

SOILS and ROCKS

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Special Issue

Deformation Characteristics of Geomaterials: Honorary & Special Lectures

Guest Editors:

Sara Rios, António Viana da Fonseca, Cristiana Ferreira
Universidade do Porto

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EDITORIAL

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Editorial to the Special Issue on “Deformation Characteristics of Geomaterials: Honorary & Special Lectures”

Sara Rios^{1#*} , António Viana da Fonseca^{1*} , Cristiana Ferreira^{1*} 

Editorial

This Special Issue contains the Honorary and Keynote Lectures on Deformation Characteristics of Geomaterials, originally delivered at the 8th International Symposium on Deformation Characteristics of Geomaterials held in Porto, Portugal, from the 3rd to the 6th of September 2023 (IS-Porto 2023).

The Symposium aimed to disseminate ideas and recent research regarding the complex stress-strain behaviour of geomaterials, comprising laboratory methods from very small to large strains; anisotropy and localization; time-dependent response of soils; characteristics of natural, treated and unsaturated geomaterials; applications in field methods; evaluation of field performance in geotechnical structures; and physical and numerical modelling in geomechanics. The Symposium was a 3-day event organized by the Faculty of Engineering of the University of Porto, under the auspices of TC101, the Technical Committee on Laboratory Stress Strain Strength Testing of Geomaterials of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). This symposium was endorsed by the Portuguese Geotechnical Society (SPG), the Faculty of Engineering of the University of Porto (FEUP), with the support of the Research Institute of R&D in Structures and Construction (CONSTRUCT) of the Portuguese Foundation for Science and Technology (FCT). Continuing the legacy established by previous symposia, from Sapporo in 1994 to Glasgow in 2019, IS-Porto 2023 convened over 240 researchers and practitioners representing 37 countries. The symposium facilitated the exchange of knowledge and experiences, stimulating discussions on advancements in testing methodologies as well as constitutive and numerical modelling of geomaterials.

This Special Issue brings together eight articles that discuss, highlight, and demonstrate cutting-edge research and advancements on the mechanics of geomaterials from very small strains to beyond failure. Each contribution offers valuable insights into the characterization, modelling, and analysis of geomaterials, spanning a wide range of scales and methodologies, towards advancing our understanding on earthworks and soil-structure interaction and improving geotechnical engineering practices.

The 7th Bishop Lecture, by Matthew Coop, discusses how the mechanics of coarse-grained geomaterials, at the meso- and micro-scales, reveal the intricate behaviour of soil particles, where localized interactions govern macroscopic responses. Romero examines the study of soils across various scales, from micro-experiments to large-scale mock-ups, providing critical insights into the deformation mechanisms and stress-strain behaviour of geomaterials under diverse conditions. Cotecchia et al. present a two-parts research on micro-to-macro investigations of natural and reconstituted clays, which are essential for informing the constitutive modelling, enabling more accurate predictions of their mechanical behaviour. Similarly, Liu et al. explain how the in-situ and laboratory characterization of stiff and dense geomaterials were applied to the design and analysis of driven piles, ensuring effective load transfer and long-term stability. The study of stress, strain, and force transfer in granular materials, from the intragranular to bulk scale, by Hall, is pivotal for understanding the multi-scale interactions that dictate the overall behaviour of geomaterials, thereby enhancing our capacity to predict soil performance in diverse geotechnical scenarios.

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*Guest Editors



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Emerging technologies and advanced non-invasive techniques are revolutionizing near-surface site characterization, as demonstrated by Abbas et al., offering new ways to investigate subsurface conditions without disturbing the soil. Beyond Earth, Delage et al. discuss how the exploration of Martian soil at the InSight landing site provides a unique opportunity to apply terrestrial soil mechanics in an extraterrestrial context, expanding the frontiers of planetary geotechnics.

These topics were carefully selected to be thought-provoking, with the aim of broadening the scope of the main focus of TC101, the deformation characteristics of geomaterials, and fostering in-depth discussion and collaboration. By addressing diverse aspects of geomaterial behaviour and innovative methodologies for its characterisation, this collection seeks to inspire new perspectives and stimulate networking among researchers and practitioners.

The editors of this special issue would like to gratefully acknowledge the dedication and effort of all the keynote speakers, and the collaboration of the Soils & Rocks Editorial Team in the successful production of this special issue.

LECTURE

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The 7th Bishop Lecture: The mechanics of coarse-grained geomaterials at meso- and micro-scales

Matthew Coop^{1*} 

Lecture

Keywords

Particle mechanics
Laboratory apparatus
Sands
Coarse-grained soils

Abstract

In order to move towards discrete analyses of soil behaviour, we need to develop a new range of apparatus and testing techniques. The lecture describes attempts to develop these new apparatus and presents data for a range of coarse-grained geomaterials at the single particle scale. The roles of the particle morphology and hence geological origin are discussed and are shown to influence both the contact mechanics and particle breakage behaviour. The mechanics of single particles are shown to be more complex than generally assumed and will require new means of modelling to account for the significant plasticity that occurs at particle contacts and patterns of breakage that are strongly influenced by the origins of particles and their effect on their morphology.

1. Introduction

Traditionally, coarse grained soils have been treated as continua using meso-scale laboratory tests such as the triaxial to derive ever more complicated constitutive models as newly discovered aspects of behaviour are included. Some of these are features that are shared with plastic soils, such as creep, rate of loading effects, anisotropy, the influence of stress rotation and of the intermediate principle stress, but others are more clearly related to the coarse grain size. While for a clay the initial depositional density only affects the behaviour at very low stress levels, for coarse grained soils this influence extends up to the onset of particle breakage when compression paths tend to converge to a unique normal compression line in the volumetric plane (Pestana & Whittle, 1995). This particle breakage has a number of consequences, for example that sands tend not to follow Rendulic's principle, the state boundary surface has an asymmetrical shape so that the flow-rule is non-associated and the stress history of a sand and the effect it has on breakage has an important influence on stiffness (e.g. Coop & Lee, 1993; Jovicic & Coop, 1997).

