ascribed to the rearrangement of the rubber-sand mixes and a reduction in the density of the overall mixes. The higher rubber content creates additional void space, resulting in reduced deviatoric stress (Manohar & Anbazhagan, 2021). Exceeding 30% rubber content leads to a significantly softer response from the sample, enhancing its deformability. At a confining pressure of 300 kPa, the sand rubber mixture exhibited peak deviatoric stresses of 616 kPa, 674 kPa, 749 kPa, and 697 kPa for rubber contents of 10%, 20%, 30%, and 40%, respectively.

The axial strain at failure was observed to increase, particularly with a higher percentage of rubber. However,

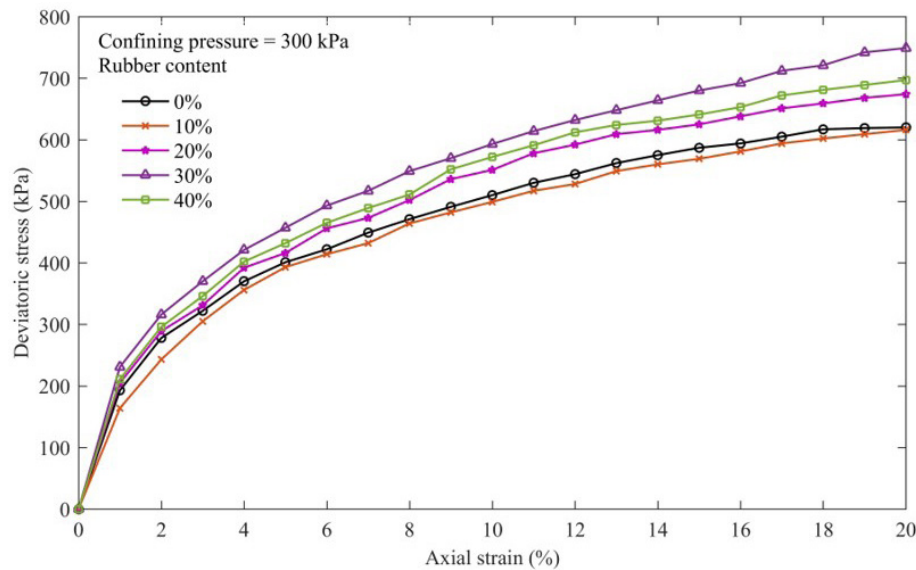


Figure 4. Variations in deviatoric stress under different rubber content, rubber size S_5 .

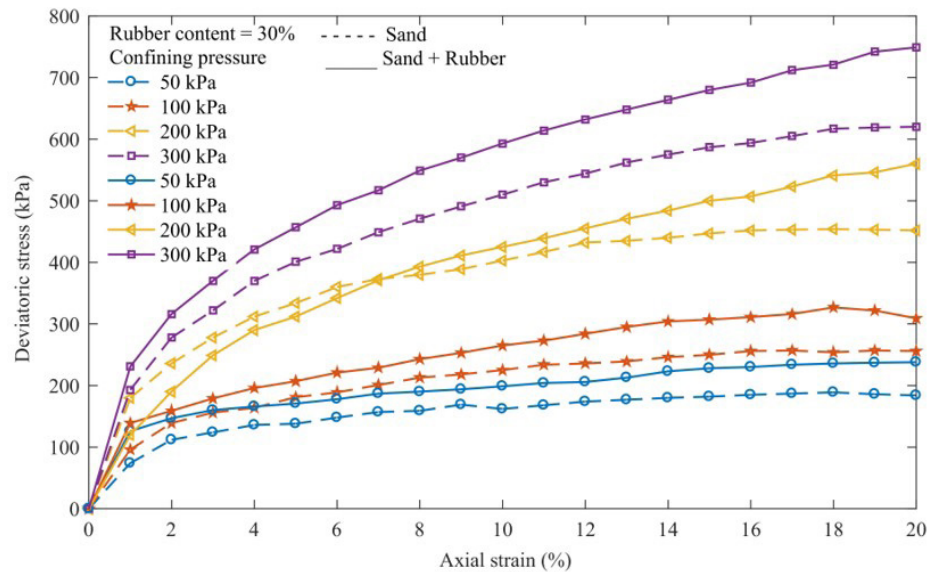


Figure 5. Variations in deviatoric stress under different confining pressure, rubber size S_5 .

due to the absence of clear-cut failure, the 20% strain level was considered for the stress-strain curve analysis. It is noteworthy that all samples experienced failure beyond the 20% strain level.

4.2 Effect of the confining pressure

Laboratory triaxial experiments were performed under varying confining pressures (50 kPa to 300 kPa) involving both clean sand (with 0% rubber content) and rubber sand mixtures. The deviatoric stress versus axial strain plots produced under the UU tests at 30% rubber content and 80% relative density

of sand are depicted in Figure 5. This illustration demonstrates that the rubber-sand mixture, possessing the same relative density and rubber content, shows an increased capacity to withstand loads as the confining pressure of the cell increases. The increased shear strength is likely a consequence of the rubber-sand mixture densifying under higher confining pressure (Ansari & Roy, 2023b; El-Sherbiny et al., 2013). The soil tested at 300 kPa revealed a stress capacity around 3.37 times higher than the soil tested at 50 kPa, despite both possessing the same relative density and rubber content. Stress-strain curves displaying nonlinearity at higher confining pressures illustrated the ductile nature inherent in rubber-sand mixtures.

4.3 Effect of the rubber sizes

UU triaxial tests were performed for investigating how the rubber sizes in the rubber sand mixture affect stress-strain behaviours. In sequence to achieve this, various rubber sizes— S_1 (425 μm to 2 mm), S_2 (2 mm to 4.75 mm), S_3 (4.75 mm to 8 mm), S_4 (8 mm to 9.5 mm), and S_5 (9.5 mm to 12 mm)—were taken and mixed with sand separately. Variations in deviatoric stress relative to axial strain are highlighted in Figure 6, considering different rubber sizes at 30% rubber content and 300 kPa confining pressure. The illustration highlights the significant impact of rubber size on stress-strain characteristics in triaxial tests conducted on rubber sand mixtures. The incorporation of rubber particles of size S_1 (425 μm to 2 mm) resulted in a minor decrease in the deviatoric stress of the sand (620 kPa to 612 kPa). This occurrence can be explained by the similarity in size between rubber and sand particles, where voids created by rubber particles may not have been filled by sand particles. Consequently, under such circumstances, rubber particles of size S_1 failed to perform as effective reinforcement in the rubber-sand mixture.

Conversely, as the rubber size increased from S_2 to S_5 in the rubber-sand mixture, a corresponding increase in deviatoric stress was observed. This suggests that larger rubber particles within the mixture contribute to reinforcement, enhancing the deviatoric stress. This result highlights the complex interconnection between rubber size and the mechanical characteristics of the rubber-sand mixture, a critical aspect for comprehending and enhancing the mixture's functionality in geotechnical engineering applications. At a confining pressure of 300 kPa and 30% rubber content, the peak deviatoric stresses for rubber sizes S_1 to S_5 were found to be 612 kPa, 668 kPa, 684 kPa, 706 kPa, and 749 kPa, respectively.

4.4 Effect of the geocell height

In this study, a comprehensive examination was carried out to investigate the influence of geocell heights on the shear strength characteristics of a rubber sand mixture through an extensive series of UU triaxial tests, covering a confining pressure range from 50 kPa to 300 kPa. The results depicted in Figure 7 present the findings for four distinct geocell heights ($H_g = 0.2H, 0.4H, 0.6H$, and $0.8H$) at a constant confining pressure of 300 kPa and a rubber content of 30%. The figure clearly indicates a consistent rise in deviatoric stress with increasing geocell height. This significant increase in deviatoric stress is credited to the geocell's effectiveness in introducing additional confining pressure, thereby enhancing the soil's load-bearing capacity while minimizing settlement. The recorded peak deviatoric stress values at geocell heights of 0.2H, 0.4H, 0.6H, and 0.8H were 799 kPa, 867 kPa, 961 kPa, and 1152 kPa, respectively.

A significant parameter, the improvement factor, was calculated as the ratio of deviatoric stress observed with geocell-reinforced soil to that without geocell reinforcement. The improvement factors corresponding to geocell heights of 0.2H, 0.4H, 0.6H, and 0.8H were determined as 1.067, 1.158, 1.283, and 1.538, respectively. These enhancement factors emphasize the positive impact of geocell reinforcement on the shear strength characteristics of the rubber sand mixture, highlighting the potential of geocell in improving the overall stability and load-bearing capacity of such soil systems.

4.5 Energy absorption capacity

The study emphasizes the critical significance of energy absorption, particularly in the context of seismic

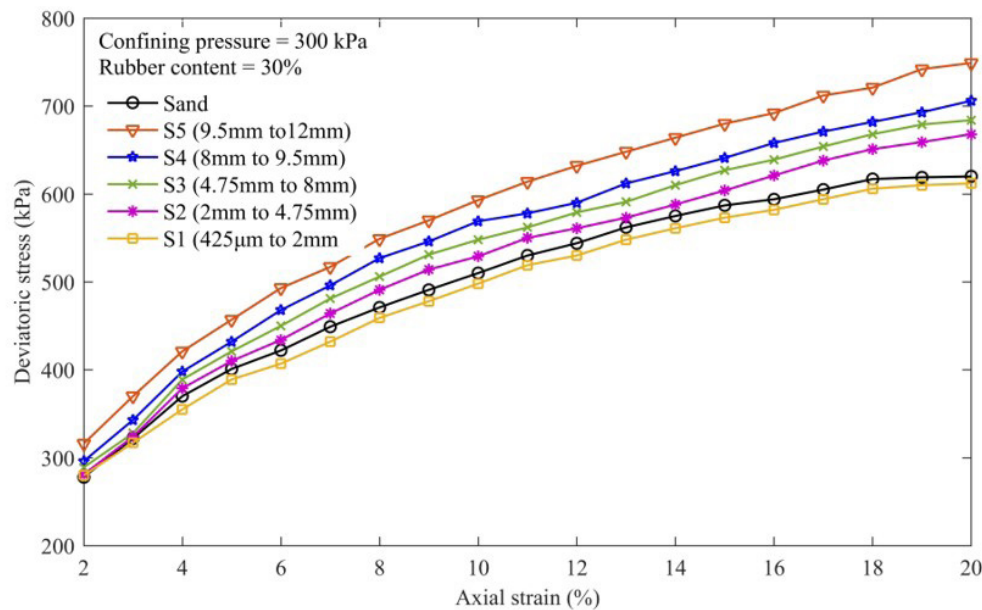


Figure 6. Variations in deviatoric stress under different rubber sizes.

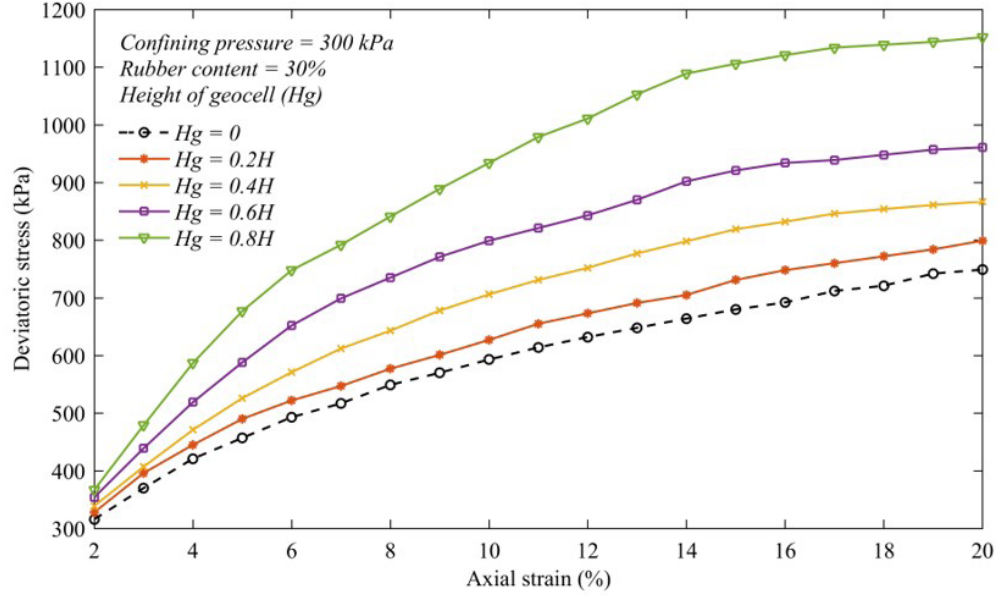


Figure 7. Variations in deviatoric stress under different height of geocell.

isolation, with a focus on the rubber-sand mixture. While existing literature extensively investigates into the static strength characteristics of rubber-sand mixtures, there exists a noticeable research gap regarding their energy absorption capacity. The assessment of energy absorption entails evaluating the total energy per unit volume absorbed by the material under specific deformation values, quantified by the area under the stress-strain plot up to a defined point (Manohar & Anbazhagan, 2021), expressed as:

$$E = \int_0^{\epsilon} \sigma(\epsilon) d\epsilon \quad (1)$$

Here, E , σ , and ϵ denote energy absorption, stress, and strain, respectively. The area beneath the stress-strain curve, obtained through triaxial tests, serves as a crucial indicator of the energy absorption capacity of the rubber-sand mixture, both with and without geocell reinforcement.

Figure 8 presents the energy absorption capacity of pure sand, a rubber sand mixture with 30% rubber content, and a geocell-reinforced rubber sand mixture with 30% rubber content and a geocell height of 0.8H. The illustration underscores a gradual increase in the energy absorption capacity of sand with the incorporation of rubber content. Furthermore, the introduction of geocells in the rubber-sand mixture leads to a further augmentation in energy absorption capacity. This observed enhancement can be attributed to the increased peak stress and ultimate stress facilitated by the inclusion of rubber, geocell, or a combination of both in the sand mixture. These findings shed light on the multifaceted contributions of rubber and geocell in enhancing the energy absorption capabilities of the sand, underscoring their potential in seismic isolation applications.

4.6 Failure mode of the samples

The failure patterns of the triaxial samples varied across different compositions: pure sand, sand-rubber mixture, and sand-rubber mixture with geocell reinforcement, as shown in Figures 9(a), 9(b), and 9(c) respectively. Pure sand samples exhibited bulging failure, likely due to their higher lateral expansion and ineffective confinement. Conversely, samples with sand-rubber mixture showed reduced bulging. Moreover, samples with sand-rubber mixture with geocell reinforcement demonstrated no clear failure plane, indicating effective lateral confinement by the geocell. Failure in these samples primarily occurred at the seam of the geocell due to its higher strength compared to the seam strength, resulting in bursting of the seam.

4.7 Applicability of rubber-sand mixtures in geotechnical field

Incorporating waste tire rubber into sand, especially when reinforced with geocell, represents a promising and sustainable solution with versatile applications in geotechnical practices. This environmentally conscious approach effectively addresses the disposal challenges stemming from the substantial volume of scrap tires generated by the automotive industry. Repurposing tire rubber in civil engineering projects not only significantly contributes to waste reduction but also plays a crucial role in preserving natural resources. Comprehensive studies on the shear strength behaviours of geocell-reinforced rubber-sand mixes highlight substantial improvements, making them advantageous for ensuring stability in diverse geotechnical structures. The adaptability of these mixtures,

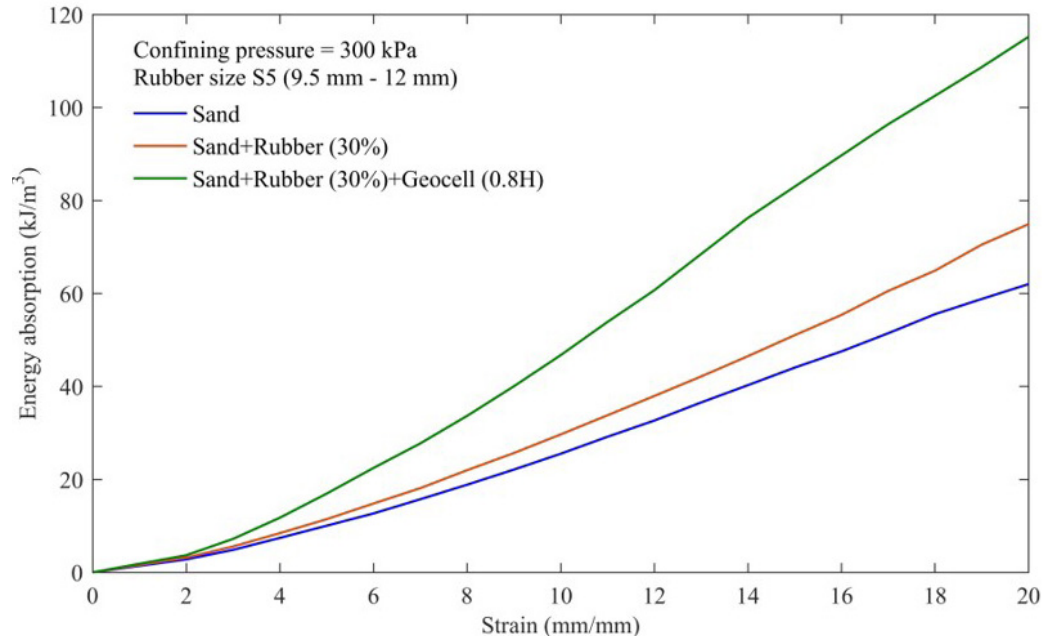


Figure 8. Variation of energy absorption capacity with strain at confining pressure of 300 kPa.

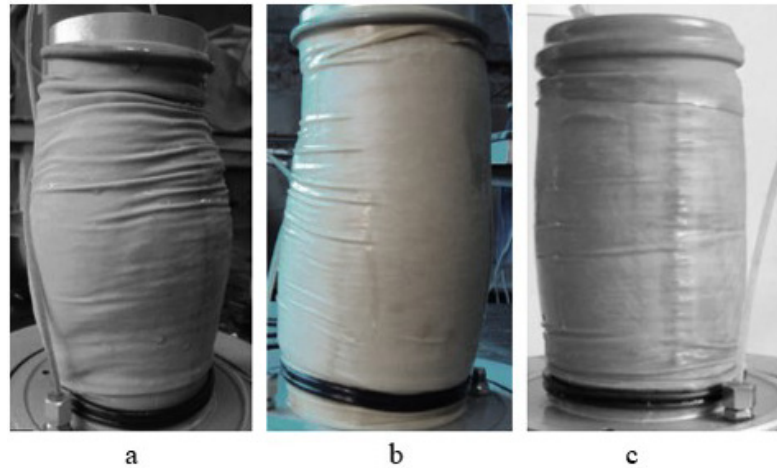


Figure 9. Failure of samples: (a) pure sand; (b) sand rubber mixture; and (c) sand rubber mixture with geocell.

complemented by their impressive energy absorption capacity essential for seismic isolation, positions them as ideal materials for a broad range of geotechnical engineering applications. Furthermore, the incorporation of geocell-reinforced rubber sand mixtures results in higher deviatoric stress and enhanced load-bearing capacity. In summary, rubber-sand mixtures emerge as durable, adaptable, and eco-friendly solutions, demonstrating significant potential for effectively addressing diverse challenges in geotechnical engineering.

The findings of this research demonstrate that a geocell-reinforced rubber sand mixture can be utilized in various geotechnical engineering projects like seismic design, lightweight filling materials in many civil engineering works,

the base material in vibration isolation for low-rise buildings, etc. In this way, a huge amount of waste tires will be easily used all over the world, which reduces significant disposal costs and prevents environmental as well as aesthetic damage.

5. Conclusion

Throughout the triaxial testing of geocell-reinforced rubber-sand mixtures, where the aim was to examine the effects of geocell heights, rubber contents, rubber sizes, and confining pressure on the shear strength characteristics of the soil, the following significant conclusions can be derived from the test results:

- The shear strength of rubber-sand mixtures is optimized at a rubber content of 30%, emphasizing the pivotal role of rubber content in determining shear strength.
- Rubber particle size significantly impacts stress-strain behaviors, with larger particles yielding higher deviatoric stress, highlighting the importance of particle size selection.
- Higher confining pressures positively correlate with shear strength, indicating densification of the mixture under increased pressure.
- Geocell reinforcement enhances shear strength by providing additional confinement, enabling the mixture to withstand higher loads with reduced settlement.
- Geocell-reinforced mixtures exhibit superior energy absorption capacity compared to conventional mixtures, suggesting enhanced performance in dynamic loading scenarios. These findings underscore the potential of geocell-reinforced rubber-sand mixtures for diverse engineering applications requiring robust and resilient materials.

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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

Md Asfaque Ansari: conceptualization, methodology, experimental program, visualization, writing – original draft. Lal Bahadur Roy: supervision, validation, review & editing. Sanjeet Kumar: review and editing, revising manuscript.

Data availability

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

List of symbols and abbreviations

ASTM	American Society for Testing and Materials
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
D_{10}	Effective Size
E	Energy absorption capacity

G	Specific gravity of soil
H	Height of the triaxial sample
H_g	Height of the geocell
LVDT	Linear Variable Differential Transformer
S_1	Rubber size of 425 μm to 2 mm
S_2	Rubber size of 2 mm to 4.75 mm
S_3	Rubber size of 4.75 mm to 8 mm
S_4	Rubber size of 8 mm to 9.5 mm
S_5	Rubber size of 9.5 mm to 12 mm
SP	Poorly graded sand
USCS	Unified Soil Classification System
UU	Unconsolidated Undrained
γ_{mim}	Minimum dry density
γ_{max}	Maximum dry density
ε	Strain
σ	Stress
ϕ	Frictional angle

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Three-dimensional numerical analysis of the effect of foundation stiffness on soil-structure interaction

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Article

Keywords

Soil-structure interaction in tall buildings
Three-dimensional numerical model
Continuous flight auger pile

Abstract

Stiffness-related geometric characteristics are closely linked to the performance of structural systems. The influence of foundation stiffness on the soil-structure interaction (SSI) mechanism was investigated by numerical modeling of different building foundations. Numerical tools (PLAXIS 3D and TQS/CAD) were used for decoupling analyses of the interaction between the structural components. Results obtained from instruments installed in a building case study were used to refine the numerical calculations. Linear elastic constitutive models and beam elements were useful in reducing computational processes and ensuring study feasibility. The simulation results provided a satisfactory representation of field observations. Interaction between piles and the environment governs the relationship between foundation stiffness and uniform settlement. Rigid pile groups are achieved with greater pile length, diameter and spacing. This study highlights the different effects of pile geometric parameters on loading and unloading rates in pillars, demonstrating that the SSI mechanism is more sensitive to less rigid foundations, particularly those with greater pile embedment. It was concluded that numerical simulations to predict settlement as realistically as possible require understanding and selecting the most appropriate model and type of calculation.

1. Introduction

In engineering projects, supports are traditionally considered rigid and fixed, while the foundation is calculated independently of the frame, considering constant loads. Analyzing the superstructure without considering the foundation (or vice-versa) disregards forces, bending moments and settlement resulting from soil-structure interaction (SSI) (Thangaraj & Ilamparuthi, 2010). Thus, it is vital to analyze the structural system considering the soil, foundation and superstructure as a compatible unit. In fact, the different parts of the structure interact and are directly associated with the system's ability to absorb and redistribute additional stress among the structural elements. This redistribution of stresses in the structure is known as soil-structure interaction (SSI), a mechanism directly linked to the performance of the construction project. Frantziskonis & Breyse (2003) reported that the performance of the structural system is also closely linked to the geometric and stiffness characteristics of the foundation. SSI may change the realistic behavior of a pile group and as such, the proper design of pile foundations must consider the interaction between the building frame, soil and pile group (Koteswara et al., 2020). This study aimed to investigate the relationship between SSI and foundation

stiffness, using PLAXIS 3D and TQS/CAD for numerical modeling of the interaction between the structural elements.

Current research on SSI is fundamentally based on formulating and using numerical methods, such as the finite element method (FEM), which makes it possible to simulate soil heterogeneity, considering stratified layers and the physical and geometric nonlinearity of the structure. Buragohain et al. (1977), Ingle & Chore (2007), Chore & Ingle (2008), Chore et al. (2009) & Chore et al. (2010) investigated pile-supported building frames using FEM to assess the effect of different foundation parameters on the superstructure. Their research emphasizes the importance of interaction between structural system components and the need to consider pile stiffness and flexibility in structural analysis. Dode et al. (2014) evaluated the effect of the spacing, diameter and number of piles in a pile group on superstructure response. Kiran & Madhav (2015) discussed the effect of parameters such as the relative stiffness of two soil layers and pile length-to-diameter ratio (L/D) on settlement behavior for a single pile under axial loading. Koteswara et al. (2020) used FEM and scale experiments to analyze the effect of a building frame on the behavior of a pile group in terms of pile settlements, cap rotation, head deflection and bending moment, by varying the pile spacing and L/D ratio, as well as the relative density of the sand bed. The study demonstrated

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that bearing capacity increases with the L/D ratio in both the single pile and pile group, especially with a rise in L , and emphasizes the importance of considering the effect of interaction between the subsoil, foundation and superstructure on pile group behavior. Rasi et al. (2020) considered SSI in FEM analyses to assess the effect of variations in foundation stiffness on settlement and concluded that greater stiffness resulted in uniform settlement. Bhartiya et al. (2020) studied pile groups and piled rafts with different geometries and configurations to determine the stiffness of these components and corresponding settlement. The authors conducted parametric studies and found that pile-soil interaction depends on pile length and diameter, as well as the elastic properties of the soil and foundation system. They also demonstrated that pile group settlement depends on the stiffness of individual piles which, in turn, depends on structural stiffness k_p (Equation 1) and stiffness resulting from pile-soil interaction (I_{ps}). This is related to pile group stiffness K_F by Equation 2, which includes both the structural and geotechnical components of stiffness. Interaction between single piles and the soil depends on the pile L/D and Young's modulus of the soil and pile.

$$k_p = \frac{A_p E_p}{L_p} \quad (1)$$

$$K_F = k_p I_{ps} \quad (2)$$

where A_p is the cross-sectional area of the pile, and E_p and L_p are its Young's modulus and length, respectively.

Studies in the literature focus on piles (single or groups) under different types of stress, with most investigations of the SSI mechanism analyzing stress in the superstructure (forces and moments). There are few experimental studies, particularly cases of real construction projects, that consider the effect of SSI and its relationship with foundation stiffness. In this context, the present study aimed to demonstrate the relationship between foundation stiffness and SSI in a real building. To that end, a parametric study was performed based on 3D linear-elastic numerical analysis via FEM, in conjunction with structural matrix analysis. For the building studied here, settlement readings from 46 pillars throughout the construction period were analyzed (Prellwitz, 2016). Numerical models validated by back-analysis with field results made it possible to implement different foundations based on pile L/D values within $\pm 10\%$ of the diameter (D) and length (L) of the piles originally designed for the building. SSI was incorporated into the analyses via iterative decoupling, with the foundation and superstructure modeled in PLAXIS 3D and TQD/CAD software, respectively.

2. Numerical models

The structure modeled is a multi-family residential building with 196 autonomous units and 19 floors. In ascending order,

it starts at the ground floor, divided into a parking garage and reception area, followed by two floors (G1 and G2) consisting of parking garages, and another floor divided into the PUC (leisure area) and a parking garage. Next, there are 14 standard floors and finally the roof slab, where the roof and access to the water tank are located. Construction time is measured in days and starts at 0 days, when the foundation is poured. Construction of the 5th slab was completed in 316 days and the 10th, 14th and 19th slabs, including the enclosing masonry, were completed on days 435, 498 and 621, respectively. After completion of the enclosing masonry, 100% of the total load of the construction was applied to the supports.

The decoupled approach adopted in the present study required constructing two-part numerical models consisting of the superstructure and foundation. The superstructure was simulated in TQS/CAD (Figure 1) and the foundation and soil mass in PLAXIS 3D. A three-dimensional model was selected in order to include the expected group effects for the foundation in the analyses. Leite et al. (2023) studied the group effect and concluded that it is significant in displacements, demonstrating increased settlement due to the overlapping stress bulbs of the piles and neighboring pile groups.

Figure 2 shows the floor plan of the ground floor pillars of the building and its different sectors under mean foundation loading. The building body consists of a reinforced concrete frame on a foundation of 18-meter-deep continuous flight auger piles 500mm in diameter in a heterogeneous sedimentary soil mass typical of the Campos dos Goytacazes region in Rio de Janeiro state, Brazil. The pillars of the body of the building were monitored and the pillars around the perimeter of the parking garage were not monitored and are not part of this study.

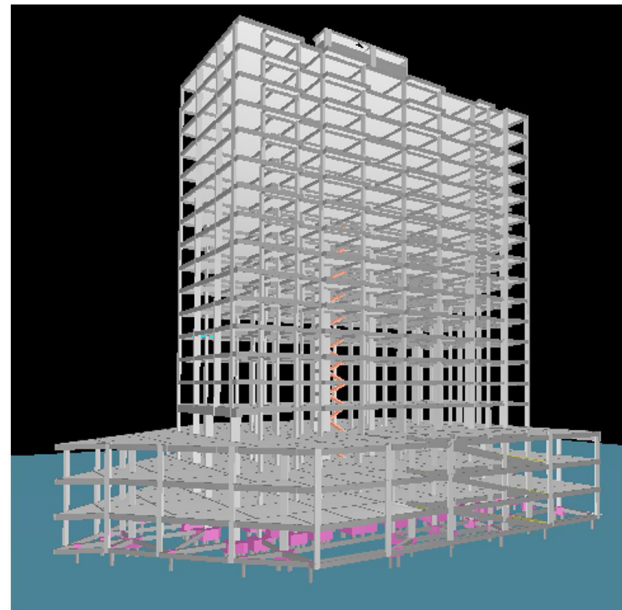


Figure 1. 3D model of the superstructure in TQS©.

Interpretations of standard penetration test (SPT) data were input into the numerical model of the foundation, which was calibrated based on results from settlement monitoring instruments in a study by Prellwitz (2016). Calibration details are presented in item 3.1. In order to monitor building settlement during construction, the author instrumented all the pillars of a 20-floor multifamily building in Campos dos Goytacazes, Rio de Janeiro state, Brazil, based on the principle of communicating vessels (see Prellwitz, 2015).

The three-dimensional FEM model was constructed to include the representative soil stratigraphy. The model boundaries were large enough to prevent the limits of the modeled from influencing the results of the problem. The structural elements were modeled considering “embedded beam” type elements for the piles and “plate” type for the capping blocks, and the contact between the soil and structural element considering an interface parameter whose value corresponds to that indicated in the technical literature (equal to 1, according to Brinkgreve & Shen, 2011). Although the program does not directly consider the effects of structural element installation, the assumed interface resistance (R_{inter}) value of one imposes a rough interface on the model, a reasonable premise since the continuous helix pile is cast in situ. This results in a lower load transfer from the pile to the soil when compared to slightly rough or smooth interfaces. The input parameters of the concrete elements are based on laboratory test results for the same building under study, reported by (Maia et al., 2019) and presented in Table 1.

3. Numerical analysis

The numerical models were used in calculations to study the effect of foundation stiffness on SSI mechanisms. This study was conducted via parametric analysis combined with iterative decoupling to assess the iterative mechanism between the structure and the calibrated model of the foundation.

Initially, stratigraphy in the numerical model was proposed based on the result of the field investigation carried out for the foundation (SPT holes), as mentioned in the previous item. The typical soil profile is shown in Figure 3. The probes indicated a surface layer of silty clay with N_{SPT} values of fewer than 10 blows, followed by a dense approximately 4-meter-thick sand layer (average N_{SPT} of 15 blows), a layer of soft clay up to 2 m thick, and a hard clay layer whose N_{SPT} increased with depth, reaching 40 blows. Figure 3 also shows the geotechnical parameters used as input in the model. Details on these values are provided below.

The superstructure model presented in Figure 1 was used to calibrate the geotechnical parameters, considering the

construction sequence. The foundation was modeled using PLAXIS 3D. Calibration was performed by back-analysis, using the pillar settlements measured in the field as reference.

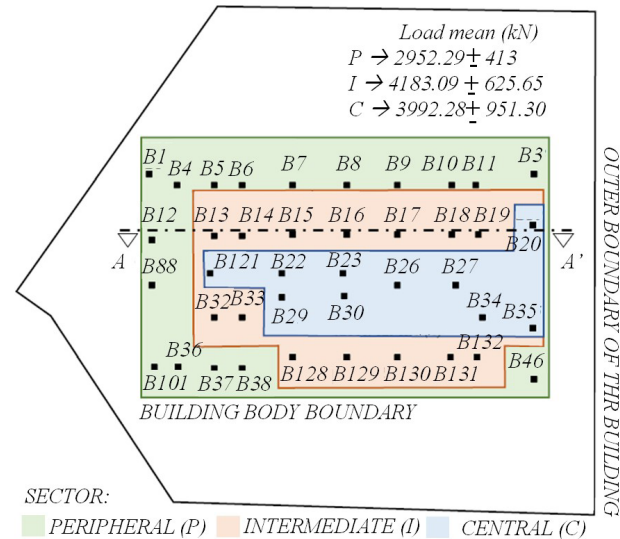


Figure 2. Floor plan of the 46 ground floor pillars of the building, its sectors under mean loading and AA cross-section.

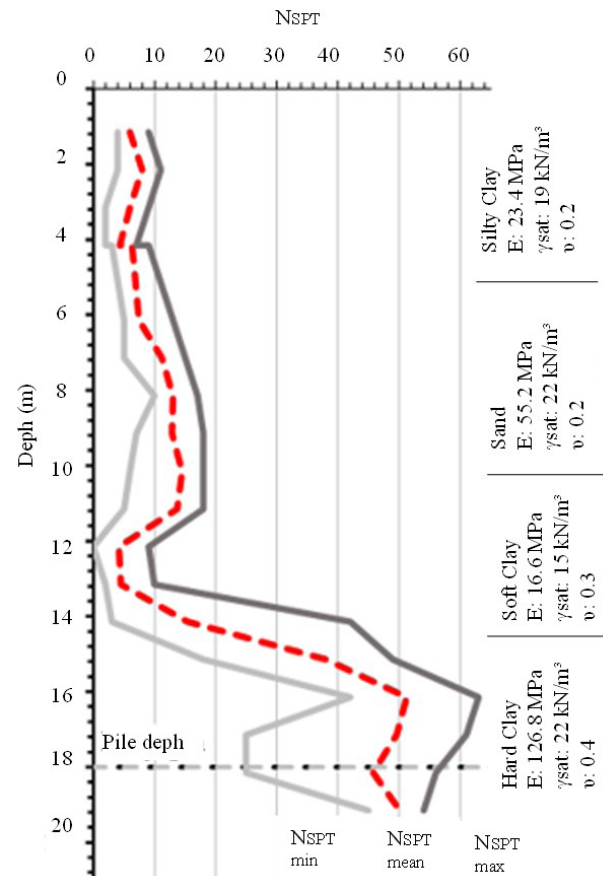


Figure 3. Equivalent soil profile.

Table 1. Pile input parameters (Maia et al., 2019).

E (GPa)	ν	γ (kN/m³)
14.8	0.11	20.69

Legend: see List of symbols and abbreviations.

Initially, parameters were determined using semi-empirical correlations based on N_{SPT} (Decourt, 1995; Marangon, 2009; Ortigão, 2007).

Next, the estimated loads were applied in line with the construction sequence. Load estimation for each loading stage of the pillars was computed using TQS/CAD software with the SISEs module (TQS Informática, 2019). Each loading stage corresponds to the concreting of a building slab. Given the considerable number of loading stages (19 slabs) and the lengthy processing time needed to include all the stages, only 4 were considered for numerical modeling (5th, 10th, 14th, and 19th slabs).

Several numerical simulations were performed at each processing stage, thus refining the geotechnical parameters until the displacements in the numerical model were consistent with the measured values. For refinement, the geotechnical parameters of the model were varied within the confidence interval obtained through the spatial variation of the properties of each layer shown in Figure 3.

The calibration results are presented in Figure 4, which shows settlement progression for each processing stage. It is evident that the proposed modeling process provided a reliable estimate of foundation behavior in terms of trend and magnitude, assuming the order of magnitude of the values obtained. The calibration results also demonstrated

that adopting field tests traditionally used in geotechnical projects (SPT) combined with back-analysis of in-situ settlement readings is a reasonable approach in numerical modeling. The foundation model consisted of an equivalent medium for the soil mass, and the linear elastic constitutive law was adopted for each soil layer, as recommended by Poulos (1989) and Mandolini & Viggiani (1997).

In Figure 4, the settlements obtained by the numerical model are overestimated. This is fundamentally justified by the simplifications inherent in the modeling process. The 3D model via FEM requires high computational effort for geometrically sophisticated models or the adoption of sophisticated constitutive models. The equivalent soil mass adopted made modeling feasible without considerably compromising deformation tendency and behavior in the foundation. As such, adopting equivalent stratigraphy and modeling with a linear elastic constitutive equation is reasonable for this case study, which, despite numerical limitations inherent to the process, made it possible to consider the group effect of the piles in determining stresses in the foundation.

3.1 Iterative decoupling methodology

Given the calibrated numerical models, a methodology was adopted to consider soil-structure interaction between the decoupled models and evaluate the effects of the proposed foundation stiffness.

The iterative decoupling methodology applied to determine loads and settlement with SSI effects for the building was based on the process described by Aoki & Cintra (2004), with the spring coefficient incorporated into the model in line with the methodology of Chamecki (1954). For each analysis, a numerical routine was devised (illustrated in Figure 5) that enabled the calculation of the loads and settlement associated with the building pillars. In this routine, SSI is considered when the settlements are induced in the TQS model. The spring coefficient introduced in the TQS/CAD model is determined based on the settlement obtained in the PLAXIS 3D model. This procedure is iterative until settlement converges between the models.

In Figure 5, the process begins with the matrix model of the superstructure (TQS/CAD) that calculates the load on the pillars in each stage of building construction. Initially, the hypothesis of zero displacements (settlements equal to zero) was considered. The load stages were simplified to four stages corresponding to the 5th, 10th, 14th and 19th slabs. In this respect, the nonlinear static calculation yields the reactions of each pile (Q) in each stage, which are applied to the foundation (FEM model). This produces the respective settlement values (w) at the top of the blocks that form the base of the pillars, thus making it possible to determine pillar base stiffness. This pillar stiffness corresponds to the vertical spring coefficient k (kN/m), which is equivalent to the ratio between the load (Q) and settlement (w).

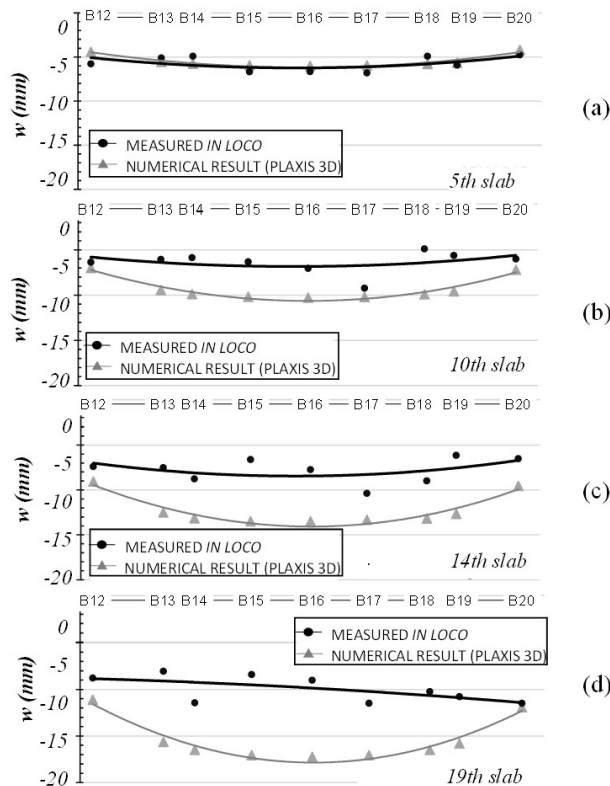


Figure 4. Foundation settlement profiles (AA cross-section) for the stages in which the (a) 5th, (b) 10th, (c) 14th, (d) and 19th slabs were constructed.

A sequence of interactions is established for each load stage considered, whereby the stiffness calculated for each pillar is input into the matrix model, which provides new pillar reactions due to the redistribution of structural stress. The process is repeated until the difference between settlement values obtained in two subsequent iterations is less than a mean error of 2%. In the present study, four to six iterations were needed for the settlement values to converge. Stress redistribution among the structural elements as loading progresses is one of the main effects of SSI. As a result, uniform settlement and smooth foundation settlement profiles are expected (Gusmão, 1990). Indeed, iterative decoupling prompted a tendency to uniform settlement on the settlement vs. time curve in the final load stages of construction.

3.2 Parametric study

Different pile geometries referred to as ‘cases’ were compiled for the parametric study of the foundation, with each case analyzed individually in a specific FEM model. The calculation routine presented in Figure 5 was used in each case to ensure that SSI was considered in the assessment of the effect of foundation stiffness.

Table 2 presents the geometry and respective stiffnesses for each case and the corresponding structural stiffnesses of the foundation (k_p) given by Equation 1, since SSI depends on pile length and diameter as well as the elastic properties of the soil and foundation system. Each case established resulted in different pile length-to-diameter ratios (L/D). Diameter (D) and length (L) were adopted at $\pm 10\%$ of the measurements of the piles originally designed for the project. Case zero corresponds to the real geometry of the construction project.

4. Study of the effect of foundation stiffness on SSI mechanisms

4.1 Settlement sensitivity

For each case in Table 2, the rate of settlement progression according to load increase was assessed and expressed in number of construction days in Figure 6. The figures indicate

an inverse relationship between the length-to-diameter ratios L/D and settlement, that is, for a same diameter, piles that are less slender produce greater settlement (case 1), while their more slender counterparts create less settlement (case 2). Cases 1 and 2 were also constant (less sensitive) during the loading process, unlike when diameter varied (cases 3 and 4), since foundation settlement is sensitive to this parameter, particularly in the early load stages. When pile length is varied, the expected increase and decrease in $w_{AVERAGE}$ is 1.072 and 0.089%, respectively, with 2.96 and 3.14% for pile diameter changes. Taken together with foundation analysis, this indicates that settlement is more sensitive to changes in pile length regardless of whether settlement tends to decline due to changes in geometry. This tendency stabilizes as the construction project progresses, unlike that observed when diameter variations are used to achieve more or less settlement in building projects. As the number of building floors increases, settlement tends to progressively decline with

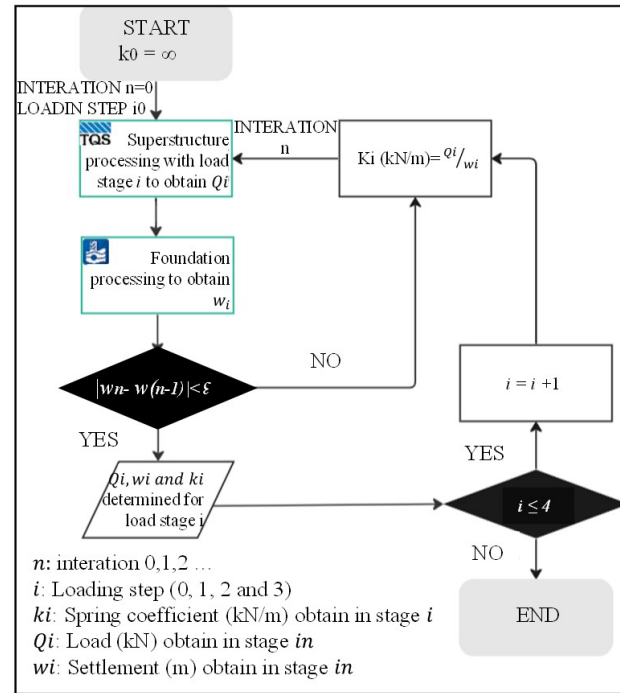


Figure 5. Iterative decoupling methodology.

Table 2. Foundation geometry for the cases considered.

Case	D (m)	L (m)	L/D	$k_p \times 10$ (kN/m)
0	0.5	18.0	36.0	1.61
1	0.5	16.2	32.4	1.79
2	0.5	19.8	39.5	1.46
3	0.45	18.0	40.0	1.30
4	0.55	18.0	32.7	1.95

Legend: see List of symbols and abbreviations.

the reduction in pile diameter. Finally, the increase in pile proportions is associated with greater bearing capacity, with piles consequently producing less settlement (Case 2 and 4).