In the volumetric plane the critical state line for sands curves to a horizontal asymptote at low stresses (Verdugo & Ishihara, 1996) the higher gradients at larger stresses generally being attributed to the onset of breakage, even if it may be found in non-plastic soils with no apparent particle damage (e.g. Carrera et al., 2011). However, what seems to be a unique critical state line is affected by prior breakage (Muir Wood, 2008; Bandini & Coop, 2011) and continued shearing to strains well beyond what a triaxial can achieve reveals continued particle breakage (Coop et al., 2004). Some sands, for example when they are fractally graded or have

particles of mixed mineralogies, may even have a transitional mode of behaviour in which the critical state line location in the volumetric plane becomes dependent on the initial density at deposition (e.g. Todisco & Coop, 2019). Added to each of these features of the behaviour of reconstituted sands are the influence of the natural fabric and bonding (Cuccovillo & Coop, 1999).

Considering also the practical difficulties of meso-scale tests on sands (e.g. particle size effects, membrane penetration, sample end restraint, strain localisation) one begins to wonder why we bother at all with continuum approaches. A discrete methodology starts to look attractive, for example using DEM. But often this is done with unrealistic particle shapes, such as spheres, and with unrealistic contact models. The normal loading behaviour at the contact is frequently assumed to be linear or following a non-linear Hertz (1882) model based on the contact of two elastic spheres. In lateral loading, the same linear stiffness as for normal loading is frequently assumed with a frictional sliding cut-off when the inter-particle coefficient of friction, μ , is reached, or at best the Mindlin & Deresiewicz (1953) extension of Hertz is used. A rolling resistance is often adopted to stop excessive rotation of the particles, but this is purely artificial and only needed because of the spheres adopted in simple models. Twisting at the contact is generally ignored and Zhai et al. (2019) have found that its effects are typically small.

Various techniques are available to model more realistic shapes such as potential particles (e.g. Ahmed et al., 2016) or using overlapping spheres (Fellerec & McDowell, 2010), which obviate the need for a rolling resistance, but it is still often the case that the input contact parameters are derived by curve fitting the DEM analyses to meso-scale laboratory

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test data (Ahmed et al., 2016). If we are to adopt a truly “micro” approach, the use of DEM should be accompanied by micro-scale tests to derive directly the particles shapes, their contact fabric and most importantly their contact mechanics as well as gaining an understanding of the occurrence and modes of particle damage. These will each require new techniques and the focus of this lecture will be the use of these new apparatus to examine the contact behaviour and damage of single particles.

2. Development of new apparatus

Previous work on the micro-mechanics of sand particles has typically used mechanical apparatus to determine inter-particle friction (Horn & Deere, 1962; Skinner, 1969; Procter & Barton, 1974) and particle strength has generally been measured in platen-to-platen loading (e.g. Nakata et al., 1999). Even recent work examining more detailed aspects of the contact mechanics has usually been in relatively simple apparatus examining one aspect of behaviour (e.g. Cole et al., 2010; Cole, 2015; Michalowski et al., 2018). Instead, the approach adopted here has been to develop a universal and computer-controlled apparatus capable of examining all aspects of loading under combined normal and tangential loading from the very small displacement stiffnesses up to failure, the equivalent of the stress path controlled triaxial apparatus for single particle contacts.

The apparatus developed are shown in Figure 1; one for sand sized grains built at the City University of Hong Kong and one for railway ballast or rock fill at University College London. Each has independent three axis control of the displacements and/or loads applied at the contact

between two particles via linear actuators. The forces and displacements in each direction are measured by load cells and non-contacting displacement transducers, with precisions for the displacements to about $\pm 0.01 \mu\text{m}$ and 0.01 N for the load on the larger apparatus which has a 1 kN capacity and about 0.001 N for the smaller one built for 100 N . One particle is mounted on an upper platen attached to the vertical load cell while the other is attached to a base pedestal mounted on a sled on a quasi-frictionless three-point bearing system. The systems were built for high rigidity to minimise any effect of apparatus compliance, especially at small displacements. Two microscope cameras monitor the particle contact from two orthogonal directions.

For the natural sands that have been tested the more spherical particles available have been chosen so that the local radius of curvature at the contact corresponds to that of the particle overall. But for materials such as ballast that are comprised of crushed rock this is not possible and typically a particle corner or point is tested against a nominal flat as in Figure 2. The cameras allow measurement of the local radius of curvature of the point for comparison with theoretical predictions of the contact stiffnesses.

This type of apparatus is suitable for testing frictional uncemented particles. It cannot measure rolling or twisting resistance, but as discussed above the former is largely fictitious for uncemented particles and the latter small. For cemented particles, however, the resistance to combined normal shear and moment loading must be measured and Figure 3 shows an apparatus developed at the City University of Hong Kong.