4.2 Sensitivity of foundation stiffness (K_F)

Analysis of the foundation as a single system considered pile group stiffness K_F , given by the ratio between working load (Q) and displacement (w) of the pile group (Equation 3).

$$K_F = \frac{Q}{w} \text{ (kN / m)} \quad (3)$$

Figure 7 presents the $K_{FAVERAGE}$ of all the pillars in the building, along with the progression of this parameter for the cases considered.

In general, foundation stiffness increases as construction advances and tends to stabilize in the final stages of building. This is justified by the greater soil strength caused by increased confinement of the adjacent soil mass. The effects of changes in pile geometric parameters are evident.

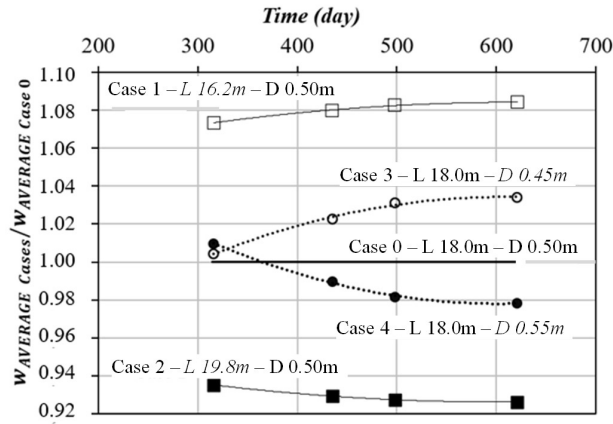


Figure 6. Normalized settlement over time for each case analyzed.

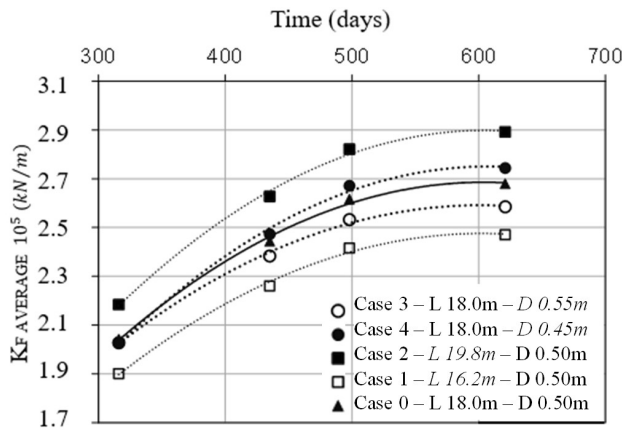


Figure 7. $K_{FAVERAGE}$ progression for the cases considered.

The lower settlement depicted in Figure 7 corresponds to higher K_F values; thus, longer and wider piles result in stiffer pile groups. This demonstrates that foundation stiffness is strongly related to the pile L or D adopted in the foundation design. Length sensitivity is greater than that observed in pile diameter analysis, making it consistent with the findings of Zampirolli et al. (2022).

4.3 Soil-structure interaction (SSI)

One of the main effects of SSI is load redistribution in the structure. The absolute settlement factor (AR) proposed by Gusmão (1990), given by Equation 4, makes it possible to analyze the effects of SSI, more specifically of load redistribution, while the *iso-AR* curves make it easier to visualize the areas on the floor plan of the ground floor pillars subjected to this effect. Figure 8 shows the spatial location of the pillars in the floor plan and the *iso-AR* curves. Pillars with a higher load level are situated in the center of the floor plan and those with lower working loads around the edge (peripheral sector), with unloading ($AR > 1$) evident in the former and overloading in the latter. This result confirms the technical literature that predicts the tendency to unloading in the most and overloading in the least loaded pillars, due to the SSI mechanism (Gusmão, 2006).

$$AR = \frac{w}{w_{AVERAGE}} \quad \begin{aligned} AR > 1 &\rightarrow \text{Unloading} \\ AR < 1 &\rightarrow \text{Overloading} \end{aligned} \quad (4)$$

Changes in D and L and, consequently, in the structural stiffness of the element, have a similar effect on AR . Figure 9 demonstrates that the sensitivity of this factor to pile stiffness is around 1% for geometry variations (L or D). When settlement decreases due to the use of longer piles, overloading in the latter ($AR < 1$).

Overloaded pillars are stiffer than their unloaded counterparts, a characteristic observed consistently across all construction stages and for both geometric variations (length, L , and diameter, D). The influence of pile L/D ratios is evident in the overloading curves (Figures 9c and 9d), with lower L/D values resulting in smaller settlements and higher values larger settlements.

By contrast, in the unloading curves (Figures 9a and 9b), smaller L/D ratios produced larger settlements and higher values smaller settlements.

It is important to discuss an interesting fact regarding AR tendencies, as shown in Figure 9 and described in the literature (Gusmão, 1990). According to the literature, AR tends to one at the end of building construction due to the expected increase in building stiffness, which acts predominantly to minimize settlement dispersion. In Figure 9, the AR coefficient curve is close to 0.90, on average, at the end of construction. This value can be improved by

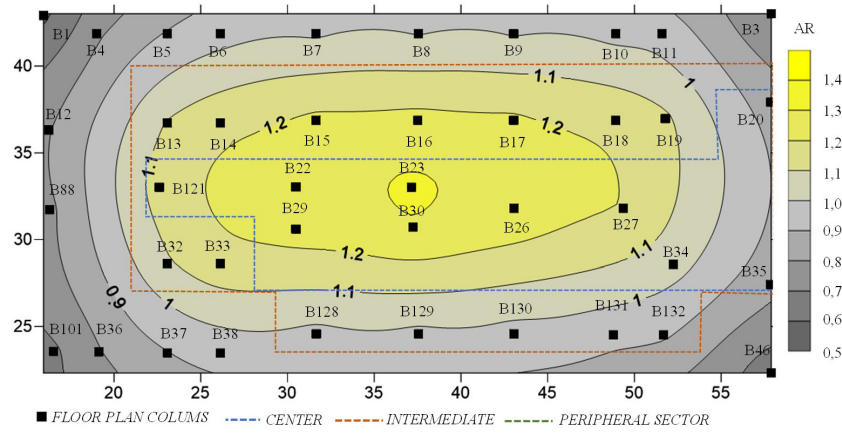


Figure 8. Iso-AR curves for case 0.

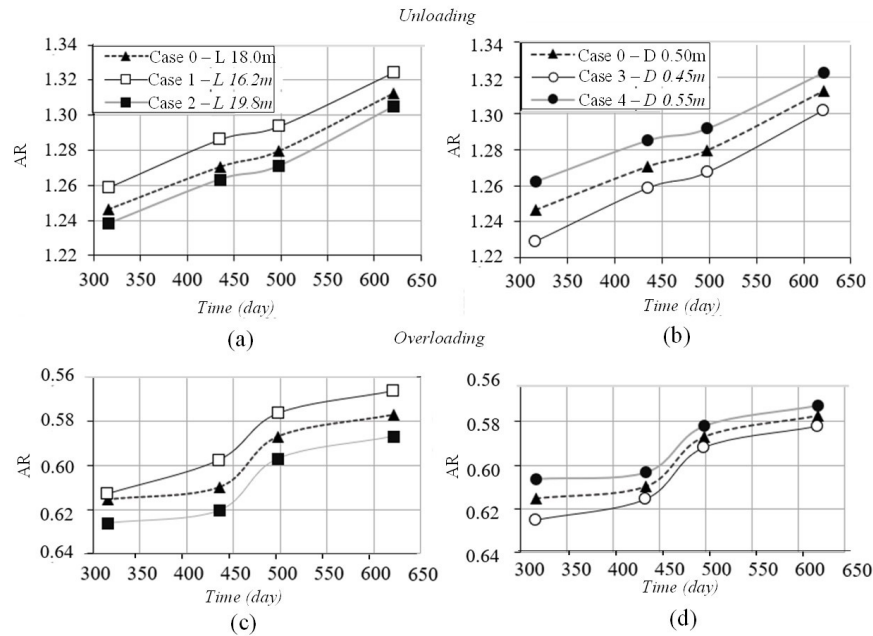


Figure 9. Progression of the AR coefficient of the pillars according to the construction sequence; (a) and (c) behavior due to the variation in pile length “ L ” (b) and (d) diameter “ D ”.

adjusting the tolerance in the decoupling approach adopted to include SSI in the analyses (shown in Figure 5). It is important to note that the numerical model makes it possible to simulate behavior observed in the field, identifying pillars with unloading ($AR > 1$, which have a higher load level, in the central sector) and overloading ($AR < 1$, with lower working loads, in the peripheral sector), in line with the technical literature (Gusmão, 1990).

Barros (2005) proposed load redistribution assessment in pillars using the load redistribution coefficient (FQ), given by Equations 5 and 6, which depict load variation between pillars as a value between -1 and 1, with FQ values from -1 to 0 indicating unloading, 0 to 1 overloading and

$FQ = 0$ (zero) no SSI. Where R_{ISE} is the reaction at the support point of pillar i , obtained based on the flexibilized model, and R_{IND} the reaction calculated considering the fixed pillars. The author also suggested that foundation behavior be assessed in relation to building geometry via the CV coefficient and that equivalent foundation characteristics be considered. Thus, for the case studied here, it is pertinent to compare the CV of isolated areas of the system (central, intermediate and peripheral sectors in Figure 2).

$$FQ = \frac{R_{ISE} - R_{IND}}{R_{IND}} \quad \text{For } R_{IND} < R_{ISE} \quad (5)$$

$$FQ = \frac{R_{ISE} - R_{IND}}{R_{ISE}} \quad \text{For } R_{IND} < R_{ISE} \quad (6)$$

In general, changes in average foundation stiffness do not affect the expected SSI trend but influence the magnitude of load redistribution among the pillars and the level of settlement dispersion in the building (FQ and CV , respectively). In fact, regardless of foundation stiffness, the central and intermediate pillars tend to unloading and those in the peripheral sector, overloading.

Figure 10 illustrates the relationship between load redistribution and the degree of absolute settlement uniformity. The FQ and CV coefficients were obtained for each stage of building construction and for each sector of the floor plan (I – intermediate, C – central e P – peripheral).

Figure 10 shows unloading in the central and intermediate pillars. However, load redistribution across the pillars in these sectors exhibits specific behavior, with practically constant unloading (3.0 to 3.4%) and increasing settlement dispersion in the intermediate section, whereas the opposite is expected in the center. Increasing unloading associated

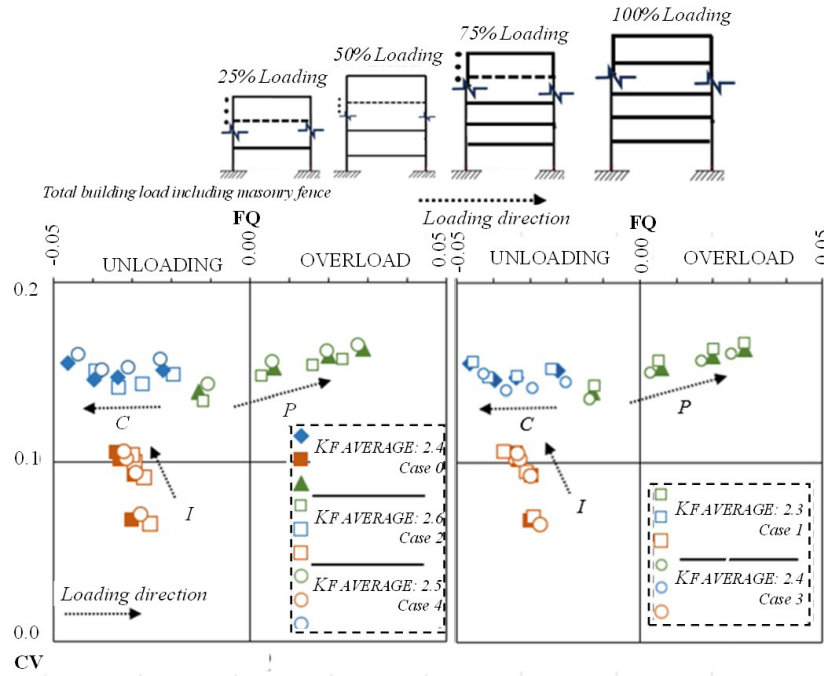


Figure 10. CV_{MEAN} vs. FQ for the building sectors and cases considered.

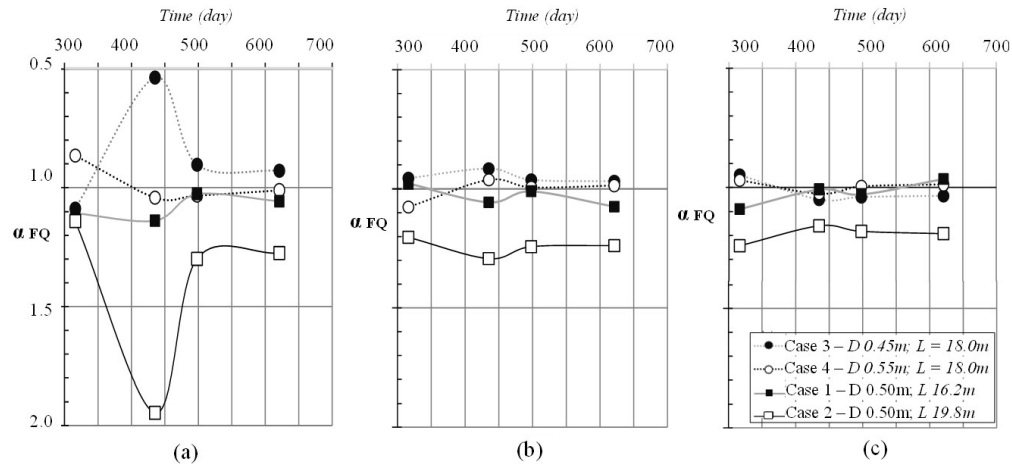


Figure 11. Progression of normalized FQ for each sector analyzed for the (a) central; (b) intermediate and (c) peripheral sectors.

with constant settlement CV_{MEAN} (around 15%) in the center of the building can be justified by the restrictions imposed on the pillars by the frame, which acts predominantly by reducing settlement dispersion. While the location of pillars is not necessarily determinant in SSI effects, the load on these structural elements plays a decisive role in this mechanism.

In general, changes in average foundation stiffness do not affect the expected SSI trend but influence the magnitude of load redistribution among the pillars and the level of settlement dispersion in the building (FQ and CV , respectively).

Indeed, regardless of foundation stiffness, the central and intermediate pillars tend to unloading and those in the peripheral sector, overloading.

Increased foundation stiffness promotes greater FQ and CV , particularly for pillars in the center of the building, albeit with a more moderate effect in the remaining sectors.

When diameter is the cause of varying foundation stiffness (cases 3 and 4), greater CV fluctuations are expected, while changes in L (cases 1 and 2) have a greater effect on FQ . This is more pronounced in the peripheral and central sectors, which seems to suggest that foundation stiffness and its effect on SSI differ according to the characteristics of the element (structural stiffness) the size and diameter of pile embedment. Quantitative assessment of the effect of the structural stiffness of the foundation on load redistribution is shown in Figure 11, demonstrating the proportional increase or decrease in load redistribution in the pillars from each sector, via α_{FQ} given by Equation 7, as building construction progresses. The further the α_{FQ} value is from one, the greater the expectation that the change in stiffness proposed by the case will contribute to load redistribution. However, when α_{FQ} is close to one, variations in foundation stiffness are expected to have little influence on FQ .

$$\alpha_{FQ} = \frac{FQ_{CASE\ n}}{FQ_{CASE\ 0}}, n = 1, 2, 3 \text{ and } 4 \quad (7)$$

In general, the effect of changes in foundation stiffness on FQ are more evident in the initial stages of construction, especially in peripheral pillars. In fact, this phase is more susceptible to SSI effects because the structure exhibits less bending stiffness, which is conferred as the number of floors increases (Gusmão, 2006). FQ variation rates are higher along the periphery than in the remaining sectors, which may be associated with the greater load redistribution in the less loaded pillars. Since the peripheral pillars are more susceptible to SSI effects, with higher overloading rates, in this case study foundation stiffness changes are expected to have a greater influence on SSI in this sector.

FQ is more sensitive to variations in L , especially in case 2, regardless of the section in which the pillars are located. The largest variations in FQ are expected at low foundation kp values (cases 2 and 3). This confirms that

FQ is more sensitive to less rigid foundations, particularly those with greater pile embedment.

In general, the intermediate sector obtained the smallest FQ variation rates when kp changes. Pillars in the intermediate section of the building are therefore less susceptible to the effects of foundation stiffness in terms of the SSI mechanism, unlike that observed in less loaded pillars.

5. Conclusion

The iterative methodology proposed here uses successive simulations of structural calculations (TQS/CAD) and stress-strain analysis (PLAXIS), which proved to be an effective approach in understanding loading and unloading rates in foundations for a 20-floor building with 46 pillars. This enabled parametric analyses to investigate the relationship between the structural stiffness of the foundation and the SSI mechanism.

Iterative decoupling also made it possible to analyze the sensitivity of foundation stiffness to changes in pile diameter and length. Greater pile embedment produced a more significant settlement reduction when compared with an increase in the contact area of the pile in this study case for these specific soil conditions, in line with the technical literature (Reul & Randolph, 2004).

Loads greater than 75% cause greater soil confinement, evident in the stable stiffness behavior of the foundation system. Moreover, foundation stiffness is directly related to how the pile interacts with the soil and not necessarily to the structural stiffness of the foundation. In fact, uniform settlement in the building depends on this interaction, since it was found that the use of more rigid parts does not necessarily result in uniformity. Fundamentally, the type of geometric parameter associated with stiffness (D and L) is a decisive factor in analysis.

This study concluded that the pursuit of low settlement in construction projects should be carefully analyzed, since pile geometric parameters have different effects on unloading and overloading rates in the pillars.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have analyzed and approved the content of the manuscript and there is no financial interest to report.

Authors' contributions

Nathani Zampirolli: conceptualization, data curation, formal analysis, investigation, methodology, validation, visualization, writing – original draft. Aldo Durand Farfán: conceptualization, data curation, supervision, validation, writing – review and editing. Paulo César de Almeida Maia: supervision, validation, writing – review and editing.

Data availability

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

List of symbols and abbreviations

k_p	Geometric coefficient of the piles
s	Spacing between piles in a block
w	Absolute settlement of the pillars
AR	Settlement factor
CV	Coefficient of variation of absolute settlement
D	Pile diameter
E	Young's modulus
FEM	Finite element method
FQ	Coefficient of load redistribution in the pillars
I_{ps}	Pile-soil interaction
K	Vertical spring coefficient
K_F	Stiffness of the foundation system
L	Pile length
N_{SPT}	Number of blows in the SPT
Q	Load
R_{inter}	Interface resistance
SSI	Soil-structure interaction
SPT	Standard penetration test
ULS	Ultimate limit state
α_{FQ}	Load redistribution factor in the pillars
γ	Soil specific weight
ν	Poisson's ratio

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Study of the influence of rheological parameters on the behavior of a submarine debris flow

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Article

Keywords

Rheological test
Submarine debris flow
Centrifuge test
Landslide soil
Geohazard

Abstract

Submarine debris flows are events that occur with great frequency on a geological scale and can cause major damage to offshore structures or even loss of human life. Understanding the mechanisms involved in the development of a debris flow requires not only knowledge of soil mechanics, but also knowledge of fluid mechanics, since the incorporation of water during the flow causes changes in shear strength. Unlike soils, the shear strength of fluids is represented by mathematical models called rheological models. The objective of this paper is to propose a new rheological model capable of representing the changes caused by water entrainment into a submarine debris flow, by correlating it with the Liquidity Index of the soil. A rheological analysis of marine clay samples from the pre-salt layer has made it possible to represent the variation in strength due to water content. The influence of water content and velocity on flow behavior has also been studied through the analysis of centrifuge tests performed in a previous study.

1. Introduction

Submarine debris flows consist of a mixture of sediment and water that is governed by gravity and is highly mobile due to voids filled with water and mud (Takahashi, 2007). These events pose a significant risk to offshore structures and can have catastrophic consequences for coastal communities (Locat & Lee, 2005; Tailling et al., 2012; Satake, 2012). However, their large size, usually associated with submarine geomorphology, and the great depths at which they occur, typically over 1000 m, make it difficult to monitor these events. As a result, most observations occur after a disaster and are only recorded as a result of damage or failure of seafloor structures such as internet cables and pipelines (Hsu et al., 2008).

Therefore, in order to mitigate environmental and financial damage, several research groups around the world have focused on understanding the development of these phenomena and their consequences. Heezen & Ewing (1952) studied the 1929 Grand Banks debris flow in Canada, caused by a magnitude 7.2 earthquake, where some telephone cables were broken. Another similar case occurred in 2006 off the coast of Taiwan (Hsu et al., 2008), where several debris flows severed internet cables, only to be assessed after the disaster. Locat & Lee (2002) report that

in 1929 an earthquake generated a debris flow that destroyed submarine communication cables about 1000 km from the epicenter. Another more recent case occurred in 1998, when a submarine landslide triggered by an earthquake caused a tsunami that killed about 2,500 people in Papua New Guinea (Heidarzadeh & Satake, 2015).

Guo et al. (2020) pointed out the need to improve the understanding of the stages of a submarine debris flow as well as the forces acting throughout the flow. There is also a need for a better calculation of the effects of water entrainment on the shear strength of the material. To this end, it is important to consider the soil from a fluid mechanics perspective (Tsugawa et al., 2019).

In this context, rheology is a significant tool for understanding the factors that govern debris flows, such as the incorporation of water and the identification of turbidity currents. It provides parameters for physical and numerical modeling (Zakeri et al., 2010; Liu et al., 2020). However, studies have shown that rheological models used to describe fluids with yield stress do not have a good correlation between rheological and geotechnical properties. Therefore, several studies have aimed to develop more suitable rheological models (Imram et al., 2001; De Blasio et al., 2003; Jeong, 2014; Campos & Galindo, 2016). The present study involves a rheological analysis of marine clay samples collected from

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different points in the Campos Basin, with water depths greater than 1000 m and where a variety of offshore pre-salt oil exploration structures are located. The objective is to propose a new rheological model capable of representing the changes caused by water entrainment into a submarine debris flow (Widjaja & Lee, 2013), by correlating it with the Liquidity Index of the soil. This parameter is especially suitable for debris flow behavior studies, since it gives a measure of how far the water content is from the liquidity limit, therefore providing a direct quantification of the capability of the flow to behave as a fluid.

The use of the proposed model in the analysis of submarine debris flows will allow a more accurate simulation of the distance traveled by the flow and the risks associated with the submarine slope failure for offshore structures, since the existing models do not take into account the water entrainment into the flow which is essential to predict its behavior.

2. Submarine debris flows

Early studies on submarine debris flows conducted by this research group using a beam centrifuge were reported by Acosta et al. (2017). Subsequently, Hotta et al. (2020) analyzed the process of transformation of debris flows into turbidity currents using a mini drum geotechnical centrifuge. This equipment has been applied in the last two decades to model a wide range of offshore problems (e.g. Oliveira & Almeida, 2010; Almeida et al., 2013; Rammah et al., 2014; Oliveira et al., 2017), but also onshore problems (e.g., Almeida et al., 2011; Fagundes et al., 2012).

2.1 The Anatomy of a submarine debris flow

The development of a submarine debris flow can be divided into 4 stages as shown in Figure 1. Stage 1, named Failure, (Figure 1a), begins with the triggering mechanism,

which can be an earthquake, erosive processes, volcanism, human activities, among others (Locat & Lee, 2009), and ends with the onset of material displacement. This phase is governed by the laws of soil mechanics.

Stage 2 consists of the formation of the flow (Figure 1b) and begins with the sliding of the soil mass down the slope, which mixes with the water in the medium. This stage is also characterized by a reduction in the shear strength of the soil caused by the increase in water content, which exhibits transitional behavior from that of a solid to that of a non-Newtonian fluid.

Stage 3 is the longest and is associated with the development of the debris flow (Hotta et al., 2020). After this stage, the flow may continue as a submarine debris flow (Figure 1e), or it may become a turbidity current (Figure 1c) with a predominance of suspended particles, depending on the relationship between gravitational and viscous forces, represented by the Reynolds number.

In stage 4, deposition of the material occurs, the rheological properties of which were studied by Liu et al. (2020), who concluded that in less dense flows (turbidity currents) there is an accelerated dissipation of kinetic energy, i.e. deposition of soil occurs during the flow (Figure 1d). In submarine debris flows, which are denser in composition, deposition occurs at the stop of the flow due to friction of the basal layer of the flow with the seabed, resulting in the formation of large deposits (Dowdeswell et al., 1996; King et al., 1996; Vorren et al., 1988) and the transport of large volumes of soil (Figure 1f).

2.2 Rheological models in submarine flows

As in slope stability analysis, the shear strength of the fluid (soil and water) acts to resist movement. However, the water entrainment during flow causes constant changes in the composition and properties of the material (Liu et al., 2023) (Figure 2), including a reduction in shear strength (Locat & Lee, 2009).

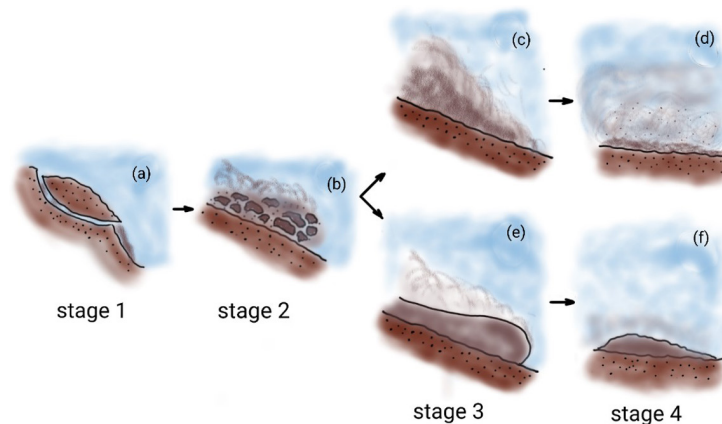


Figure 1. Stages of a submarine debris flow: (a) failure; (b) flow formation; (c) turbidity current; (d) deposition of suspended particles; (e) submarine debris flow; (f) formation of debris deposits.

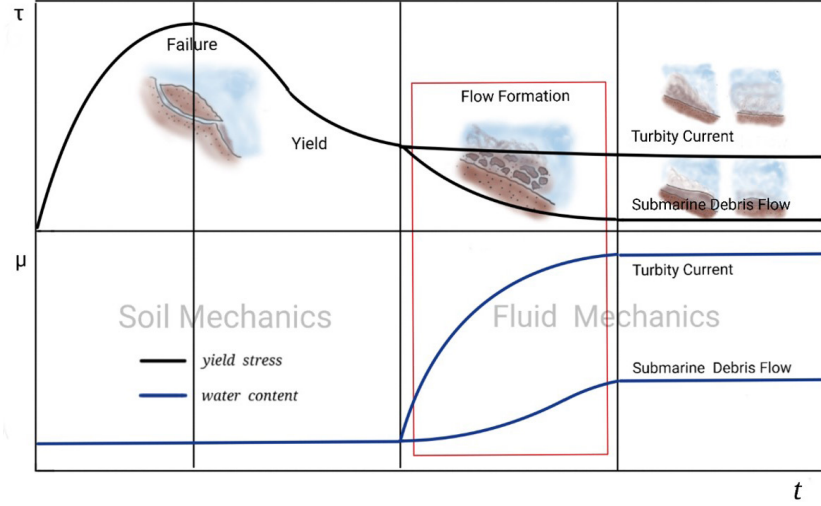


Figure 2. Shear strength behavior and variation in water content of the material during the development of a submarine debris flow.

Debris flows behave like a non-Newtonian fluid (Jeong et al., 2010), and like a fluid, their shear strength can be represented by rheological models. The Bingham model (Equation 01) is a widely used model due to its simplicity (Barnes, 1999). This model (Figure 3) has two rheological parameters: viscosity (μ) and yield stress (τ_0). The yield stress is the minimum shear stress required to initiate flow (Tsugawa et al., 2019). When the fluid is subjected to lower stresses, it behaves like a rigid elastic solid. Viscosity is the portion of the resistance that varies as a function of shear rate.

$$\tau = \tau_0 + \mu \dot{\gamma} \quad (1)$$

Another widely used model for debris flows is the bilinear model (Jeong, 2014), in which the fluid has two flow regimes: one for shear rates lower than the critical shear rate ($\dot{\gamma} < \dot{\gamma}_c$), in which the material behaves like a Newtonian fluid (Regime I, Equation 02), and another for shear rates higher than the critical value (Regime II, Equation 03), in which the material behaves like a Bingham fluid (Salmanidou et al., 2018). Figure 3 shows the plot of the Bingham and bilinear models with Regimes I and II for the same flow curve. Thus, for ($\dot{\gamma} < \dot{\gamma}_c$):

$$\tau = \mu_1 \dot{\gamma} \quad (2)$$

and for ($\dot{\gamma} \geq \dot{\gamma}_c$):

$$\tau = \tau_0 + \mu_2 \dot{\gamma} \quad (3)$$

where: τ_c is the critical stress and represents the shear strength corresponding to the critical shear rate.

Debris flows are governed by the rheological parameters of the material, but other factors such as velocity, specific gravity of the soil, and channel geometry also affect the flow behavior.

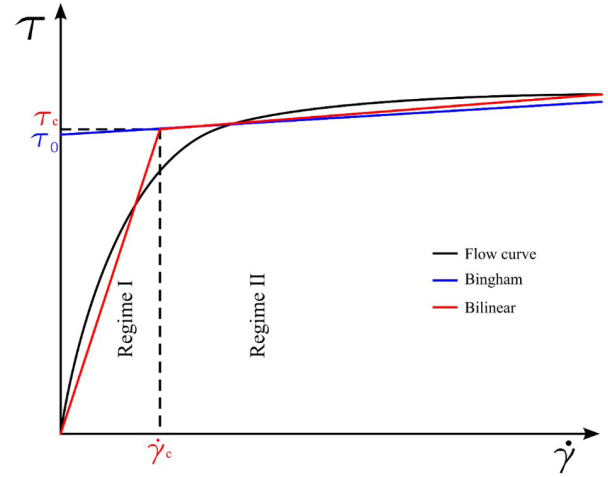


Figure 3. Flow Curve Example, Bingham Model, Bilinear Model - Regime I and II.

A dimensionless number that expresses the effect of varying each of these parameters is the Bingham Reynolds number ($R_{eBingham}$), calculated from Equation 04 (Madlener et al., 2009; Haldenwang & Slatter, 2006). A flow with a $R_{eBingham}$ greater than 1500 characterizes a semi-turbulent regime and indicates the formation of a turbidity current (Hotta et al., 2020), which causes an increase in water entrainment into the flow.

$$R_{eBingham} = \frac{\rho_s D_e v_m}{\mu + \frac{\tau_0(w)}{\dot{\gamma}(w)}} \quad (4)$$

where: ρ_s is the density of the slurry; v_m is the frontal velocity of the flow; μ is the dynamic viscosity; τ_0 is the yield stress; $\dot{\gamma}$ is the shear strain rate; and D_e is the equivalent diameter,

which is the ratio between the channel wet cross-section area and wet perimeter.

D_e is a relevant parameter in the calculation of the Reynolds number because, in an open channel, it indicates that reducing the flow height cause the increase in R_e , which may lead to a change in the flow behaviour from laminar to turbulent.

3. Rheological tests

This section describes the tests correlating the rheological parameters of marine clay samples from the Brazilian continental shelf with their respective liquidity indexes. Based on the results, a bilinear rheological model is proposed to account for water entrainment in debris flows.

3.1 Materials and methods

The rheological study of fine marine sediments is a topic addressed by several authors (Jeong et al., 2009, 2010; Zakeri et al., 2010). In this study, 6 soil samples from the Santos Basin, on the Brazilian continental shelf, were used for this purpose. The Santos Basin is the largest sedimentary offshore basin in Brazil and is one of its most important oil and gas exploration regions where the deepest platforms are installed.

The samples were taken in the seabed at depths of up to 30 m in regions with water depths of more than 1000 m and have the following average particle size distributions: 60% silt, 38% clay, and 2% sand. Analysis by electron microscopy and X-ray diffraction showed that the soil is composed mainly of SiO_2 and CaCO_3 . Table 1 summarizes the geotechnical properties of these samples, which are classified as high plasticity inorganic clays according to the unified soil classification system (USCS) (ASTM D2487-2017), with the soil's Liquidity Index (LI) calculated using Equation 05.

$$LI = \frac{w - w_p}{w_L - w_p} \quad (5)$$

where: w is the water content; w_p is the plastic limit; and w_L is the liquid limit.

Six reconstituted samples were prepared by remoulding the Shelby samples in a mixer for 30 minutes by adding destiled water. Then each of the 6 samples was tested with 3 different water contents, for a total of 18 tests.

A Brookfield RST-CC rheometer was used to perform the ramp and vane tests at a controlled laboratory temperature of 24°C. The well-known equations to compute shear stresses and shear strains for concentric cylinder (CC) and vane geometries are given by Steffe (1996). The ramp tests used a smooth concentric cylinder geometry (Figure 4a) consisting of an inner rotor and an outer cup of varying dimensions selected based on the magnitude of the apparent yield stress (τ_0) of the sample being tested, as each geometry is suitable for different stress ranges. The tests consisted of continuously and linearly increasing and decreasing the shear rate from 0 s^{-1} to 100 s^{-1} (acceleration) and from 100 s^{-1} to 0 s^{-1} (deacceleration) and measuring the corresponding shear stresses. The shear rate variation was 1 s^{-1}/s . This range of shear rate values was defined based on recommendations in the literature for submarine debris flow events (Kobayashi et al., 2015; Santoro et al., 2012). The vane tests (Barnes & Nguyen, 2001; Steffe, 1996) were carried out at a constant shear rate of 0.1 s^{-1} and a constant temperature of 24°C to measure the shear strength τ_p of the samples. The vane geometries are shown in Figure 4b. The geometry chosen for the ramp and vane tests depends on the range of shear stresses being studied. In this case, given the specimens studied and the assumed fluidity index values, the CCT-25 and VT 30-15 geometries were used for the ramp and vane tests, respectively.

During ramp tests, flow curves are obtained that describe the behavior of soil as the shear rate varies from zero to the established limit and back to zero, i.e., when the soil is accelerating and decelerating, respectively. The authors aim to fill a gap in the existing literature by providing a detailed understanding of the curves that best represent each stage of the debris flow. Figure 5 shows the flow curves obtained in the ramp tests for soil samples 1, 2, 3, and 5, that correspond to the deceleration stretch and represent the behavior of the material during most of the flow.

Table 1. Geotechnical properties of the soil samples and adopted water content.

Sample	G_s	w_p (%)	w_L (%)	w_1 (%)	w_2 (%)	w_3 (%)
S1	2.69	39.0	117.0	107.1	124.3	140.5
S2	2.68	36.0	90.0	98.2	112.1	128.4
S3	2.70	30.0	82.0	97.0	112.4	132.2
S4	2.67	28.0	114.0	95.0	122.0	135.0
S5	2.72	23.0	67.0	99.7	117.8	133.1
S6	2.72	27.0	104.0	104.0	130.1	141.3

G_s = Specific gravity of soil; w_p = Plastic limit; w_L = Liquid limit.

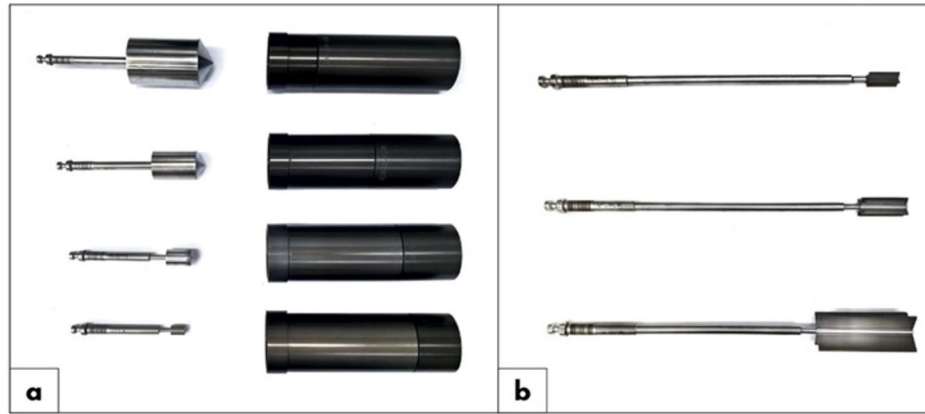


Figure 4. Geometries used in the tests (a) concentric cylinders (from top to bottom) CCT-40, CCT-25, CCT-14, CCT-8; (b) vane (from top to bottom) VT 20-10, VT 30-15, VT 60-30.

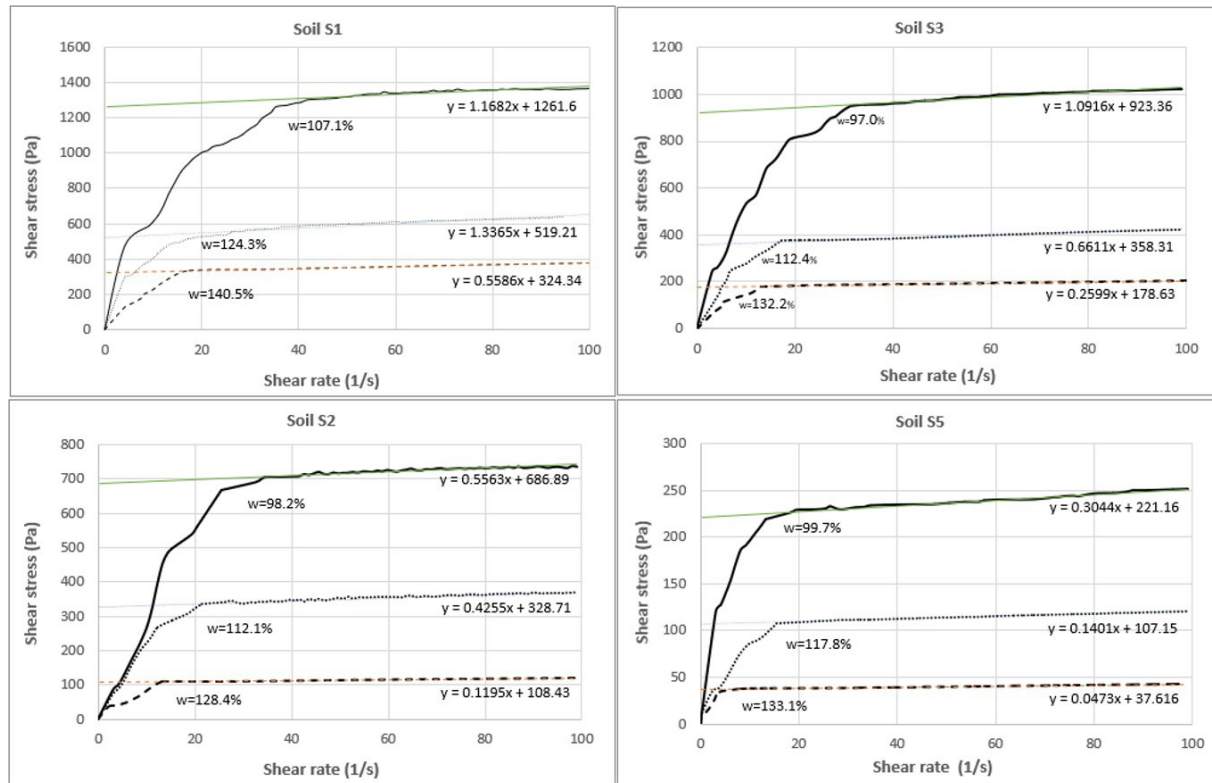


Figure 5. Flow curves for soil samples S1, S2, S3 and S5 and corresponding Bingham model lines.

3.2 Results of the rheological tests

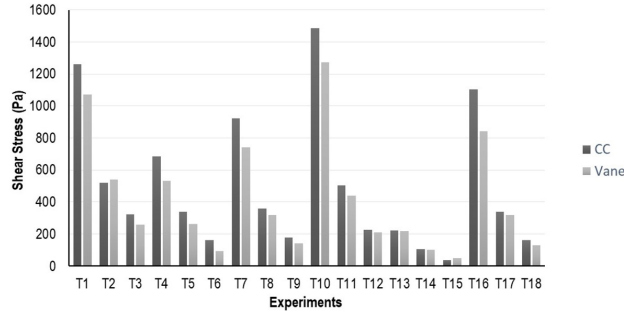
The values of Bingham yield stress (τ_0), viscosity (μ) and critical flow rate (γ_c) obtained in the ramp tests for each of the samples tested are shown in Table 2. As can be seen from the flow curves shown in Figure 5, the straight line assumed for regime II of the bilinear model and the straight line of the Bingham model are practically identical.

Therefore, to simplify the modeling, $\mu_2 = \mu$ and $\tau_0 = \tau_c$ are assumed (Table 2).

Figure 6 compares the remolded shear strength values (τ_p) from the vane tests with the yield stress measurements (τ_0) from the concentric cylinders (CC) tests. The average values differ by less than 15%. However, in tests with the lower water content (T1, T4, T7, T10, and T16), the concentric cylinder tests showed higher shear stresses due

Table 2. Parameters of the rheological model.

Samples		w (%)	τ_0 (Pa)	μ (Pa. s ⁻¹)	$\dot{\gamma}_c$ (s ⁻¹)	Samples		w (%)	τ_0 (Pa)	μ (Pa. s ⁻¹)	$\dot{\gamma}_c$ (s ⁻¹)
S1	T ₁	107.1	1261.6	1.168	40.0	S4	T ₁₀	95.0	1486.2	0.302	34.0
	T ₂	124.3	519.2	1.337	25.0		T ₁₁	122.0	503.3	0.589	24.0
	T ₃	140.5	324.3	0.559	20.0		T ₁₂	135.0	226.1	0.322	16.0
S2	T ₄	98.2	686.9	0.556	32.0	S5	T ₁₃	99.7	221.2	0.304	18.0
	T ₅	112.1	328.7	0.426	24.0		T ₁₄	117.8	107.2	0.140	15.0
	T ₆	128.4	108.4	0.120	12.0		T ₁₅	133.1	37.6	0.047	8.0
S3	T ₇	97.0	923.4	1.092	24.0	S6	T ₁₆	104.0	1092.4	0.509	25.0
	T ₈	112.4	358.3	0.661	17.0		T ₁₇	130.1	339.2	0.493	20.0
	T ₉	132.2	178.6	0.260	14.0		T ₁₈	141.3	163.8	0.260	13.0


Figure 6. Comparison between the apparent yield stress (sweep tests) and remolded shear strength (vane tests) values for soil samples – “Bingham model”.

to greater viscous effects (Sigh & Mitchell, 1968; Bjerrum, 1972; Chandler, 1988; Bowles et al., 2003; Yang et al., 2015).

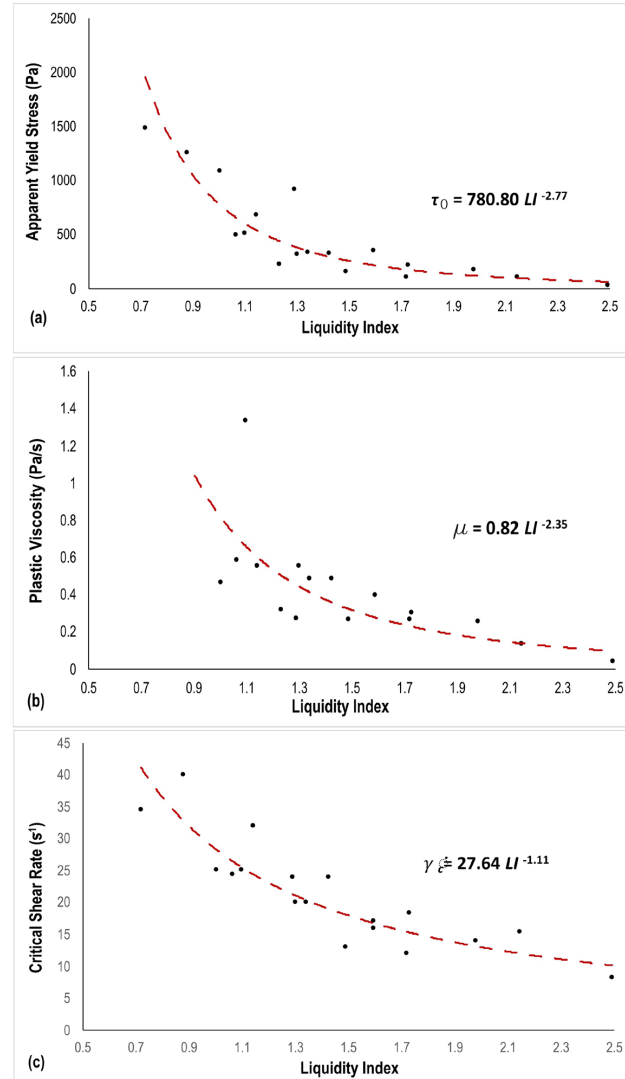
The values of the rheological parameters τ_0 , μ and $\dot{\gamma}_c$ (Table 2) and the correlation curves of these parameters with the liquidity index of the material are given in Equations 06, 07 and 08.

$$\tau_0 = 780.8 LI^{-2.77} \quad (6)$$

$$\mu = 0.82 LI^{-2.35} \quad (7)$$

$$\dot{\gamma}_c = 27.64 LI^{-1.11} \quad (8)$$

As can be seen in Figure 7, the three rheological parameters decrease exponentially with the Liquidity Index of the material. Considering that, for a given soil, all the parameters of the bilinear model can be calculated as a function of the LI (Equations 06, 07 and 08), the rheological parameters can be correlated with the water content of the sample.