It is based on the apparatus in Figure 1 but applies the horizontal or shear loads through reaction arms, the height of which may be adjusted to give a moment. These

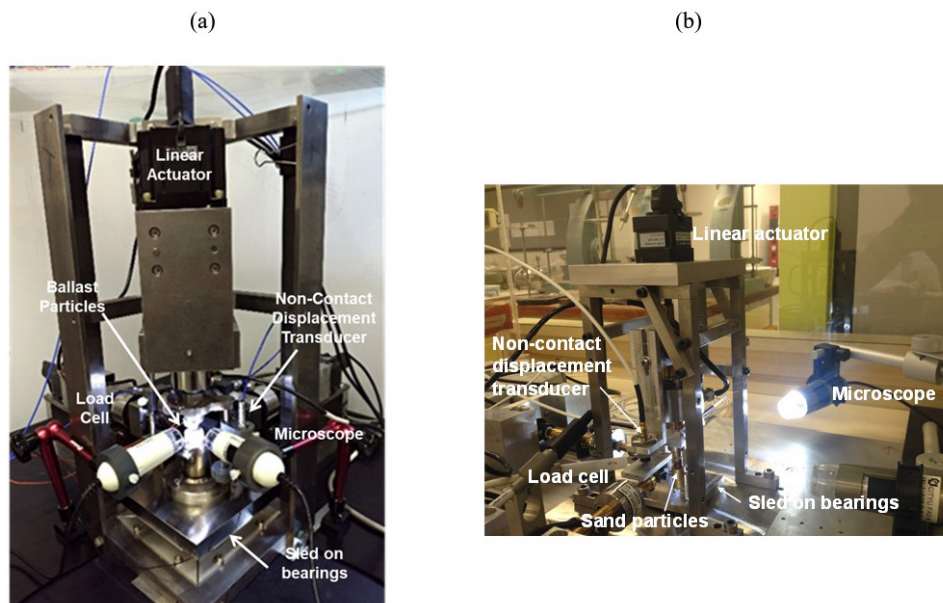


Figure 1. Inter-particle loading apparatus for: (a) ballast/rock fill; (b) sands.

act via a knife edge against L-shaped mounts that transfer the loads to the top and base of the cemented particle pair. The axial loads are applied to the mounts through pin joints. This style of loading system was used by Jiang et al. (2012) to investigate the contact behaviour of bonded artificial particles. The horizontal loading in this case is in 2D only and the displacements are measured by LVDTs, two being mounted one above the other in the horizontal direction so that the shear displacement and rotation can both be resolved.

To gain insight into the mechanisms of particle breakage a simplified version of the loading apparatus was constructed for vertical loading only that enabled high speed imagery of the tests (Figure 4) in which a microscope lens was attached to a high-speed camera.

3. Normal loading behaviour

Selected data for a variety of sands are given in Figure 5a. The Leighton Buzzard sand is a quartz sedimentary sand

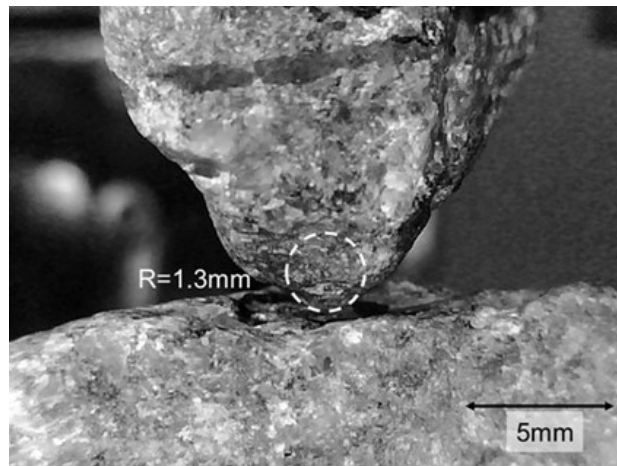


Figure 2. Point to flat contact for a railway ballast (after Wong & Coop, 2023).

from Southeast England. The Philippines carbonate sand is biogenic in origin and the particles tested were selected to be solid coral fragments. The crushed limestone was an artificial sand created by crushing a young limestone from China, while the completely decomposed granite was from Hong Kong, and was created by weathering. The most spherical of the particles was tested in each case so that the sphericities assessed from the Krumbein & Sloss (1963) identification chart were all 0.7–0.8. They were all well rounded with values of roundness of 0.6–0.7 apart from the crushed limestone which had a mean roundness of 0.3 arising from the crushing process.

All the data in Figure 5a are for particle sizes in the range 1.18–2.36 mm and to some extent the differences between the curves result from the variation of sizes. Also important though are any local shape variations at the contact, especially because of the low roundness of the crushed limestone and also the mineralogy that controls the Young's modulus of each material, E . Quartz has an E of around 94–98 GPa while for the calcite of the carbonate sand and crushed limestone $E = 73$ –84 GPa (Mavko et al., 1998; Jaeger et al., 2007). The decomposed granite has a mixed mineralogy mostly of quartz and feldspar, but the main cause of its very soft contact behaviour is the coating of clay on the particles that results from the weathered origin. These large displacements were therefore mostly plastic and were also not generally present when the clay coating was washed off as can be seen in the data labelled WCDG on Figure 5b.

Most of the data on Figure 5b are for a granite railway ballast (Wong & Coop, 2023) supplied from the Mountsorrel quarry in the UK. Because of the very irregular shapes a nominal point was generally tested against a nominal flat, as in Figure 2, the contact radius of the point being derived from the microscope camera images. While there is little direct correlation between the contact radii that are indicated for each test on the figure, it is clear that the load-deflection curves are very much softer than for the contact of smooth spheres of gneiss tested by Cole et al. (2010) and are much closer to the contact behaviour of the much smaller Leighton Buzzard sand from Nardelli & Coop



Figure 3. Inter-particle loading apparatus for cemented particles: (a) photo; (b) schematic (modified from Wang et al., 2019).

(2019). This is largely because the contact radii of the granite are more similar to the sand than the 14.7 mm diameter spheres Cole et al. (2010) used, but also probably because of the very high roughness of the ballast (mean $S_q = 18.2 \mu\text{m}$).