Figure 7. Rheological parameters versus liquidity index (a) apparent yield stress; (b) plastic viscosity; and (c) critical shear rate.

The correlation between the yield stress τ_0 and the LI (Figure 7a) given by Equation 6 is similar to that found for other soils (Locat, 1997; De Blasio et al., 2003; Issler et al., 2005; Sørli et al., 2022). Other rheological studies (Campos & Galindo, 2016; Jeong et al., 2010; Jeong, 2014; Sørli et al., 2022) have shown a similar relationship between the viscosity and the Liquidity Index (Figure 7b), for the range of LI values greater than 1.

3.3 Proposed bilinear model considering the soil Liquidity Index

Based on the tests presented, the influence of the water content of the material on the shear strength of the soil can be expressed by adopting a bilinear model in which the rheological parameters vary as a function of the water content of the material. For marine clays from the Santos Basin, the model is defined by equations 09, 10 and 11.

In regime II ($\dot{\gamma} > \dot{\gamma}_C$), substituting (06) and (07) in (03), we have:

$$\tau = 780.8 LI^{-2.77} + 0.82 LI^{-2.35} \dot{\gamma} \quad (9)$$

τ_C can be calculated by substituting (08) into $\dot{\gamma}$ from (09):

$$\tau_c = 780.8 LI^{-2.77} + 22.61 LI^{-3.45} \quad (10)$$

Finally, the shear strength τ corresponding to Regime I ($\dot{\gamma} < \dot{\gamma}_C$) can be calculated by:

$$\tau = 0.82 LI^{-2.35} \dot{\gamma} \quad (11)$$

The parametric analysis of equation 08 shows that the higher the water content of the mixture, the lower the

transition shear rate ($\dot{\gamma}_C$) between regimes I and II, decreasing until the behavior of the material approaches the Bingham model (Figure 3). Therefore, the use of the bilinear model is recommended for the analysis of flows with low average velocity, i.e. with an average shear rate below the critical shear rate. In other cases, it is possible to use the Bingham model (Guo et al., 2020), which has the advantage of mathematical simplification since it contains only one regime.

The results show a good agreement between the model curves and the flow curves obtained in the tests, especially for the samples with the highest water content, which characterize a submarine debris flow. Figure 8 shows the comparison of a representative group of the flow curves obtained in the rheological tests (T2, T14 and T17) with those obtained using the adapted bilinear model, assuming different water contents.

The equations used to predict the flow curves in Figure 8 are valid only for the tested soil which parameters were obtained in the rheological tests. For different soils, new rheological tests are needed, and a new set of equations will be defined.

4. Assessment of the relevance of the model base on previous results

Given the difficulty in evaluating the geotechnical and rheological parameters of a real submarine flow, as mentioned above, centrifuge physical modeling was used to evaluate the relevance of the assumptions considered here. To that end, the tests carried out by Hotta et al. (2020) to study the dynamics of submarine flows were used to investigate the conditions for the formation of a turbidity current evaluating the relevance of water content to the behavior of this phenomenon, based on $Re_{Bingham}$. Centrifuge tests were conducted using Speswhite kaolin clay, a commonly used soil in centrifuge modeling.

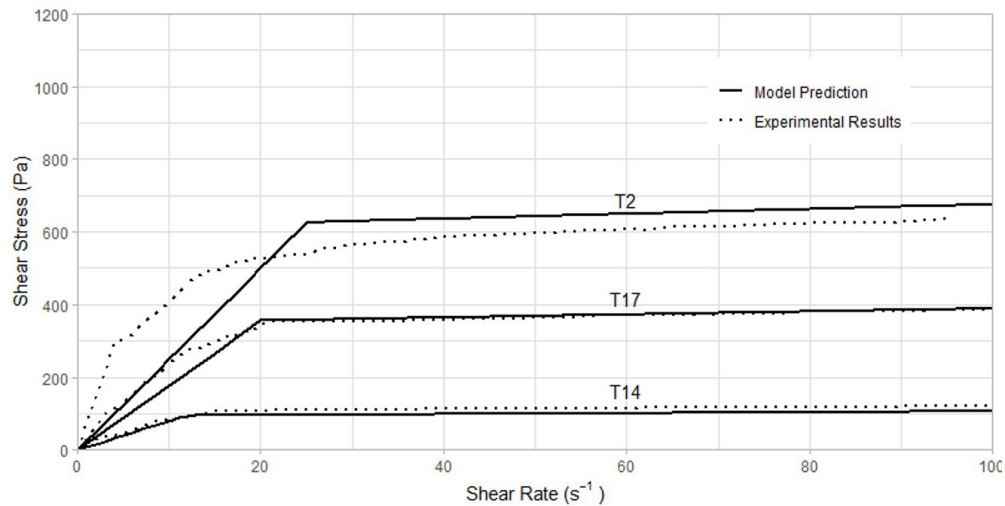


Figure 8. Comparison between model predictions and experimental results.

Table 3 shows the rheological parameters of the Speswhite clay for each water content value in the three tests performed by Hotta et al. (2020), obtained using the rheometer equipment and procedures described above.

Figure 9 depicts the test scheme analyzed according to the approach proposed in this article, and Table 3 shows the parameters of each test. The initial flow velocity, measured at checkpoint 1 (P_1 in Figure 9), was controlled by applying different mud launch pressures, and the rheological parameters were modified with the variation of the water content.

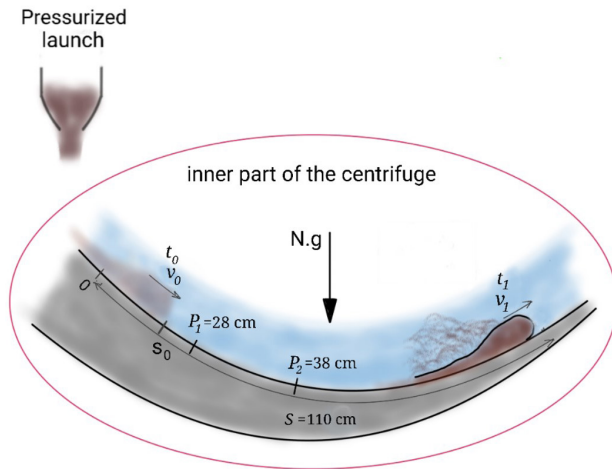


Figure 9. Representation of the kinematic parameters of a submarine debris flow in a centrifuge test. Based on Hotta et al. (2020).

Average velocity (v_m), yield stress (τ_0), viscosity (μ) and Bingham Reynolds number for each sample (Hotta et al., 2020) are listed in Table 3. It is important to note that both the viscosity and the yield stress are a function of the LI of the material.

As already explained, the most appropriate model to study the flows in stage I (Figure 1) would be the bilinear model. However, since the tests represent a high initial velocity flow, the use of the Bingham model satisfies the initial conditions of the problem, since flows with high initial velocity characteristics correspond to stages 3 and 4, where the shear rate is greater than the critical shear rate.

The comparison of tests w90_p0.8 and w100_p0.8 (Figures 10 and 11) shows the influence of the variation of the water content on the flow behavior since both were launched at the same pressure (same initial velocity) and with different water contents (Table 3). The flow velocities measured at checkpoint 1 (Table 3) were 0.39 m/s in test w90_p0.8 and 1.00 m/s in test w100_p0.8. In real scale, these two tests are equivalent to two flows formed by the same material, with the same initial velocity, developed on slopes with the same inclination (same inertial component), but with different water contents. This condition can be caused, for example, by the difference between the triggering mechanisms.

In test w90_p0.8, the flow developed as a submarine debris flow, represented by stage 3b in Figure 1. However, in test w100_p0.8, as a result of the increase in initial water content, the flow regime became turbulent and developed into a turbidity current, represented by stage 3a in Figure 1.

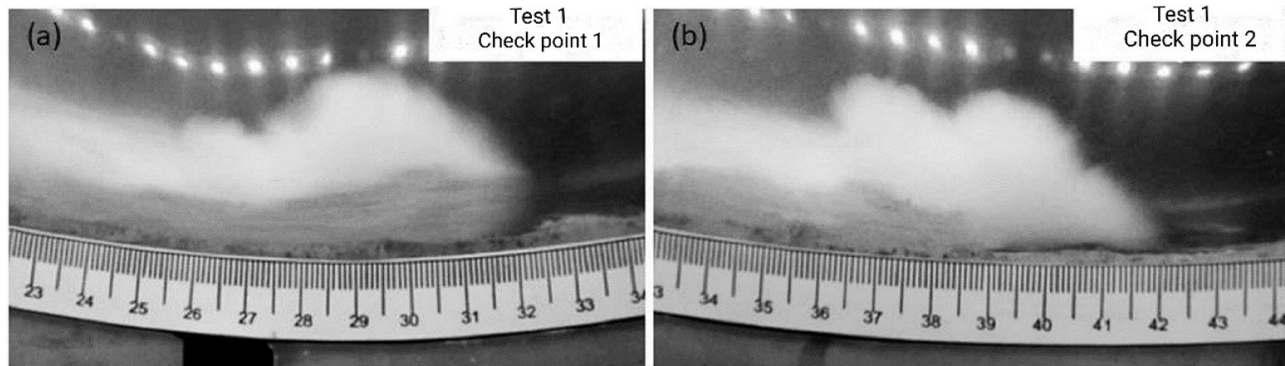


Figure 10. Image of test w90_p0.8 ($w = 90\%$). (a) checkpoints 1; (b) check point 2. Adapted from Hotta et al. (2020).

Table 3. Centrifuge test parameters (Hotta et al., 2020).

Test identification	Water content w (%)	Slurry yield stress τ_0 (Pa)	Slurry dynamic viscosity μ (Pa.s)	Applied pressure (kPa)	Velocity (m/s)	Bingham Reynolds Number
w90_p0.8	90	231	5.89	80	0.39	212.7
w90_p0.9	90	231	5.89	90	0.42	282.4
w100_p0.8	100	200	4.57	80	1.00	1816.2

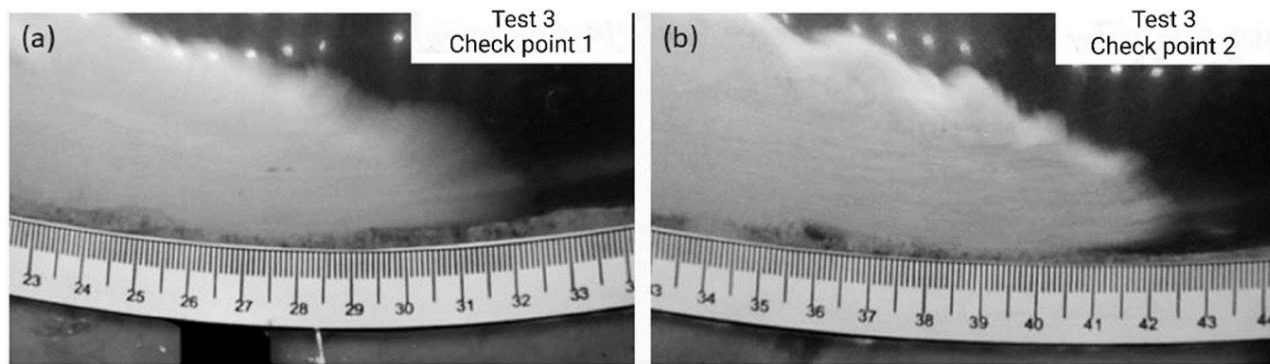


Figure 11. Image of test w100_p0.8 ($w = 100\%$). (a) check point 1; (b) check point (2). Adapted from Hotta et al. (2020).

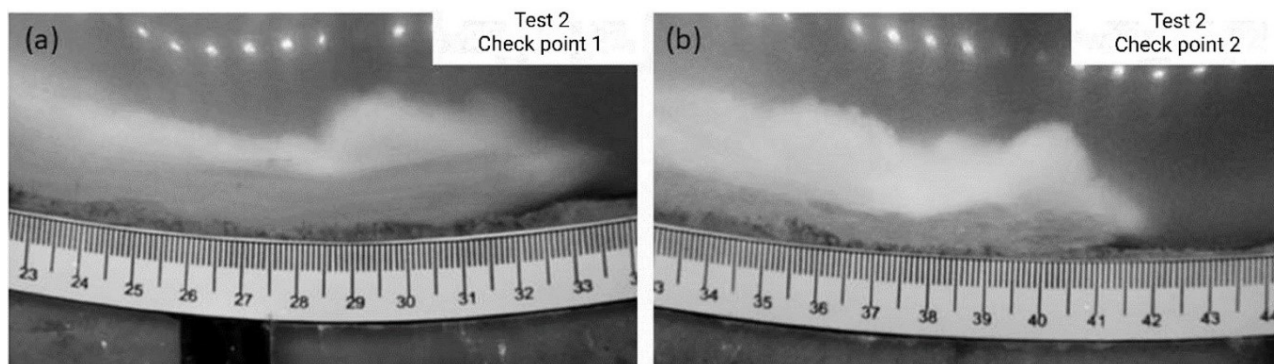


Figure 12. Image of test w90_p0.9 ($w = 90\%$). (a) check point 1; (b) check point 2. Adapted from Hotta et al. (2020).

These results are consistent with the predictions based on the Bingham Reynolds number analysis ($Re_{Bingham} > 1500$).

By comparing tests w90_p0.8 and w90_p0.9 in Figure 10 and Figure 12, it is possible to see the isolated effect of the initial velocity of the flows, since they differ only in the launch pressure (initial velocity), the former being 80 kPa and the latter 90 kPa. The flow velocities measured at checkpoint 1 were close, despite the different launch pressures: 0.39 m/s for test w90_p0.8 and 0.42 m/s for test w90_p0.9. At real scale, these tests are equivalent to two flows of the same material with the same water content, differing only by different initial velocities. The comparison between Figures 11 and 12 shows that in neither test did the debris flow become a turbidity current, again confirming the predictions of a change in flow regime based on the Bingham Reynolds number.

The centrifuge tests confirm that, due to the exponential relationship between LI and shear strength, an 11% change in the water content of the material caused a change in the flow regime, which did not occur with the 12.5% increase in launch pressure from test w90_p0.8 to test w90_p0.9.

The results can be better understood by analyzing the influence of the variation of the velocity and the water content separately in the $Re_{Bingham}$. Equation 4 shows that $Re_{Bingham}$ increases linearly with velocity. In contrast, as

has been shown for the marine clay from the Brazilian pre-salt (Equations 10 and 11) and other soils studied in similar investigations, $Re_{Bingham}$ increases exponentially with increasing water content.

The transformation of the submarine debris flows into a turbidity current (Figure 1) causes an accelerated dissipation of kinetic energy, which translates into greater deceleration and lower run out. This confirms the need to use adaptive rheological models, such as the one presented, for the modeling, study and analysis of the risks caused by submarine debris flows in offshore installations.

5. Conclusions

The main conclusions of this research are:

- The proposed rheological model makes it possible to model the different stages of a debris flow, since both yield stress and viscosity (Bingham and Bilinear models) are replaced by a function correlated solely with the LI of the material. As such, it integrates the changes in rheological properties resulting from the incorporation of water into the medium.
- The bilinear model should preferably be used when modeling submarine debris flows in their initial phase,

since after the slope failure the velocity developed by the flow is not yet high, resulting in lower shear rates. In other cases, the model can be simplified to Bingham, which has the advantage of mathematical simplicity.

- The behavior of a submarine debris flow is very sensitive to changes in the water content of the material, since the Reynolds number increases linearly with increasing flow velocity but increases exponentially with increasing water entrainment.
- The use of the proposed model applied to the centrifuge tests conducted by Hotta et al. (2020) to calculate the Bingham Reynolds number allowed to interpret the changes in flow regime and better understand the behavior of the debris flow. The application of these concepts in numerical modeling and analysis of the risks caused by submarine debris flows provides greater security for offshore installations.

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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’ contribution

Marcelo Muta Hotta: conceptualization, investigation, data curation, visualization, writing – original draft. Marcio de Souza Soares de Almeida: project administration, methodology, supervision, validation, writing – review & editing. José Renato Moreira da Silva de Oliveira: formal analysis, supervision, writing – review & editing. Ricardo Garske Borges: funding acquisition, review.

Data availability

Data sets generated and analyzed during the current study are available from the corresponding author upon reasonable request.

List of symbols and abbreviations

a	Acceleration
g	Gravity
t_0	Initial time
t_f	Final time
v_0	Initial velocity
v_f	Final velocity
v_m	Average velocity
w	Water content
w90_p0.8	Centrifuge test with water content 90% and pressure 80 kPa
w90_p0.9	Centrifuge test with water content 90% and pressure 90 kPa
w100_p0.8	Centrifuge test with water content 100% and pressure 80 kPa
w_p	Water content at plasticity limit
w_l	Water content at liquidity limit
CaCO_3	Calcium carbonate
CC	Concentric cylinder
CCT	Concentric cylinder geometry
D_e	Equivalent Diameter
LI	Liquidity index
N	Scale factor
P_1	Checkpoint 1
P_2	Checkpoint 2
Re	Reynolds number
Re_{Bingham}	Bingham Reynolds number
SiO_2	Quartz
S	Flow displacement
S_0	Initial flow displacement
Si	Soil sample i
Ti	Rheological test i
USCS	Unified soil classification system
VT	Vane geometry
$\dot{\gamma}$	Shear rate
$\dot{\gamma}_c$	Critical shear rate
μ	Viscosity
μ_1	Viscosity in Regime I of the Bilinear Model
μ_2	Viscosity in Regime II of the Bilinear Model
ρ	Mass density of the fluid
ρ_s	Mass density of the solid
τ	Shear strength of the material
τ_0	Yield stress
τ_c	Critical yield stress
τ_p	Remolded shear strength

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Analysis of field monitoring of precipitation and pore pressure over seven years at Morro do Boi of Serra do Mar/Brazil

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Article

Keywords

Field monitoring
Pore water pressure
Stability of slopes
Geotechnical analysis

Abstract

The stability of slopes or the occurrence of mass movements is directly related to the incidence of rainfall. In this work a qualitative analysis of the historical series was made for the period between May 2012 and February 2019, where the development of pore pressures was evaluated, analyzing the behaviors of the instruments, correlating their readings with precipitation and with variations in the water table. Pore pressure was measured using vibrating wire piezometers, and precipitation data was collected using pluviometers with tipper buckets, whose readings were obtained by a datalogger model ML1-FL. From the data collected, it was possible to estimate the duration of a rainfall event for the soil, as that time interval between the occurrence of rain and the dissipation of excess pore pressure. These intervals are between 3 and 26 days in duration. The main results were: the period between the end of June and August is the driest and the highest precipitation occurs from December to early June; Precipitation of up to 100 mm of daily accumulation is well distributed throughout the year and occurs in all seasons. The pore pressure values developed were low, with a maximum of 9.3 kPa for the deepest piezometer on elevation 92.40 m. There is a delay between the occurrence of precipitation and the recording of the peak pore pressure. The in-depth knowledge of variations in precipitation and pore pressure can support the development of more robust geotechnical engineering projects. This preventive approach, based on concrete data and correlations, offers a more reliable strategy for mitigating the risks associated with landslides, directly contributing to the safety and continuous operation of the BR-101/SC highway.

1. Introduction

Serra do Mar Mountain Range is one of the most prominent orographic features of the south and southeast coast of the country. It measures 1500 km in length and is the site of the highest number of landslides in Brazil (Listo et al. 2021). In the period between 1991 and 2012, there were several mass movements along the BR-101 highway, in Serra do Mar. One of these movements was the landslide at km 140+700 m, in Morro do Boi, which interrupted the traffic, causing damage to society (Pretto, 2014a). This occurrence motivated the implementation of a containment system and an instrumentation plan in 2012 that served as the basis for the research project to which this work is linked.

This slope has already been the subject of some research. Sestrem et al. (2015) analyzed changes in groundwater levels and their relationship with rainfall in sixteen months of monitoring, Lazarim (2012) conducted a geotechnical characterization of the colluvial layer, González et al. (2017) evaluated the first 10 months of results and gave statistical treatment, Pretto (2014a) obtained permeability parameters

and water retention curves in the soil and Joly (2018) evaluated the depth of the wetting front.

Santos (2014) reports 94 occurrences of landslides in 2008 and 31 in 2011 along the BR-101. Only in the area that is the object of this study, 14 occurrences of landslides were recorded between January 2015 and December 2018 related to rainfall events (Salah et al., 2019).

Therefore, the contribution and objective of this article was the general evaluation of the behavior of Morro do Boi in relation to precipitation and the development of pore pressures. Data from 7 years of monitoring were analyzed, estimating the duration of a pluviometric event for the soil, and establishing correlations between precipitation and pore pressure variations, in order to better understand the mechanisms that promote slope instability and provide more technical information to engineers working on site trying to avoid catastrophes.

2. Materials and methods

The Serra do Mar region in Brazil experiences a high frequency of landslides. This study investigates the impact

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of rainfall on pore pressure development, analyzing data collected from May 2012 to February 2019, using Excel software. Although the analysis of time series is defined as a technique for observing trends, seasonality, and predicting values of historical series, in this work a qualitative analysis of trends and seasonality was performed with no aim of predicting future values.

Pore pressure was measured using vibrating wire piezometers (Geokon sensor model 4500S), and precipitation data was collected using pluviometers with tipper buckets was model TB4/0.2 from Hydrological Services, whose readings are obtained by a datalogger model ML1-FL. Detailed calibration and data collection procedures are described. Readings from both types of instruments were collected automatically.

The vibrating wire piezometers were installed at the residual soil layer center, at the interface soil-weathered rock, and at the contact between the weathered rock and bedrock. This configuration was used in two sections of the slope, one uphill of the stabilized area (cluster 1 with elevation 102.5 m), which includes piezometers 04, 05, and 06, and the other inside it (cluster 2 with elevation 92.40 m), which includes piezometers 01, 02 and 03. This use enabled a comparative analysis between both places (Sestrem et al., 2015). It is also noteworthy that piezometers have an accuracy of 0.35 kPa (Sestrem, 2012).

A pluviometer of the tipping bucket type was set up on cluster 2 to monitor rainfall, capable of recording events for every 0.25 mm of precipitation. The recorded data was stored in a data logger with a maximum reading capacity of 700 mm/h, and it also had the capability to record the date and time of each event (Hydrological Services, 2014a, b).

The monitoring of all sensors commenced in May 2012 through an automated data logger (Geokon, 2014),

with initially configured time intervals of 8 hours. However, between 11/08/2012 and 12/12/2012, data collection was hindered due to technical issues.

2.1 Description of the place

The study site was a slope on the banks of the BR-101 on the south lane at km 140+700 m, with latitude of -27.0494 and longitude of -48.6122, having already been the subject of detailed descriptions by several authors (Sestrem et al., 2015; González, 2013; Pretto et al., 2014b), being a natural slope, belonging to the Serra do Mar, in the State of Santa Catarina/Brazil.

The Morro do Boi area features two main types of rock: the intrusive Nova Trento granites and Morro do Boi migmatites. These lithologies are lightly fractured and maintain relatively high strength when not exposed to significant climatic variations. The region is influenced by NE-SW and NW-SE shear surfaces and sub-horizontal fractures that segment the rock mass into blocks, significantly weakening the mechanical stability of the slope. Additionally, the presence of colluvial/talus soils creates a highly heterogeneous structure with a high degree of weathering and very low cohesion (Fiori, 2011). A detail is presented in Figure 1.

Additional characterization and laboratory tests on shear strength were conducted by Lazarim (2012). The findings revealed an average grain density of 2.66 g/cm³, with an average liquid limit of 32% and an average plastic limit of 27%, indicating materials with low plasticity and compressibility. The particle size distribution demonstrated that the soils were predominantly sandy (60.4%), with 27.0% silt, 8.3% gravel, and only 4.3% clay. Direct shear tests on four different block samples yielded effective friction angles ranging from 28° to 39° and cohesion intercepts between

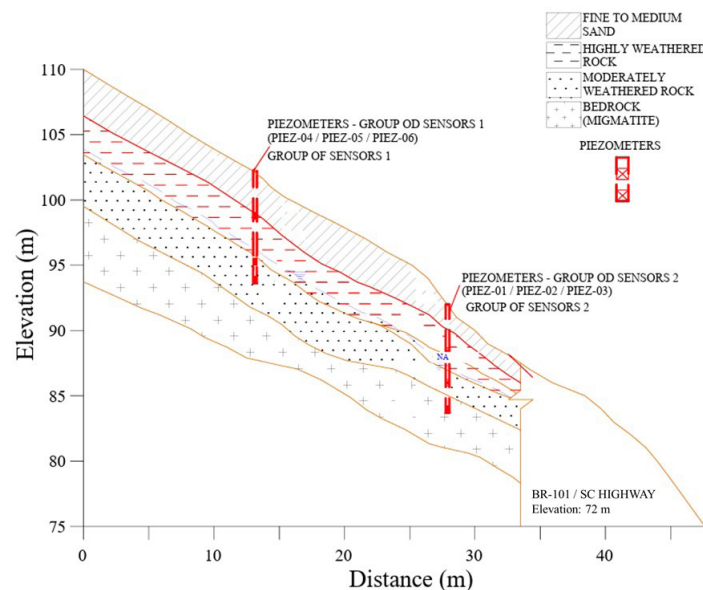


Figure 1. Geological and geotechnical profile of the studied slope.

1 and 17 kPa. The samples were collected at depths ranging from 0.25 to 1.27 m, with water content between 3% and 10% and saturation levels between 2% and 8%.

2.2 Precipitation

In the precipitation analysis, a characterization of the rains was made according to their daily accumulation, and statistical treatment was given for a better understanding of the rains in the analyzed period. The lack of data was observed, and a linear correlation was made with three nearby stations to evaluate the possibility of filling them.

A correlation was made between the data from Morro do Boi and from each of the stations for the period with equivalent dates through simple linear regression. The station was chosen according to the existence of the data and the correlation obtained with the data from Morro do Boi. Preferably, data from station C-2927 - Laranjeiras was used, and in the absence of records at this station, C-2385 - Tijucas was used, and in the absence of data from this station, C-2033 - Rio Canoas with the respective linear correlations 0.83, 0.68 and 0.65.

The data were completed preferring the station with the best linear correlation. Analysis of monthly averages was carried out to identify the seasonality of rainfall and analysis of their distribution by daily accumulated, in addition to the distribution according to the CIRAM classification (2016 apud González et al., 2017).

2.3 Pore pressure

Pore pressure readings were taken using piezometers. As with precipitation data, there was no data in certain periods

for positive pore pressure as well. Where there was no data from all instruments, these data were ignored, and when there was only one instrument, data was filled in through simple linear regression with data from the instrument with the highest linear correlation.

2.3.1 Analysis of the relationship between water level variation (WL) and pore pressure readings

For comparison between piezometer readings and manual WL readings, both were transformed into total potential values. To obtain measured potential values, or measured elevation (ME), in piezometers, add the instrument elevation value (IE) to the ratio between the measured pore pressure value (PP) and the unit weight of water (γ_w). The instrument elevation (IE) is the subtraction of the piezometer depth (PD) from the elevation of the terrain where the instrument is positioned (TE). Urciuoli et al. (2016) use a similar approach but adopt measured depth rather than total potential. Table 1 shows the elevation values of the instruments and water level reading wells and Figure 2 shows a scheme of the adopted methodology.

2.3.2 Linear correlation between pore pressure readings and precipitation records

A linear correlation analysis was performed between the values of precipitation and pore pressure, where daily accumulated values of 1, 2, 3, 5, 7, 10, 15, 21, 30, and 45 days and delays in readings of 2, 3, 4, 5 and 7 days. The data were organized in such a way that accumulated precipitation could be obtained in the periods analyzed and delays in the readings could be imposed as suggested by Buzzi (2007).

Table 1. Elevation of instruments.

	PZE-01	PZE-02	PZE-03	MP-04	PZE-04	PZE-05	PZE-06
PD (m)	8.65	6.40	3.90	-	8.60	7.20	3.70
TE (m)	92.40	92.40	92.40	92.40	102.50	102.50	102.50
IE (m)	83.75	86.00	88.50	-	92.90	94.30	97.80

PD: piezometer depth; TE: elevation of the terrain; IE: instrument elevation.

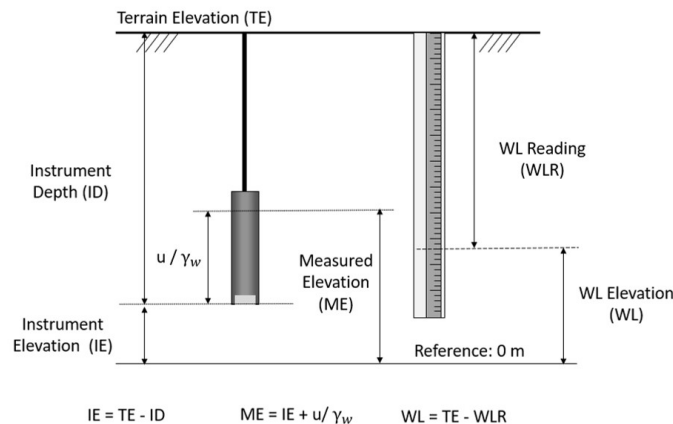


Figure 2. Scheme for transforming WL and pore pressure readings into elevation values in meters or total potentials.

2.3.3 Qualitative analysis of pore pressure increases

Time intervals were analyzed where the dates of the highest daily precipitation records and the date of the highest pore pressure records were chosen. These dates were called “events”. The analysis interval was defined as the one where it is possible to observe a significant variation in pore pressure values, starting from zero or a minimum value to a peak and returning to the initial value, which would represent the duration of a pluviometric event. Table 2 presents the chosen events. Event 3 represents the date with the highest pore pressure record and the others represent events with daily rainfall above 100 mm. Events 7 and 8 were divided into 7 and 7A and 8 and 8A due to the low variation in the reading of some instruments in the main event.

The time to reach peak PP in the interval was obtained by comparing the date of the event with the date of the highest pore pressure (PP) record. The increase in pore pressure generated after the event was obtained by comparing the PP reading on the day before the event with the peak reading of the interval.

To estimate the time needed to dissipate the increase in pore pressure, the best reduction trend was analyzed after reaching the PP peak in the period. For this purpose, the first trend after the peak was chosen, and this same trend was extended until its projection reached the PP level of the day before the event. This criterion was adopted, as there is a clear change in the tendency and speed of dissipation, which was possibly caused by later rains. Therefore, in order to disregard the possible effect of these later rains, the second dissipation trend was disregarded. The crossing between dissipation trends and the PP level on the previous day can occur between two dates. Therefore, it was adopted as a criterion, to choose the closest date, and in doubt, the smallest date.

3. Analysis and results

3.1 Precipitation

Figure 3 presents precipitation data summarized in monthly terms. Despite the variation over the years, it is

Table 2. Range of events chosen for qualitative analysis.

Event	Date of the event	Precipitation in the Day (mm)	Analyzed ranges	
			Start	End
1	04/03/2013	155.20	03/23/2013	05/09/2013
2	01/11/2014	121.40	12/27/2013	02/16/2014
3	11/04/2015	96.00	08/29/2015	12/06/2015
4	03/25/2016	238.60	03/20/2016	05/02/2016
5	04/13/2016	189.20	03/20/2016	05/02/2016
6	10/25/2016	123.00	10/20/2016	11/13/2016
7	05/31/2017	119.80	05/18/2017	07/02/2017
7A	06/05/2017	67.40	05/18/2017	07/02/2017
8	01/10/2018	106.20	01/04/2018	02/12/2018
8A	01/15/2018	18.20	01/04/2018	02/12/2018

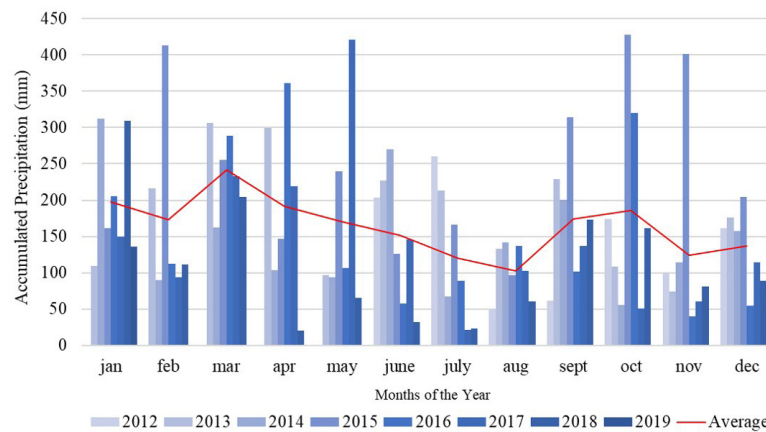


Figure 3. Monthly rains.

possible to observe that the rainiest periods are from January to April and between September and October. The rainiest season is summer (end of December to end of March) and the least rainy is winter (end of June to end of September), but with small differences in relation to the other seasons, making the region with frequent rain all year round.

It was found that the highest number of rainfall occurrences was recorded in the summer (end of December to end of March), and the highest daily accumulated records occurred in the spring (end of September to end of December). It is noteworthy that the analysis without filled data showed rainfall distribution in 27% spring, 30% summer, 23% fall, and 20% winter. Also, the accumulated in mm/day was 47% of 0 mm, 17% between 0 and 1 mm, 21% between 1 and 10 mm, 9% between 10 and 25 mm, 4% between 25 and 50 mm, 2% of 50 and 100 mm, 0% above 200 mm. It is observed that about 84% of the rainfall events had daily accumulations of less than 10 mm.

The analysis with the filled-in data showed rainfall distribution in 25% spring, 28% summer, 25% fall, and 22% winter. Also, the accumulated in mm/day was 43% of 0 mm, 17% between 0 and 1 mm, 25% between 1 and 10 mm, 9% between 10 and 25 mm, 4% between 25 and 50 mm, 2% of 50 and 100 mm, 0% above 200 mm. With the data filled in, a reduction of 4 percentage points is observed in the proportion of days without rain. This happens because there is a linear coefficient in the regression equation and, therefore, even on days without rain in the correlated stations, the regression will indicate a value greater than zero to fill in the data. But, in general, no significant change in the distribution of rainfall is observed with filling in the missing data. Following the CIRAM classification (2016 apud González et al., 2017), the analyzed period can be divided as shown in Table 3.

3.2 Positive pore pressure

Evaluating the pore pressure values measured through the 6 piezometers, it is observed that the magnitude of the

readings is small, with maximum values that do not reach 10 kPa and averages close to zero. This observation is consistent with the draining capacity of the geotechnical profile, whether due to the type of soil and its permeability or the presence of fractures in the layers of rock alteration, as observed in Domingues (2020). Table 4 presents the maximum, minimum, mean, and standard deviation values for the values measured in the 6 piezometers.

Figures 4 and 5 show the frequency diagrams for pore pressure and daily precipitation values, with the piezometers grouped by instrumentation cluster. Salah et al. (2019) identify the dates of occurrences of landslides in the Morro do Boi area between January 2015 and December 2018. These dates are identified on the charts. But, despite the low magnitude recorded, the development of positive pore pressures is observed in all instruments, except for piezometer 2 (PZE-02). The averages of all instruments are close to 0 kPa, except the PZE-02 which is close to -5 kPa.

It is observed that the frequency diagram for the PZE-02, although it has variations consistent with the precipitation and with the other instruments, is at a differentiated pore pressure level, presenting an average of values around -5 kPa. A possible justification would be the possibility of this instrument not being influenced by the elevation of the water level (WL) or the wetting front.

3.2.1 Variation of pore pressure with water level

Figures 6 and 7 show the frequency diagrams of pore pressure values and WL measurements in terms of total potential for clusters 2 and 1, respectively. It is noteworthy that, for Figure 7, the WL of inclinometer 2 from Domingues (2020) was used for the analysis.

On cluster 2, the total potential change according to the WL readings was 5.11 m while the potential change for the piezometers was 1.35 m, 0.35 m, and 0.78 m for the PZE-01, PZE-02, and PZE-03 respectively. It is observed that

Table 3. Classification according to CIRAM (2016 apud González et al., 2017).

Classification	Precipitation Range (mm/day)	Events (days)	Events (%)
Dry Day (DD)	$P < 0.2$	1055	42.40
Drizzle (D)	$0.20 < P < 0.60$	284	11.41
Light Rain (LR)	$0.60 < P < 3.20$	476	19.13
Moderate Rain (MR)	$3.20 < P < 13.0$	367	14.75
Heavy Rain (HR)	$13.0 < P < 45.6$	249	10.01
Violent Rain (VR)	$P > 45.6$	57	2.29

Table 4. Maximum, minimum, average and standard deviation values of positive pore pressure.

u (kPa)	PZE-01 (8.65 m)	PZE-02 (6.40 m)	PZE-03 (3.90 m)	PZE-04 (8.60 m)	PZE-05 (7.20 m)	PZE-06 (3.70 m)
Max	9.30	-2.31	4.64	6.56	3.40	2.30
Min	-3.91	-5.75	-3.05	-4.07	-3.40	-2.27
Average	0.01	-5.07	-0.83	-0.69	-0.92	-0.61
SD	2.21	0.66	0.87	2.12	0.76	0.59

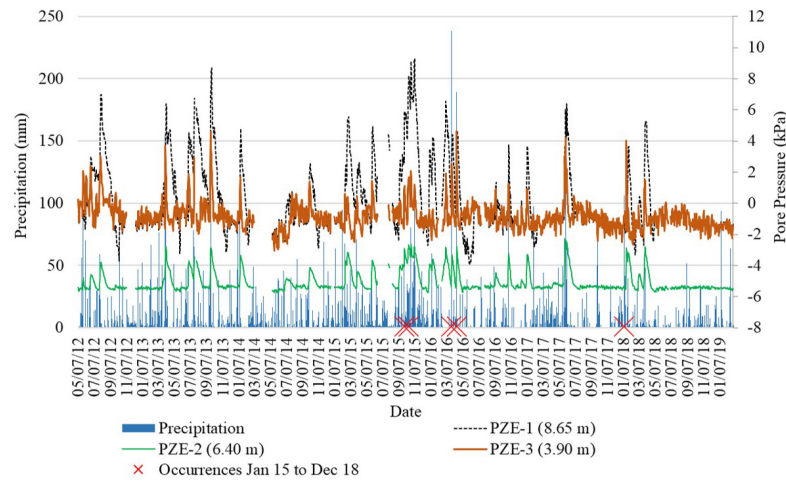


Figure 4. Piezometer frequency diagram for cluster 2.

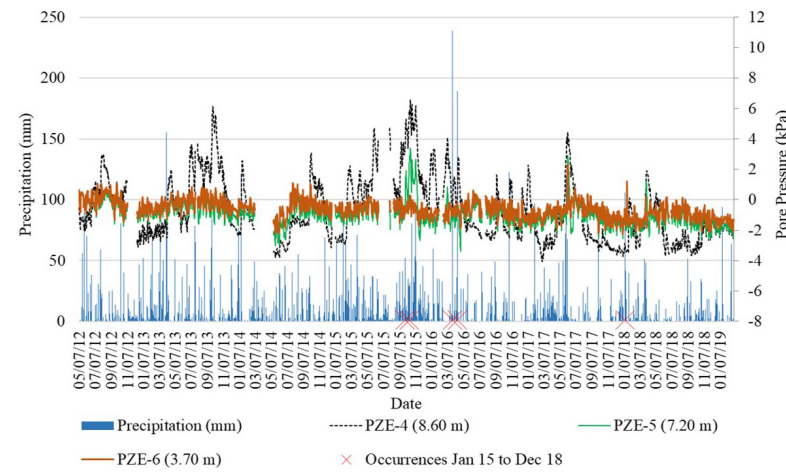


Figure 5. Piezometer frequency diagram for cluster 1.

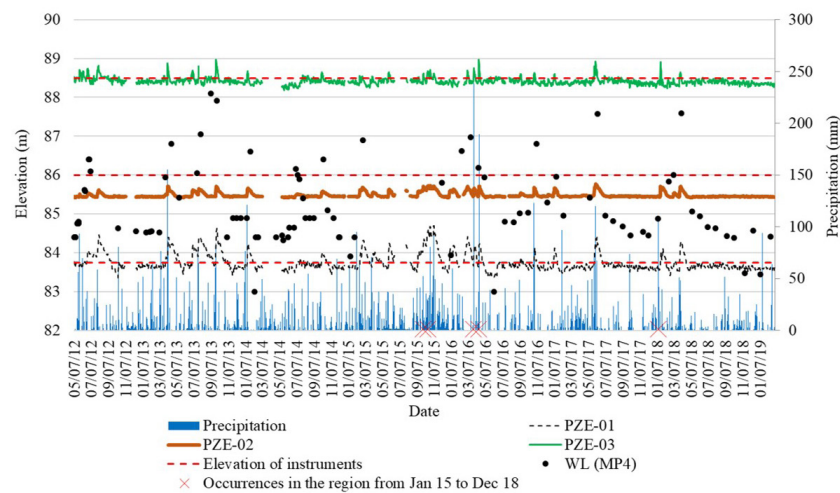


Figure 6. Frequency diagram of pore pressures and WL measurements in terms of elevation for cluster 2.

the WL was above the PZE-01 quota practically the entire period. In relation to the PZE-02, the WL alternates higher quota records than this Instrument with periods with lower quotas. For the PZE-03, WL depth measurements indicate that it was always below the instrument's height.

Based on these observations, it is possible to argue that the PZE-02 may have an error in its readings or that it is positioned on a lens made of less permeable material to the point of imposing this condition of differentiated pore pressure in relation to the other instruments. It can also be seen that this instrument is installed at the interface between a layer of weathered rock and a thin layer of 1.1 m of silty sand with mica and gravel, according to Domingues (2020). It is also observed that there is a relationship between rainfall and variations in the readings in the piezometers and the water level. The potential values of the WL readings and the PZE-01 and PZE-02 piezometers showed good linear correlation (0.72 and 0.70), while the correlation between WL and PZE-03 was lower (0.36).

In cluster 1, whose frequency diagram is shown in Figure 7, there is a smaller variation in the measured potential values in the PZE-05 and PZE-06 instruments. The potential variation measured in the WL readings was 3.7 m while for

the piezometer readings, it was 1.08 m, 0.69 m, and 0.47 m for PZE-04, PZE-05, and PZE-06, respectively. The WL records were almost the entire period below the instrument installation quota, except on 3 dates where they exceeded these quotas, indicating that on cluster 1, even the deepest instrument is rarely submerged.

The linear correlation coefficients between the measured potential values in the PZE-04, PZE-05, and PZE-06 piezometers and the WL values were, respectively, 0.65, 0.40, and 0.30. A lower correlation can be seen in relation to cluster 2 and the PZE-04 has a better correlation with WL than the PZE-05 and PZE-06 piezometers.

3.2.2 Correlation of pore pressure values with precipitation

The linear correlation values obtained were relatively low, ranging from 0.10 to 0.61. Table 5 presents the best values obtained from linear correlation combining daily accumulated variations and delays in readings.

For the piezometers on cluster 1 (PZE-04 to PZE-06), the correlation values were higher for 45 days. For the cluster 2 piezometers, the best correlations were for daily accruals

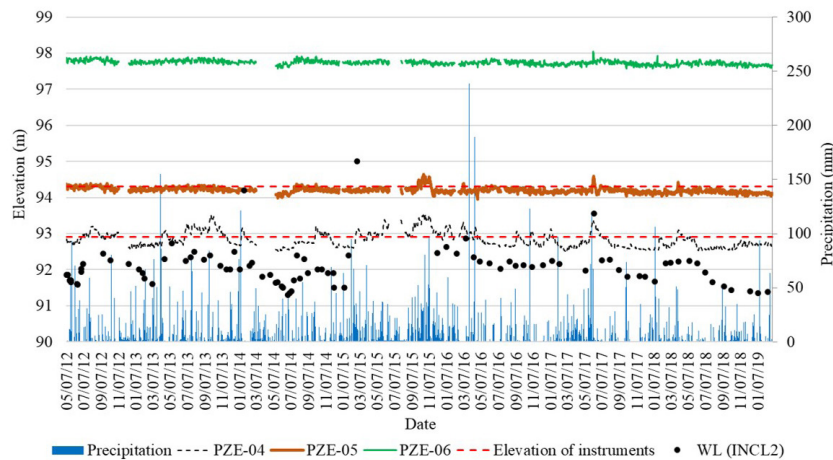


Figure 7. Frequency diagram of pore pressures and WL measurements in terms of elevation for cluster 1.