The simplest theory with which the data may be compared is that of Hertz (1882) which gives the mutual displacement δ at the contact of two elastic spheres:

$$\delta = \frac{a_H^2}{R} = \left(\frac{9N^2}{16RE^{*2}} \right)^{1/3} \quad (1)$$

where a_H is the radius of the contact area, R the equivalent radius ($1/R = 1/R_1 + 1/R_2$), N the normal force and E^* the

equivalent Young's modulus, taken as $1/E^* = (1 - \nu_1^2)/E_1 + (1 - \nu_2^2)/E_2$ in which ν_1 , E_1 , ν_2 and E_2 are the Poisson's ratios and Young's moduli of the two contacting materials. The Poisson's ratio of quartz is 0.065–0.068 (Mavko et al., 1998; Jaeger et al., 2007). Hertz assumes that there are elastic strains in the vicinity of the contact but zero strain in the bulk of the particle.

To account for the roughness of the particles which increases the contact displacements Greenwood et al. (1984) defined a normalised roughness, α :

$$\alpha = \frac{\sigma R}{a_H^2} \quad (2)$$

where σ is the combined roughness of the two surfaces $\sigma = (S_q^2 + S_{q2}^2)^{1/2}$ in which S_{q1} and S_{q2} are the RMS roughnesses of the surfaces, which for the Leighton Buzzard sand was measured by white light interferometry to be $0.29 \pm 0.09 \mu\text{m}$ (Nardelli & Coop, 2019). The apparent area of the contact between two rough surfaces is then a^* which replaces a_H in Equation 1 which, based on experimental data for metals, Yimsiri & Soga (2000) suggested could be given by:

$$\frac{a^*}{a_H} = \frac{-2.8}{\alpha + 2} + 2.4 \quad (3)$$

The Hertz predictions and those of “Hertz with roughness” are compared in Figure 6 with typical data for a pair of Leighton Buzzard sand particles. At low loads the roughness of the particles plays an important role and Hertz is far too stiff, while Hertz with roughness gives a perfect

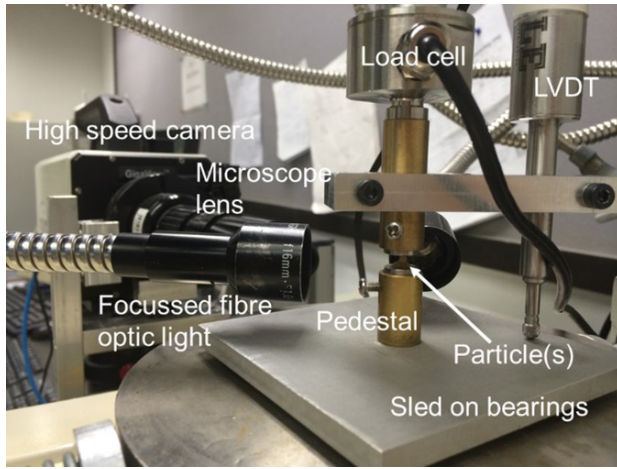


Figure 4. Particle crushing apparatus (modified from Todisco et al., 2017).

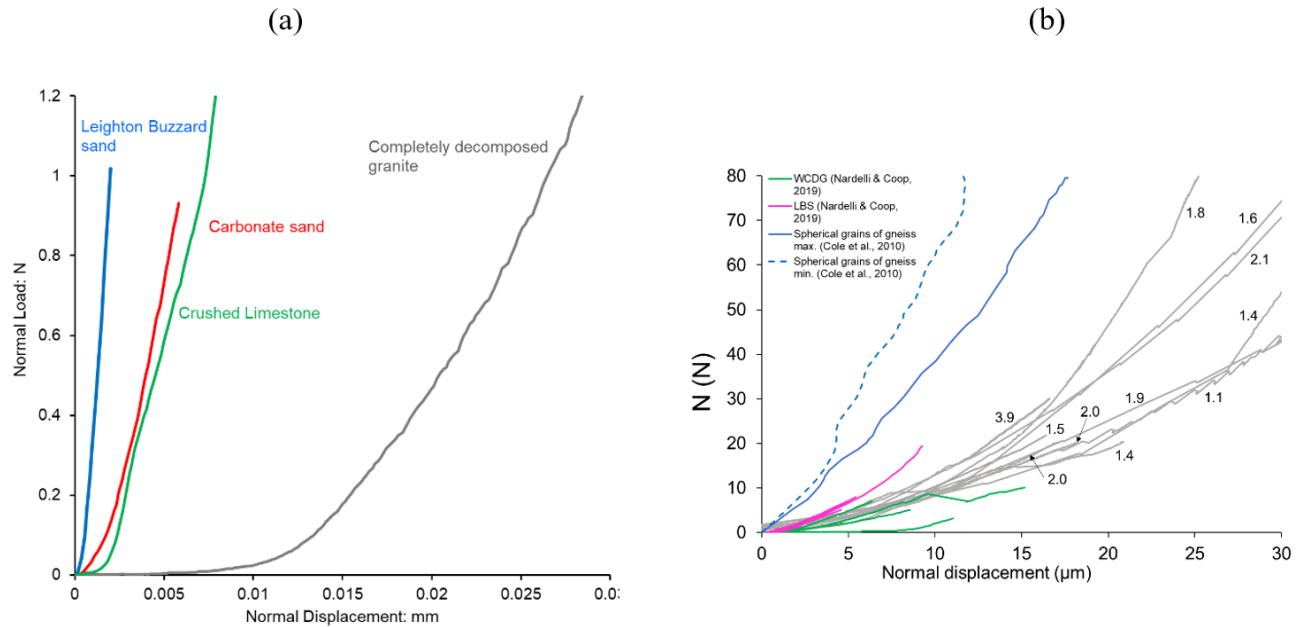


Figure 5. Normal loading behaviour of various materials: (a) sands (data from Nardelli & Coop, 2019); (b) granite ballast (after Wong & Coop, 2023).

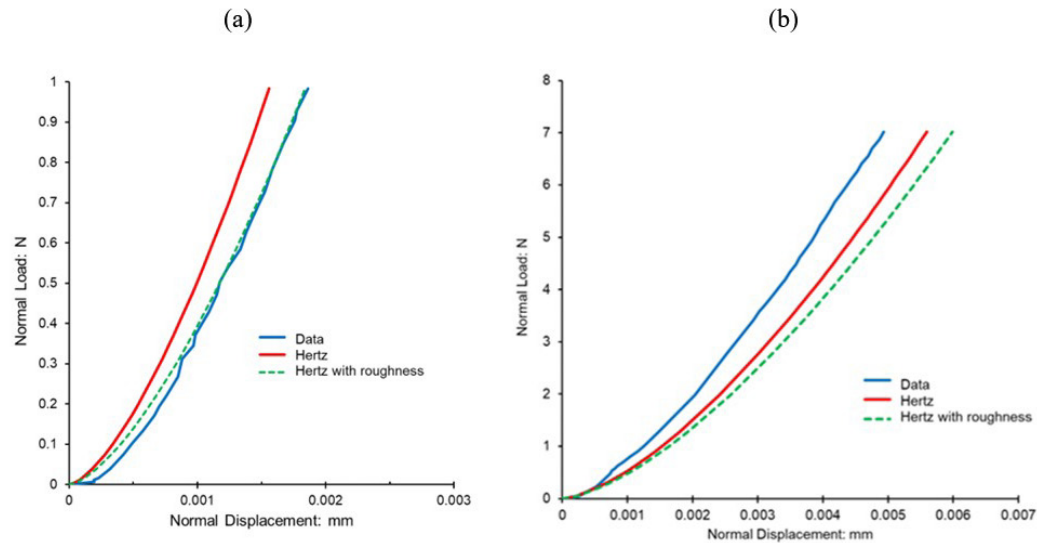


Figure 6. Comparisons between data for Leighton Buzzard sand and Hertz with and without roughness: (a) low loads; (b) higher loads; (modified from Nardelli & Coop, 2019).

fit. But at higher loads, presumably as the asperities flatten, the data show a behaviour that becomes stiffer than Hertz with roughness and after about 4 N the gradient of the curve is closer to that of Hertz.

If cycles of load are carried out at moderate loads and on stronger particles such as quartz, then on unloading there is generally only a small amount of plastic displacement evident (Figure 7). At about 2 μm this plastic deformation is considerably larger than could be accounted for by asperity flattening. Tests on slightly larger Leighton Buzzard sand particles of mean diameter 2.36 mm were carried out by Yao et al. (2022) in particle to platen tests. Interferometry profiles by Yao et al. (2022) of a typical particle in Figure 8 clearly show extensive plastic deformations in the vicinity of the contact and in fact up to 80 N the roughness only reduced by around 10% from an initial value of about 0.416 μm so the asperity flattening contribution to the plastic displacements was small. In contrast for the weaker carbonate particles even before overall particle breakage there was significant fracture damage at the contact, giving an irregular loading curve and irrecoverable displacements that are an order of magnitude larger (Figure 7).

For the granite ballast, Hertz deviates from the data much more than for the sands because of the high roughness (Figure 9). The prediction is shown for a mean contact radius of 2 mm and $E = 70$ GPa for granite (Vasconcelos et al., 2008; Domede et al., 2019). Hertz with roughness (labelled Hertz*) still fits the data well, but unlike the sands unload-reload data from test SC15 indicate very large plastic displacements, which in this case would be consistent with plasticity within the asperities since $S_g = 18.2$ μm . So although an elastic model such as that of Greenwood et al. (1984) / Yimsiri & Soga (2000) fits the data well, it cannot capture this plasticity and the unloading is better modelled by Hertz.

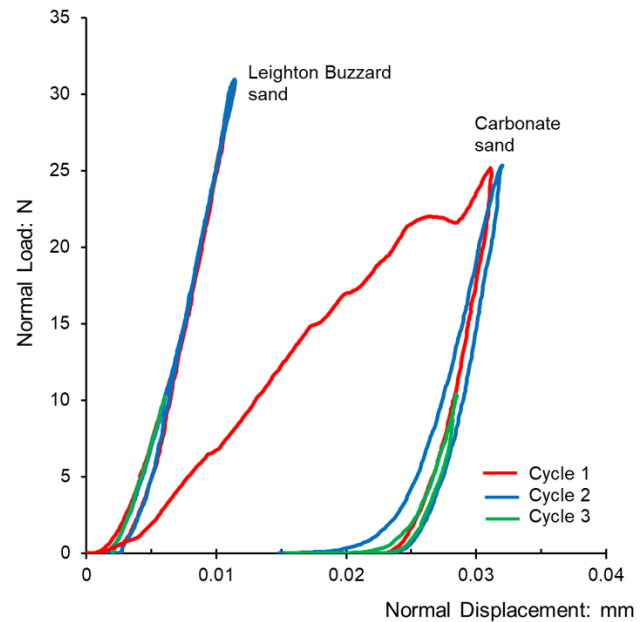


Figure 7. Cyclic normal loading of Leighton Buzzard and Carbonate sand particles (data from Nardelli & Coop, 2016 and 2019).