Table 5. Better correlations obtained between piezometer readings and precipitation.

Instruments		Late			Without delay		Better Correlation with delay
PZE	Depth	Delay (days)	Accumulated (days)	r	Accumulated (days)	r	
1	8.65 m	1	30	0.55	30	0.55	0.64%
2	6.40 m	1	21	0.61	21	0.60	1.61%
3	3.90 m	1	7	0.39	7	0.38	4.46%
4	8.60 m	2	45	0.47	45	0.46	2.34%
5	7.20 m	1	45	0.23	45	0.22	2.09%
6	3.70 m	7	45	0.13	45	0.10	31.23%

Values other than 0 with significance level, $\alpha = 0.05$; r: linear correlation.

of 30, 21, and 7 days for instruments PZE-01, PZE-02, and PZE-03 respectively. By imposing delays on the readings, small improvements were obtained in the values of the linear correlation coefficient. From the point of view of statistical analysis, these values are low and from the point of view of the phenomenon involved, many days of daily accumulation or delay do not seem to make much sense because, depending on the rainfall regime in the region, long periods later or earlier are affected by other events. Therefore, it is assumed that there is a low correlation between the values of the piezometer readings and the precipitation records.

This absence of a good correlation is expected since not all precipitation infiltrates. A part of the rain is intercepted by the vegetation, other drains superficially, another evaporates through the plants (evapotranspiration), and other infiltrates (Urciuoli et al., 2016).

Another factor is that the relationship between infiltration and rainfall is not linear, since it depends on the soil moisture condition prior to the rain, the intensity and duration of rainfall, and the slope (Tsaparas et al., 2003; Rahardjo et al., 2005; Cuomo & Della Sala, 2013). Rahardjo et al. (2004) report a relationship between infiltration and precipitation of 40% to 74%, Rahardjo et al. (2005) found infiltration rates ranging from 40% to 100% of precipitation and Li et al. (2005) suggest a relationship of 59% to 75%, however, this proportion is variable and maybe 100% for small accruals and at the beginning of an event until the runoff starts and the ratio begins to fall. In addition, in sandy soils, the moisture absorption capacity is high when the soil is dry (Ridley et al. 2003), but its permeability in this condition is very low, in the levels of fine soil (Urciuoli et al., 2016). However, it is possible to analyze and identify the relationship between precipitation and the development of pore pressures qualitatively.

3.2.3 Qualitative analysis of the development of pore pressures

Table 6 presents the chosen events, which are the dates with rainfall records greater than 100 mm and the dates with

the highest pore pressure record in the series. In addition to the precipitation values for each event (daily record), the highest pore pressure values measured after the event (pore pressure peak) are presented.

It should be noted that some observations on analyses carried out and the results obtained are presented below. Figure 8 represents event 1 on 04/03/2013 for instrumentation cluster 2. In this event, the rain on 04/03/2013 with a daily accumulation of 155.2 mm is followed by rain on the following day with a daily accumulation of 85.80 mm. It is actually the same rain event, as the hourly records indicate that the rain started on 04/03/2013 between 1:01 pm and 2:00 pm and ended on 04/04/2013 between 5:01 am and 6:00 am.

Figure 9 shows event 1 for instrumentation cluster 1, where in addition to the pore pressure magnitudes being smaller than that recorded on cluster 2, the PZE-05 and PZE-06 instruments were practically not influenced by these rains and recorded increases in pore pressure of 0.10 kPa and 0.12 kPa, respectively, values lower than the precision of the instrument. Therefore, for this event, analysis was performed only for PZE-04.

In event 3, on 11/04/2015, with a daily accumulation of 96 mm, there was the highest pore pressure record, which was 9.30 kPa in the PZE-01. Figure 10 shows the frequency diagram for cluster 2. The sequence of increments in the pore pressure values generated by smaller rainfall prior to the event is observed but with short intervals between one and the other. The dates where there are records of landslides in the region from January 2015 to December 2018 are marked, according to Salah et al. (2019).

The days in which landslides were recorded in the region, in this interval, do not coincide with days of greater precipitation or greater pore pressure. The frequency diagram for the same period relative to event 3 for cluster 1 is shown in Figure 11. For the PZE-04 piezometer, the development of pore pressures is similar to that recorded in the PZE-01 on cluster 2, but with a peak of lower pore pressure. It is observed that the PZE-05 is less influenced by the rains of

Table 6. Events chosen for qualitative analysis.

Event	Date	Rain (mm)	Highest pore pressure value of the range (kPa)					
			PZE-01	PZE-02	PZE-03	PZE-04	PZE-05	PZE-06
1	04/03/2013	155.20	6.40	-2.80	3.71	1.05	-0.43	-0.17
2	01/11/2014	121.40	4.81	-3.36	1.70	2.56	-0.55	-0.09
3	11/04/2015	96.00	9.30	-2.79	1.21	6.16	2.66	-0.22
4	03/25/2016	238.60	4.41	-3.31	2.51	1.52	-0.71	-0.45
5	04/13/2016	189.20	4.66	-2.81	4.64	2.81	-0.34	-0.15
6	10/25/2016	123.00	3.75	-3.21	1.24	1.37	-0.28	0.06
7	05/31/2017	119.80	6.03	-2.31	4.22	4.13	0.48	-0.25
7A	06/05/2017	67.40	6.37	-2.49	2.18	4.48	2.90	2.30
8	01/10/2018	106.20	-0.78	-5.33	-0.51	-2.88	-0.88	-0.59
8A	01/15/2018	18.20	2.59	-2.86	4.01	-3.29	-1.24	1.23

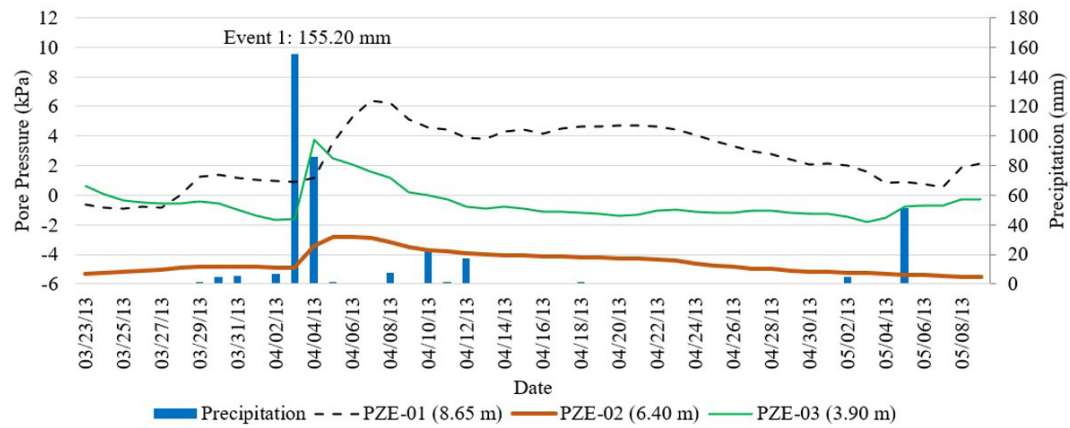


Figure 8. Frequency diagram of pore pressures and precipitation in event 1 on cluster 2 - period from 03/23/2013 to 05/28/2013.

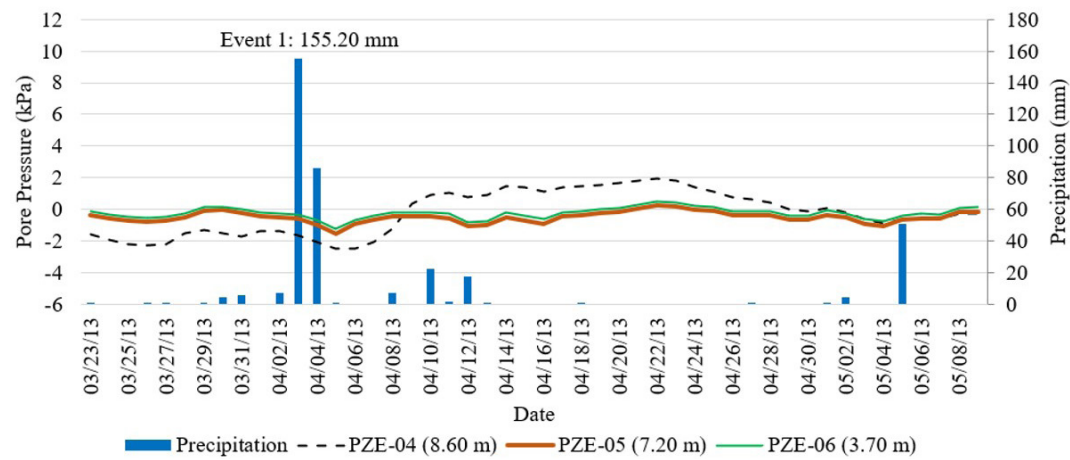


Figure 9. Frequency diagram of pore pressures and precipitation in event 1 on cluster 1 - period from 03/23/2013 to 05/09/2013.

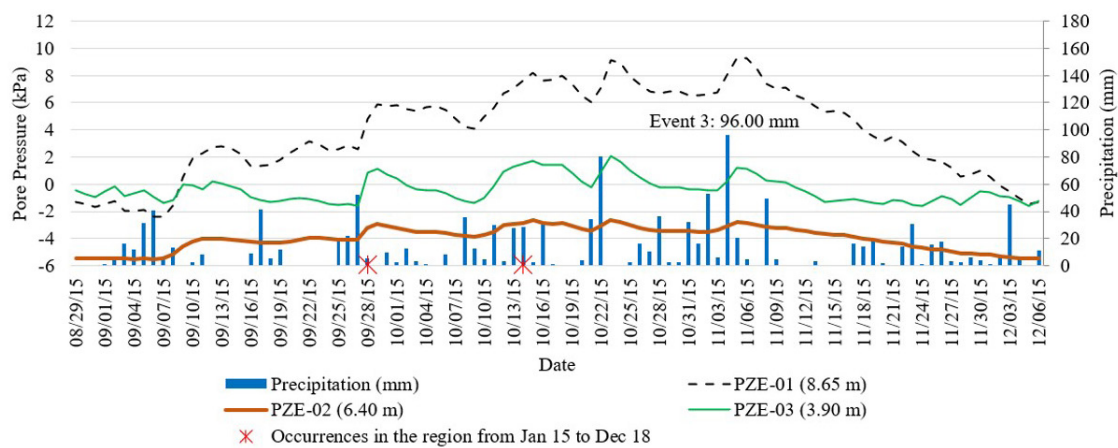


Figure 10. Event 3 frequency diagram on cluster 2.

that period and that the PZE-06 is practically not influenced by these rains. The pore pressure peak of the analyzed period occurred on 08/29/2015, before event 3, on 11/04/2015.

Event 4 on 03/25/2016 and event 5 on 04/13/2016 for cluster 2 are shown in Figure 12. As in the events presented above, there are pore pressure peaks clearly related to the

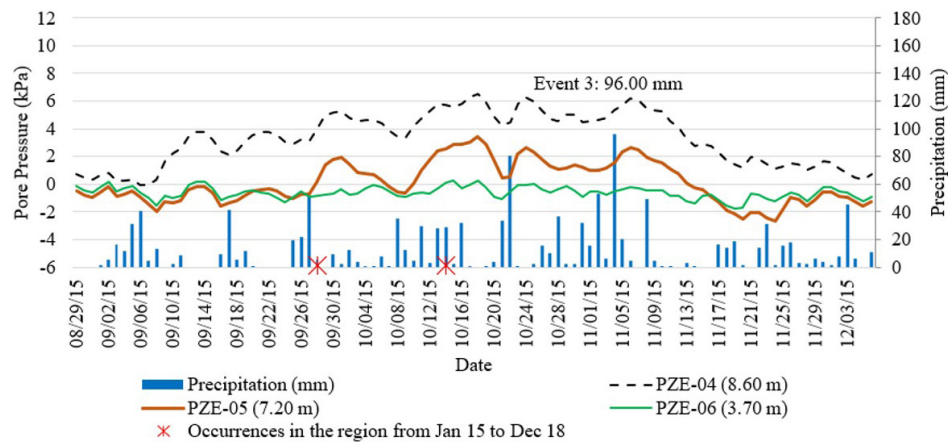


Figure 11. Event 3 frequency diagram on cluster 1.

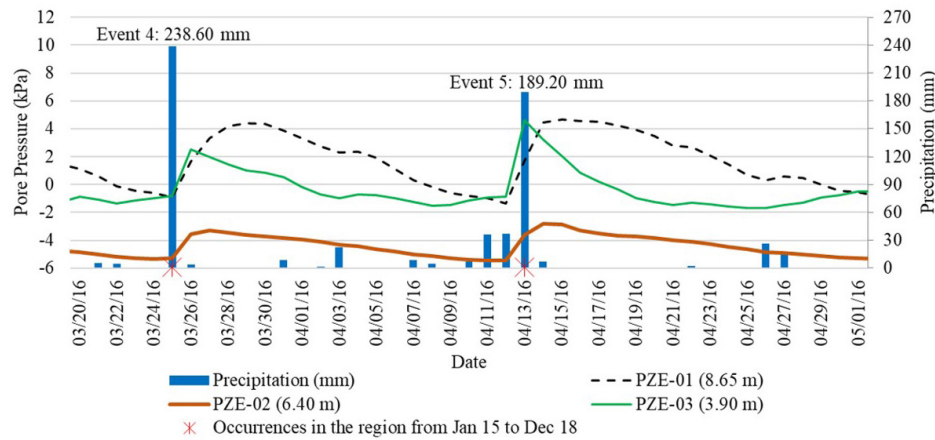


Figure 12. Cluster 2 frequency diagram for events 4 and 5.

events, and trends of fall after the events. On these two dates, there were also records of landslides in the region and they do not coincide with the dates of the pore pressure peaks.

Figure 13 presents the frequency diagram for the period comprising events 4 and 5, referring to the piezometers on cluster 1. It is observed that the magnitudes of the increases and pore pressure peaks are relatively small, even for the two largest records of accumulated daily rainfall throughout the analyzed period.

The pore pressure additions of the PZE-05 and PZE-06 piezometers were 0.29 kPa and 0.26 kPa, which are lower than the precision of the instrument, therefore, they were disregarded for this analysis.

Ferreira et al. (2018) analyzed the rainfall preceding events 4 and 5 and identified that the difference in instrument response between events can be explained by the difference in accumulated rainfall in the previous days. While in event 4 the accumulated for the previous 7 days was 40 mm for event 5 it was 100 mm.

Events 7 and 8 were divided into 7 and 7A and 8 and 8A due to the little influence of events 7 and 8 on the instruments. In event 7 for cluster 2, there was an influence on the instruments, but the later event called event 7A, despite having less precipitation, raised the pore pressure value in PZE-01 beyond the peak generated by event 7. Figure 14 shows the frequency diagram for these events.

For instruments on cluster 1, event 7 generated a pore pressure increase of 1.62 kPa and 1.25 kPa and peaks of 4.12 kPa and 0.48 kPa for PZE-04 and PZE-05, respectively. The increase in pore pressure of the PZE-06 was lower than the accuracy of the instrument and therefore was disregarded. Figure 15 presents the frequency diagram for these cluster 1 readings.

Figure 16 shows events 8 and 8A for cluster 2. Event 8 on 01/10/2018, with 106.20 mm of precipitation, generated pore pressure increases of 0.07 kPa, 0.01 kPa, and 0.37 kPa in the PZE-01, PZE-02, and PZE-03, which are smaller values or very close to the accuracy of the instruments.

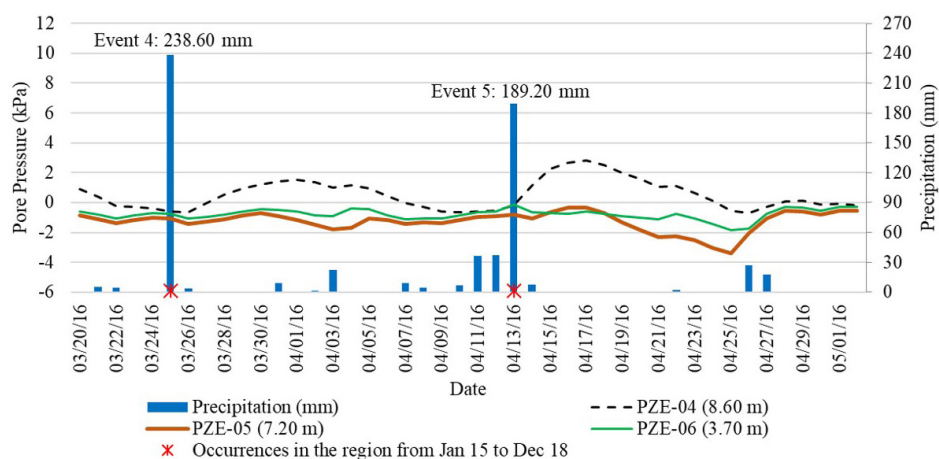


Figure 13. Frequency diagram of piezometers on cluster 1 in events 4 and 5.

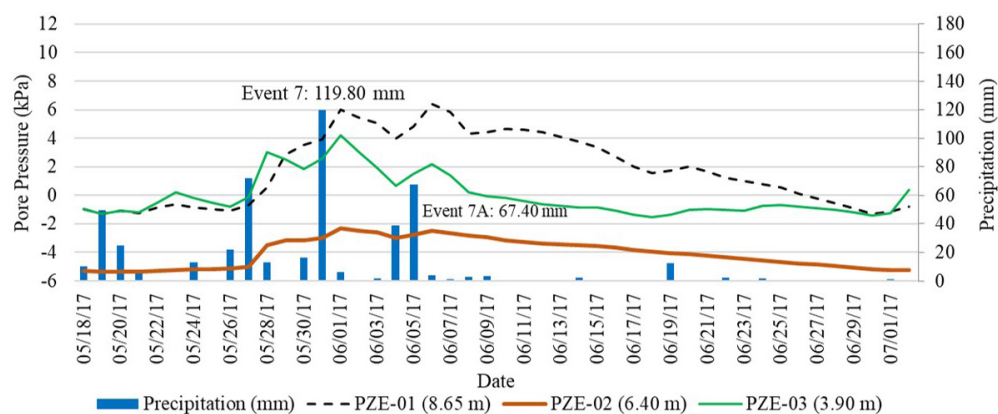


Figure 14. Frequency diagram for events 7 and 7A on cluster 2.

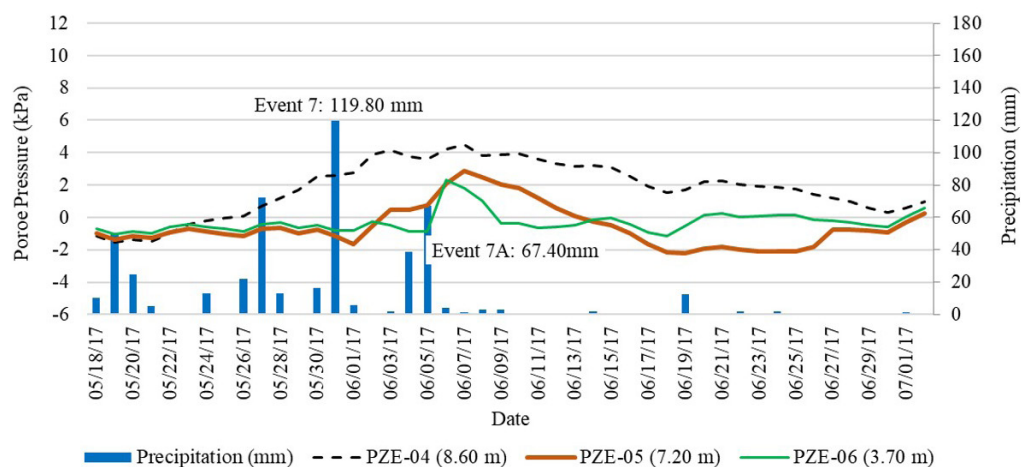


Figure 15. Frequency diagram for events 7 and 7A on cluster 1.

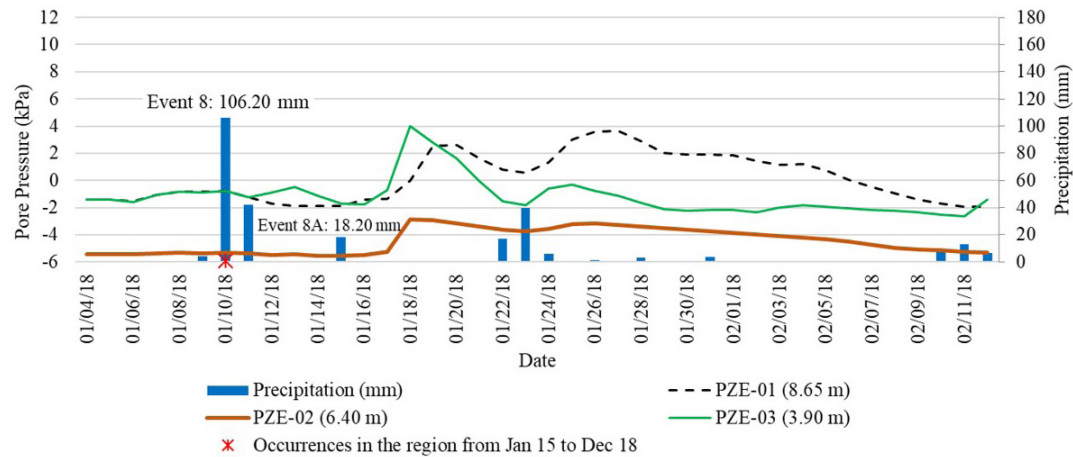


Figure 16. Frequency diagram for events 8 e 8A on Cluster 2.

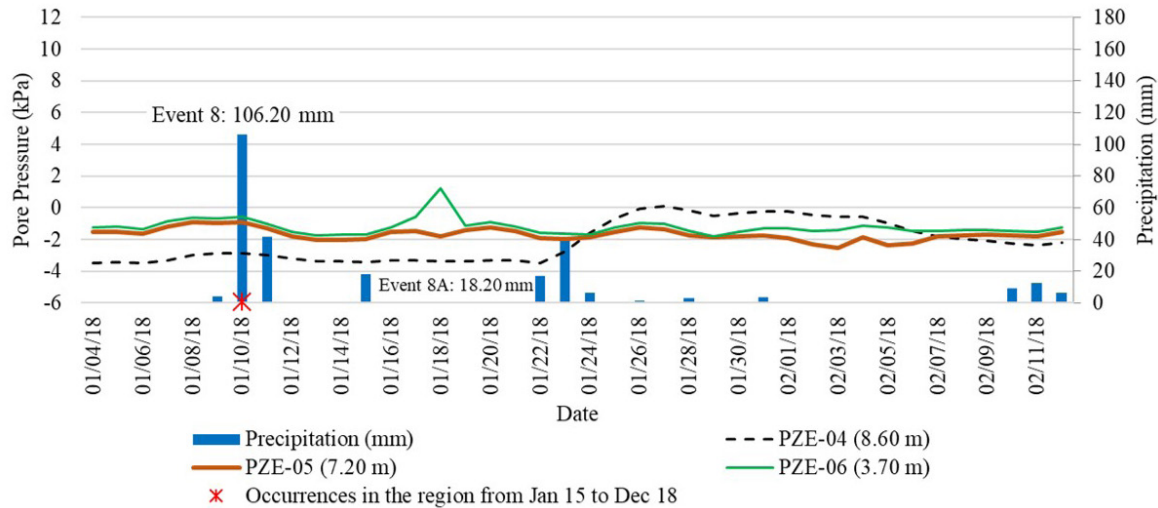


Figure 17. Frequency diagram for events 8 e 8A on Cluster 1.

Therefore, although landslides were recorded in the region on that day, this analysis considers that there were no effects on the piezometers due to this event.

Figure 17 presents events 8 and 8A for cluster 1. As in cluster 2, event 8 did not influence the pore pressure records in the piezometers and in event 8A, only PZE-05 and PZE6 suffered pore pressure and small magnitude increases.

Analyzing events 7, 7A, 8, and 8A, significant differences are identified. On 01/10/2018 (event 8) there was an accumulation of 106.20 mm with rainfall records in 22 hours and with an average intensity of 4.83 mm/h, while on 01/15/18 (event 8A) there was an accumulation of 18.20 mm in two hours, with an average intensity of 9.10 mm. On the other hand, event 7A, with rainfall intensity similar to that of event 8, generated increases in pore pressure. In addition

to the daily cumulative being different, the highest hourly cumulative is also different. In event 8A, the highest hourly accumulation is 14.2 mm, while in event 8 it is 52.6 mm, which could indicate that the distribution of rainfall throughout the day would influence the impact of the increase in pore pressure. However, in event 5 on 04/13/2016, the highest hourly accumulation was 51.0 mm and this was one of the events with the highest increase in pore pressure.

In Domingues (2020) the measures of pore pressure increase, the time to reach the pore pressure peak after the event, and the estimated time for dissipation of the pore pressure increase are presented. Where there was a large dispersion of data and an absence of clear trends since heavy rainfall does not necessarily generate the greatest increases in pore pressure, nor is it possible to establish a clear relationship with the intensity or duration of rain. Frequency diagrams

Table 7. Summary of times to peak and dissipation of pore pressure.

Instrument	Time to reach peak pore pressure (days)		Time to dissipate increased pore pressure (days)	
	Smaller	Bigger	Smaller	Bigger
PZE-01 (8.65 m)	1	5	2	14
PZE-02 (6.40 m)	1	3	3	23
PZE-03 (3.90 m)	1	3	1	10
PZE-04 (8.60 m)	2	8	1	16
PZE-05 (7.20 m)	1	4	1	5
PZE-06 (3.70 m)	0	4	1	5

and pore pressure dissipation graphs for all events are fully presented in the same work. Table 7 presents a summary of observations of the delay to reach peak pore pressures and time to dissipate excess pore pressure.

4. Conclusion

The instrumentation used for this study provided important information regarding precipitation and pore pressure variations over nearly seven years on a slope in a tropical area of the Southern part of the Serra do Mar in Brazil. The following conclusions can be drawn from the present research:

The pluviometric regime recorded by the pluviograph of Morro do Boi is widely distributed throughout the year, making it impossible to clearly define dry and wet periods, which according to Monteiro (2001) is a characteristic of the rainfall regime of Santa Catarina. Despite this, it was possible to identify the period between June and August as the driest and that the highest rainfall occurs in summer and fall.

Precipitation of up to 100 mm of daily accumulation is well distributed throughout the year and occurs in all seasons. Approximately 73% of the daily records are classified as, from dry day to light rain, and only 27% as, from moderate to heavy rain, according to the CIRAM classification (2016 apud Kormann et al., 2016).

Finally, the pore pressure values developed were low, with a maximum of 9.3 kPa for the deepest piezometer on Cluster 2. Furthermore, it was not possible to establish a direct correlation with precipitation, which was expected, since it is only a portion of rain that infiltrates. This proportion depends on the intensity and duration of the rain, the condition of soil moisture before the rain, slope, and vegetation cover.

The results obtained in this study on precipitation and pore pressure in Morro do Boi have direct implications for the maintenance of road infrastructure and the safety of the people who use it. Understanding how climatic phenomena influence the geotechnical properties of the soil, especially precipitation, will allow identifying potential zones of instability with greater precision. This will enable the implementation of more effective preventive measures. Identifying the areas most prone to pore pressure variations could lead to the installation of real-time monitoring systems, such as additional

sensors and piezometers at critical locations. In addition, the development of drainage and vegetation control strategies could be optimized to reduce water infiltration and minimize the impact of seasonal precipitation variations. Such preventive measures, based on concrete data and identified correlations, would offer a more effective approach to mitigating risks associated with landslides.

However, the study has some limitations, such as the lack of a direct correlation between precipitation and pore pressure, which can be attributed to factors such as rainfall intensity and soil condition. Future research should focus on expanding the data collection to include these additional variables and explore different types of weather events and their interactions with soil properties.

In addition, the results of this study provide a solid foundation for developing more advanced predictive models on the relationship between precipitation and pore pressure. Future studies could use this information as a starting point for creating statistical and machine learning models that integrate additional variables such as rainfall intensity and duration, pre-rain soil moisture, slope, and vegetation cover. Advanced statistical techniques such as time series analysis and multiple regression models, along with machine learning algorithms such as neural networks and decision trees, could be applied to identify more subtle patterns and more accurately predict pore pressure variations based on weather conditions. Integrating these methods can further refine the analysis, revealing complex relationships and improving the ability to predict instability events.

Finally, in-depth knowledge of variations in precipitation and pore pressure can support the development of more robust geotechnical engineering projects. This preventive approach, based on concrete data and correlations offers a reliable strategy for mitigating the risks associated with landslides, directly contributing to the safety and continuous operation of the BR-101/SC highway.

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

Giancarlo Domingues: conceptualization, methodology, project administration, data curation, formal analysis, investigation, visualization, writing - original draft. Vitor Pereira Faro: data curation, resources, methodology, supervision, validation, writing - original draft. Elisangela Aparecida Mazzutti: methodology, visualization, supervision, validation, writing - reviewing and editing.

Data availability

Data generated and analyzed in the course of the current study are available in the UFPR Digital Collection: <https://hdl.handle.net/1884/76056>.

List of symbols and abbreviations

CIRAM Information Center for Environmental Resources and Hydrometeorology of Santa Catarina

D	Drizzle
DD	Dry day
HR	Heavy rain
ID	Instrument depth
IE	Instrument elevation
LR	Light rain
ME	Measured elevation
MP	Mixed Probe
MR	Moderate rain
P	Precipitation
PD	Piezometer depth
PP	Pore pressure
PZE	Piezometer
R	Linear correlation
TE	Terrain elevation
VR	Violent Rain
WL	Water level
WLR	Water level reading
γ_w	Unit weight of water

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TECHNICAL NOTE

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Integrating downhole seismic testing into dynamic probing light (DPL): first results

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Technical Note

Keywords

Site characterization
In situ test
DPL
Downhole seismic test
Hybrid test

Abstract

Dynamic probing light (DPL) is available among a wide range of dynamic penetrometers. This equipment can be used in locations where conventional sounding equipment may have limited access. Here, the shear wave velocity (V_s) determination by the downhole seismic technique is incorporated to the DPL to develop a hybrid test, the seismic DPL test, for the preliminary characterization of tropical soils. The corresponding equipment, test procedure and data interpretation of the seismic DPL (S-DPL) test are presented and discussed. S-DPL test campaigns were carried out at two experimental research sites where crosshole seismic tests were conducted to provide reference V_s profiles. The V_s profiles determined with the S-DPL test are in accordance with those determined with the crosshole test for both sites. The proposed hybrid test (S-DPL) has the potential to serve applications in preliminary site characterization, particularly at sites including unsaturated tropical soil profiles.

1. Introduction

Dynamic probing light (DPL) is a dynamic penetrometer known for its simplicity, easy manufacturability and ease-of-use (Figueiredo et al., 2012). The main advantage of the DPL test is that it is a compact piece of equipment that can access sites where conventional sounding equipment may have some restrictions, such as steep slopes, flooded areas, swamps, dense forests, and areas where heavy vehicles cannot access (Cunha & Nilsson, 2004). The DPL test is predominantly used in preliminary site investigations (Duarte et al., 2004) to obtain correlations with other in situ tests (Dos Santos & Bicalho, 2017; Hashemi & Nikudel, 2016; Mohammadi et al., 2008), to define the site stratigraphy from the variation in the cone penetration resistance index (Butcher et al., 1996), foundations, road design and slope stability analysis (Cunha & Nilsson, 2004).

The interpretation of the DPL test is generally based on the number of hammer-blows to drive the probe every 0.10 m (N_{10}) depth interval, which gives an idea of the soil resistance, similar to the N_{SPT} determined by the standard penetration test (SPT). It is important to know the energy transmitted to the system and the real energy delivered to the probe. This approach is known to be rational and has been used in the interpretation of the SPT. The unit tip resistance (q_p) can be determined by knowing the energy transmitted to the probe, which is responsible for the penetration of the DPL tip. It is therefore possible to apply these values to

shallow and deep foundation designs. Bastos (2016) and Almeida (2023) demonstrated that the q_p profile has the same trend and order of magnitude as the cone tip resistance profiles (q_c) obtained from cone penetration tests (CPTs) on an unsaturated tropical soil site.

The shear wave velocity (V_s) is an input parameter in soil dynamic and static analyses. Moreover, V_s can be used in soil classification, state parameter estimation, microstructure identification and collapsible soil identification (Robertson, 2016; Rocha et al., 2022, 2023; Schnaid et al., 2020). It would therefore be valuable to combine the potential of DPL test with the determination of V_s to serve applications in geotechnical design. In this way, this paper presents and describes a system and technique for performing and interpreting seismic DPL (S-DPL) test. The feasibility of combining the DPL test with the downhole seismic technique was evaluated at two experimental research sites where crosshole test data are available.

2. Background

2.1 Dynamic Probing Light (DPL)

Penetrometers are sounding devices that are used to estimate the resistance of soils and soft rocks by measuring the dynamic cone penetration resistance index (Hashemi & Nikudel, 2016; Mohammadi et al., 2008).

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The DPL test involves quasi continuous penetration, does not collect soil samples and the objective is to estimate the resistance and deformability parameters of the tested site as well as to define the soil profile from the variation in the penetration resistance index. The current standard that contains the components, dimensions and test procedures of the DPL test is EN ISO 22476-2:2005 (ISO, 2011). The number of blows that penetrated 0.1 m into the depth interval is registered during the test and is denoted as N_{10} .

The EN ISO 22476-2:2005 (ISO, 2011) standard does not restrict the test depth. Borowczyk & Frankowski (1981) used the concept of critical depth, the point at which the influence of friction between the soil and the guide rods starts to compromise the accuracy of the data. Sanglerat (1972) noted that the friction between the soil and the guide rods is low compared to the resistance of the soil and, in most cases, can be ignored up to a depth of 10 m.

Studies have been conducted with dynamic penetrometers all over the world in the last 50 years, most of which have sought empirical correlations with SPT and CPT (Dos Santos & Bicalho, 2017). Researchers have also attempted to correlate DPL test data with the soil shear strength, California Bearing Ratio (CBR) and degree of compaction (Butcher et al., 1996). Recent advances in DPL data interpretation have involved the use of dynamic instrumentation to determine hammer efficiency (ISO, 2011) and rational interpretation of the test.

The rational interpretation of the test consists of using the hammer efficiency values to calculate the energy transferred to the probe, as described by Odebrecht et al. (2005) and Ibáñez et al. (2012), to obtain the unit tip resistance (q_p) and the unit friction resistance (τ), as described by Lobo (2005).

Lobo (2005) proposed a method for calculating the load bearing capacity of piles using the average dynamic force obtained through the rational interpretation of the SPT, which can be extrapolated to other dynamic penetration tests. The method includes dividing the average dynamic force into two parts, one representing the contribution of the lateral dynamic force mobilized by the sampler or the probe and the other representing the dynamic force of the tip of the sampler or the probe, where:

$$F_d = F_{d,l} + F_{d,p} \quad (1)$$

where F_d is the average dynamic force, $F_{d,l}$ is the lateral dynamic force, and $F_{d,p}$ is the dynamic tip force.

The average dynamic force is calculated by Equation 2:

$$F_d = \frac{E_{probe}}{e} \quad (2)$$

where E_{probe} is the energy delivered to the sampler or the probe and e is the penetrometer advance per blow. The E_{probe} can be theoretically obtained or measured using instrumentation (Lobo, 2005; Odebrecht et al., 2005).

Using average estimated parameters, theoretical formulations of load capacity and spherical cavity expansion, Lobo (2005) observed that approximately 70% of F_d comes from the mobilization of tip resistance ($F_{d,p}$) and that the unit tip resistance (q_p) can be calculated by Equation 3:

$$q_p = \frac{F_{d,p}}{a_p} = \frac{0.7 \times F_d}{a_p} \quad (3)$$

The unit tip resistance (q_p) values obtained with the rational interpretation of the DPL test are analogous to the cone resistance values obtained with the CPT, as shown by Bastos (2016) and Almeida (2023).

2.2 The downhole test

A downhole test can be conducted to obtain the S-wave and P-wave velocities in the soil strata by using the theory of elasticity and assuming that the soil is an elastic, homogeneous and isotropic material.

The downhole seismic test measures the time it takes for a wave to propagate from the ground surface to the depth where the seismic transducer is located. The crosshole seismic test measures the time for a wave to propagate from a borehole to two seismic transducers located in two other boreholes at the same depth as the seismic source (ASTM, 2007). The crosshole seismic test is used as a reference for V_s values considering the horizontal path of the seismic wave.

The main devices used to perform a downhole seismic test are as follows (Butcher et al., 2005):

- Seismic transducer: a geophone or an accelerometer used to indicate the moment the seismic wave reaches the probe. Only accelerometers can be used in dynamic tests because geophones cannot withstand hammer impacts without being damaged;
- Seismic source: a device that generates the seismic wave; a shear beam and a hammer;
- Trigger: a system capable of identifying the time when the hammer hits the seismic source. It can be an electric, or an acceleration, force or impact transducer attached to the seismic font;
- The data acquisition system includes measurement devices, sensors, a computer, and data acquisition software to trigger, capture the traces and generate the data files.

3. Materials and methods

3.1 Research sites

Site investigation campaigns were conducted at two well-documented research sites: USP – São Carlos and Unesp – Bauru. These sites were selected because crosshole test data were available to use as a reference.

3.1.1 The USP Site

The USP (University of São Paulo) research site is located in the city of São Carlos, São Paulo State, Brazil. The city of São Carlos lies on rocks from the São Bento Group, which consists of sandstones from the Botucatu Formation and basic migmatites from the Serra Geral Formation (De Mio, 2005; Machado, 1998). The site profile has a sandstone residual soil layer covered by an unsaturated lateritic clayey sand layer (colluvial soil). A layer (0.2 m to 0.5 m) of pebbles separates the upper layer from the residual soil. The groundwater level varies seasonally between 9 m and 12 m below the ground surface. Both layers are classified as clayey sand (SC) according to the Unified Soil Classification System (USCS). Several site characterization campaigns with SPT, CPT, DMT, downhole, crosshole and laboratory tests were conducted at the site (Machado, 1998; Rocha, 2018; Rocha et al., 2021).

3.1.2 The Unesp Site

The Unesp (São Paulo State University) research site is in the city of Bauru, São Paulo state, Brazil. The city of Bauru is located in Paraná Sedimentary Basin. Sedimentary rocks from the Bauru Group (Marília and Adamantina Formations) cover the volcanic rocks (basalts) of the Serra Geral Formation that crop out toward the Tietê River valley (De Mio, 2005). The site profile has red clayey fine to medium sand with high porosity, and the relative density increases with depth. The upper layer (up to a depth of approximately 13.0 m) is a colluvial Neo-Cenozoic deposit with lateritic behavior (LA') over residual sandstone soil. The groundwater level was not found up to 30 m depth. These soils are classified as SM-SC by the Unified Soil Classification System (USCS) and have undergone pedogenic and morphogenetic processes typical of tropical zones. Several in situ (CPT, SPT, DMT, downhole and crosshole) and laboratory tests were conducted to characterize the site and determine the physical and mechanical properties of the soils (Giacheti et al., 2019; Rocha & Giacheti, 2018).

3.2 Crosshole tests

Two crosshole tests (CH1 and CH2) were previously performed up to approximately 9 m depth at the USP site (Giacheti et al., 2003; Giacheti, 1991), and one crosshole test (CH1) was performed up to approximately 14 m depth at the Unesp site (Giacheti, 1991) following the ASTM D4428-07 (ASTM, 2007). Three boreholes 150 mm in diameter and spaced 3 m apart were installed for each test. The two receiver boreholes were cased with PVC pipes 75 mm in diameter, and the source borehole with PVC pipes 85 mm in diameter. A grout mix that closely matched the soil density was used. The data were taken at every 1 m depth interval for test CH1, every 0.5 m depth interval for test CH2 at the

USP site, and every 0.5 m depth interval at the Unesp site. The test procedure consisted of generating reverse polarity shear waves, first by impacting one end of a hammer source and then by impacting the other end. Volts *versus* time traces, corresponding to each impact, were registered on an oscilloscope and recorded in a notebook for subsequent processing and analysis.

3.3 DPL

3.3.1 Equipment

The DPL model DPL-T_SM-v1.2.0 was used, and this device was made by SOLOMAP following the EN ISO 22476-2:2005 (ISO, 2011) specifications (Figure 1). The hammer is raised and released manually, and it has a wood cushion between the hammer and the anvil.

3.3.2 Seismic DPL probe

A conical probe with a slot to insert the sensor was manufactured and named the seismic DPL probe (Figure 2). A seismic transducer was incorporated into this probe to determine the shear-wave velocity.



Figure 1. The DPL equipment used in this study.

The seismic transducer was a high-sensitivity ICP accelerometer from PCB Piezotronics, Model 333B32. This device has a 100 mV/g sensitivity, ± 50 g measurement range and a frequency range from 0.5 Hz to 3000.0 Hz (PCB PIEZOTRONICS, 2002). The cable is also from PCB Piezotronics, the 003C model.

The seismic source has a shear beam made with a piece of wood reinforced with steel and has steel blades on its base to prevent it from sliding during blows applied by a small sledgehammer.

Two different triggers were tested to start recording the seismic traces by the data acquisition system immediately after the sledgehammer hit the seismic source: a) an impact sensor from Murata Electronics, model PKS1-4A10, with a 40 mV/g sensitivity attached to the seismic source, and b) an electrical circuit that closes and the electrical signal triggers the data acquisition system. The electrical trigger is simple and reliable to use, as discussed by Campanella & Stewart (1992), and it was selected for use.

The accelerometer and the trigger traces (when the impact sensor was used) were captured using a signal amplifier model MX410 from HBM. The amplifier has four data acquisition channels, an individual sample rate of up to 100000 samples/second, and a 40 kHz bandwidth and is compatible with ICP sensors (HBM, 2020). HBM MX410 can be grounded to prevent signal disturbances due to electromagnetic interference (HBM, 2020).

CatmanEasy AP software version 3.5 was used with the HBM MX410 to visualize and interpret the data because it has a data acquisition module and a data visualization module for the data collected from the sensors connected to the signal amplifier.

3.3.3 The S-DPL test procedure

The DPL test is conducted as described by EN ISO 22476-2:2005 (ISO, 2011) to the depth at which the downhole seismic test is to be conducted. The data acquisition software module is turned on after the system is appropriately grounded. The sledgehammer is then struck on the right side of the seismic source. After three blows to one side of the seismic source, another three blows are delivered to the other side of the seismic source. The arrival time of the seismic wave is the average time interval determined for the two directions. When the shear beam is struck in opposite directions, it produces shear waves of opposite polarity (Butcher et al., 2005) making it easier to identify the time of arrival of the wave. The traces of the two polarized waves are shown in Figure 3.

The S-DPL test is resumed until the next depth at which the V_s will be determined, after completing the acquisition of the time-of-arrival data the test is resumed and so on to the desired depth. The S-DPL test arrangement is shown in Figure 4, all the tests were conducted up to 8 m depth, with seismic waves generated every 0.5 m depth interval.

The shear wave velocity is determined by the pseudo interval method (Butcher et al., 2005) because the seismic DPL probe has only one sensor. The seismic test data were exported, and an anti-aliasing filter was used during data acquisition, as reported by Butcher et al. (2005). The data were analyzed by using commercial SPAS software to analyze the seismic traces and to calculate the shear wave velocity using the cross-correlation method (Butcher et al., 2005). All the waves in each test were initially adjusted to a total length of 400 ms. Waves with poor quality recordings or with any identified problem in a visual assessment were excluded. This exclusion process also ensured that the waves were correctly polarized for the strokes on opposite sides of the source. A Butterworth filter was then applied. The bandpass filter cutoffs were set at 40 Hz to 400 Hz, order 3, for the test conducted at the USP

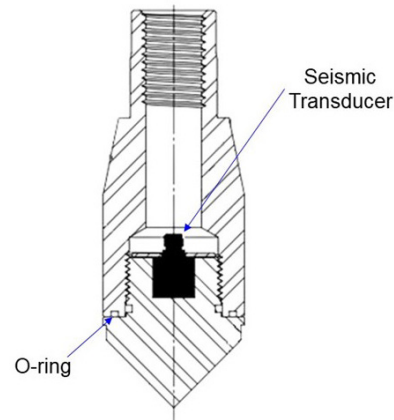


Figure 2. Schematic representation of the seismic DPL probe.