4. Tangential loading

Typical tangential loading data for a variety of sands of diameters 1.18–2.36 mm are shown in Figure 10a. Just as for normal loading there is considerable scatter, partly arising from different diameters, partly the different mineralogies and hence different Young's moduli, but also from the various different coefficients of inter-particle friction of the sands, μ , that control the shear stress at

sliding failure. Some scatter is also undoubtedly caused by local variations in the contact geometry that cannot be

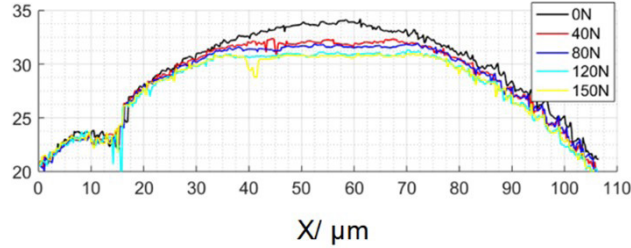


Figure 8. Contact deformation of a Leighton Buzzard sand particle (after Yao et al., 2022).

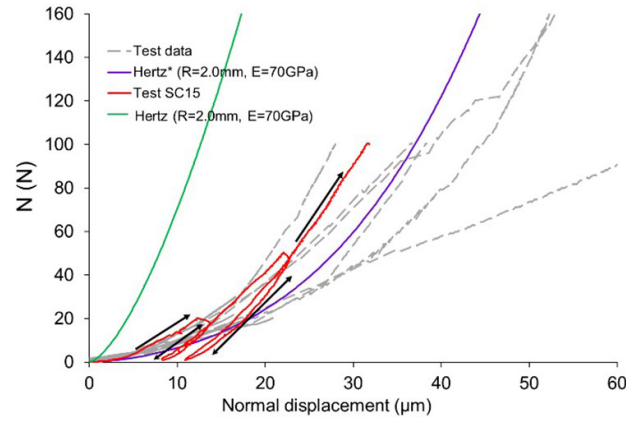


Figure 9. Normal loading test data for granite ballast (modified from Wong & Coop, 2023; Hertz* is Hertz with roughness).

observed from external microscope observation but would need nano-X ray CT to resolve. For the granite ballast in Figure 10b the tangential loads T have been normalised by the normal load N and also by μ so that each curve tends to unity at large displacements.

Stiffnesses may be derived from the force-displacement plots by taking local tangents calculated by regression over a short interval of the data points. These are given in Figure 11 for the data from Figure 10. In each case these are highly non-linear tending towards constant values at very small displacements less than around $1 \mu\text{m}$ and to zero at sliding failure which can occur at any displacement from a few microns to a few hundred microns depending on the particles. Apart from the nature and geometry of the particles the load level has a clear effect, as seen in Figure 11b for the ballast, and the large difference of stiffness between the ballast and sand particles is mostly a function of the higher load levels because the contact radii are not so different, even if the particle sizes are.

Simple predictions for tangential stiffness may be made from Mindlin & Deresiewicz (1953), who derived the initial elastic tangential stiffness, K_{T0} as:

$$K_{T0} = 8a_H \left(\frac{2-\nu_1}{G_1} + \frac{2-\nu_2}{G_2} \right)^{-1} \quad (4)$$

where G_1 and G_2 are the shear moduli of the two contacting materials. Their derivation is an extension of Hertz and so assumes elastic smooth spheres. The decay of stiffness K_T with displacement is then hysteretic and empirical and the equation used depends on the direction of loading and the load T^* at the last turning point:

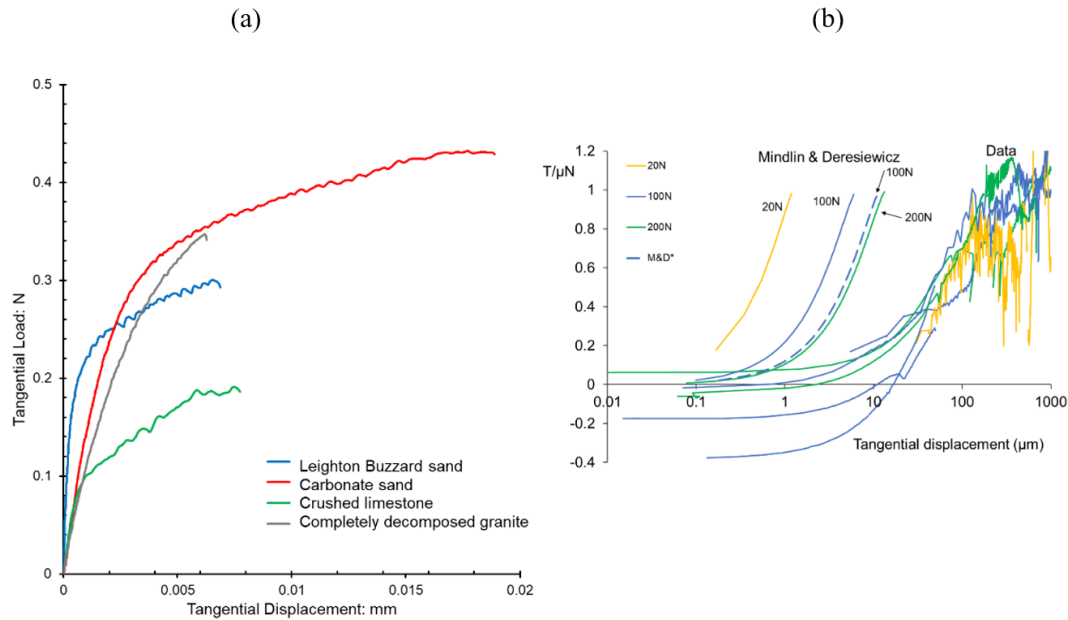


Figure 10. Tangential loading tests: (a) various sands at 1 N normal load (diameters 1.18–2.36 mm, using data from Nardelli & Coop, 2019); (b) granite ballast (modified from Wong & Coop, 2023; M&D* with roughness).

$$T \text{ increasing, } |T| \leq |T^*| K_T = K_{T0} \left(1 - \frac{T - T^*}{2\mu N} \right)^{1/3} \quad (5)$$

$$T \text{ decreasing, } |T| > |T^*| K_T = K_{T0} \left(1 + \frac{T}{\mu N} \right)^{1/3} \quad (8)$$

$$T \text{ increasing, } |T| > |T^*| K_T = K_{T0} \left(1 - \frac{T}{\mu N} \right)^{1/3} \quad (6)$$

$$T \text{ decreasing, } |T| \leq |T^*| K_T = K_{T0} \left(1 - \frac{T^* - T}{2\mu N} \right)^{1/3} \quad (7)$$

Comparisons between the Mindlin and Deresiewicz predictions and the data for Leighton Buzzard sand and given in Figure 12. The μ values used in the predictions were those measured for each test. For one test at 7 N normal load (Figure 12a) it is clear that the prediction is significantly too stiff. To check if the decay equation is correct, a prediction is

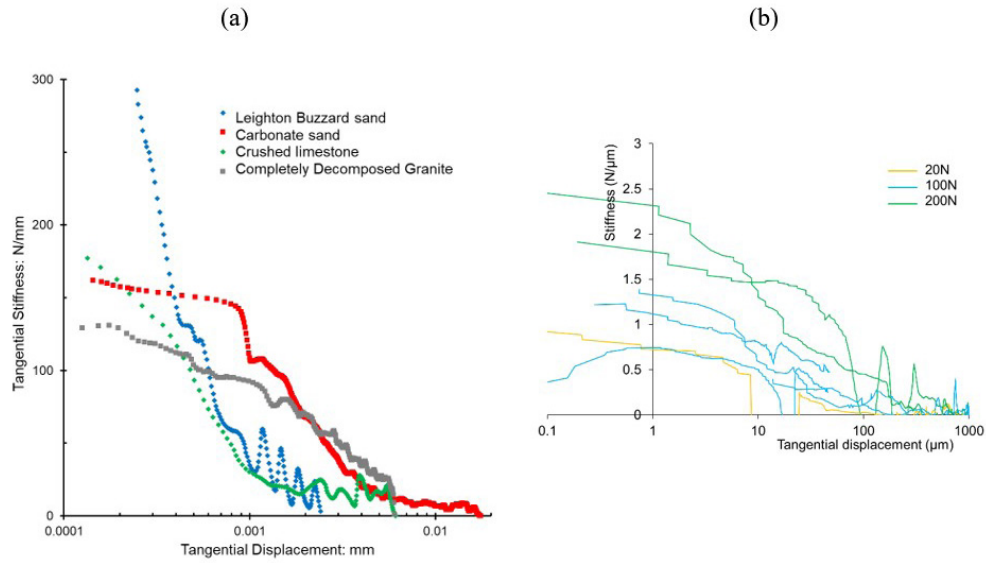


Figure 11. Tangent stiffnesses: (a) various sands at 1 N normal load (using data from Nardelli & Coop, 2019); (b) granite ballast at different normal loads (modified from Wong & Coop, 2023).

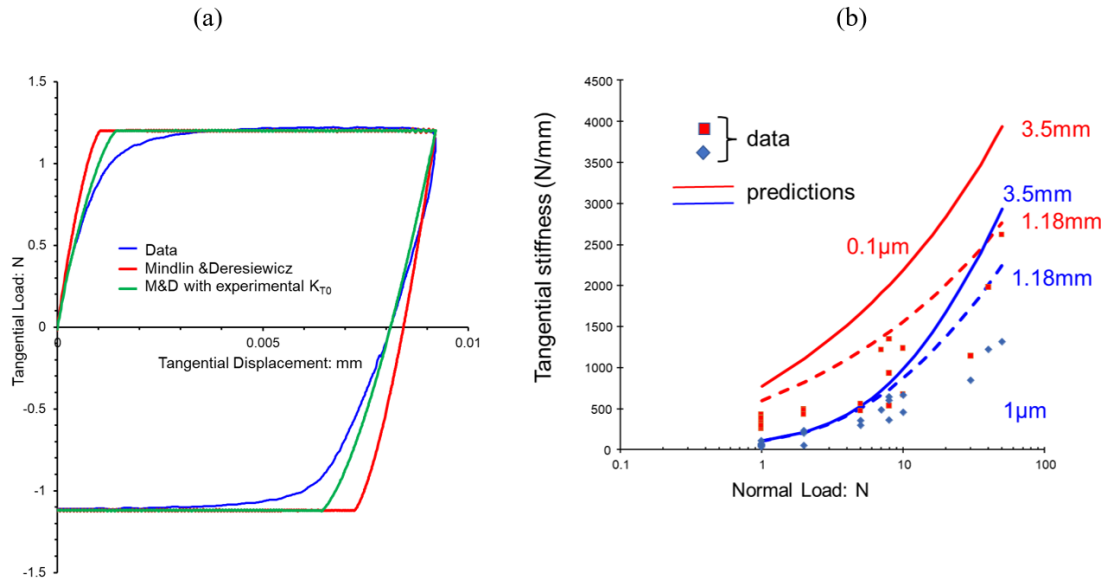


Figure 12. Mindlin and Deresiewicz predictions for Leighton Buzzard sand a) comparison of force-displacement curves at 7 N normal load, b) measured and predicted stiffnesses (modified from Nardelli & Coop, 2019).

also shown starting from the measured rather than predicted K_{T0} . The decay is then reasonably matched for the first part of the curve up to about $T = 0.5$ N. All of the Leighton Buzzard sand data are shown for two displacements, 0.1 and 1 μm , on Figure 12b. These are compared with Mindlin and Deresiewicz predictions for the largest and smallest particles tested which were 3 mm and 1.18 mm. The general trend of increasing stiffness with load level is matched quite well but the predictions are consistently around a factor of about two too high. In comparison the predictions for the granite ballast in Figure 10b are much worse, being over an order of magnitude, which is too stiff. For these a mean contact radius of 2 mm was used for the upper, pointed particle with an infinite radius for the lower flat one. For the ballast there is again a load level effect on stiffness as is evident in Figure 11b, but from Figure 10b it is clearly much less than predicted.