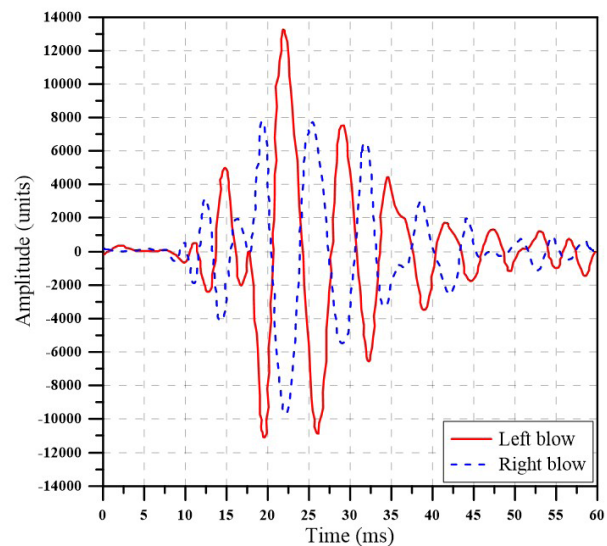


Figure 3. Traces of two polarized waves generated during one seismic test at the same depth at the Unesp site.

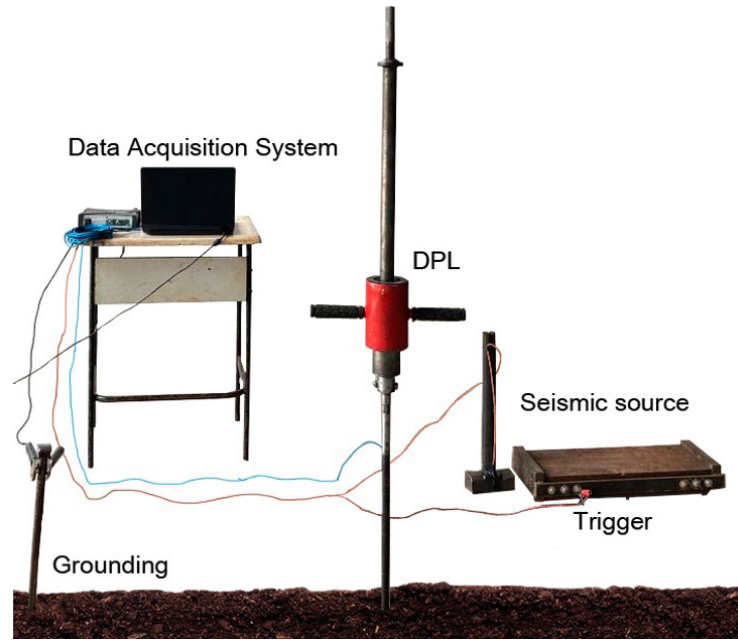


Figure 4. The S-DPL test arrangement.

site. The recordings of the tests conducted at the Unesp site are noisier, so in this case, a bandpass filter of 50 Hz to 300 Hz, order 4, was used. A typical V_s profile obtained with the SPAS software as previously described is shown in Figure 5.

4. Test data and analysis

4.1 The USP site

Three S-DPL tests were conducted at the USP site in July 2023, near one of the two crosshole tests conducted by Giacheti (1991) and by Giacheti et al. (2003). The unit tip resistance (q_p) profiles obtained with the DPL test using rational interpretation and the shear wave velocity (V_s) profiles obtained with the S-DPL test are presented together with the two crosshole V_s profiles in Figure 6.

The q_p profiles are quite repeatable, and the V_s profiles determined with the S-DPL are in accordance with the V_s profiles determined with crosshole tests by Giacheti (1991) and Giacheti et al. (2003). The shear wave velocities have an average relative error difference of 9.8% when compared with Giacheti (1991) data and 11.9% when compared with Giacheti et al. (2003) data. The relative error was calculated from Equation 4 (ISO, 1994).

$$\delta_y = \left(\frac{\Delta y}{y} \right) \cdot 100 = \left(\frac{y_i - y}{y} \right) \cdot 100 \quad (4)$$

where δ_y is the relative error; Δy is the absolute error; y is the true value of V_s (the assumed reference values from the crosshole test); and y_i is the measured value of V_s from the S-DPL.

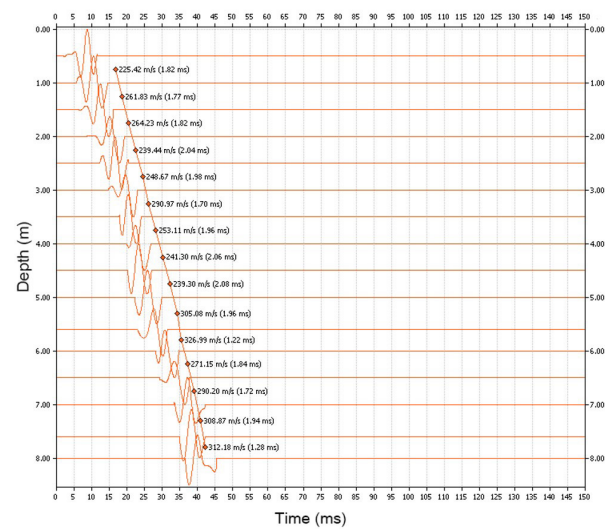


Figure 5. Typical seismic traces after applying the filters using the SPAS software for one downhole test conducted at the USP site to determine the V_s profile. The V_s values and the time intervals for each depth are also presented.

4.2 The Unesp site

Three S-DPL tests were conducted at the Unesp site, in July and in September 2023, near the crosshole test conducted by Giacheti et al. (2003). The unit tip resistance (q_p) profiles obtained with the DPL test using rational interpretation and the V_s profiles obtained with the S-DPL test are presented together with the crosshole V_s profile in Figure 7.

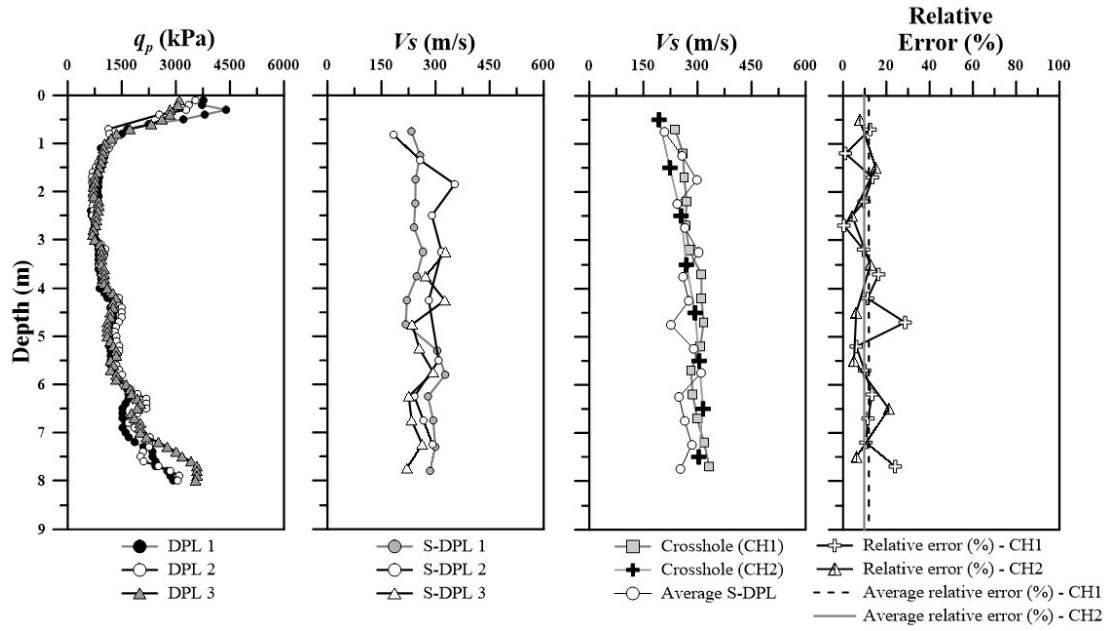


Figure 6. q_p and V_s profiles from the S-DPL tests together with crosshole seismic data collected at the USP site.

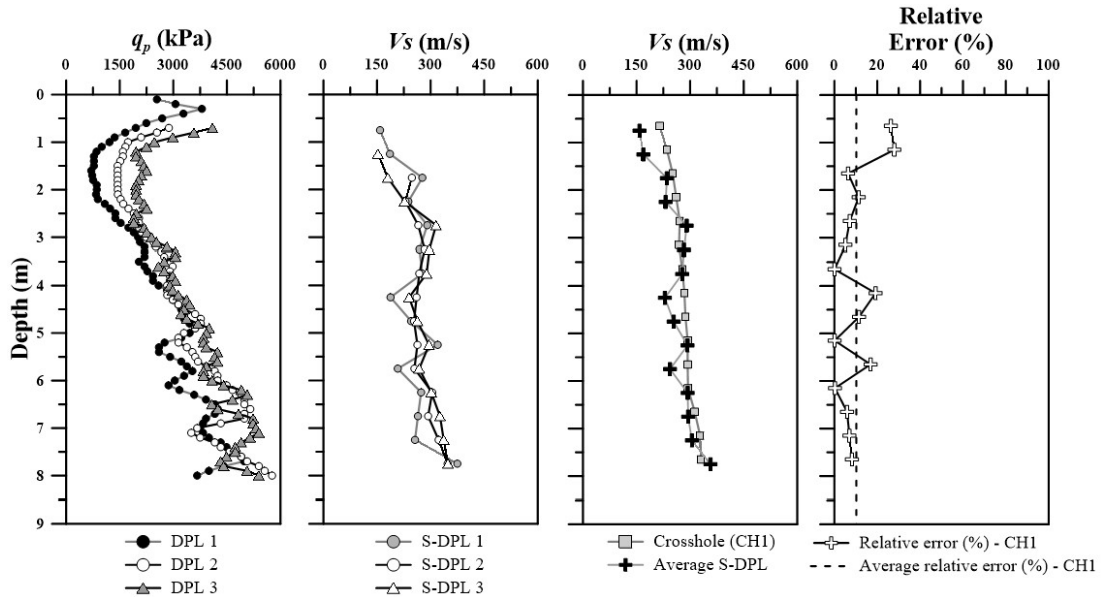


Figure 7. q_p and V_s profiles from the S-DPL tests together with crosshole seismic data collected at the Unesp site.

The q_p profiles are more variable than those observed at the USP site, and the V_s profiles determined with the S-DPL are in accordance with the V_s profile determined with the crosshole test by Giacheti et al. (2003), with an average relative difference of 9.1%, similar to what was found at the USP site.

5. Conclusions

A system to conduct the seismic DPL test and a method to interpret the seismic test data are presented and discussed. The V_s profiles determined with the seismic DPL have an average relative difference equal to 9.8% for one crosshole

test and 11.9% for the other crosshole test at the USP site. An average relative difference of 9.1% was found for one crosshole test conducted at the Unesp site. These findings indicate that the equipment, procedure, and data interpretation of the seismic downhole test conducted together with the DPL test have the potential to be used as preliminary tests in the site characterization of soils.

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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

Caio Henrique de Castro Almeida: conceptualization, data curation, formal analysis, writing – original draft, writing – review & editing. Heraldito Luiz Giacheti: conceptualization, methodology, supervision, funding acquisition, writing – original draft, writing – review & editing. Breno Padovezi Rocha: conceptualization, data curation, formal analysis, methodology, writing – original draft, writing – review & editing. Edmundo Rogério Esquivel: supervision, 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.

List of symbols and abbreviations

e	Penetrometer advance per blow
q_c	Cone tip resistance
q_p	Unit tip resistance
y	True value
y_i	Measured value
CPT	Cone Penetration Tests
DMT	Dilatometer Tests
DPL	Dynamic Probing Light
E_{probe}	Energy delivered to the sampler or the probe
F_d	Average dynamic force
$F_{d,l}$	Lateral dynamic force
F_d	Dynamic tip force
N_{10}	Number of blows to penetrate 0.1 m

N_{20}	Number of blows to penetrate 0.2 m
N_{SPT}	SPT N value
SPT	Standard Penetration Tests
S-DPL	Seismic DPL
SC	Clayey sand
SM	Silty sand
USCS	Unified System Classification Soils
V_s	Shear wave velocity
τ	Unit friction resistance
δy	Relative error
Δy	Absolute error

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CASE STUDIES

Soils and Rocks
v. 47, n. 4

A simple method for assessing the probability of liquefaction of tailings dams

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Case Study

Keywords

Tailings dam
Liquefaction
Probabilistic analysis
Risk assessment

Abstract

In recent years, two tragic accidents in Brazilian tailings dams resulted in a lack of trust in the performance of these structures. This paper aims at proposing a new procedure for estimating the liquefaction risk of existing tailings dams, with the use of well-known probabilistic methods as a valuable tool in the decision-making process for triggering an Emergency Plan. The combination of a rigorous limit equilibrium stability method (Spencer, 1967) and the FOSM probabilistic procedure (Christian et al., 1992) has proved to adequately predict the imminent liquefaction risk of both Fundão and Feijão tailings dams, which recently collapsed in Mariana (2015) and in Brumadinho (2019), both in Minas Gerais State, Brazil. Considering the triggering mechanisms known to have occurred, the estimated probabilities of liquefaction failure at the time of the accidents were 36% for Fundão Dam, and 47% for Feijão Dam. The results show that the risk associated to the collapse of both structures was well above the acceptable level suggested by current standards.

1. Introduction

In recent years, significant accidents have occurred in tailings dams in Brazil, raising doubts about the safety of these structures and lack of trust in their performance (Morgenstern, 2018). The catastrophic failure of Fundão tailings dam, which occurred in Mariana, in December 2015, was one of Brazil's major environmental accidents, leaving a serious scar on the mining industry worldwide. A few years later, the world was again shocked by another tragedy, this time captured on the security video cameras showing the abrupt failure of the tailings dam at the Feijão mine, in Brumadinho, in January 2019, and the subsequent massive devastation caused by the uncontrolled flow of the saturated tailings. These two accidents were surely due to static liquefaction of the dams' tailings.

The accidents raised questions about the methodologies for analyzing dam stability in Brazil, as well as the acceptability of the risk by society.

Concepts of risk analysis can be applied to dams, but they are not yet widely used in Brazil. Espósito & Palmier (2013) applied two risk analysis methods to Brazilian dams and concluded that this type of analysis allows a better comprehension of dam behavior. At the same time, the use of probabilistic procedures has proved to be a valuable tool in assessing the safety of these structures. Regulatory guides usually specify a minimum acceptable factor of safety (*FS*)

of 1.50 as the design criteria of dam structures. CDA (2013) indicates this value as the minimum adequate in static analyses of downstream slopes under steady state (long-term) conditions, as seen in Table 1. Australian National Committee on Large Dams (ANCOLD, 2022) presents, beyond the safety factor related to each case, the strength parameters that should be adopted, as displayed in Table 2.

In recent years, however, the use of probabilistic analysis of dam safety has been growing rapidly, as emphasized by CDA (2013). In Brazil, probabilistic analysis has been still limited to academic studies and ignored as a technical tool in the decision-making process of assessing the safety of existing structures.

The aim of this work is to contribute to a better understanding of the risk of failure of existing tailings dams. In this context, this paper presents a simple procedure for estimating the probability of liquefaction failure at the time of the accident, indicating that such analysis could have been of great help in anticipating these tragic occurrences.

2. Liquefaction probabilistic analysis

Liquefaction may classically occur in saturated contractive loose sand deposits. Olson & Stark (2003a) suggests that a liquefaction analysis consists of three main tasks: (1) an analysis of flow failure susceptibility, (2) a triggering analysis, and (3) a post-triggering failure stability analysis.

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Table 1. Minimum Factor of Safety in static analyses recommended by CDA (2013).

Loading Conditions	Minimum FS	Slope
End of construction, before the reservoir filling	1.30	Upstream, downstream
Long-term steady state	1.50	Downstream
Full or partial rapid drawdown	1.20	Upstream

Table 2. Minimum safety factors recommended by ANCOLD (2022).

Loading condition	Minimum safety factor for tailings dams	Strength parameters
Long term – drained	1.5	Drained strength
Short term – undrained (potential contaminant loss)	1.5	Undrained consolidated strength
Short term – undrained (without potential contaminant loss)	1.3	Undrained consolidated strength
Post seismic	1.0-1.2 ^a	Post Seismic Strength ^b

Legend: ^aRelated to the reliability on the residual parameters selection. $SF = 1.0$ can be applied in cases where conservative parameters are adopted. ^bResidual strength after several loading cycles or liquefied strength in the case of liquefiable materials.

The first step of a liquefaction analysis of tailings deposits is to investigate if the material is loose and contractive (i.e., susceptible to flow failure). Simple methodologies for assessing the soil's susceptibility to liquefaction have been described by Olson (2001) and Robertson (2010), with applications to practical cases.

As dam liquefaction usually occurs under undrained conditions, the key factor in computing the probability of liquefaction failure is to estimate the probability of undrained failure. Martin & McRoberts (1999) have proposed that the results of undrained analysis must be understood in the context of the potential triggers of undrained shear. These authors state that the stability margin against the shear failure of tailings dams is better expressed in terms of both the probability of undrained failure and a safety factor from a rigorous limit equilibrium method.

Martin & McRoberts (1999) suggest there are several potential triggers for undrained failure, including erosion, high impoundment rate, fast steepening of the downstream slope, seepage breakout at the dam's face, foundation and/or embankment movement, sudden reduction of mean effective stress, severe storm runoff, blockage and failure of spillways, earthquake, and seismic deformation.

Olson & Stark (2003a) stated that the strength parameter mostly used in stability analyses is the undrained stress ratio s_u / σ'_v , which is the ratio of undrained strength (s_u) to initial vertical effective stress. This stress ratio parameter offers a rational way to account for effective stress variations, state parameter (or void ratio), and grain size characteristics (Olson & Stark, 2003b).

This parameter may be defined in undrained triaxial tests in the laboratory or estimated from field piezocone (CPTu) tests results (Olson & Stark, 2002, 2003a; Sadrekarimi, 2014; Robertson, 2010).

After defining the parameters needed for the analysis, the probability of undrained failure can be obtained in several ways. In this paper, Spencer's limit equilibrium method and

the FOSM probabilistic method (First Order Second Moment, described by Christian et al., 1992) were adopted, for being practical and suitable in previous applications (Dell'Avanzi & Sayão, 1998; Sayão et al., 2012; Vecchi & Sayão, 2019; Yokozawa et al., 2019).

The Spencer limit equilibrium method (Spencer, 1967) was chosen for this analysis because it is a widely used rigorous method. In this scenario, the rupture surface is free and can be either circular or non-circular, satisfying the forces and moments equations of equilibrium. The main hypothesis adopted in this method is the parallel inter-slice forces, which means that the forces between the slices can be replaced by a resultant Q_i inclined δ to the horizontal (Spencer, 1967).

Christian et al. (1992) and Sayão et al. (2012) indicate that the partial derivatives of the safety factor function in the FOSM method may be obtained by a small variation of each random variable. They suggested that each variable should be distinctly subjected to an increase of 10% on its average value while keeping all other variables fixed. In this way, the resulting variation of the safety factor is adequately assessed. In order to estimate this probability of failure by the FOSM method, the average and standard deviation values of each random variable must be previously obtained, from a proper number of reliable field or lab test results. This requirement may impose a serious difficulty to use probabilistic methods in smaller, low-budget projects.

In the tailings dam's stability analyses herein presented, the ratio s_u / σ'_v was considered the only random variable, due to its notorious higher relevance.

If the tailings are susceptible to liquefaction, the probability of liquefaction may be given by Equation 1:

$$P_L = P_{FOSM} * P_t \quad (1)$$

where P_L is the probability of liquefaction, P_{FOSM} is the probability obtained by the FOSM method, and P_t is the

Table 3. Classification of consequences of dam failures (CDA, 2013).

Dam class	Population at risk (note 1)	Loss of life (note 2)	Environmental and cultural value losses	Infrastructure and economic losses
Low	None	0	Minimal short-term loss; no long-term loss.	Low economic losses; area contains limited infrastructure or services
Significant	Temporary only	Unspecified	No significant loss or deterioration of fish or wildlife habitat; loss of marginal habitat only; restoration or compensation in kind highly possible.	Losses to recreation facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	Significant loss or deterioration of important fish or wildlife habitat; restoration or compensation in kind highly possible.	High economic losses affecting infrastructure, public transportation, and commercial facilities
Very high	Permanent	100 or fewer	Significant loss or deterioration of critical fish or wildlife habitat; restoration or compensation in kind possible but impractical.	Very high economic losses affecting important infrastructure or services (e.g., highways, industrial facilities, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of critical fish or wildlife habitat; restoration or compensation in kind impossible.	Extreme losses affecting critical infrastructure or services (e.g., highways, industrial facilities, storage facilities for dangerous substances)

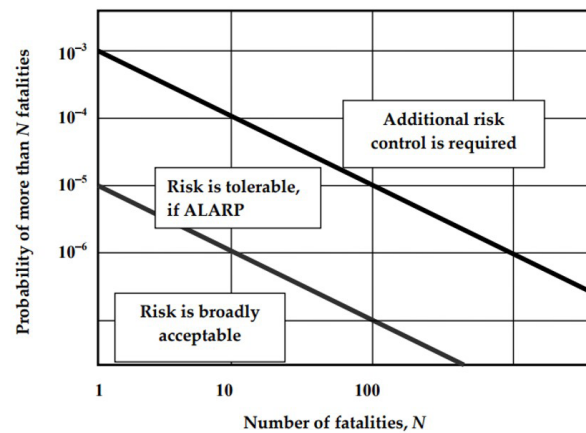
Legend: Note 1. Definitions for the population at risk: None – There is no identifiable population at risk, so there is no possibility of loss of life other than through unexpected misfortune; Temporary – People are only temporarily present in dam-breach inundation zone (e.g., in cottages for seasonal use, passing through transportation routes, or participating in recreational activities); Permanent – The population at risk is usually located in the dam-breach inundation zone (e.g., as permanent residents); three consequence risk classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if appropriate analysis is carried out). Note 2. Implications for loss of life unspecified – the appropriate level of safety required at a dam where people are momentarily at risk depends on many conditions: the number of people, the exposure time, the nature of their activity, etc. A higher class may be appropriate, depending on the conditions. The design flood requirement, for example, could not be higher if the temporary population is likely to be absent during the flood season.

probability of occurrence of a trigger, as indicated by D'Hyppolito (2023).

After careful analysis of three Brazilian tailings dams, D'Hyppolito (2023) concludes that the probability of trigger for liquefaction in Brazilian tailings dams may be taken 1.0, due to frequent problems, such as insufficient geotechnical investigation, fast tailings disposal, malfunction of drainage systems (Ávila et al., 2021) and divergences with design parameters.

CDA (2013) suggests that the probability of liquefaction be obtained from Equation 1 and compared to reference values, which depend on the consequences of a potential dam failure (Table 3). This CDA classification table regulates the necessary safety criteria in terms of design parameters and proposes the risk assessment for dam safety considering the approach shown in Figure 1, with life safety risk guidelines consistent with values used in other hazardous industries and with the principle that risks should be made as low as reasonably practicable (ALARP). In Figure 1, the probability of more than N fatalities may be considered equal to the probability of liquefaction obtained from Equation 1.

Recently, the Australian National Committee on Large Dams released a review of the guidelines on risk assessment (ANCOLD, 2022), presenting a chart to evaluate the societal risks for existing dams, as shown in the Figure 2. It is interesting to note the similarities between CDA and ANCOLD charts.

**Figure 1.** Risk assessment for dams, as a function of number of fatalities (CDA, 2013).

2.1 Fundão dam

On the afternoon of November 5th, 2015, Fundão Dam started to collapse, in Mariana, an important iron mining center in Minas Gerais State, Brazil. This dam was built by the upstream method and was 110 m high, by the time of the accident. This event has been reported in detail by Morgenstern et al. (2016).

Considered as one of the Brazilian's worst environmental disasters, the failure of Fundão Dam resulted in massive

damages and caused 19 deaths. Until January 2020, a total of R\$ 8.2 billion (or US\$ 1.8 billion) had been already disbursed in repairs and compensations, as reported by Renova (2022). The number of deaths would have been much higher if the accident had occurred at night, as most people would have been sleeping in houses devastated by the massive flow of saturated tailings released by the dam's failure.

The cross-section of Fundão Dam (Figure 3), reported by Morgenstern et al. (2016), has been considered in the analyses reported in this paper. Two types of slurry tailings may be noted, both delivered in separate pipelines to the Fundão impoundment. Sandy tailings, or simply Sands, are a mixture of sand-sized and finer silt particles and are relatively free-draining, but susceptible to liquefaction when loose and saturated. The slimes, on the other hand, are much finer and clay-like in nature - soft and compressible tailings, with low permeability.

In Figure 3, a mixture of sand and slimes can be noted, but the exact proportion of this material is not clear. A unique parameter, chosen from field piezocone (CPTu) test results, was adopted to represent the tailings.

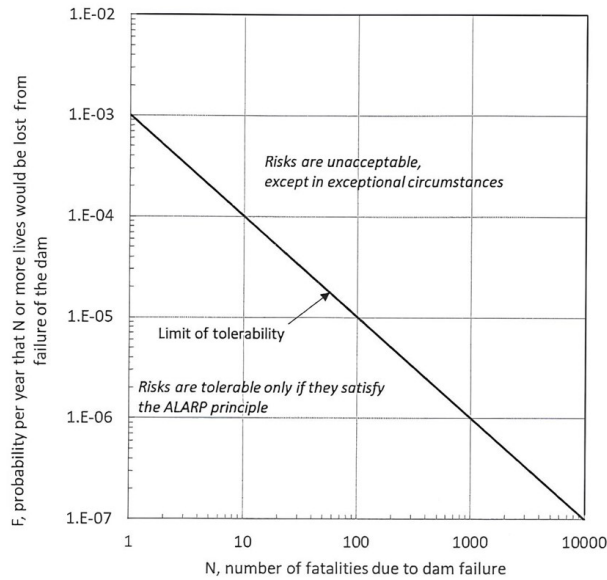


Figure 2. Societal risk assessment for existing dams (ANCOLD, 2022).

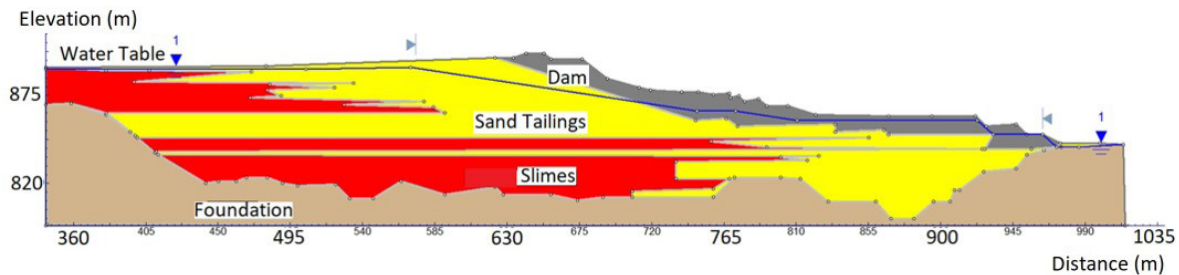


Figure 3. Cross section of Fundão dam from Morgenstern et al. (2016).

Regarding previous studies related to similar tailings dams (Sayão et al., 2012; Braga, 2019), it was possible to observe that the variation of the undrained strength ratio is responsible for more than 90% of the failure probability. Additionally, Duncan (2000) indicates this parameter presents the most expressive variability of all soil's strength parameters, with a coefficient of variation reaching about 40%. Consequently, it may be suggested that adopting the undrained strength ratio as the only random variable is justified, considering the simplification of the proposed procedure, without compromising the accuracy of the result.

Regarding the in-situ tests performed on the dam, over 50 different expeditions were reported containing a wide range of tests, including field piezocone test (CPTu), standard penetration test (SPT), and permeability test. Morgenstern et al. (2016) report that at that time there was no CPTu or SPT made from the dam's crest until the bedrock foundation. After the failure, additional tests were performed in the Germano dam, similar to and close to Fundão dam.

The analysis of CPTu data showed that about 70% to 80% of the sandy tailings within 75 m of the dam crest were noted to be contractive, based on the liquefaction criterion proposed by Robertson (2010). At greater distances from 180 m, 95% of the sandy tailings are contractive. This indicates that the hydraulically discharged Fundão sandy tailings satisfied the contractiveness requirement for liquefaction flow sliding.

Additionally, a back analysis was carried out of a failure incident that occurred in September 2014. At that time, there was a local failure of the dam. The strength parameters resulting from this back analysis may be used to compare with the parameters from CPTu tests. From this analysis, Morgenstern et al. (2016) obtained a value of 0.22 for $(s_u / \sigma'_v)_{peak}$.

The average results for each undrained strength ratio in Fundão tailings are shown in Table 4, obtained from the results reported by Morgenstern et al. (2016).

The present work adopted as average value the result obtained in the back analysis from the 2014 incident. Since there was a located failure in this incident, it can be considered that the undrained strength was mobilized. Furthermore, this value is similar to the CPTu results, as shown in Table 4.

The standard deviation from the results of CPTu tests may be regarded as evidence of the variability of the material at depth. It is important to consider the fact that various problems were reported in the tests performed in Fundão, which led to doubts about the reliability of the results and their representativeness related to all the material contained in the dam, as presented in Morgenstern et al. (2016). Nevertheless, the analysis of Morgenstern et al. (2016) results shows that it can be obtained a standard deviation of 0.03 considering all the CPTu results available in the area. Briefly, the adopted values are: $(s_u / \sigma'_v)_{peak} = 0.22$ e standard deviation = 0.03.

2.2 Feijão dam

At 12:28 pm local time on January 25th, 2019, the tailings dam B1 at Córrego do Feijão Iron Ore Mine (Vale, Brazil), in Brumadinho, Brazil, suffered a sudden failure, releasing 12.7 million m³ of tailings, and resulting in a catastrophic mudflow that traveled rapidly downstream. The mudflow destroyed several structures (refectory, offices, homes, and a bridge). This dam failure is unique for having high-quality video images that provide insight into the failure mechanism (Robertson et al., 2019).

At the time of the failure, Feijão dam was 86 m high, with a 720 m long crest. More details of the dam and the causes of its failure were reported by Robertson et al. (2019).

The main dam tragedy ever seen in Brazil has resulted in 270 deaths, severe damage to the environment and about US\$ 8 billion already spent by Vale for social and environmental repair. The cross-section of Feijão dam is presented in Figure 4. This was the highest and the most critical section, as indicated by Robertson et al. (2019) and Polícia Federal (Minas Gerais, 2019).

Like the Fundão case, there were two types of tailings, coarse and fine tailings, and the exact location/proportion of these materials is not clear. This paper adopted a unique parameter to the tailings in the analysis, and the parameter was chosen based on the CPTu tests, which reflect the real situation in the field. In Brumadinho's case, a larger amount of test data is available, and Robertson et al. (2019) presented a statistical analysis of the CPTu results, as shown in Figure 5. Considering the adoption of a single representative parameter, the option made was to consider the average value equivalent to the weighted average according to the frequency of each type of tailings, and the standard deviation adopted was obtained for the sand tailings due to the greater variability observed in this case. In this scenario, these are the selected parameters: $(s_u / \sigma'_v)_{peak} = 0.22$ e standard deviation = 0.12. Additionally, the value adopted is consistent with another analysis described in Braga (2019), Polícia Federal (Minas Gerais, 2019) and TUV SUD (2018).

Table 4. Average values of undrained strength ratio (D'Hyppolito, 2023).

Type	$\left(\frac{s_u}{\sigma'_v} \right)_{peak}$	Standard Deviation
Value defined by Morgenstern et al. (2016) based on CPTu tests	0.25	0.03
Back analysis from 2014 incident	0.22	0.03
CPTu Test – Estimated by Olson & Stark (2003a)	0.25	0.03
CPTu Test - Estimated by Sadrekarimi (2014) - Triaxial Compression	0.25	0.03
CPTu Test - Estimated by Sadrekarimi (2014) - Simple Shear	0.21	0.02

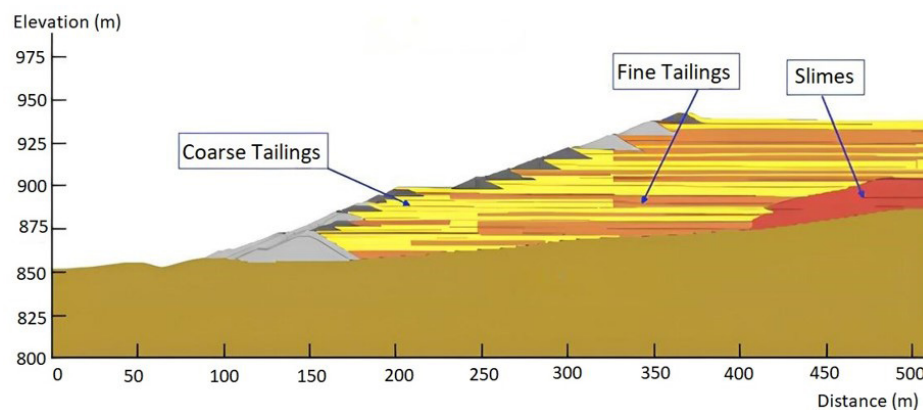


Figure 4. Critical section from Feijão dam (Robertson et al., 2019).

3. Results

3.1 Fundão dam failure

As presented, the average elected for $(s_u / \sigma'_v)_{peak}$ was 0.22, with 0.05 for standard deviation. The other parameters were adopted equal to Morgenstern et al. (2016) because they have a small impact on the results. The stability

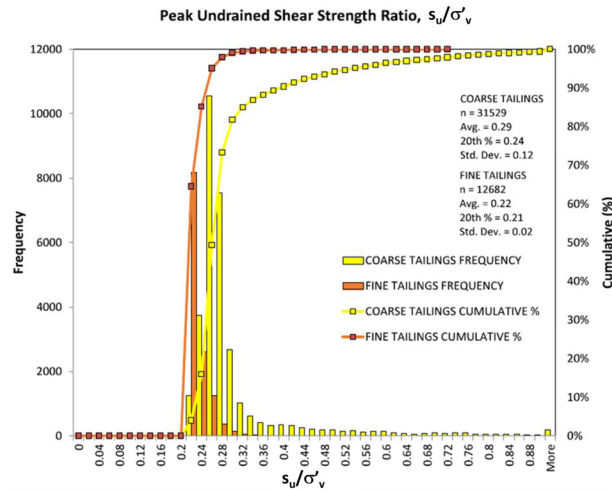


Figure 5. Statistical analysis of the peak undrained shear strength ratio (Robertson et al., 2019).

analysis performed for the average parameters using the Spencer method is presented in Figure 6. Table 5 exhibits the parameters adopted for the analysis.

The combination of the FOSM probabilistic method and the Spencer limit equilibrium method, considering that the random variable has a normal (Gaussian) distribution, resulting in 0.37 for the Reliability Index β (Christian et al., 1992) and the probability of failure $P_{FOSM} = 36\%$.

The calculated probability is quite high and inconsistent with any kind of engineering construction. Based on the results, the dam was not stable and could be subjected to an undrained collapse because of a trigger.

Regarding the occurrence of triggers, according to Morgenstern et al. (2016), the dam was showing (i) serious issues related to drainage; (ii) disobedience to the width of the beach designed, leading to a fine waste settlement in unforeseen places; (iii) small displacements through the instrumentation; and (iv) quick loading to heighten the structure. Those problems are known for their possibility to trigger the tailings liquefaction, as already commented.

Considering that the tailings were subjected to liquefaction and the triggers occurred, the Equation 1 shows that: $P_{liquefaction} = 36\%$. The failure consequences can be classified as Extreme, according to Table 2 (CDA 2013), due to the resultant environmental impact, which was expressive and unreturnable.

The risk associated to the Fundão dam was way beyond the acceptable. In this scenario, measures should have been taken to mitigate this risk through probability reduction and consequence control.

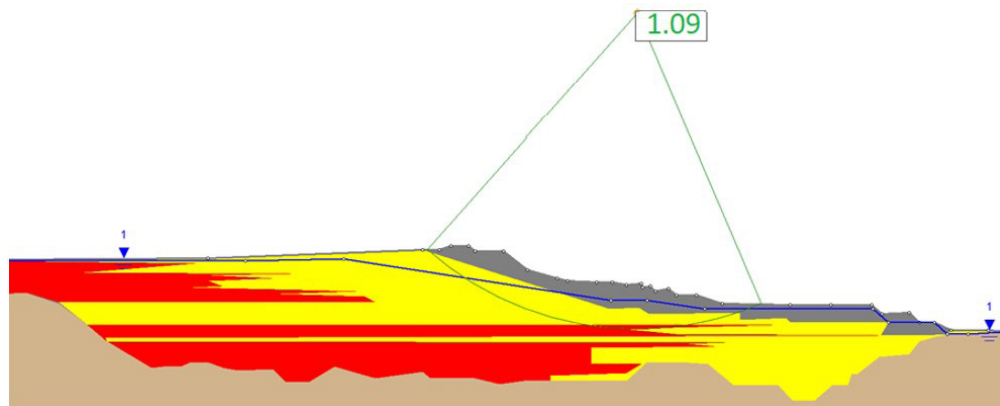


Figure 6. Stability analysis of Fundão Dam.

Table 5. Parameters adopted in Fundão Dam analysis (D'Hyppolito, 2023).

Material name	Color	Unit Weight (kN/m ³)	Cohesion (kPa)	ϕ (deg)	Vertical stress ratio (average)
Dam – raisings	Gray	22	5	35	-
Coarse tailings	Yellow	22	-	-	0.22
Slimes	Red	22	-	-	0.22
Foundation	Brown	22	40	32	-

3.2 Feijão dam failure

The stability analysis performed by the Spencer method with average parameters is presented in Figure 7. Table 6 exhibits the parameters adopted in the analysis. The average selected for $(s_u / \sigma'_v)_{peak}$ was 0.27 and 0.12 for standard deviation. These other parameters were adopted as indicated by Morgenstern et al. (2016) because they have a small impact on the results.

The combination of the FOSM probabilistic method and the Spencer limit equilibrium method, considering that the random variable has a normal (Gaussian) distribution, achieves 0.07 for the reliability index β (Christian et al., 1992) and the failure probability is 47%.

As presented by Robertson et al. (2019), (i) the dam was presenting significant issues related to drainage; (ii) there were layers of fine tailings in areas supposed to have tailings with larger particle size; (iii) the foundation data and information were very limited; and (iv) slow displacements were observed, consistent with a possible creep, through one of the instruments (InSar). In this scenario, the liquefaction triggers were present.

Considering the tailings were subjected to liquefaction and the triggers occurred, the use of Equation 1 results in $P_{liquefaction} = 47\%$. The failure consequences can be classified as Extreme, according to Table 2 (CDA, 2013), due to the associated numbers of deaths.

The risk associated to the Feijão dam was not satisfactory. In this scenario, actions should have been taken to mitigate

this risk by reducing the probability of failure and controlling the consequences.

4. Discussion

Brazilian dams have specific problems demanding special caution, such as drainage systems (Ávila et al., 2021), quick loading, and lack of attention to the design conception elements, like the beach width (Martin et al., 2002). These problems may be seen frequently in tailing dams and open the chances for the occurrence of triggers.

Considering the information presented in this study, the most important element in the liquefaction risk analysis is the undrained probability of failure. The estimated probabilities in the two cases reviewed herein were -36% at the time of the Fundão dam (Mariana) failure, which collapsed in 2016, and 47% at the time of the Feijão dam (Brumadinho) failure, which collapsed in 2019.

These two cases discussed in this paper, Fundão and Feijão dams could be considered to present very high levels of associated consequences. For the Feijão dam, which caused over 200 deaths, the acceptance limit as suggested by CDA (2013) would be 10^{-7} and tolerability of 10^{-5} . In Fundão dam, in which much lower fatality level (19 deaths), the acceptance limit as suggested by CDA (2013) would be 10^{-6} and tolerability of 10^{-4} . The limits of tolerability proposed by ANCOLD (2022) are similar. However, it was not the worst case. As previously mentioned, the number of deaths in the Fundão case could have been

Table 6. Parameters adopted in Feijão dam analysis (D'Hyppolito, 2023).

Material name	Color	Unit Weight (kN/m ³)	Cohesion (kPa)	ϕ (deg)	Vertical stress ratio
Dam raisings	Yellow	19	5	36	-
Dam raisings	Orange	28	5	36	0.22
Tailings	Gray	22	-	-	0.27
Foundation	Green	20	16	30	-

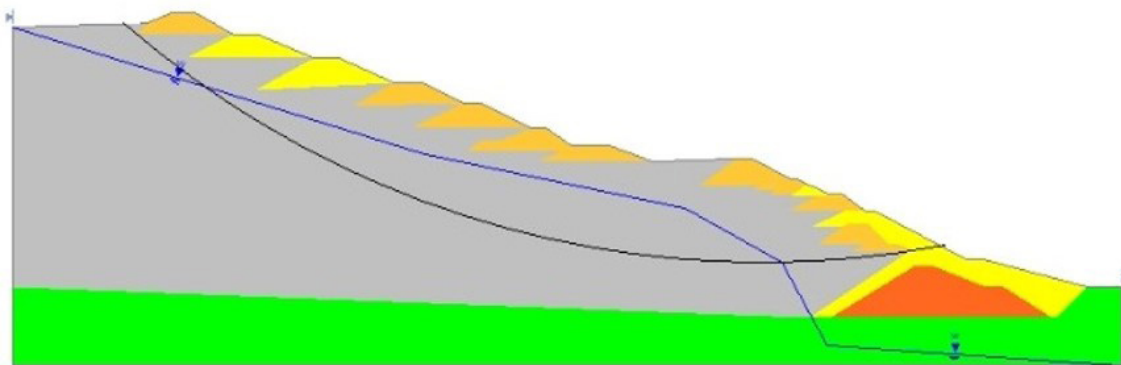


Figure 7. Feijão dam stability analysis with average parameters, obtaining a safety factor of 1.03.

much higher, and similar to the Brumadinho case, had the accident occurred at night. The computed results for liquefaction failure probability were much higher than these reference values and would be unacceptable for any kind of engineering construction.

5. Conclusions

This paper presents a simple procedure for estimating the risk associated to tailings dams, using simple well-known methods. The practical application of the proposed procedure was illustrated by two recent cases of dam failure, suggesting that the risk associated with the collapsed structures was higher than the acceptable level indicated by current standards. The results of the analysis presented in this paper indicate that in the case of the Fundão Dam, the estimated probability of failure was 36%, while in the case of the Feijão Dam, it was 47%. In both scenarios, the probabilities suggest a level of risk deemed intolerable for dam structures.

The probability of failure may be estimated with the procedure suggested in this paper and may be considered in the decision-making procedures, regarding the need for intervention and mitigation of the risk of accident and its consequences by triggering an Emergency Plan.

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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

Lays Cristina de Souza D'Hyppolito: conceptualization, data curation, methodology, visualization, writing - original draft preparation. Alberto Jardim Sayão: formal analysis, funding acquisition, supervision, validation, writing - reviewing and editing. Anna Laura Lopes Nunes: formal analysis, methodology, supervision, validation.

Data availability

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

List of symbols and abbreviations

ALARP	As low as reasonably practicable
ANCOLD	Australian National Committee on Large Dams
CNEN	Brazilian Commission of Nuclear Energy
CDA	Canadian Dam Association
CPTu	Field Piezocone Test
FS	Factor of safety
FOSM	First-Order Second Moment
P_L	Probability of liquefaction
P_{FOSM}	Probability obtained by the FOSM method
P_t	Probability of occurrence of a trigger
PUC	Pontifical Catholic University of Rio de Janeiro
s_u / σ'_v	Undrained stress ratio
$(s_u / \sigma'_v)_{peak}$	Peak undrained stress ratio
β	Reliability index
ϕ	Friction angle

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The relationship between geophysical and geotechnical data: a temporal analysis of an iron ore tailings dam

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Case Study

Keywords

Electrical resistivity
Precipitation
Time analyses
Piezometry

Abstract

Iron ore tailings dams must be constantly monitored for safety purposes. Among the aspects traditionally monitored are geotechnical protocols, such as piezometric levels along the dam. Recently, geophysics, a science that studies, among other things, dynamic processes in the Earth, has contributed to non-invasive monitoring using electro-resistivity methods. Geophysical methods provide a large amount of data in relatively quick samples, but there is little information on the integration of traditional geotechnical and geophysical techniques. Therefore, this work aimed to evaluate the relationship between geophysical data, monitored every 15 days between May 2022 and March 2023, and geotechnical data from an iron ore tailings dam located in the Iron Quadrangle. The geophysical method used was two-dimensional electrical imaging with a Schlumberger array. Analysis of variance and multiple regression allowed us to evaluate the relationship between resistivity values, piezometer level, sampling time, and rainfall. Piezometry data explained the electro-resistivity significantly, while sampling time and rainfall did not significantly explain electro-resistivity. Therefore, the dam's electro-resistivity patterns have not changed significantly over the period, and local rainfall does not imply an immediate response in resistivity. These conclusions can support new directions in decisions about monitoring iron ore tailings dams and may contribute to advances in dam safety in the mining sector.

1. Introduction

The construction of dams has been frequent in the world for centuries, and their failure may cause life loss as well as severe environmental impacts (Mainali et al., 2015). Data on global hazards indicate a high probability of dam-break per year (Paiva et al., 2019). In Brazil, the failures of Fundão and Córrego do Feijão tailing dams are remarkable due to the killed people (more than 250 in these two failures) and disaster extension (Santamarina et al., 2019; Milanez et al., 2021).

Dam monitoring is essential to ensure the dam's safety throughout its lifetime. Instruments and visual inspections monitor these structures (Machado, 2007), such as surface markers, inclinometers, water level indicators (NA), and piezometers. These methods are efficient and quite traditional in the mining industry, but they offer specific information about the amount of dam water.

On the other hand, geophysical methods applications are more frequent now, and their non-destructive nature and cost-effectiveness stand out (Mainali et al., 2015). Geophysics works with physical phenomena, such as Earth's magnetic field, geothermal flow, seismic wave propagation, gravity, electric and electromagnetic fields, telluric currents, and radioactivity. The four main groups of Geophysics methods

are: 1) gravimetric, 2) magnetometric, 3) geoelectric, and 4) seismic (Braga, 2006). Geoelectric methods include electro-resistivity (ERT), induced polarization, and spontaneous potential (SP) (Telford et al., 1990). Except for the SP method, which measures the natural potential in the subsurface, all the others depend on the electrical transmission of the current into the soil (Dentith & Mudge, 2014).

Geophysical methods have been used to study dam conditions (Mainali et al., 2015; Martini et al., 2016; Nikonow et al., 2019; Mollehuara-Canales et al., 2021). In such conditions, changes in natural potential in the subsurface occur due to water movement through the dam and changes in resistivity reflect changes in the electrical properties of the dam materials. Therefore, measuring the natural potential over time is a powerful method for detecting leaks, and time series resistivity measurements provide valuable information about changes in dam conditions over time. However, the time-lapse ERT is relatively uncommon in mining waste monitoring (Dimech et al., 2022).

Among the geophysical methods applied to dam monitoring, ETR and SP stand out. ETR is a method that uses an artificial current, introduced by two electrodes (called A and B) in a terrain, to measure the potential generated in two other electrodes (called M and N) in the vicinity of the

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current flows, thus allowing the real or apparent resistivity in the subsurface to be determined (Lago et al., 2006). SP is a passive geophysical method that measures natural variations in electrical potential due to the spontaneous polarization of the earth, the causes of which are diverse (Telford et al., 1990).

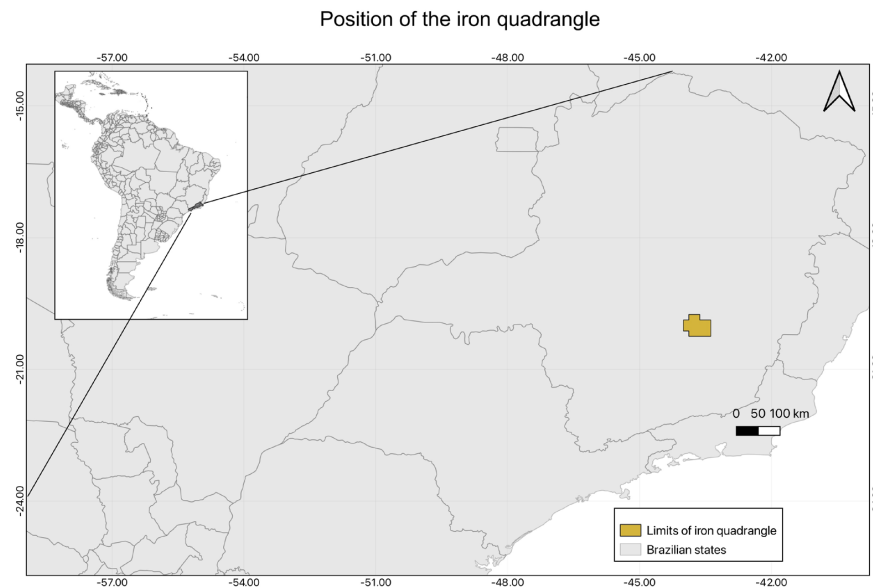
Despite the increasing use of geophysical methods in tailings dams, few studies have presented an integrated analysis between data collected by traditional monitoring methods and geotechnical information on these structures. Besides that, the role of geophysical testing with geotechnical monitoring has been discussed for a long time (Jamiolkowski, 2012). Therefore, we aimed to investigate the relationships between data obtained by geophysical methods and those obtained by traditional dam safety monitoring methods. Thus, we compared the monitoring of an iron mining tailings dam, which applied the ERT method between May 2022 and March 2023, to the monitoring of traditional techniques such as

piezometers, water indicators, and meteorological stations. The ERT method arrangement was Schlumberger, and the data was analyzed using statistical techniques of variance analysis and multiple regression, evaluating resistivity values, piezometer level, sampling time, and precipitation.

2. Materials and methods

2.1 Study area

The dam is in Nova Lima municipality (Brazil), which is the Belo Horizonte metropolitan region (Figure 1a). It is also in the north-central portion of the Iron Quadrangle, which is in the Brazilian Precambrian area with large mineral reserves and great structural complexity (Alkmim & Marshak, 1998). The stratigraphy of the Iron Quadrangle



a)



b)

Figure 1. (a) Study area in Minas Gerais (Brazil) map; (b) dam overview.

encompasses the gneissic complexes of the basement, the volcano-sedimentary sequence of the Rio das Velhas Supergroup, and the metasedimentary sequences of the Minas Supergroup and Itacolomi Group (Dorr, 1969; Machado et al., 1996; Almeida et al., 2005).

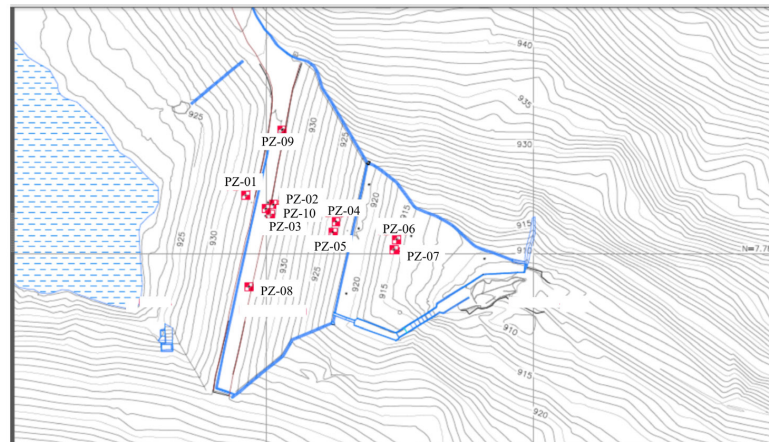
The water dam flows in the Rio das Velhas watershed, which runs through 51 municipalities, and its tributary is called the Taquaras stream. The dam is 24 meters high and 95 meters long. Its crest is 5.25 meters wide, and its intermediate berm is 3.00 meters wide. The downstream slopes have a slope between berms of 1V: 2H and the upstream slope has a shape of 1V: 1.7H (Figure 1b). The dam was built in a single stage and made of fine sandy silt to clayey silt soil. The foundation contains colluvial soil, residual phyllite soil, altered phyllite rock, and phyllite material.

2.2 Geotechnical monitoring

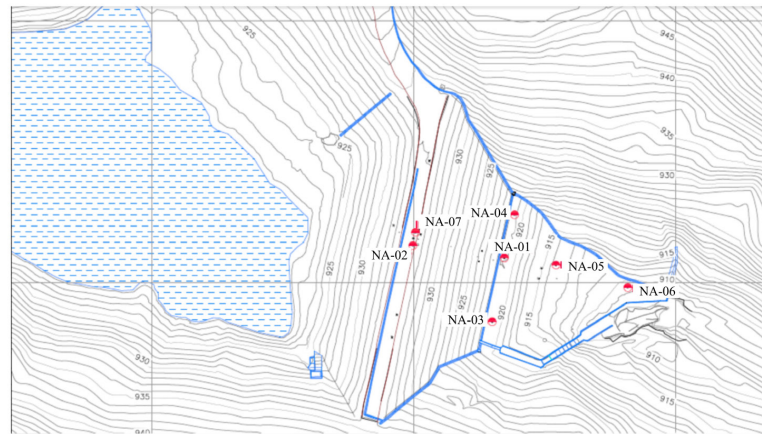
The dam has ten piezometers and seven water level indicators to measure the water and piezometric levels of the structure (Figure 2 and Table 1).

Table 1. Name and type of geotechnical monitoring instrument.

Instrument ID	Instrument type
PZ-01	piezometer
PZ-02	piezometer
PZ-03	piezometer
PZ-04	piezometer
PZ-05	piezometer
PZ-06	piezometer
PZ-07	piezometer
PZ-08	piezometer
PZ-09	piezometer
PZ-10	piezometer
NA-01	water level indicator
NA-02	water level indicator
NA-03	water level indicator
NA-04	water level indicator
NA-05	water level indicator
NA-06	water level indicator
NA-07	water level indicator



a)



b)

Figure 2. The location of ten piezometers (a) and seven water level indicators (b) in the dam.

2.3 Geophysical monitoring

The ERT method applied the two-dimensional electrical imaging technique (also known as two-dimensional Electrical Tomography) in the Schlumberger array. The spacing between the electrodes was 5 m. The survey comprised four electro-resistivity sections parallel to the dam axis, which is 320 meters long (Figure 3a). Interpolations allowed to creation of the three-dimensional model of ERT data (Figure 3b). Periodic surveys were carried out on the same sections as the initial survey, e.g. four sections parallel to the dam axis every 15 days, starting on 17th May 2023 and ending on 31st March 2023, totaling 22 surveys (Table 2).

2.4 Database

The Leapfrog software (Version 2022.1.0) performed the analyses of periodic electro-resistivity surveys (in grid format) and the dimensional solids of electro-resistivity measurements. The instruments (piezometers and water level indicators) also had geographical coordinates (North/East coordinate and elevation), and so the data importation had the respective date readings equal to the days of the geophysical surveys (Figure 4a).

After the model creation, we defined data pairs (electro-resistivity and water level/piezometric level data) for each instrument and that date. It is important to note that if the instrument is dry, we selected the instrument base as the survey point (Figure 4b).

Table 2. Date of ERT surveys in the dam.

Monitoring number	Monitoring date
1	17-May-22
2	30-May-22
3	9-Jun-2022
4	17-Jun-2022
5	27-Jun-2022
6	8-Jul-2022
7	30-Jul-2022
8	8-Aug-2022
9	29-Aug-2022
10	5-Sep-2022
11	30-Sep-2022
12	17-Oct-2022
13	28-Oct-2022
14	9-Nov-2022
15	29-Nov-2022
16	8-Dec-2022
17	23-Jan-2023
18	10-Feb-2023
19	27-Feb-2023
20	10-Mar-2023
21	20-Mar-2023
22	31/03/2023

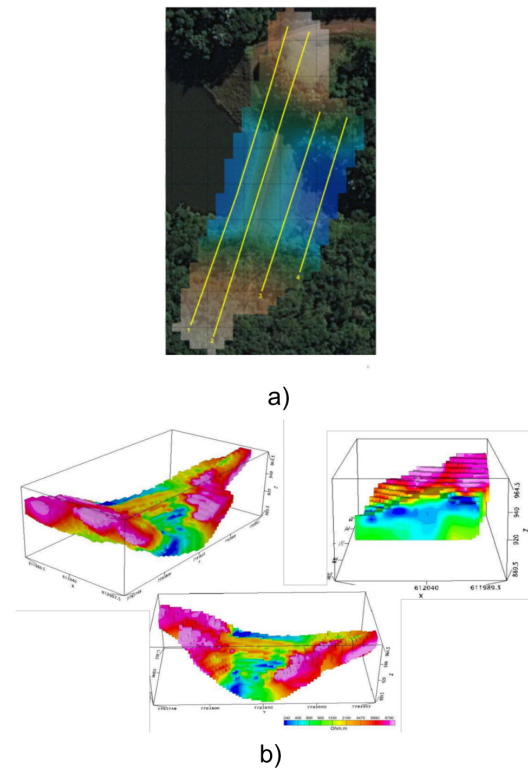


Figure 3. (a) the ERT sections parallel the dam axis; (b) the dam three-dimensional model with electro-resistivity data.

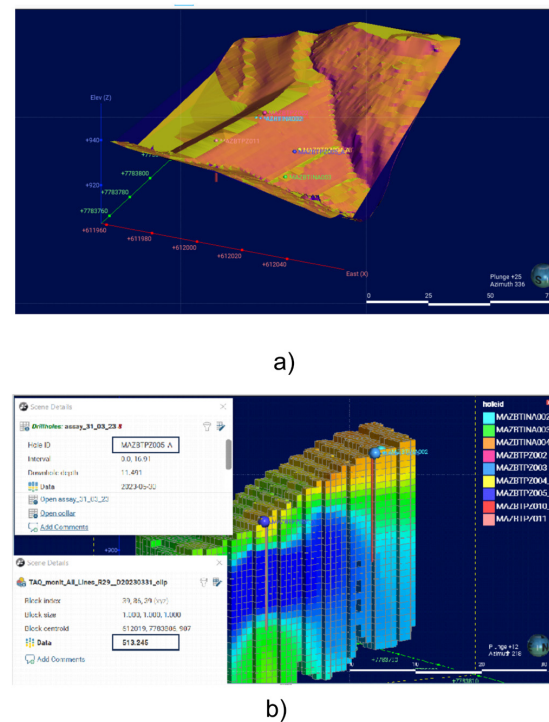


Figure 4. (a) The dam three-dimensional model with electro-resistivity data and location of the geotechnical instrument on 30th March 2023; (b) the instrument position concerning the elevation level.

It was then possible to generate the database for evaluating the relationship between the structure's geotechnical and geophysical data. In addition, the rainfall database from a rain gauge, which is near the structure, was also used. In addition to the daily spot reading, we evaluated the accumulated rainfall between geophysical surveys.

2.5 Statistical analysis

The analysis of variance, Pearson's correlation, multiple regression without interaction, multiple regression without interaction, and linear regression allowed to investigation of the relationship between geophysical data and geotechnical data, statistical analyses of the data were carried out using. The R program (version 4.2.1) ran the analyses, and the probability of 95% rejects the null hypothesis ($p\text{-value} < 0.05$).

The ANOVA analysis evaluated if the electro-resistivity data may be explained by the occurrence of precipitation (in mm) and the time (number of days that sampling). Pearson's correlation aimed to identify the relationship between electro-resistivity and piezometer reading, as well as electro-resistivity and precipitation survey. We used multiple regressions to explain electro-resistivity data by the number of days of rain, as well as the linear regression.

3. Results

3.1 Database

The PZ-04 instrument showed the greatest variation in geotechnical data compared to the other piezometers (Figure 5a). The others were dry or had low variation over

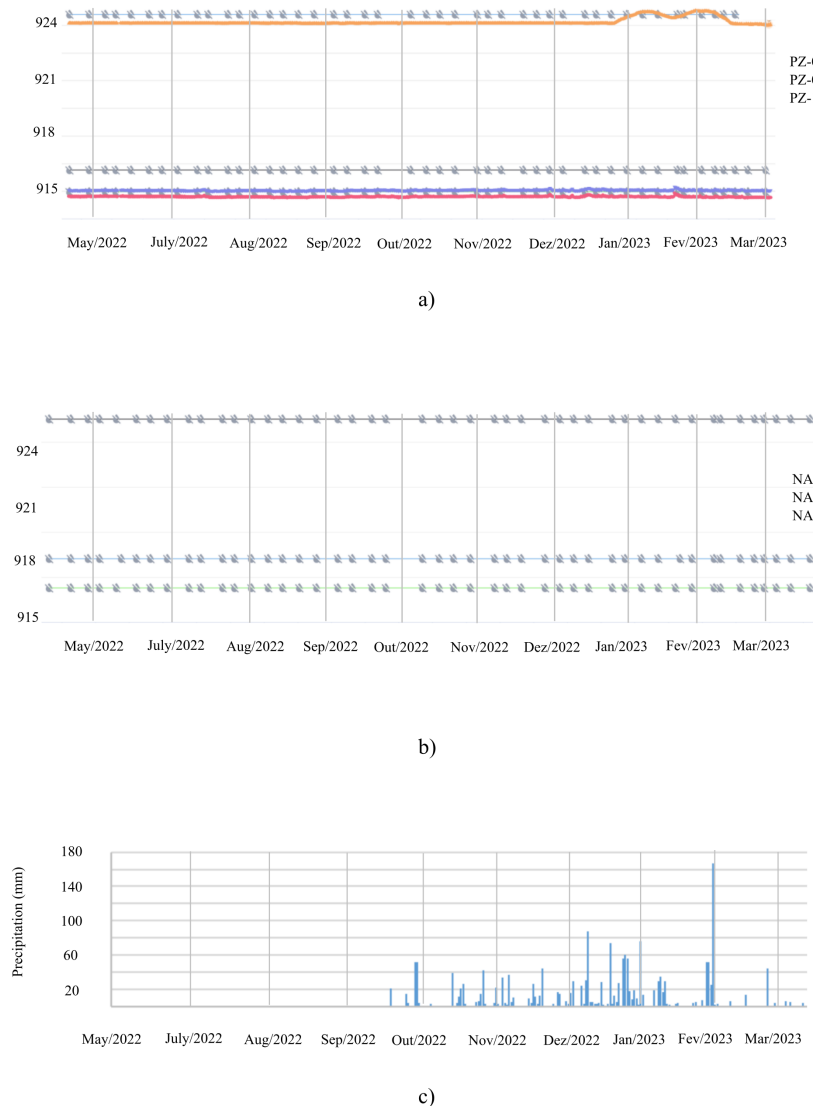


Figure 5. (a) Data by main piezometers (m); (b) data by water level indicator (m); (c) precipitation (mm) along the 11 monitoring months.

time. The water level gauges installed in the structure remained dry throughout the survey period (Figure 5b). During the sampling period, the highest rainfall occurred in February, around 160 mm (Figure 5c).

The PZ04 piezometer is 8.34 m deep, and its top level is on the downstream slope (Figure 6). The electrical

resistivity values for the reading points of this instrument ranged from 39 ohms to 57 ohms, without much variation over the rainy/dry period. The instrument's reading of the piezometric level varied over the period, and it was possible to observe that a lower piezometric load is associated with lower electro-resistivity.

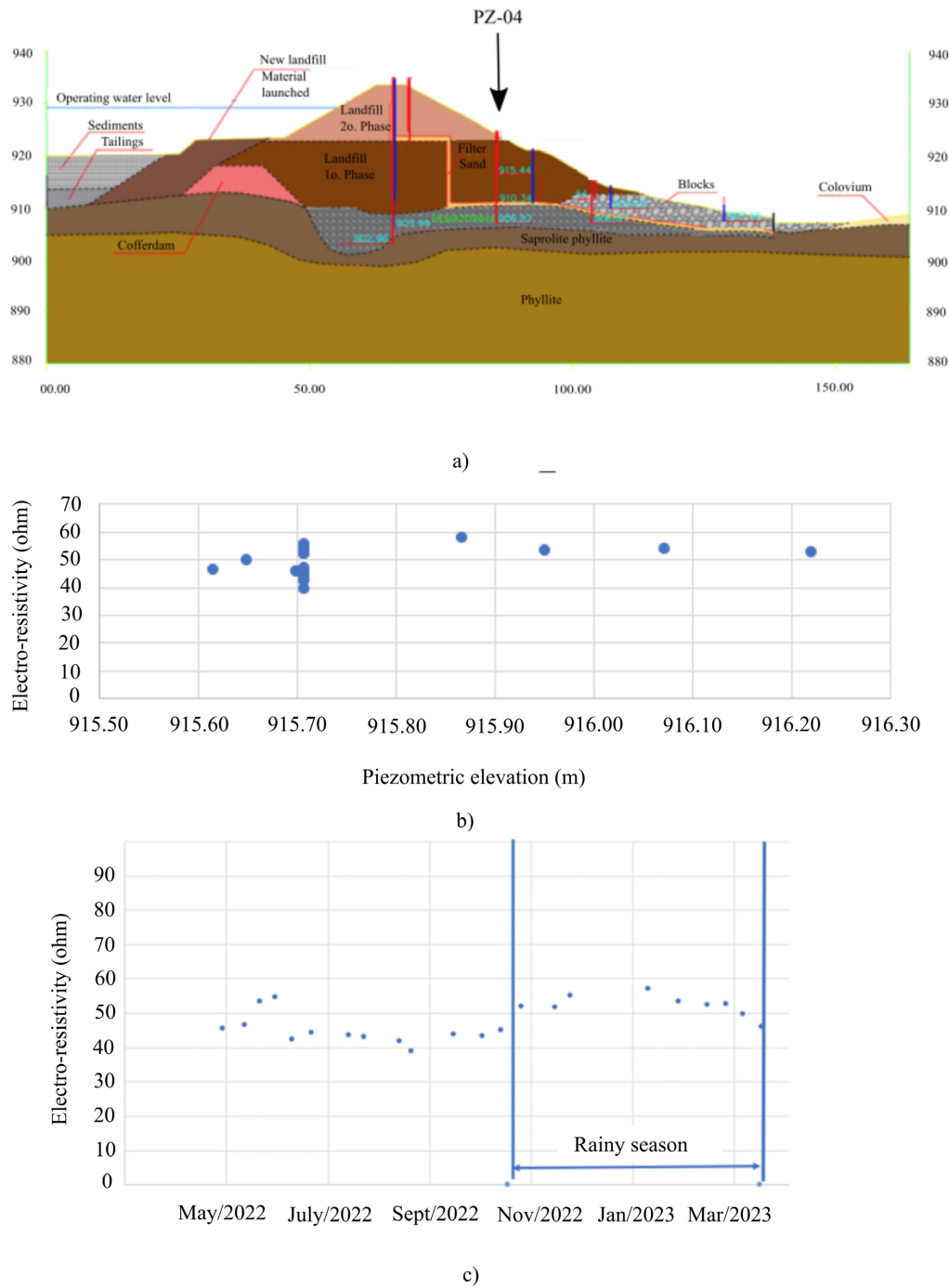


Figure 6. (a) Location of PZ-04 in the dam; (b) electro-resistivity versus piezometric elevation in PZ-04; (c) electro-resistivity versus date in PZ-04.

The NA-03 water level indicator is 6.34m deep, and its top level is located on the middle berm downstream (Figure 7). The electro-resistivity values for the reading points of this instrument ranged from 286 ohms to 442 ohms, with the lowest measurements at the end of the rainy season. The instrument had no variation in water level readings during the period.

3.2 Statistical results

According to the analysis of variance, neither the precipitation factor nor the time factor significantly explained the variation in the average electro-resistivity values ($p\text{-value} < 0.05$) (Table 3). The result of the regression

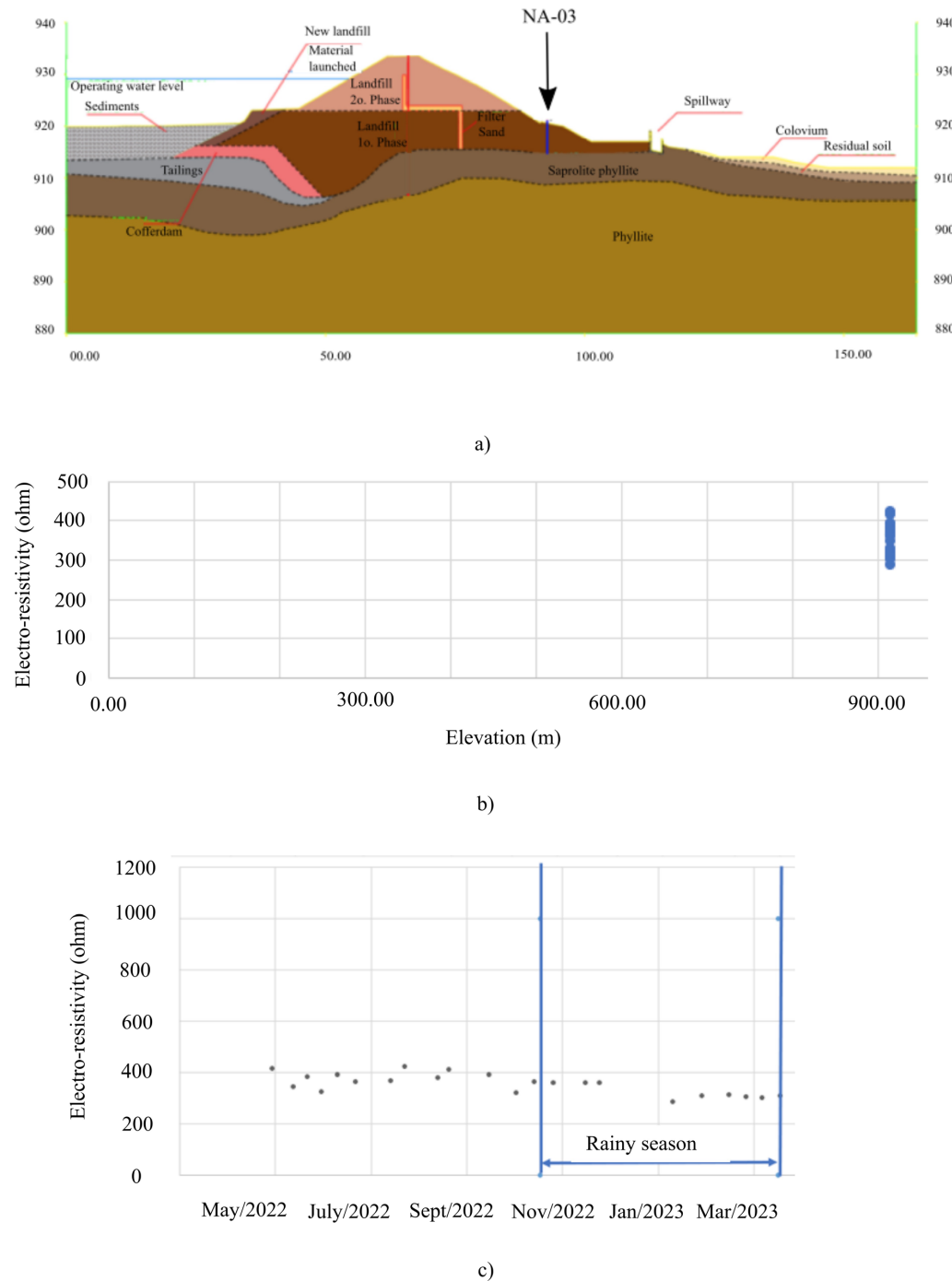


Figure 7. (a) Location of INA-03 in the dam; (b) electro-resistivity versus piezometric elevation in PZ-04; (c) electro-resistivity versus date in PZ-04.

indicated that electro-resistivity is significantly explained by piezometric variation (Table 4). The multiple linear regression with interaction aimed to check whether the variables had a combined influence on the final result of the analysis. The interaction between the variables was unable to explain (p -value > 0.05) the electro-resistivity values found (Table 5).

4. Discussion

The use of indirect methods applied to direct methods is of paramount importance since there is a significant gain in the laterality of the information and the possibility of extrapolating it. Oliveira (2018) states that geophysical data generates continuous subsurface images, making it possible to identify possible irregularities in the dam structure. The methods are highly capable of detecting failure modes such as internal erosion (Mainali et al., 2015). It clearly shows the potential of using geophysical data integrated with geotechnical data to gain confidence in the information on structures.

By analyzing the data collected over the eleven-month monitoring period, it was possible to obtain the following results: 1) electro-resistivity was significantly explained by piezometric variation; 2) precipitation had no significant relationship with electro-resistivity and 3) electro-resistivity showed no variation concerning time in the sampling collected.

We expected the relationship between the electro-resistivity and piezometry data since low electrical resistivities are associated with the presence of water. Electro-resistivity, on the other hand, did not respond immediately to rainfall. According to Canatto (2021), in the study of a small dam located in Minas Gerais (Brazil), there was variability in the electro-resistivity values between the surveys, with the months with low rainfall (August, September, and October) showing higher electro-resistivity amplitudes, while the months with the highest rainfall (November, December, and January) showed low electro-resistivity values with little amplitude. The large influence of rainfall on the saturation patterns of dams may explain these results. It is important to note that the response time of the measurements and the variation in the electro-resistivity can lead to different results. In other words, for future work, the temporal correlation between the data and the variation in time could be related.

We also expected the correlation between the variation in electro-resistivity over time. However, we did not find any temporal correlation between the geophysical data. It was possible to observe that in the year of the surveys, there were no successive days with rainfall, but isolated peaks of rain, which did not generate a constant saturation of the soil with significant changes in electro-resistivity.

Table 3. Analysis of variance of the electro-resistivity data by precipitation and time factors.

Variable	Sum	Sum SQ	Sum F	F	p -value
Precipitation (mm)	1	31588	31588	0.81	0.37
Time (days)	20	73038	3652	0.09	1.00
Residues	176	69000817	39209		

Table 4. Result of the multiple linear regression of electro-resistivity by the dependent variables precipitation, piezometric variation, and time.

	Estimate	Std. Error	t -value	p -value
Intercept	-7307.49	2043.00	-3.58	<0.05
Precipitation (mm)	-0.65	1.39	-0.47	0.65
Piezometric variation (m)	8.42	2.38	3.76	< 0.05
Time (days)	-0.12	0.13	-0.93	0.35

Table 5. Results of the multiple linear regression of electro-resistivity considering the interaction between the independent variables precipitation, piezometric variation, and time.

	Estimate	Error	t -value	p -value
Intercept	-1567.00	3690.00	-0.43	0.67
Precipitation (mm)	335.00	1936.00	0.17	0.86
Piezometric variation (m)	2.13	4.04	0.53	0.60
Time (days)	-36.91	22.70	-1.63	0.11
Precipitation (mm) and Piezometric variation (m)	-36.59	2.12	-0.17	0.86
Precipitation (mm) and Time (days)	-1.82	9.57	-0.20	0.85
Piezometric variation (m) and Time (days)	0.04	284.60	1.62	0.11
Precipitation (mm), Piezometric variation (m) and Time (days)	<0.01	0.01	0.19	0.85

Geophysical monitoring proved to be important and could provide additional information for geotechnical structures. Further research could be carried out to verify the applicability of the correlations between electro-resistivity and piezometry in surveys of other dams, to identify whether the correlation applies to other structures, which have different boundary conditions, such as construction method, construction material, variations in internal piezometry, etc.

5. Conclusion

This study aimed to correlate geotechnical data with geophysical data from periodic surveys of a tailings dam in an ore mine. The main findings of this work are: 1) In terms of variations in water levels as well as piezometric heights, the survey points did not have significant variations, with many instruments having dry readings for a large part of the surveys. As a result, it was not possible to observe any great dispersion in the geotechnical data; 2) the electro-resistivity data is explained by the piezometric level; 3) the date of the survey does not produce significant variation in electro-resistivity data, and 4) the electro-resistivity data do not present direct relation with precipitation. These conclusions may contribute to advances in dam monitoring of the mining sector and, consequently, to improving dam savings policies.

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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

Isabela Vasconcelos Leite de Freitas: conceptualization, data curation, investigation, software, funding acquisition, writing – original draft. Hersilia Andrade e Santos: conceptualization, data curation, methodology, supervision, validation, writing – original draft, writing – review & editing.

Data availability

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

List of symbols and abbreviations

p -value	level of marginal significance within a statistical hypothesis test
t	value of student's t-test
ERT	electro-resistivity
F	value of F-test
H	horizontal increment
NA	water level indicators
PZ	piezometer
SP	spontaneous potential
SQ	square
V	vertical increment

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Effect of weathering on the stability of rock slope at a water intake canal of Simplicio Hydroelectric Power Plant

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Case Study

Keywords

Alterability
Rock and joint behavior
Slope stability analysis

Abstract

Rock and joint alteration leads to modifications in their properties, resulting in a reduction of rock mass strength and consequently a decrease in the safety of engineering structures. The alteration process changes the material and alters its geomechanical behavior. This research presents the studies conducted to evaluate the mechanical behavior of rock joints in the gneiss massifs that make up the hydraulic circuit of the Simplicio Hydroelectric Plant due to their alteration. Compilation and analysis of data from previous research were carried out to parameterize the joints at different levels of alteration. Deterministic stability analyses showed that the alteration process of gneiss joints can significantly reduce the slope safety factor by up to half of its initial value. Furthermore, probabilistic analyses revealed an increase in failure probability of approximately 10% among the studied alteration classes. The results underscore the importance of assessing the level of joint alteration in engineering projects.

1. Introduction

The alteration of rock materials results from their exposure to weathering agents, thus being the effect of a geological process. Physical and chemical changes induced by this exposure can modify the engineering properties of rock masses, leading to their weakening due to the appearance of fissures, mineral alterations, and a reduction in material strength (Guidicini & Nieble, 1983; Teixeira et al., 2000; Maia, 2001; Maia et al., 2002, 2003; Press et al., 2006).

The alteration of rocks and their discontinuities may lead to a decrease in material strength and deformability, together with block disintegration and increased permeability. Understanding this process enables the prediction of the behavior of rock masses over time, thus enabling the valuation of the required time for maintenance and preventive actions.

Thus, the present study seeks to assess the influence of gneiss joint alteration on the mechanical behavior of a water canal at the Simplicio Hydroelectric Power Plant (AHE Simplicio). It is part of a series of investigations carried out over the past two decades by the Alterability Research Group at PUC-Rio and COPPE-UFRJ.

Among the previous research conducted by the group, notable works include those by Maia (2001), Maia et al. (2002, 2003) on the rock masses of the Marimbondo and Serra da

Mesa dams, in Brazil, as well as the research conducted on the rock masses of the Simplicio Hydroelectric Project by Machado (2012), Salles (2013), Oliveira (2017), Steffens (2018), Lopes (2019) and Lopes (2023). These studies were conducted following recurrent failures during the excavation of the gneissic canal slopes at the Simplicio Hydroelectric Power Plant. This research aimed at assessing the influence of altered rock on the stability both rock and residual soil slopes.

The compilation and comprehensive analysis of the results from the previous research, particularly those by Oliveira (2017), Steffens (2018), and Lopes (2019), served as a basis for defining the parameters of rock joints at different levels of alteration. The main objective of the present research is to establish the influence of gneiss alteration on the geotechnical parameters of Simplicio rock which can affect the stability of the water intake canal slopes within the project.

2. Shear strength of joints

Roughness is associated with the morphology of discontinuity walls. This feature of joints, in conjunction with the compressive strength of the walls, can significantly influence the mechanical behavior of the rock mass. However, its importance is reduced as more criteria on joint opening, filling, and degree of prior displacement are considered.

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Shear strength of joints is directly linked to their roughness, especially in unfilled discontinuities. The magnitude of shear strength of joints is related to the peak or residual friction angle, combined with the contribution of roughness. Patton (1966) proposed a failure criterion that takes into account the influence of this roughness in estimating the strength of discontinuities. Thus, two failure envelopes were proposed, one for low nominal loads (Equation 1), i.e., when there is no shearing of roughness, and another for high nominal loads (Equation 2), when shearing of irregularities occurs, resulting in a reduction in the friction angle.

$$\tau = \sigma_n \cdot \tan(\phi_b + i) \quad (1)$$

$$\tau = c_j + \sigma_n \cdot \tan \phi_r \quad (2)$$

where, σ_n = effective normal stress, ϕ_b = basic friction angle, i = average angle of deviation of particle displacements from the direction of the applied shear stress, c_j = cohesion intercept of the joint, ϕ_r = residual friction angle.

Barton (1973) developed a criterion that considers the influence of roughness and the compressive strength of discontinuity walls on the shear strength of rock joints. The criterion accounts for two components of strength: the portion related to the rock mineralogy and the portion linked to discontinuity roughness, as presented in Equation 3.

$$\tau = \sigma_n \cdot \tan \left(JRC \cdot \log \left(\frac{JCS}{\sigma_n} \right) + \phi_b \right) \quad (3)$$

where, JRC = Joint Roughness Coefficient, JCS = Joint Compressive Strength.

The basic friction angle (ϕ_b) represents the portion of the strength associated with the mineralogy of unaltered, dry, and flat joints, meaning those that exhibit no visible roughness components. According to Bandis (1980), the basic friction angle can be determined from the residual shear strength of flat rock surfaces. This parameter can be determined through direct shear tests, tilt tests, and push or pull tests, using sawn or sandblasted surfaces.

Subsequent contributions enhanced the criterion presented by Barton (1973), incorporating additional considerations, such as the use of the residual friction angle (ϕ_r) for altered joints (Barton & Choubey, 1977) and the scale effect (Barton & Bandis, 1982).

The residual friction angle (ϕ_r) was incorporated into the criterion to broaden its applicability to altered saturated joints. This parameter corresponds to the residual friction angle of altered and/or saturated joints. Its determination can be achieved through direct shear tests with sufficiently long displacements to eliminate the effects of joint roughness (Woo et al., 2010). Equation 4 presents the generalized criterion for the shear strength of joints.

$$\tau = \sigma_n \cdot \tan \left(JRC \cdot \log \left(\frac{JCS}{\sigma_n} \right) + \phi_r \right) \quad (4)$$

The residual friction angle (ϕ_r), like the basic friction angle (ϕ_b), depends on the constituent materials of the joint. However, the former also depends on the level of alteration of these materials. Therefore, the level of alteration is one of the most important factors influencing the residual friction angle for rock joints with the same mineralogy and grain size (Woo et al., 2010).

The roughness component of the joint depends on three main elements: the joint roughness coefficient (JRC), the joint compressive strength of the discontinuity walls (JCS), and the acting normal stress on the assembly. Joints with lower roughness are less affected by the value of JCS , as roughness has less influence when the joint roughness coefficient (JRC) is reduced (Barton, 1978).

According to Kveldsvik et al. (2008), for low-acting normal stresses, joint roughness has a significant impact on assembly strength. This phenomenon is due to joint dilation caused by roughness during shearing. For higher normal stresses, the effect of joint roughness is reduced due to the decrease in the ratio between the joint compressive strength and the acting normal stress (JCS / σ_n).

Most stability problems related to rock slopes occur under low-acting normal stresses. Therefore, roughness is a fundamental component in the study of shear strength for unaltered and unfilled joints (Barton, 1978).

Joint wall compressive strength (JCS) was defined by Barton (1973, 1978) as equal to the uniaxial compressive strength of the rock (σ_c) for unaltered joints or as a portion of (σ_c) for altered joints, which can amount to 1/4 of σ_c .

According to Barton (1978), joint wall compressive strength is an essential component of shear strength. Consequently, processes that could reduce this portion, such as alteration and moisture variation, may decrease the shear strength of the assembly.

Joint wall compressive strength (JCS) can be estimated through uniaxial compression tests or Schmidt hammer tests (Barton & Choubey, 1977).

Among the parameters involved in the strength criterion defined by Equation 4, the joint roughness coefficient (JRC) is the most challenging to determine. Its estimation can be carried out by comparing roughness profiles, measuring roughness, or through testing methods (Leite & Dantas Neto, 2020; Dantas Neto et al., 2022).

Methods for measuring roughness include the use of profilometers, photogrammetry, laser scanning, among others. Regarding JRC estimation methods through testing, direct shear tests, tilt tests, and push or pull tests can be conducted.

Direct shear tests can be performed on circular or square samples, providing shear stress versus normal stress curves, shear stress versus displacement curves, and dilation versus

displacement curves, as outlined in Figure 1 (Barton et al., 2023).

The JRC value can be estimated by retro-analyses of the data obtained from the direct shear test of rock joints through the rearrangement of Equation 4.

2.1 Scale effects

JRC and JCS values are dependent on the scale of the tested sample. In general, larger blocks tend to have a lower JRC value compared to smaller blocks, considering the same rock joint. Barton & Bandis (1982) proposed Equations 5 and 6 to correct the scale effect.

$$JRC_n \approx JRC_o \left(\frac{L_n}{L_o} \right)^{-0.02 * JRC_o} \quad (5)$$

$$JCS_n \approx JCS_o \left(\frac{L_n}{L_o} \right)^{-0.03 * JRC_o} \quad (6)$$

where, $JCS_o = JCS$ measured in the laboratory, L_n = joint in situ length, L_o = joint length in laboratory.

According to Barton & Bandis (1982), the scale effect tends to decrease as the sample dimensions exceed the natural size of blocks found in the field.

Furthermore, the scale effect alters the behavior of the joint in terms of dilation and the required displacement to mobilize peak resistance. Samples with larger dimensions experience reduced dilation during shearing, while the displacement required to mobilize peak resistance increased.

As observed by Goodman (1970), Barton (1973), and Pratt et al. (1974), the peak dilation of joints occurs simultaneously with or shortly after the peak shear resistance

of the sample, demonstrating a strong correlation between these quantities.

2.2 Barton-Bandis failure criterion

The shear strength criterion for joints presented earlier is also known as the Barton-Bandis failure criterion. The Barton-Bandis failure criterion has been used over the past decades and provides a reliable approximation for the behavior of unfilled rock joints.

The Barton-Bandis (BB) model is a nonlinear method and is considered more accurate when compared to the linear Mohr-Coulomb (MC) method. Numerous studies have been conducted to assess the effectiveness of applying the BB model to predict shear resistance in joints and the stability of their slopes, with these research efforts yielding favorable results and approximations (Barton et al., 2023).

Barton et al. (2023) emphasize that despite all the advancements achieved, the determination of joint roughness remains complex and continues to be a challenge even today.

3. Stability analysis

The stability of rock slopes is influenced by numerous factors such as stress and drainage conditions, mechanical properties of the massif, and the distribution of discontinuities. In general, the presence of joints represents a zone of weakness in which the properties of these discontinuities govern the stability of the rock mass in terms of shear and deformation (Barton et al., 2023).

According to Barton et al. (2023), there are two primary approaches for evaluating the stability condition of rock slopes, considering the influence and properties of joints: the discontinuous and continuous approaches.

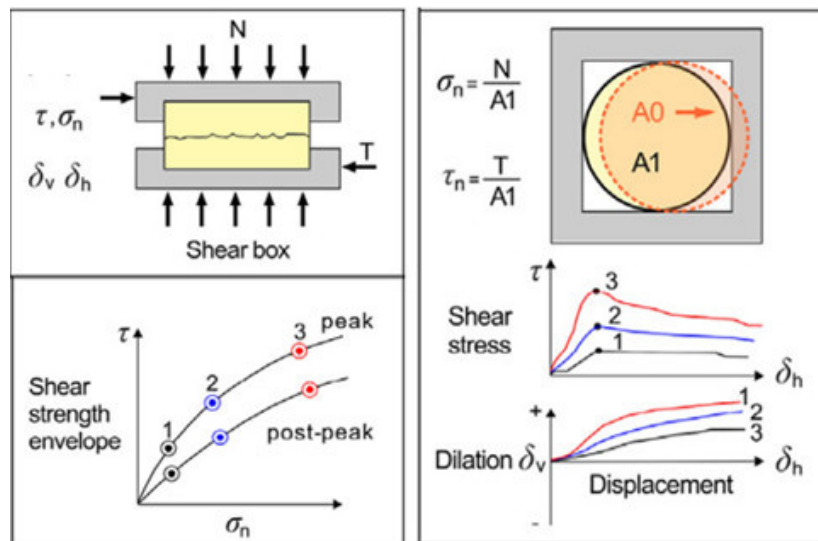


Figure 1. Typical direct shear test results (Barton et al., 2023).

The discontinuous approach models the geological structure within the massif, allowing the present discontinuities to control deformation and failure modes. The continuous approach corresponds to simulating the rock mass as a medium that assumes the properties of the joints, transforming the slope discontinuity into a hypothetical continuous medium.

According to Barton et al. (2023), both approaches are applicable under specific conditions. The discontinuous approach is more appropriate when the geological structure controls anisotropy, deformation, and massif strength. On the other hand, the continuous approach is more suitable when the frequency and orientation of the joints are such that there are no preferential paths of rupture and/or deformation.

Generally, stability analyses in geotechnical engineering are conducted considering deterministic analyses, in which a safety factor is obtained as the ratio of resisting applied forces. In deterministic analyses, the use of average parameters disregards the uncertainties surrounding the stability analyses of a structure or slope, such as strength parameters, site stratigraphy, groundwater level, amongst others.

Probabilistic analyses emerge to account for all these uncertainties, complementing deterministic analyses and providing probability distribution of failure related to each value of the safety factor (SF). Thus, it enables the assessment of the distribution of SF as a variable dependent on the distribution of various independent variables.

In the probabilistic approach, a variable or performance indicator y (SF , settlement, flow rate, among others) is described through a mathematical function, whose statistical distribution allows for the calculation of the most probable value and the failure probability associated with each value (Assis et al., 2018).

Probabilistic methods enable the determination of the probability distribution of a dependent variable (performance indicator) based on the statistical distributions of its independent variables. Some of the primary probabilistic methods include the First Order Second Moment (FOSM) and the Monte Carlo Method (MCM).

3.1 First Order Second Moment (FOSM)

The FOSM method was developed as an expansion of the Taylor series of the equation for the variance of a dependent variable y in terms of independent random variables, where this expansion is truncated at its first-order terms.

According to Griffiths & Fenton (2007), this method provides approximations for the mean and standard deviation of the dependent variable based on the means and standard deviations of all independent random variables. Therefore, it is not necessary to determine the probability distribution function of the independent variables. In other words, the calculation of failure probability is based solely on the moments of the statistical distribution (mean and standard deviation) of the random variables (Sayão et al., 2012).

Considering $x_1, x_2, \dots, x_n$ as random variables in the calculation of the safety factor (SF), the mean value of X and its performance function are represented by $\bar{X}$ and $F(\bar{X})$. The truncated Taylor series for the expansion of the function $F(X)$ around the point $X = \bar{X}$ is given by Equation 7. Equation 8 presents the variance of the safety factor for N random variables.

$$F(X) = f(\bar{X}) + f'(\bar{X})(X - \bar{X}) \quad (7)$$

$$V(F) = \sum_{i=1}^N \left(\frac{\partial SF}{\partial x_i} \right)^2 V(x_i) \quad (8)$$

Christian et al. (1992) introduced a methodology for approximating partial derivatives through a simplification based on the divided differences method. The methodology involves an incremental variation of each parameter defined as a random variable and the corresponding change in the safety factor (SF). The approximation of the partial derivative with respect to each parameter is obtained by the ratio of the SF variation to the parameter variation, as indicated in Equation 9.

$$\left(\frac{\partial SF}{\partial x_i} \right) = \frac{SF(x_i + \delta x_i) - SF(x_i)}{\delta x_i} \quad (9)$$

3.2 Monte Carlo Method (MCM)

The Monte Carlo Method seeks to find an approximate numerical solution for the probability distribution of the dependent variable by using a technique in which a sequence of random numbers is generated to characterize the function $F(X)$ in a representative manner. As the number of generated values tends to infinity, the result becomes more precise and accurate.

After repeating the calculation process N times for all values of the independent variables, a sample of discrete values of the variable $F(X)$ is obtained. This allows for the calculation of its mean, standard deviation, the best-fitting probability distribution, and failure probability.

4. Study area and material

The Simplicio Hydroelectric Power Plant is located between the municipalities of Três Rios (RJ), Sapucaia (RJ), Chiador (MG), and Além Paraíba (MG), extending parallel to the banks of the Paraíba do Sul River (Figure 2). The project consists of the small Anta Hydroelectric Plant (PCH Anta), where the river is dammed, and the Simplicio Hydroelectric Plant (UHE Simplicio). After damming, a portion of the flow is diverted through an intake circuit that

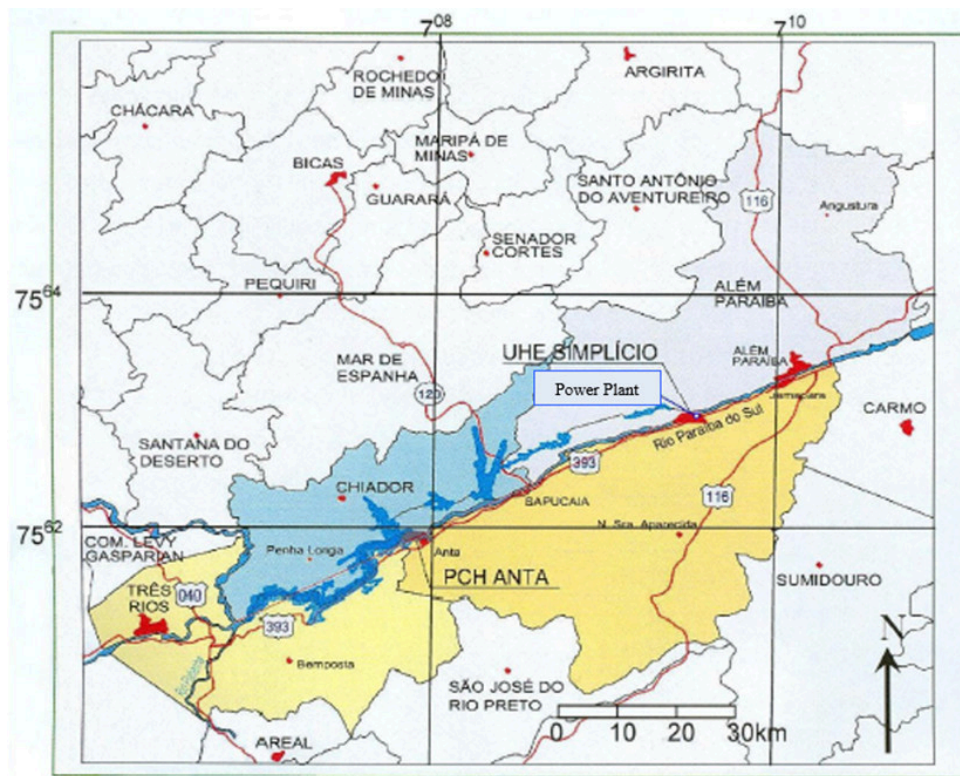


Figure 2. Location of Simplicio Hydroelectric Power Plant (Marchesi, 2008).

spans approximately 30 km and includes canals, tunnels, reservoirs, and dikes.

This circuit is responsible for conveying water to the powerhouse of the UHE Simplicio, where energy generation occurs through a single drop, harnessing a natural elevation difference of 115 meters (Machado, 2012; Steffens, 2018).

The project site is situated within the Além Paraíba Lineament unit, also known as the Paraíba do Sul River Shear Zone, and features high-grade igneous and metamorphic rocks (Dehler & Machado, 2002; Viana, 2010). This shear zone is a transcurrent fault zone characterized by a prominently N60E-aligned topography. The surrounding geology consists of crystalline substrates of gneisses, migmatites, and granitoid rocks intruded by diabase dikes, partially overlain by deposits of alluvial, colluvial, and residual soils (Valeriano, 2006; Marinho, 2007).

Regarding climate, the region experiences a humid climate with annual precipitation exceeding 1200 mm, leading to significant weathering of rocks along fractures, resulting in well-developed weathering mantles that become more susceptible to sliding and erosion (Marinho, 2007).

The samples tested by Oliveira (2017) and Steffens (2018) were obtained from rotary drillings conducted within the project area and tested in the laboratory by Salles (2013). The block samples tested by Lopes (2019) in the laboratory were collected from excavation areas of the project canals, primarily from Canal 1 excavations. These samples were separated and stored in sealed containers, which remained

in an open area of the Geotechnical Laboratory of COPPE-UFRJ for 10 years, exposed to weathering agents such as temperature and humidity variations.

5. Definition of geotechnical parameters and geometry

Using Barton's (1973) shear strength criterion for rock joints, as presented in Equation 4, and the results obtained by Steffens (2018) and Lopes (2019) through direct shear tests and Schmidt Hammer tests, along with relevant literature references on the subject, it was possible to determine the geomechanical parameters of the gneiss joints at the Simplicio Hydroelectric Plant (AHE Simplicio) region.

Steffens (2018) assessed the mechanical behavior of the rock joints at the AHE Simplicio for various levels of natural and artificial alteration. This involved conducting direct shear tests and Schmidt Hammer tests on core samples taken from drillings in Canal 1 of the hydropower project, which had been altered in the laboratory by Salles (2013). Salles (2013) subjected the samples to accelerated percolation for various alteration times: 0 hours, 600 hours, 1200 hours, 2400 hours, 4800 hours, and 83000 hours.

Lopes (2019) conducted Schmidt Hammer tests in both field and laboratory settings for different levels of joint alteration and developed a proposal to predict behavior over time for the gneiss in the region of the Simplicio Hydroelectric Project.

According to Barton's (1973) equation for the shear strength of rock joints (Equation 4), the variables to be defined for the use of the Barton-Bandis failure criterion are JCS , JRC , and the residual friction angle (ϕ_r), in addition to the rock unit weight (γ) and the joint friction angle (ϕ).

To evaluate the influence of the level of joint alteration on the stability of rock masses, the approach chosen was to treat the massif as a hypothetical continuous medium, with joint parameters governing the geomechanical behavior.

Furthermore, five levels of alteration were defined to determine the parameters, encompassing the alteration times presented in Table 1. Subsequently, these levels of alteration will be used to define five stability analysis scenarios.

5.1 Joint Compressive Strength (JCS)

Joint Compressive Strength (JCS) can be obtained through the Schmidt Hammer test, in which the rebound of the joint surface is measured, and its compressive strength is deduced through correlation.

Based on the results obtained by Steffens (2018) and reevaluated by Lopes (2019), the average JCS value for each level of alteration of the laboratory samples (ranging from 0 hours to 8300 hours) can be seen in Table 2.

5.2 Residual friction angle

The residual friction angle of joints can be obtained through direct shear tests and tilt tests, in which unfilled joints are sheared under low stresses (self-weight of the upper part). It can also be derived through correlation with the basic friction angle and the rebound values of the altered joint and the intact rock.

Table 1. Level of alteration defined according to the alteration time.

Level	Weathering Level	Time of alteration
Level 1	Unaltered Joint	Field Test (Fresh Joint)
Level 2	Very Slightly Altered Joint	0 hours of laboratory alteration
Level 3	Slightly Altered Joint	600 hours of laboratory alteration 1200 h of laboratory alteration
Level 4	Moderately Altered Joint	2400 h of laboratory alteration 4800 h of laboratory alteration
Level 5	Moderately Altered Joint	8300 h of laboratory alteration

Table 2. JCS , JRC and unit weight considering the scale effect.

Level	JCS_n (MPa)	JRC_n	γ (kN/m ³)
Level 1	139.3	7.1	27.0
Level 2	46.6	5.8	25.7
Level 3	38.7	5.6	24.3
Level 4	25.1	5.6	21.6
Level 5	19.5	6.4	21.6

Based on the results obtained by Steffens (2018) for flat gneiss joints that were not subjected to accelerated alteration in the laboratory (samples with 0 hours of alteration), a value of ϕ_r equal to 35° was determined.

5.3 Joint Roughness Coefficient (JRC)

For the calculation of the joint roughness coefficient, Equation 4 was employed, replacing the shear and normal stress with their peak values, and substituting the basic friction angle with the residual angle, as illustrated in Equation 10.

$$JRC = \frac{\tan^{-1}\left(\frac{\tau_{peak}}{\sigma_{n_{peak}}}\right) - \phi_r}{\log\left(\frac{JCS}{\sigma_n}\right)} \quad (10)$$

where, τ_{peak} = peak shear strength, $\sigma_{n_{peak}}$ = effective normal stress at peak shear strength.

Using Equation 10 and the raw data obtained by Steffens (2018) from direct shear tests on gneiss joints, it was possible to calculate the corresponding JRC value for each set of peak shear stress to normal stress. For the unaltered joint tested in the field using the Schmidt Hammer, the JRC value was assumed to be the highest JRC value found for the samples with 0 hours of laboratory alteration, resulting in $JRC = 13.3$. Therefore, the average JRC values for each level of alteration are presented in Table 2.

5.4 Rock unit weight

In this study, the unit weight of unaltered gneiss will be considered as 27 kN/m³. Using the unit weight reduction ratio proposed by Barton & Choubey (1977), the unit weight values for each level of alteration studied in this research were obtained. The values are presented in Table 2.

5.5 Joint friction angle

Based on the results obtained by Steffens (2018) in the direct shear tests of gneiss joints, it was possible to construct the Mohr-Coulomb strength envelopes to determine the joint friction angle for each laboratory alteration time. The friction angle values obtained for each level of alteration and their respective averages are presented in Table 2. For the unaltered joint tested in the field, the highest friction angle value from the presented envelopes was adopted, which is equal to 37°.

5.6 Field parameters – Scale effect

In order to compare the safety factors obtained by considering the laboratory-tested parameters with the field parameters suggested by Equations 5 and 6, Table 2 presents

the field results for *JCS*, *JRC*, and unit weight for each level of alteration. It should be noted that for this study, the block size in the field (L_n) was considered to be equal to 0.5 m.

5.7 Geometry

Due to the impossibility of accessing the geological-geotechnical section of the Simplicio canals, a schematic geometric section was adopted in this work, based on images of Canal 1 excavations and information published by Motta et al. (2009).

According to Motta et al. (2009), part of the slopes of Canals 1, 2, 3, and 4 were built on slopes of 1(V):1.3(H), with a berm height equal to 10 m and a width of 4 m. Furthermore, from images from Channel 1, it was possible to notice that in some points, the three lower banks received rockfill berms, and the others were made up of soil. The lower banks are precisely the gneiss banks that suffered recurrent failures and whose stability needed to be increased through the placement of rockfill berms.

Considering the available information, the schematic geometry of the slopes was considered with a slope of 1(V):1.3(H), with a berm height of 10m and a width of 4m. In order to analyze the influence of the joint behavior on the stability of the massif and how its alteration reduces the safety of the canals, the three lower benches in rock were modeled. In other words, the region of the slopes consisting of residual gneiss soil was not considered in the analyses. The final geometry analyzed is presented in Figure 3.

5.8 Stability analysis scenarios

In this research, the rock mass was modeled as a continuous medium, with parameters based on the geomechanical behavior of intact and/or altered joints in the gneiss of the Simplicio region. Namely, the properties of the joints were assumed to govern the behavior of the rock mass.

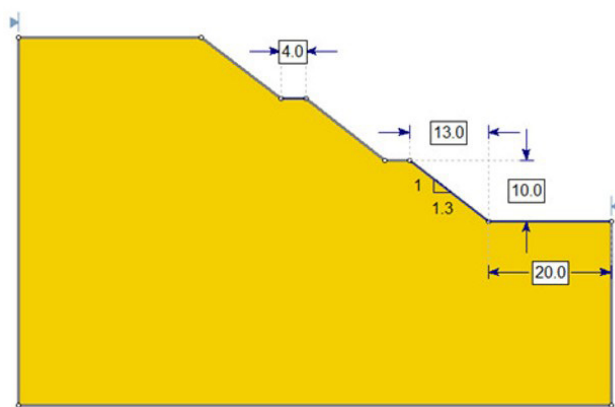


Figure 3. Schematic geometry defined for the stability analyses in this study.

This approach was adopted to understand how joint alteration can influence the stability of the Simplicio canals and other structures with similar material characteristics.

As previously discussed, deterministic and probabilistic stability analyses were conducted using the limit equilibrium approach and the Barton-Bandis failure criterion. Rocscience Slide2 software was used for the analyses, and the safety factors were obtained for non-circular surfaces using the Spencer and Morgenstern-Price methods.

Five scenarios were considered for the analysis of rough joints, with one scenario for each level of alteration defined previously in Table 1. Additionally, for all scenarios, the analyses were conducted both with and without considering the influence of the water level, as presented in Table 3. The standard deviations considered in the probabilistic analyses for each scenario are shown in Table 4.

6. Analysis and results

6.1 Geotechnical parameters

The dispersion of *JRC* values, peak friction angle, and friction angle obtained in the direct shear tests is presented in Figure 4, according to the level of alteration of the samples.

There is a noticeable trend of reduction in both the peak friction angle and the friction angle of the gneiss joints. Additionally, a slight increase in the dispersion of *JRC* values, peak friction angle, and friction angle can be observed as the level of alteration increases.

As depicted in Figure 4, there exists a discernible trend of friction angle reduction as the level of joint alteration escalates. Furthermore, an escalation in result dispersion is noticeable for more substantial levels of alteration. Figure 4 also indicates a tendency towards a decrease in the peak friction angle when considering the average of laboratory-derived values. The *JRC* value experiences a marginal decrease for initial alteration levels and a significant increase at the highest alteration level, along with an escalation in dispersion.

According to Woo et al. (2010), both the peak friction angle and the friction angle tend to diminish as the alteration of granite joints intensifies, while the *JRC* value generally rises. Despite this increase in the joint roughness coefficient, shear resistance tends to be lower for altered joints due to the reduction in other parameters, namely the peak friction angle, friction angle, and *JCS*.

In general, the results presented herein align with the behavior trends described in the literature, with the substantial dispersion and some divergent points attributed to the number of shear tests performed, which is considered below the ideal.

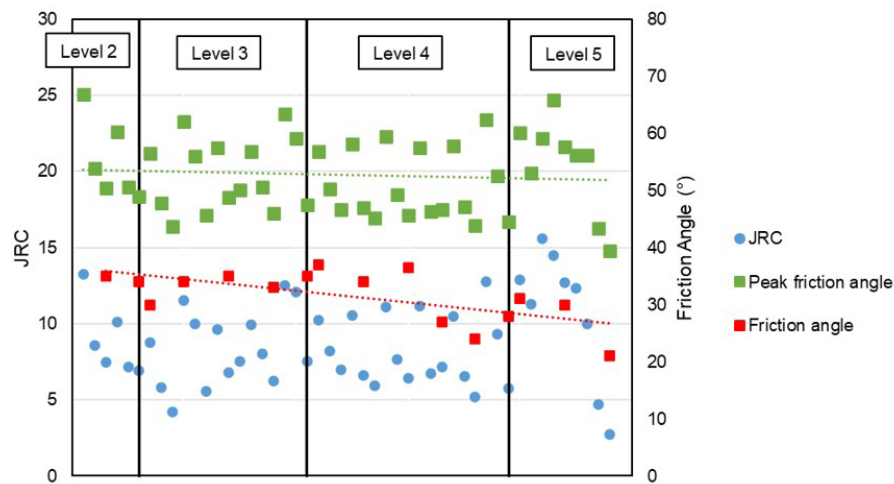
Figure 5 illustrates the graph of peak shear stress and ultimate shear stress against the normal stresses applied during direct shear tests. Additionally, four shear stress trajectories were plotted on the graph using the Barton-Bandis model: two trajectories employing laboratory-derived parameters for intact joints (green trajectory) and highly altered joints

Table 3. Analysis scenarios for rough joints and their respective levels of alteration.

Scenarios	JCS (MPa)	JRC	γ (kN/m ³)	ϕ (°)	Water level
Scenario 1	139.3	7.1	27	37	-
Scenario 2	46.6	5.8	25.7	35	-
Scenario 3	38.7	5.6	24.3	33	-
Scenario 4	25.1	5.6	21.6	31	-
Scenario 5	19.5	6.4	21.6	27	-
Scenario 6	139.3	7.1	27	37	12 m
Scenario 7	46.6	5.8	25.7	35	12 m
Scenario 8	38.7	5.6	24.3	33	12 m
Scenario 9	25.1	5.6	21.6	31	12 m
Scenario 10	19.5	6.4	21.6	27	12 m

Table 4. Standard deviation considered in the probabilistic analyses for each scenario.

Scenarios	σ_{JCS} (MPa)	σ_{JRC}	σ_{γ} (kN/m ³)	σ_{ϕ} (°)	$\sigma_{water\ level}$ (m)
Scenarios 1 to 5	10.4	2.4	2.4	4.2	-
Scenarios 6 to 10	10.4	2.4	2.4	4.2	1.2

**Figure 4.** Dispersion of the JRC values, the peak friction angle, and the friction angle.

(blue trajectory), as well as two trajectories using JCS and ϕ values identical to the first two trajectories but altering only the JRC values to 10 and 0.5 (red and black trajectories, respectively). These JRC values were chosen to encompass a broader range of data points, aligning with values typically available in studies on the subject.

It is discernible from Figure 5 that the peak shear stress values are higher and, in general, lie within the range delineated by the trajectory based on intact joints (green) and highly altered joints (blue). Conversely, the ultimate

shear stress values fall within the bounds of trajectories with JRC values of 10 (red) and 0.5 (black), with most of them situated below the blue trajectory (representing joints with a higher level of alteration).

This disparity primarily arises due to the pivotal role played by roughness in the initial shear resistance of these geological structures. Following the peak resistance, roughness assumes a diminishing role, consequently reducing the shear resistance of the samples.

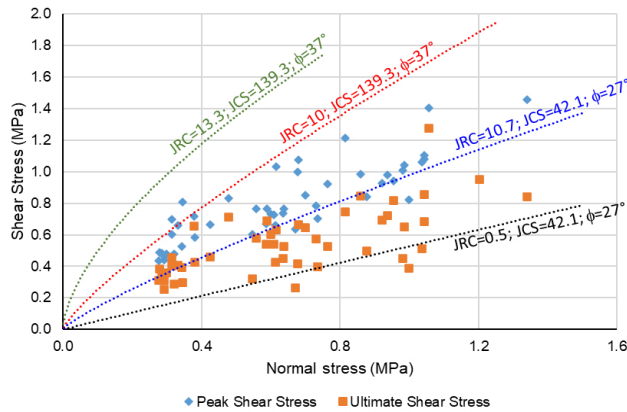


Figure 5. Peak and ultimate shear stress by normal stress and strength envelopes estimated by the Barton-Bandis model for parameters obtained in the laboratory.

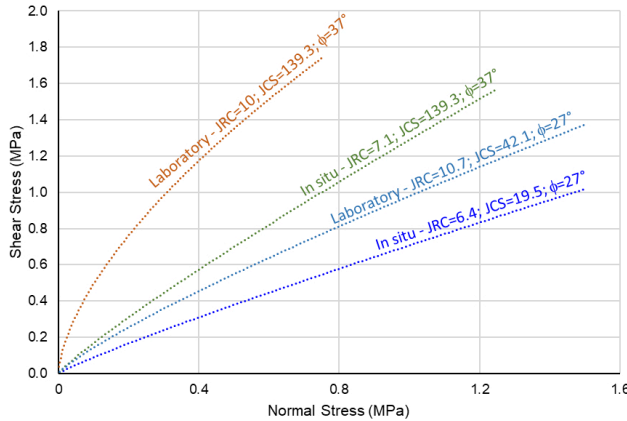


Figure 6. Strength envelopes by the Barton-Bandis model for laboratory and field parameters.

When employing field-derived parameters estimated through correlation with laboratory values, the trajectories become even more pronounced, as depicted in Figure 6.

As observed in Figure 6, the field trajectories exhibit a shallower slope when compared to the laboratory curves. As previously discussed, the smaller the tested specimen, the greater the influence of joint roughness on its shear resistance. Furthermore, it is evident from the trajectories of the intact rock, both for the laboratory (orange) and field (green), that the JRC value exerts a significant influence on the stress trajectories.

6.2 Deterministic stability analyses

Deterministic stability analyses were conducted using the Barton-Bandis criterion, considering the geometry and parameters defined in Section 5. The simulations were executed with Rocscience Slide2 software, employing the Spencer and Morgenstern-Price (GLE) methods for non-

Table 5. Safety Factors – *SF*.

Scenarios	Spencer	GLE
Scenario 1	2.82	2.80
Scenario 2	1.94	1.93
Scenario 3	1.74	1.74
Scenario 4	1.58	1.57
Scenario 5	1.46	1.45
Scenario 6	2.76	2.74
Scenario 7	1.89	1.88
Scenario 8	1.69	1.69
Scenario 9	1.53	1.52
Scenario 10	1.41	1.41

circular surfaces. The results obtained from the stability analyses are presented in Table 5.

From the results presented in Table 5, a considerable reduction in the Safety Factor (*SF*) can be observed with an increasing level of alteration, for field-derived parameters.

Considering the field scenarios, the percentage reduction in *SF* becomes even more substantial, with the difference between Scenario 1 and Scenario 5 being approximately 48% and between Scenario 6 and Scenario 10, 49%.

It is important to highlight that between scenarios that used parameters for intact joints (Scenario 1 and 6) and those that used parameters for slightly altered joints (Scenario 2 and 7), there was a reduction in *SF* of approximately 31% for field scenarios. This significant decrease demonstrates that even for low levels of alteration, the rock mass strength can be considerably affected.

Regarding the water level in the sections, a reduction in *SF* ranging from 2% to 4% is observed for all scenarios considered in this study.

It should be noted that for field scenarios, joint alteration reduced the safety factors to values below 1.5, a threshold that may not be acceptable depending on the type of construction, as per Brazilian standards.

Furthermore, Lopes (2019) demonstrated that joints subjected to 8300 hours of accelerated percolation, according to Salles (2013), exhibit characteristics similar to those of naturally altered joints over a period of 10 years. Thus, a reduction of approximately 50% in *SF* for naturally altered samples over a decade represents a very significant decrease in strength, potentially leading to the failure of these altered slopes.

6.3 Probabilistic analyses

This chapter presents the results obtained from probabilistic analyses conducted using the First Order Second Moment (FOSM) and Monte Carlo methods. The equilibrium limit methods and configurations used for deterministic analyses remained consistent; therefore, simulations were executed using the Spencer and Morgenstern-Price (GLE) methods for non-circular surfaces, employing the Barton-Bandis model.

6.3.1 FOSM analyses

As discussed in the literature review, the First Order Second Moment (FOSM) method allows for the determination of the standard deviation of the SF (Safety Factor) function and its corresponding reliability index (β), thereby obtaining failure probability (Pr). To achieve this, the method calculates the SF variance by varying each parameter while keeping the others fixed.

Following Dell'Avanzi (1995) recommendation, a $\pm 10\%$ variation was employed for each parameter value. For scenarios without a water table, four parameters were considered variable: JCS , JRC , ϕ and γ . In contrast, for scenarios with a water table, five parameters were considered variable: JCS , JRC , ϕ , γ and *water level*.

In accordance with Vecchi (2018, 2023), the fixation or lack thereof of the critical failure surface has a relatively minor impact on failure probability in a problem analyzed by FOSM. Therefore, this study adopted the assumption of a free failure surface (without fixation).

From the SF values obtained for each parameter variation, the variance and standard deviation of the safety factor were calculated for each scenario, yielding the values of β and Pr . Table 6 presents the results obtained from the

FOSM analyses for each of the scenarios, using the Spencer and GLE methods.

It is evident that for both the Spencer and Morgenstern-Price methods, for field scenarios considering parameters for intact joints (Scenarios 1 and 6) exhibit failure probability close to 5%.

The failure probability increases as the level of alteration increases and a significant increase in the failure probability can be observed between Scenarios 1 and 5, as well as between Scenarios 6 and 10.

As aforementioned, the FOSM method allows for the quantification of the relative contribution of each parameter to the failure probability calculated by the method. This application provides engineers with greater sensitivity to the problem and the most influential parameters, thereby guiding more effective engineering solutions.

Table 7 provides a summary of the relative contribution of each random variable to the variance of SF for each of the scenarios analyzed using the FOSM method, specifically for the Morgenstern-Price method.

From the results presented in Table 7, it can be observed that JRC is the parameter with the greatest influence on SF for the configuration analyzed in this study, followed by ϕ

Table 6. Results obtained in the FOSM analyses.

Scenarios	Spencer			GLE		
	SF	β	Pr (%)	SF	β	Pr (%)
Scenario 1	2.815	1.66	4.86	2.804	1.63	5.12
Scenario 2	1.939	1.56	5.97	1.931	1.57	5.87
Scenario 3	1.740	1.42	7.75	1.735	1.40	8.14
Scenario 4	1.578	1.27	10.18	1.572	1.26	10.45
Scenario 5	1.459	1.11	13.27	1.452	1.12	13.16
Scenario 6	2.757	1.61	5.33	2.743	1.61	5.41
Scenario 7	1.887	1.52	6.44	1.880	1.52	6.45
Scenario 8	1.691	1.36	8.76	1.685	1.34	8.95
Scenario 9	1.525	1.21	11.25	1.519	1.17	12.12
Scenario 10	1.411	1.02	15.46	1.405	1.02	15.28

Table 7. Percentage influence of each parameter on SF variation – GLE.

Scenario	JCS	JRC	γ	ϕ	<i>Water Level</i>
Scenario 1	0.1	82.6	0.1	17.2	-
Scenario 2	0.5	78.1	0.1	21.3	-
Scenario 3	0.7	77.2	0.1	22.0	-
Scenario 4	1.6	75.0	0.1	23.3	-
Scenario 5	3.4	71.3	0.2	25.1	-
Scenario 6	0.1	82.7	0.1	17.1	0.0
Scenario 7	0.4	78.4	0.0	21.1	0.1
Scenario 8	0.6	78.0	0.0	21.3	0.1
Scenario 9	1.7	75.5	0.0	22.7	0.1
Scenario 10	3.6	72.9	0.0	23.4	0.1

Table 8. Results from the analyses conducted using the Monte Carlo method. – GLE.

Scenario	SF_{det}	SF_{avg}	RI_{normal}	$RI_{lognormal}$	Pr (%)
Scenario 1	2.804	2.857	2.377	3.774	0.0
Scenario 2	1.931	2.044	1.720	2.314	0.4
Scenario 3	1.735	1.825	1.561	1.978	1.8
Scenario 4	1.572	1.637	1.401	1.672	3.6
Scenario 5	1.452	1.509	1.288	1.469	7.2
Scenario 6	2.743	2.800	2.338	3.678	0.0
Scenario 7	1.880	1.996	1.675	2.224	0.6
Scenario 8	1.685	1.779	1.504	1.877	2.3
Scenario 9	1.519	1.587	1.330	1.556	5.3
Scenario 10	1.405	1.463	1.213	1.354	9.9

and JCS . For the modeled problem, unit weight and water level exhibited no significant influence on the variance of SF .

Furthermore, it is noticeable that as the level of alteration increases, the influence of JRC decreases, while that of ϕ and JCS increases. This phenomenon aligns with the information presented in Figure 5, where a variation of JRC on the order of 3.3 (green and red curves) appears to have generated a greater reduction in strength than a variation on the order of 10.2 (blue and black curves).

6.3.2 Monte Carlo method analyses

Due to the significant computational effort required by the Monte Carlo method, this study opted to utilize Rocscience Slide2 software, which already incorporates the method in its statistical analysis options.

According to Silva (2015), in a normal distribution, approximately 99% of the sampling is covered when considering three standard deviations for each parameter. However, based on the mentioned value, the JRC value would become negative or exceed 20. Therefore, this study adopted the use of two standard deviations for each parameter. For Field Scenarios 5 and 10, only one standard deviation was employed to ensure that the JCS value did not become negative. Similar to the FOSM method, the assumption of a free failure surface, with no fixation, was maintained. The “overall slope” option was used, conducting the search for all possible probabilistic surfaces.

Table 8 presents the results obtained from the Monte Carlo method analyses for each of the field scenarios, using the Morgenstern-Price method.

As presented in Table 8, the failure probability for Scenarios 1 and 6 is zero and increases as the level of alteration rises, reaching 7.2% for Scenario 5 and 9.9% for Scenario 10.

The reliability index values, and failure probability obtained by the Spencer and Morgenstern-Price methods

are consistent with each other for both probabilistic analysis methods.

Generally, the reliability index obtained by FOSM has values lower than those obtained by the Monte Carlo method. However, it can be observed that the magnitude of values obtained by FOSM is generally consistent with those obtained by Monte Carlo, especially when compared to the RI obtained through normal distribution.

Regarding the failure probability, the FOSM method yielded values higher than those obtained by Monte Carlo, which was expected as the latter method is more rigorous. For Field Scenarios 1 and 6, the failure probability calculated by FOSM was approximately 5%, while for Monte Carlo, it was zero. For Scenarios 5 and 10 (more altered), the difference in the PR percentages was also on the order of 5%, with values ranging from approximately 13% to 15% for FOSM and 7% to 10% for Monte Carlo.

Despite the differences, the use of the FOSM method for probabilistic analysis can be considered more interesting than Monte Carlo, as the associated error is not very high, and it allows engineers to identify which random variables have the greatest influence on the studied problem, thereby providing better guidance for engineering solutions.

7. Conclusion

The assessment of the alteration of rock materials in the study of geomechanical behavior of rock masses is of utmost importance and necessity. This work aimed to evaluate the influence of the alteration of gneiss joints on the stability and safety of the rock slopes at the Simplicio Hydroelectric Power Plant.

Regarding the results obtained in the parameterization of gneiss joints at different levels of alteration, the main conclusions are as follows:

- A trend of reducing the friction angle with an increase in the level of joint alteration and an increase in result

- dispersion for more significant alteration levels was observed.
- ii. For the peak friction angle, there is a less significant reduction trend, but the increased result dispersion for more altered samples led to an increase in the average peak friction angle for samples with a higher level of alteration.
 - iii. The JRC value obtained for each level of alteration underwent a slight initial reduction with a significant increase for moderately altered joints, along with increased dispersion.
 - iv. The dispersion of results is associated with the number of samples tested by direct shear, considered below the ideal.
 - v. Peak shear stress values are higher than ultimate shear stress values. This difference mainly arises due to the influence of joint roughness on joint strength at the onset of shearing. After the peak strength, the influence of roughness is less significant, as the irregularities rupture, reducing the shear strength of the samples.
 - vi. For field parameters obtained using the Barton & Choubey (1977) Method, the stress trajectories exhibit less inclination when compared to laboratory curves. This is a result of the sample size, as larger samples experience less influence from joint roughness on shear strength.
 - vii. The stress trajectories for intact and altered rock substantiate the significant influence of JRC values.

As for the results of deterministic analyses, the following conclusions can be drawn:

- i. Safety Factor (SF) undergoes a considerable reduction with an increase in the level of alteration.
- ii. For field scenarios, the percentage reduction in SF is even greater, reaching 50% when comparing scenarios with intact joints to the scenario with the most altered joints.
- iii. Considering the water level in the sections led to a reduction in SF ranging from 2% to 4% for all scenarios in this study.
- iv. For field scenarios, joint alteration reduced the safety factors to values below 1.5.
- v. The results indicate a reduction of approximately 50% in SF for naturally altered samples over a decade, representing a very significant decrease in strength, which can lead to the failure of altered slopes.

The results obtained from the probabilistic analyses allow us to conclude that:

- i. For scenarios considering intact joint parameters, the FOSM method provided a failure probability close to 5% for field scenarios.
- ii. With the FOSM method, the failure probability increases as the level of alteration of the samples increases.
- iii. In the FOSM method, JRC is the parameter with the most influence on the SF for the configuration

analyzed in this research, followed by the friction angle and JCS . Unit weight and water level have no significant influence on SF variance.

- iv. In the FOSM method, the influence of JRC decreases with an increase in the level of alteration, while the influence of the friction angle and JCS increases.
- v. Both FOSM and Monte Carlo methods indicate a significant increase in the failure probability between field scenarios with intact joints and moderately altered joints.
- vi. The reliability index and failure probability obtained with the Spencer and Morgenstern-Price methods are convergent.
- vii. The reliability index obtained by FOSM has values lower than those obtained by the Monte Carlo method, but the values are of the same order of magnitude, especially when compared to the reliability index obtained through normal distribution.
- viii. Regarding the failure probability, the FOSM method provides values higher than those of Monte Carlo.
- ix. For scenarios with intact joints, the failure probability calculated by FOSM is 5%, while for Monte Carlo, it is zero. For moderately altered scenarios, the difference between the PR percentages is also 5%, with values ranging from approximately 13% to 15% for FOSM and 7% to 10% for Monte Carlo.
- x. The use of the FOSM method for probabilistic analysis can be considered more advantageous than Monte Carlo, as its associated error is not very high, and it allows engineers to identify which random variables have a greater influence on the studied problem, leading to more precise engineering solutions.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have evaluated the paper contents in detail and there are no financial advantages and no conflicts of interest to declare.

Authors' contributions

Patrícia Martins Lopes: conceptualization, data curation, formal analysis, investigation, software, writing - original draft. Alberto de Sampaio Ferraz Jardim Sayão: methodology, supervision, validation, visualization, writing - review & editing. Anna Laura Lopes da Silva Nunes: data curation,

methodology, resources, supervision, validation, visualization, writing – review & editing.

Data availability

The datasets generated analyzed in the current study are available by the corresponding author upon request.

List of symbols

c_j	Cohesion intercept of the joint
c	Cohesion intercept
i	Average angle of deviation of particle displacements from the direction of the applied shear stress.
JCS	Joint Compressive Strength
JCS_n	JCS extrapolated to in situ sample size
JCS_o	JCS measured in the laboratory
JRC	Joint Roughness Coefficient
JRC_n	JRC extrapolated to in situ sample size
JRC_o	JRC measured in the laboratory
L_n	Joint in situ length
L_o	Joint length in laboratory
R	Rebound number
γ	Dry unit weight of rock
σ_n	Effective normal stress
σ_C	Uniaxial compression strength
τ	Peak shear strength
ϕ_b	Basic friction angle
ϕ_r	Residual friction angle
ϕ	Friction angle

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Experimental and numerical analysis of instrumented caissons in tropical soil

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Case Study

Keywords:

Caisson
Numerical analysis
Load-settlement
Static load test

Abstract

This paper presents an experimental and numerical study on the behavior of three (3) instrumented caissons in a tropical laterite soil in the Planalto Central, DF, Brazil. The concrete caissons have a shaft diameter of 0.6 m, a bell diameter of 1.2 m and 1.3 m and lengths of 8.9 to 14.4 m. Static load tests were conducted out to investigate the behavior of the caissons. The results of load vs displacement and their interactions with the tropical soil were measured and evaluated using the methodology of Camapum de Carvalho et al. (2008, 2010) and numerical modeling with back-calculation of static load tests in the Plaxis 3D software. The paper presents the analysis of the load transfer lines and the comparison with the proposed methods. The results pointed to a constant load transfer along the shaft, with a significant contribution from the skin friction, developed for a fraction of the displacement necessary for its mobilization. The accuracy of using the Camapum de Carvalho et al. methodology stands out (2008, 2010), identifying the region dominated by skin friction, tip and plasticization of the foundation, confirming the authors' predictions for a tropical soil in the Distrito Federal/DF, Brazil. The numerical method using the Plaxis 3D software, proves that for the limit test load there was a significant contribution from skin friction, which must be consider in the analysis of the behaviour of caissons.

1. Introduction

In foundation engineering, when there is a need to design a construction with the transmission of high loads to the substrate, it is necessary to use deep foundations such as displacement piles, bored piles, caissons, and micro-piles. These types of piles are current standards for deep foundations, especially when to support high loads. The caisson is a special type of deep foundation, with a load capacity increase system obtained through the widening of the bell and one of the most important aspects is the understanding of its behavior when subjected to high compression loads. According to Berberian (2017), in Brazil caisson is used in many works, mainly due to aspects of geotechnical performance, as it is an alternative foundation with the possibility of using light equipment, and in porous soil it is easy to excavate and stable in vertical cuts. Such advantages are possible due to the stability of the excavations and the absence of ground water. However, even with the significant amount of work carried out on caissons, there is little studies on behavior through instrumented static load tests and numerical modeling. This occurs due to the peculiarity of this type of foundation, since the mobilization of skin friction is developed from small displacements, which

can cause an excess of load on the shaft before this load can be transferred to the bell.

According to Brown et al. (2018); Das (2016); Herrmann et al. (2013), Murthy (2002); Bowles (1997) and Johnson et al. (1981), the execution of foundations in caissons has the main advantages: greater load capacity concentrated in a single caisson; enable changes to design criteria during excavation, depending on the type of soil found; possibility of direct inspection of the bearing capacity of the soil and the foundation where the base will be laid; easy construction in cohesive soils and rock mass, without deflection of the shaft; suitability in distinct types of soil; visual inspection of the shaft material so that the nature of the substrate can be confirmed; possibility of extremely high resistance; ease of increasing the shaft to allow greater bending moments; low noise and vibration level; reduction in the number of piles excavated under concentrated loads and can be easily adjusted to accommodate variable conditions encountered during its execution.

To understand the compression behavior of caissons built in a tropical laterite soil in the Planalto Central, DF, Brazil, based on static load tests, Camapum de Carvalho et al. (2008, 2010) proposed a methodology considering the characteristic

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behavior of each soil and its interaction mechanisms between the foundation elements. It is a valuable tool for analyzing load tests, without instrumentation along the shaft. Silva (2011) and Castro Neto et al. (2019), applied the aforementioned methodology to tropical soil, validating the predictions obtained from the method. For numerical simulations in the Plaxis 3D software, the initial model validation procedure was based on back-calculation, based on empirical correlations of soil parameters, of data obtained from the Standard Penetration Test (SPT), making a variation of these parameters in the input data of the software, until the load vs displacement curve shows the same behavior as the static load tests.

According to Albuquerque (2001), it is difficult to understand and complex to understand the load distribution mechanism in foundations. According to the author, the evaluation of the tip load and distribution of skin friction along the shaft, the characteristics of the soil where the foundations will be built, the changes in the initial conditions caused by their execution, the complicated behavior of the pile-soil assembly, difficult to numerical or analytical modeling, support the need to use static load tests in foundations.

In caisson, static load tests aim to prove the element's ability to resist the axial load for which it was designed and to obtain detailed information on the settlements, to obtain values of skin friction resistance and bell resistance. In Brazil, static load tests are standardized by NBR 16903 (ABNT, 2020a), replacing NBR 12131 (ABNT, 2006) which can be applied to all types of piles, both vertical and inclined, regardless of the execution process and installation on the ground.

2. Experimental study

In the Planalto Central, DF, Brazil region there is a notable predominance of residual porous lateritic silty clays, overlying a rocky mantle made up of quartzites, slates, phyllites and siltstones. The texture and mineralogical composition of these soils are a function of the underlying rock and weathering processes, such as laterization.

At the study site, twenty (20) Standard Penetration Test (SPT), were carried out, in accordance with NBR 6484 (ABNT, 2020b), to characterize the subsoil conditions. The soil stratigraphy is shown in Figure 1. A layer of red sandy clay (lateritic soil), very soft to soft, with a thickness of up to 5 m and N_{SPT} between 2 and 7 blows covers a large deposit of silty sand, with N_{SPT} values of up to 40 blows. The final layer is characterized by silty sand, interspersed with sandy silt, compact to very compact and N_{SPT} values greater than 45 blows (residual soil). The ground water was found at an average depth of 13 m below the surface.

For the experimental study, three (3) static load tests were carried out, identified as PC05, PC10 and PC11, in caisson with a shaft diameter of 0.6 m and bases of 1.2 m (PC10 and PC11) and 1.3 m (PC05).

For execution, a drill on a truck was used to excavate the shaft and widen the base manually. The concrete used

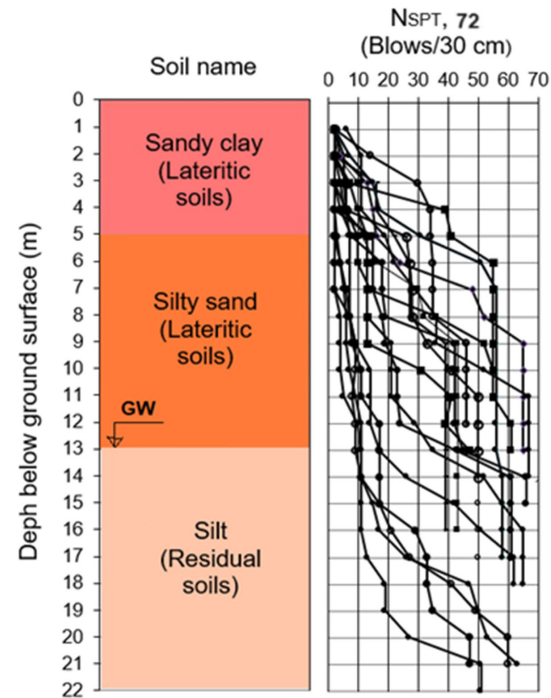


Figure 1. Soil profile of the study area.

for filling was 35 MPa. Figure 2 shows the distribution of static load tests in the study site, the depth at which the caissons were executed, and Figure 3 presents details of the type test caisson.

The reaction system included an iron beam anchored in two reaction caissons, with a shaft diameter of 1.0 m, without enlarging the bell, 2.65 m apart from axis to axis of the tested caisson, as shown in Figure 3. The lengths varied between 8 and 18 m, filled with 35 MPa concrete.

The two reaction caissons have INCOTEP brand anchors, type INCO 90D, throughout their entire length that function as traction elements, with the capacity to resist at least 50% of the maximum load predicted for the static load test, with a minimum distance between the reaction caissons, the system of caissons and tested caisson with three times (3x) the diameter of the largest cross-section, according to NBR 16903 (ABNT, 2020a).

To apply the load, a hydraulic jack with a capacity of 2000 kN was used. The applied load was read using a load cell with a capacity of 2000 kN. To read the displacements, six deflectometers were used, four at the top, diametrically opposed, and two on the sides, as shown in Figure 4.

The static load tests were carried out with reading loading (SML – Slow Maintained Load), in accordance with NBR 16903 (ABNT, 2020a). Albuquerque (2001) mentions in his paper that, even though the slow test (SML) requires a lot of time for its execution, the resulting load-displacement curve more adequately represents the behavior of the foundation at a given loading stage. For Velloso and Lopes (2002), the

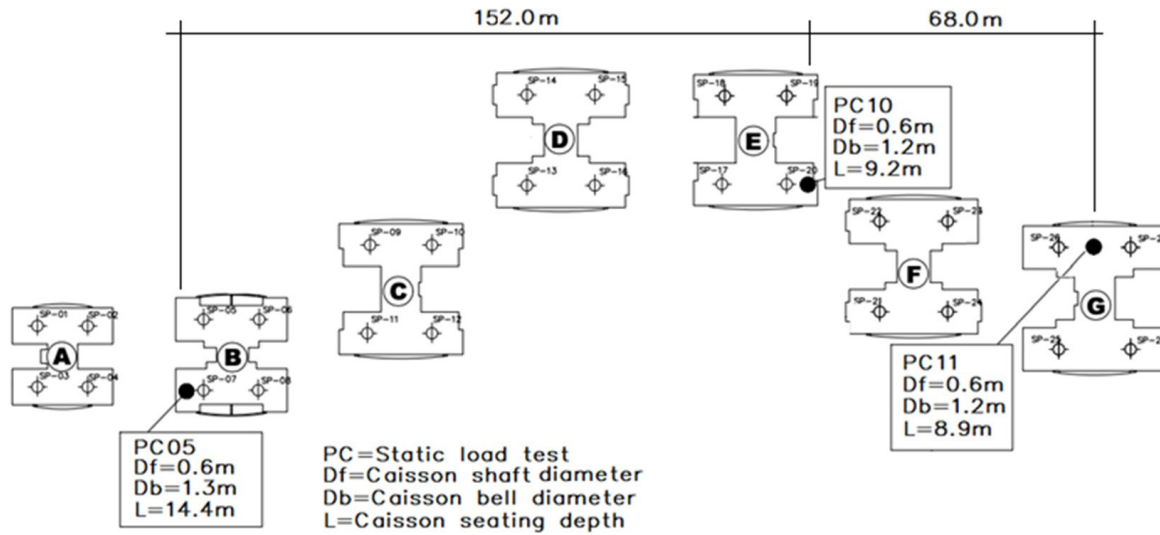


Figure 2. Distribution of static load tests at the study site.

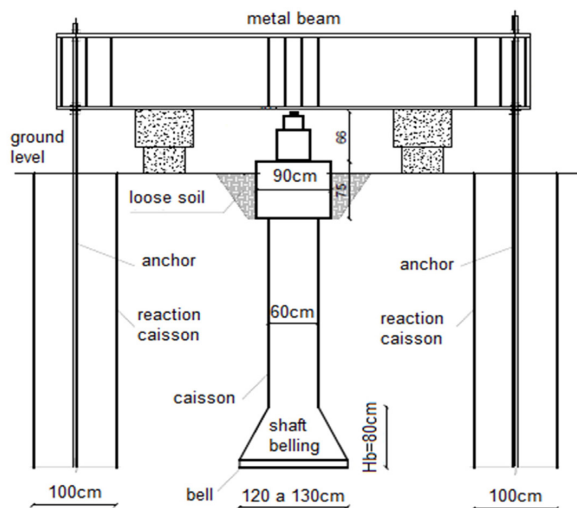


Figure 3. Layout of the static load test reaction system.

slow load (SML) is the one that best approximates the load to which the pile will be subjected under the future structure. According to the authors, when the soil is sheared quickly, it presents lower deformations and greater resistance due to its viscosity. As a result, longer loading stages, a lower loading speed leads to greater settlements and lower load capacities. Table 1 presents data on the static load tests performed.

The instrumentation of the caissons was carried out using a set of CA-50 steel bars, at different levels along the depth. The installed instrumentation consisted of electrical resistance strain gauges type KFG2 -120 – D16 -11 from Kyowa Eletronic Instruments CO. Ltd. glued to CA-50 steel bars, 1/2" in diameter and 0.4 m long and installed in pairs at the same level, in a diametrically opposite position, with a full bridge type connection between the strain gauges.

The connections between the bars were made with a threaded system at the end of each bar alongside steel sleeves. To ensure the centralization of instrumentation in the caissons, steel centralizers were used at the tip of the caissons. To connect the instrumented bars to the reader unit, a 4 x 26 AWG electrical cable was used. The strain gauges were protected against humidity and mechanical shocks using electrical insulation resin, silicone adhesive and adhesive tape. After placing the instrumented bar, concreting was carried out to consolidate the instrumented bar-caisson system. The instrumented bars were calibrated at UNICAMP, and this calibration aimed to make it possible to correct any differences in deformation readings and possible installation errors. After calibration, the bars were, during installation, joined using threads to form a continuous bar.

For the PC05 static load test, the levels were 0.2 m (reference section), 9.1 m and 13.8 m (located in the shaft) and 14.4 m (at the bell). For PC10, the levels were 1.0 m (reference section); 2.4 m; 5.1 m and 8.8 m (located in the shaft) and 9.2 m (at the bell). For PC11, the levels were 1.0 m (reference section); 2.3 m and 5.2 m (located in the shaft) and 8.9 m (at the bell).

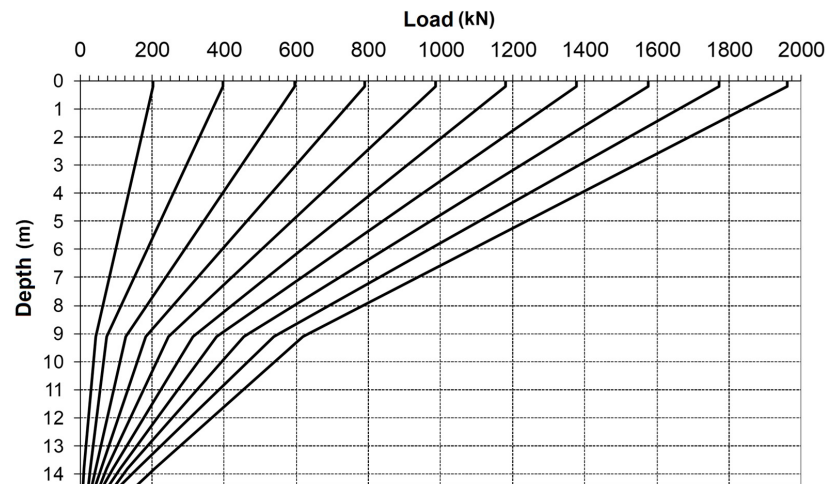
3. Analysis and results

Figure 5 to Figure 7 presents the results obtained by the instrumentation during the static load tests. To analyze the instrumentation, the modulus of elasticity obtained in the reference section of the caisson (extensometer positioned at the top of the caisson) was adopted, using stress-strain curves according to deformation ranges and the Fellenius (2023) methodology.

For PC05, with a total load of 1964 kN, a maximum load of 1804 kN was transferred to the instrumented shaft,

Table 1. Data on the static load tests performed on caissons.

Static load test	Caisson			Working load (according to the project) (kN)	Maximum load test (kN)	Total displacement (mm)
	Depth (m)	Shaft (cm)	Bell (cm)			
PC5	14.4	60	130	982	1963	6.29
PC10	9.2	60	120	981	1962	6.97
PC11	8.9	60	120	982	1964	11.47

**Figure 4.** a) general view; b) detailed view.**Figure 5.** Load transfer along the shaft and bell of the caisson – PC05.

equivalent to 91.9%, the difference being 8.1% absorbed by the bell. The mobilization of the bell of the caisson in the first loading (202 kN) is equivalent to 4.4% of the total load applied in that stage. For the other load stages, the values were in the order of 5.6% to 6.5%, reaching a maximum value of 8.1% in the last stage.

In the case of the PC10 load test, the total load was 1992 kN, transferring a maximum load of 1496 kN to the instrumented shaft, equivalent to 76.2%. The load measured at a depth of 9.2 m was 467 kN equivalent to 23.8%. There was mobilization of the bell of the caisson from the first loading

(199 kN) with values of the order of 14 kN equivalent to 7.0%, reaching a maximum value of 23.8% in the last stage.

For PC11, the total load was 1964 kN, transferring a maximum load of 722 kN to the instrumented shaft, equivalent to 36.8%. The load measured at a depth of 8.9 m was 1242 kN equivalent to 63.2%. There was mobilization of the bell of the caisson from the first loading (204 kN) with values of around 48 kN equivalent to 23.8%, reaching a maximum value of 63.2% in the last stage.

To analyze the instrumented static load tests, the finite element Plaxis 3D software was used, developed for

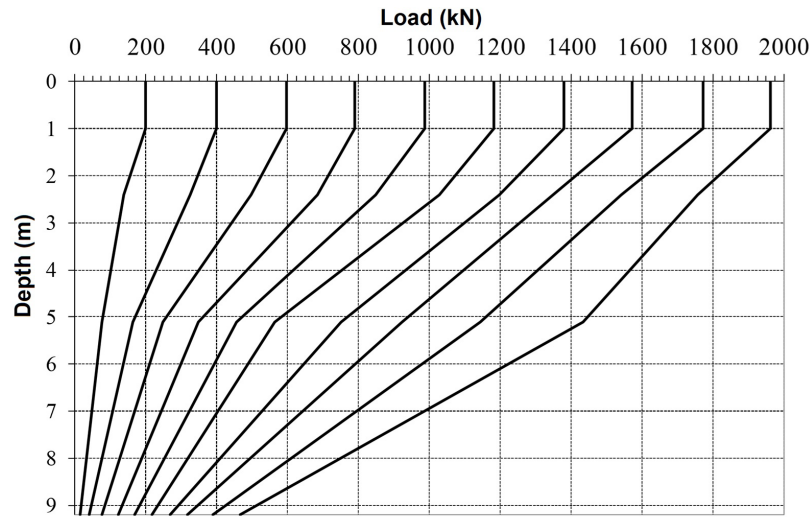


Figure 6. Load transfer along the shaft and bell of the caisson – PC10.

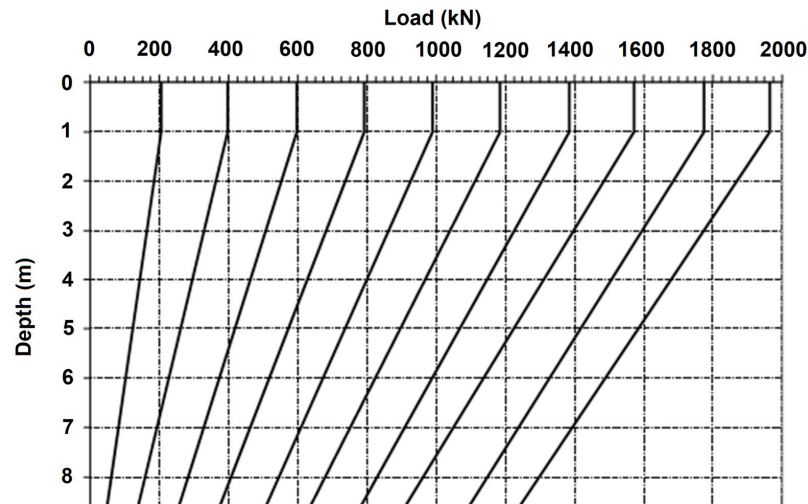


Figure 7. Load transfer along the shaft and bell of the caisson – PC11.

analyzing deformations, stability, and ground water level in geotechnical engineering. For the numerical simulation, the perfectly plastic Mohr-Coulomb linear elastic model was used, with the main input parameters: elastic modulus (E), Poisson's ratio (ν), friction angle (ϕ), cohesion (c) for soil plasticity and soil specific weight (γ). This Mohr-Coulomb model represents a “first order” approximation of soil or rock behavior. Soil parameters were determined by empirical correlations with N_{SPT} values from Table 2 of Joppert Jr. (2007). These parameters were used as initial input data within the Plaxis 3D software.

Then, back-calculations were carried out, with variation of soil parameters, to arrive at an approximation of the load vs displacement curve presented in the static load tests. For the concrete of the caissons, the linear elastic model was used, which is based on Hooke's Law, of isotropic

elasticity, which involved the input elastic parameters: the modulus of elasticity (E_c) = 35000 MPa, the Poisson's ratio (ν) = 0,20 and the specific weight of the concrete (γ_c) = 25 kN/m³. Table 3 shows the parameters resulting from the back-calculations of the Plaxis 3D software.

Figure 8 to Figure 10 presents the curves vs displacements resulting from the back-calculation.

The load vs displacement curves using the Plaxis 3D software showed conventional rupture displacements, with continued deformations following new load increases. According to NBR 6122 (ABNT, 2022), this behavior of the load vs displacement curve occurs in the following circumstances: when there is a limitation of the reaction load, or the load capacity of the caisson is greater than the applied load and when the caisson is loaded until it presents high displacements without a clear rupture configuration.

Table 2. Average geotechnical parameters of soils (adapted, Joppert Junior, 2007).

Soil	NSPT	E (MPa)	Specific weight		ϕ (°)	c (kPa)
			γ (kN/m ³)	γ_{sat} (kN/m ³)		
Slightly silty sand/ Lightly clayey sand	0-4	20-50	17	18	25	-
	5-8	40-80	18	19	30	-
	9-18	50-100	19	20	32	-
	19-41	80-150	20	21	35	-
	>=41	160-200	20	21	38	-
Medium sand and very clayey fine sand	0-4	20	17	18	25	0
	5-8	40	18	19	28	5
	9-18	50	19	20	30	7.5
	19-41	100	20	21	32	10
Red and yellow porous clay	0-2	2-5	15	17	20	7.5
	3-5	5-10	16	17	23	15
	6-10	10-20	17	18	25	30
	>=10	20-30	18	19	25	30 to 70
Slightly sandy silty clay (tertiary soil)	0-2	1	17	18	20	7.5
	3-5	1-2.5	18	19	23	15
	6-10	2.5-10	19	19	24	20
	11-19	5-10	19	19	24	30
	20-30	30-100	20	20	25	40
	>=30	100-150	20	20	25	50
Lightly silty sandy clay	0-2	5	15	17	15	10
	3-5	5-15	17	18	15	20
	6-10	15-20	18	19	18	35
	11-19	20-35	19	19	20	50
	>=20	35-50	20	20	25	60
Turf/ Organic clay	0-1	0.4-1	11	11	15	5
	2-5	1-5	12	12	15	10
Lightly clayey sandy silt (residual soil)	5-8	8	18	19	25	15
	9-18	10	19	20	26	20
	19-41	150	20	20	27	30
	>=41	200	21	21	28	50

Table 3. Geotechnical design parameters of the caissons resulting from the back-calculations of the Plaxis 3D.

Static load test	Soil type	Material Model	Drainage type	γ (kN/m ³)	γ_{sat} (kN/m ³)	c (kPa)	ϕ (°)	E (MPa)	ν
PC05	Sandy Clay	Mohr-Coulomb	Drained	17	18	22	15	15	0.30
	Compact silty sand	Mohr-Coulomb	Drained	19	19	30	18	18	0.30
	Clayey silt	Mohr-Coulomb	Drained	20	21	40	25	24	0.30
PC10	Sandy Clay	Mohr-Coulomb	Drained	17	18	22	15	15	0.30
	Silty sand	Mohr-Coulomb	Drained	19	19	32	28	40	0.30
	Sandy silt	Mohr-Coulomb	Drained	20	21	45	28	60	0.30
PC11	Sandy Clay	Mohr-Coulomb	Drained	17	18	22	15	15	0.30
	Silty sand	Mohr-Coulomb	Drained	19	19	32	28	34	0.30
	Sandy silt	Mohr-Coulomb	Drained	20	21	40	28	60	0.30
Caisson	Concrete	Linear elastic	Non-porous	25	-	-	-	35000	0.20

Legend: Weight (γ); saturated specific weight (γ_{sat}); Cohesion (c); Soil friction angle (ϕ); Elastic Modulus E ; Poisson's ratio (ν).

In these cases, the interpretation of the caisson load tests configured conventional rupture in all cases. The loading speed, stabilization of displacements and the nature of the terrain are considerations raised that affect the results of load

tests for cases of conventional ruptures as recommended by NBR 12131 (ABNT, 2006).

The small displacements configured at the beginning of loading up to the limit of the static test load demonstrate that

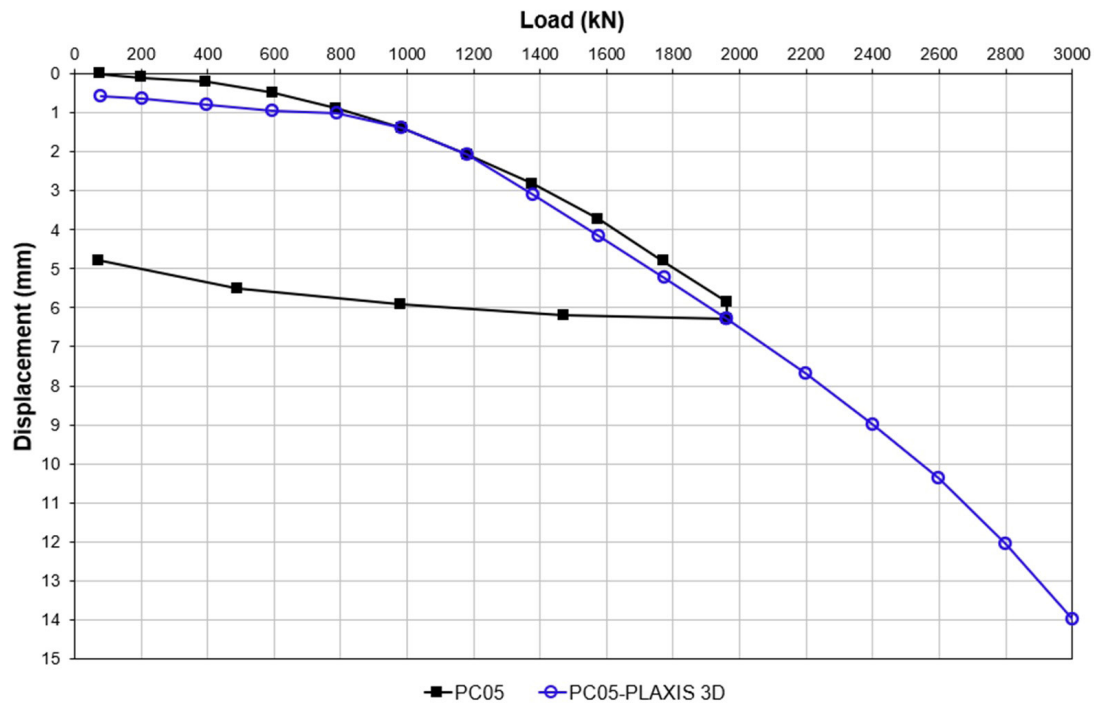


Figure 8. Back-calculation of the load vs displacement curve in the PLAXIS 3D – PC05.

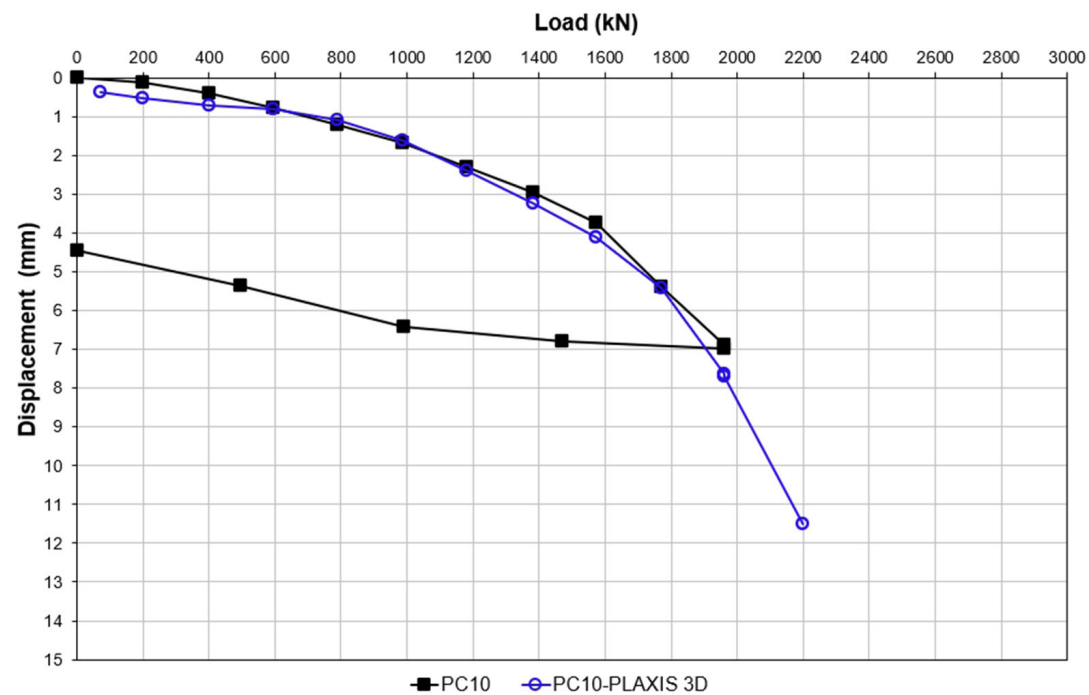


Figure 9. Back-calculation of the load vs displacement curve in the PLAXIS 3D – PC10.

there is a significant load being mobilized by skin friction along the axis of the caissons for PC05, PC10 and PC11. Given that the working load is between 981 kN and 982 kN, the initial displacements that occurred were mobilized by

the skin friction portion at the beginning of the linear-elastic deformation of the displacement curve.

The result of the PC05 instrumentation shows that the length of the caisson has a significant contribution to the

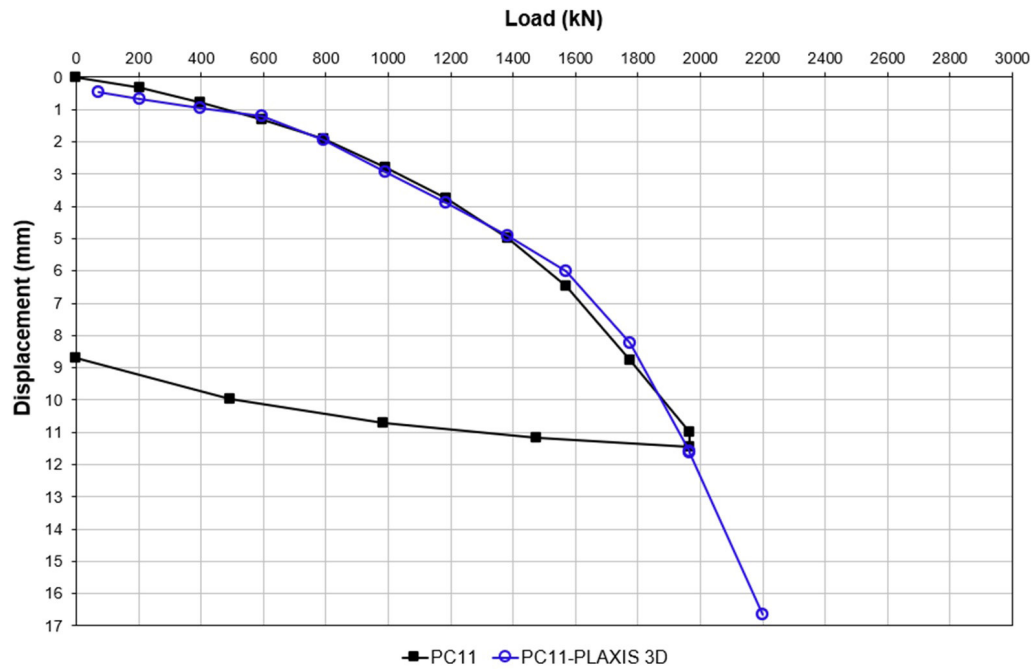


Figure 10. Back-calculation of the load vs displacement curve in the PLAXIS 3D – PC11.

mobilization of skin friction, since a small portion of the load was mobilized at the bell. For PC10 and PC11, the skin friction load had a smaller contribution due to the length of the caissons, however, this portion must be considered when analyzing the behavior of the caissons for tropical soils of the same geological feature.

In the paper of Pereira & Garcia (2023) an extensive study was carried out on the contribution of resistance to skin friction in caissons and confirms that the load capacity of caissons has a significant increase when the portion of resistance to skin friction is considered.

In instrumented static load tests, based on the methodology of Camapum de Carvalho et al. (2008, 2010), soil mobilization at the interface with the foundation and plasticization of the entire complex were evaluated.

The methodology proposed by Camapum de Carvalho et al. (2008, 2010), analyzes the settlement/displacement coefficient (α) plotted in relation to the loads and is based on the laboratory consolidation test. The coefficients are extracted from the graph plotted on a logarithmic scale, in a similar treatment to the consolidation test, for each load, from the linear final part of the settlement curves (mm) as a function of time.

The resulting graph presents three straight segments, where the 1st section defines the beginning of the plastic deformations of the shaft-soil system, that is, the friction domain. The 2nd section is characterized by mobilization of the tip and the 3rd represents the beginning of plastic deformations of the soil at the tip of the pile (Figure 11).

From the results of the static load tests, it was possible to obtain graphs in terms of ultimate settlement, measured as a function of the applied load. Figure 12, Figure 13 and Figure 14 show the accumulated immediate settlements and accumulated settlements due to consolidation and/or secondary, depending on the applied loads, for the load tests. The curves indicate the changes in the behavior of the caissons.

The mobilization of the skin friction of the caisson begins up to the 1st point of intersection, suffering small settlements with the applied load. From this load onwards, the shaft and bell of the coffin begins to work in an integrated manner, with the beginning of the mobilization of the tip, which is preponderant in relation to the load to which the foundation is subjected. The 2nd intersection point with the 3rd section corresponds to the beginning of plastic deformations of the soil at the base of the foundation, however, it does not mean the occurrence of rupture but rather that, from this point onwards, the settlements become more relevant.

Analyzing the graphs presented in Figure 12, Figure 13 and Figure 14, four Intersections are indicated that show the behavior of the foundation, as shown in Table 4.

Intersection 1 – Load with which work on the shaft begins (with few settlements occurring).

Intersection 2 – Value at which the contribution of the shaft and bell set begins in an integrated manner up to the indicated load.

Intersection 3 - In the accumulation and/or secondary settlements accumulated due to the loads applied to the caisson, it was observed that the shaft begins to work with small settlements until close to the indicated load.

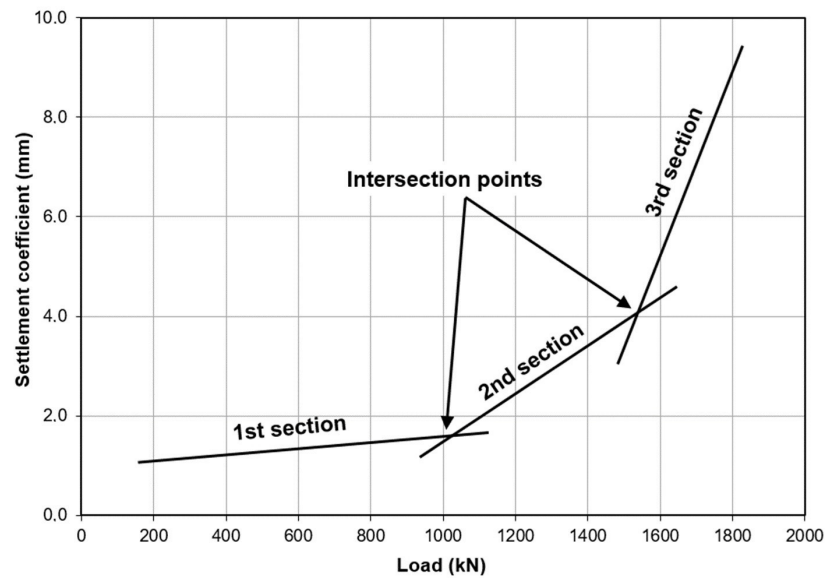


Figure 11. Load vs settlement coefficient (Camapum de Carvalho et al., 2010).

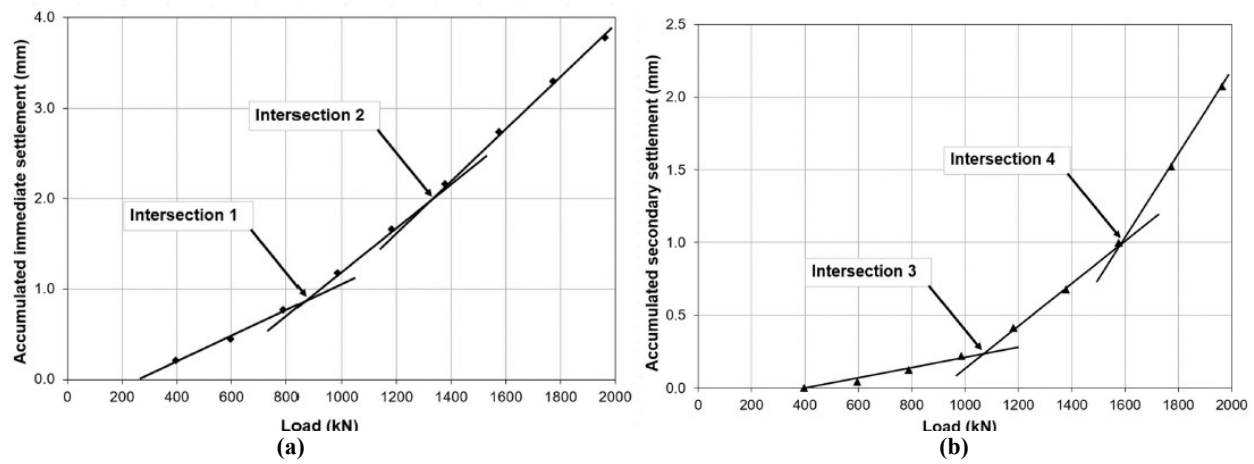


Figure 12. Curves load vs accumulated immediate settlement (a) and load vs accumulated secondary settlement (b) – PC05.

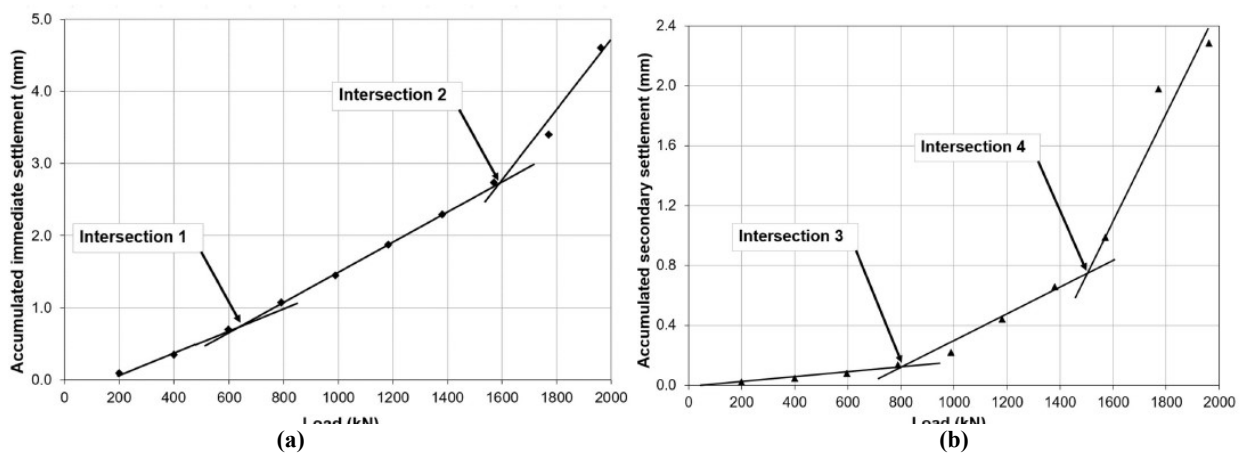
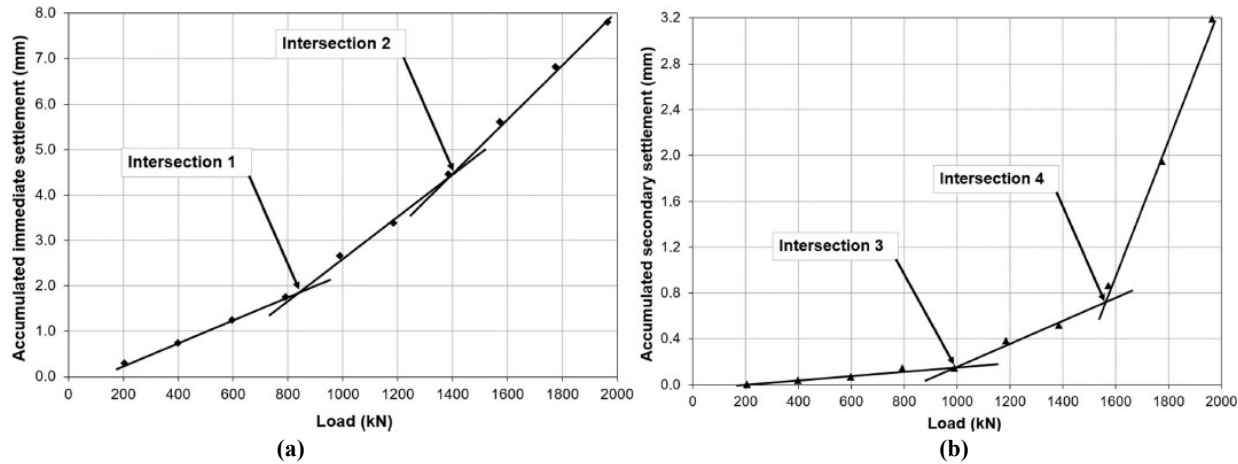
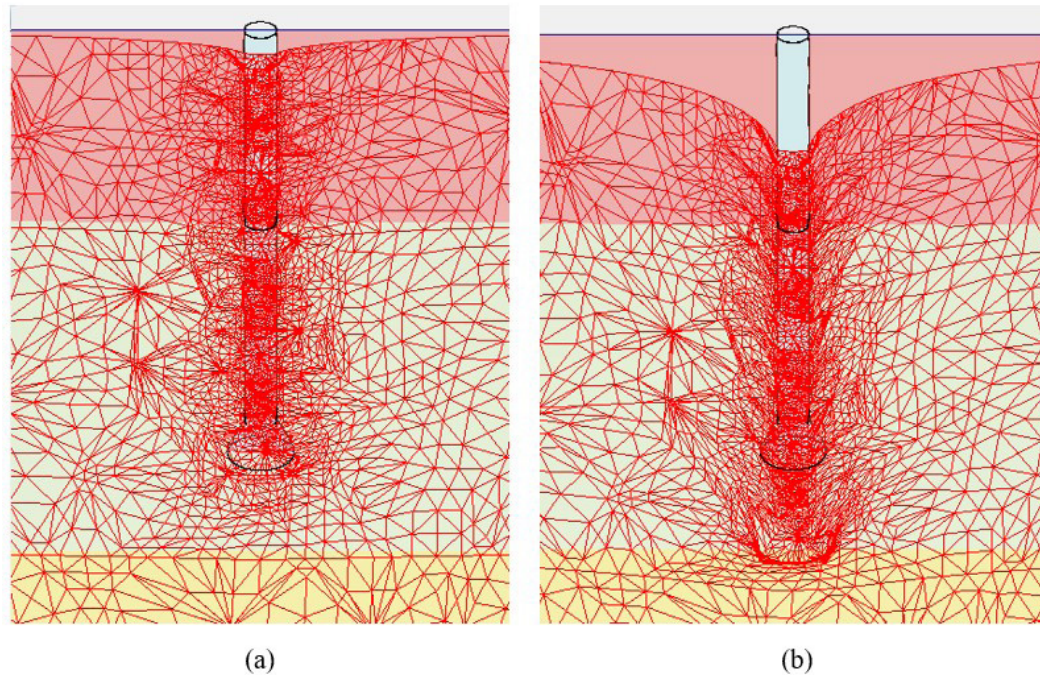


Figure 13. Curves load vs accumulated immediate settlement (a) and load vs accumulated secondary settlement (b) – PC10.

Table 4. Loads identified in the loading phases.

Static load test	Intersection 1	Intersection 2	Intersection 3	Intersection 4
PC 05	870 kN	1380 kN	1070 kN	1580 kN
PC 10	640 kN	1590 kN	800 kN	1510kN
PC 11	840 kN	1410 kN	990 kN	1570 kN

**Figure 14.** Curves load vs accumulated immediate settlement (a) and load vs accumulated secondary settlement (b) – PC11.**Figure 15.** Numerical analysis of loads in Plaxis 3D. a) Mobilization of the shaft; b) shaft+bell mobilization.

Intersection 4 - After the load indicated in Intersection 3, the shaft and bell set compete in an integrated manner until approximately the indicated load.

From the back-calculation that validated the numerical model for analyzing the behavior of the caissons, it was possible to identify when the mobilization of skin friction occurs and

when the mobilization of loads towards the bell of the caissons begins. In Figure 15 it is possible to see the beginning of these mobilizations using Plaxis 3D software. Table 5 shows the loads identified in the loading phases in Plaxis 3D.

To determine the skin friction load and initiate the mobilization of the load in the caisson bell for the Plaxis

Table 5. Loads identified in the loading phases in the Plaxis 3D.

Static load test	Resistance shaft	Resistance shaft+ resistance bell
PC 05	788 kN	1575 kN
PC 10	596 kN	1381 kN
PC 11	597 kN	1185 kN

3D software, the reductions in displacement increments were analyzed with the application of the load in each loading phase. It was noted that from the 3rd loading phase onwards (approximately 596 kN and 597 kN) for load tests PC10 and PC11, the displacements undergo small and constant increments, which indicates the beginning of load mobilization at the bell. For PC05 this only occurs in the 4th loading phase due to the length of the caisson shaft (14.4 m). As the caisson is considered a rigid reinforced concrete element in the simulations, soil mobilization is clear, both in the shaft and at the bell of the caisson, as shown in Figure 15.

The instrumentation results for load tests PC05, PC10 and PC11 indicate that from the 1st load the load mobilization on the bell begins. For PC10 and PC11, total mobilization of the shaft occurs for loads of 1496 kN and 1242 kN respectively, which corresponds to the 2nd point of intersection according to the methodology of Camapum de Carvalho et al. (2008, 2010), where the bell mobilization begins. For PC05, the maximum skin friction load was 1804kN, which indicates that the length of the caisson directly affects the mobilization of skin friction.

In the Plaxis 3D results, the total mobilization of skin friction occurs between the 3rd and 4th loads, with mobilization of the bell starting from the 6th and 7th loads.

Given the results of the instrumented load tests PC05, PC10 and PC11 and Plaxis 3D, the set of analyzes allowed us to verify that the Camapum de Carvalho et al, methodology. (2008, 2010) presents good accuracy, identifying the region dominated by skin friction, bell and plasticization of the foundation.

4. Conclusion

The results of the in-depth instrumentation of the caissons show a significant contribution from the portion of skin friction resistance developed for a fraction of the displacement necessary to mobilize the resistance, which should be considered in the analysis of the behavior of open pit caissons for soils of the same characteristic from the Planalto Central, DF, Brazil.

With the results of the static load tests, it was also observed that increasing the bell by up to two times (2x) the shaft diameter contributes significantly to the increase in resistance or decrease in displacement. The resistance to skin friction has a constant value, with the proportion of load decreasing as the bell increases. This means that in caissons, the rupture of the bell has no practical definition, as the

displacement necessary for their mobilization is excessive and the structures would not support such displacements for their complete mobilization.

When comparing the results obtained by the method of Camapum de Carvalho et al. (2008) and the Plaxis 3D software, it can be stated that the loads are close together, confirming the methodology proposed by numerical methods and the authors' predictions for a tropical soil in the Distrito Federal. Therefore, the method constitutes an important tool for analyzing the loads mobilized along the shaft and bell of the caissons.

It is important to highlight that the analysis of the results is not restricted to determining the load capacity, but goes further, establishing the domains of friction, tip and plastification, information that is difficult to measure and preponderant in the design of foundations, especially in non-static load tests led to failure.

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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

Regina Maria Ferreira: conceptualization, methodology, validation, resources, software, writing – original draft. Paulo Jose Rocha de Albuquerque: formal analysis, data analysis, supervision, validation, writing – review & editing. Neusa Maria Bezerra Mota: formal analysis, methodology, supervision, validation.

Data availability

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

List of symbols and abbreviations

3D	Three-dimensional modeling
<i>c</i>	Cohesion
ABNT	Brazilian Association of Technical Standards
<i>E</i>	Elastic Modulus
<i>E_c</i>	Elastic Modulus of concrete
NBR	Brazilian standard
SPT	Standard Penetration Test
SML	Slow Maintained Load

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γ	Weight
γ_c	Specific weight of the concrete
γ_{sat}	Saturated specific weight
ν	Poisson's ratio
ϕ	Soil friction angle

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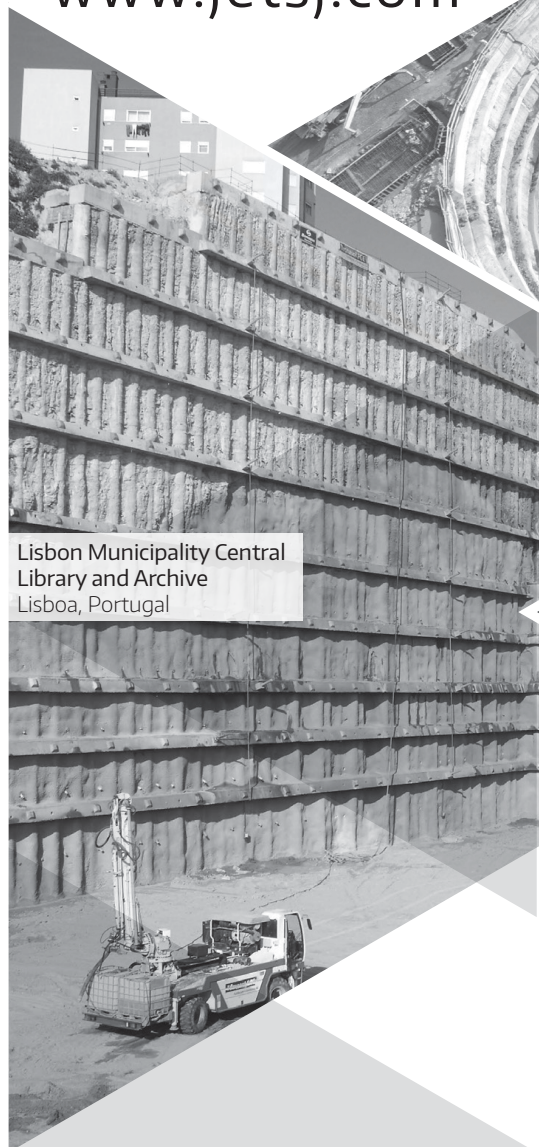
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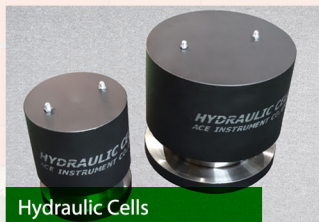
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Figures, tables and equations should be numbered in the sequence that they are mentioned in the text.

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:

<https://csl.mendeley.com/styles/583367631/soilsandrocks-4>

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 1: 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. 19^o 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.