The much worse predictions for the ballast are likely to arise from the much greater particle roughness, Hertz-Mindlin assuming smooth particles. Otsubo et al. (2015) tried to account for this using the same ratio of the initial shear stiffness to the current normal stiffness at the applied normal load (K_T/K_N) for a rough surface as for smooth surfaces. The new prediction at 100 N normal load using K_N from Greenwood et al. (1984) for the ballast is labelled M&D* on Figure 10b, but the differences with the measured data remain very large. It is interesting that while Greenwood et al. (1984) is able to capture the normal loading well, its extension to tangential loading in this way is not successful.

Since there is significant plastic displacement in normal loading, it might be expected that pre-loading would affect the tangential stiffness. In Figure 13 the same pair of carbonate sand particles have been sheared tangentially at

1 N and then at 1 N after pre-loading to 10 N, which clearly has a significant effect on the stiffness decay.

Within Mindlin and Deresiewicz the decay of stiffness is assumed to result from the onset of local micro-slips within the contact area and there is no assumption of plastic deformation of the particles or their asperities. An attempt was made to examine if plastic deformation rather than simply local slipping could be shown to occur by shearing two particles of Leighton Buzzard sand around a circular force-controlled path of 0.6 N radius under a normal load of 4 N, so that the displacements were all small and much less than required for overall sliding failure (Figure 14). The displacement increment vectors at various points around the imposed force circle have some scatter because of the difficulty in resolving the tiny displacements, but generally they do not indicate any non-coaxiality that might be a symptom of, for example, plastic asperity deformations.

5. Sliding failure

Typical data for the inter-particle forces at sliding failure are very scattered, as in Figure 15a and a valuable test is then the “swipe test” in which the interface is taken to failure and then the normal force increased slowly while sliding continues, which gives a more continuous failure envelope. The values of μ are still scattered but this does confirm that for each particle pair the envelope is generally straight with a fairly constant μ , at least over a limited range of forces. To resolve the μ values for any sand therefore takes a large number of tests, but when that is done clear differences emerge between sands (Figure 15b). The Eglin sand that has been included here is a sedimentary sand with a mixed mineralogy, mostly quartz and feldspar (Nardelli et al., 2017).

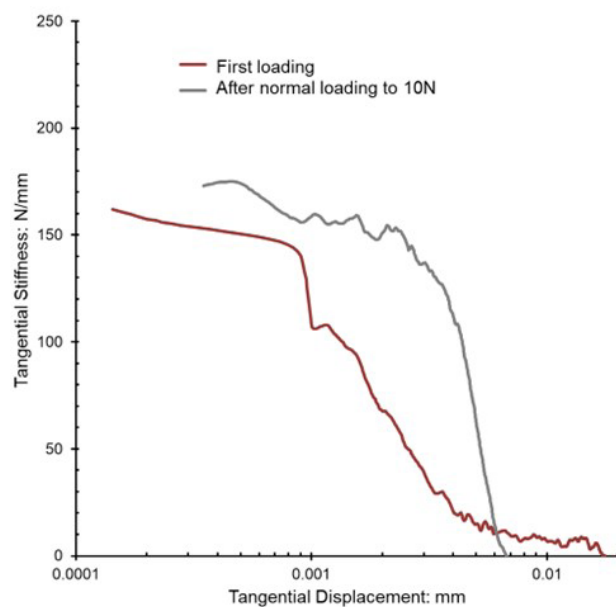


Figure 13. Effect of normal pre-loading on tangential stiffnesses for carbonate sand particles at 1 N (after Nardelli & Coop, 2019).

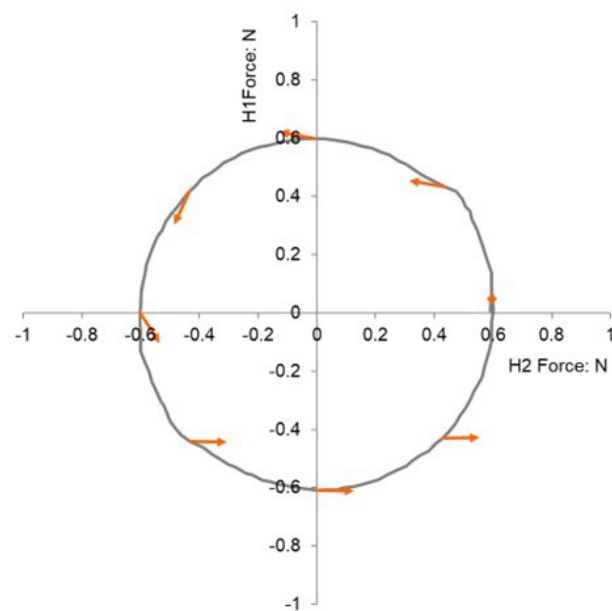


Figure 14. Displacement increment vectors for Leighton Buzzard sand under a normal load of 4 N (after Nardelli & Coop, 2019).

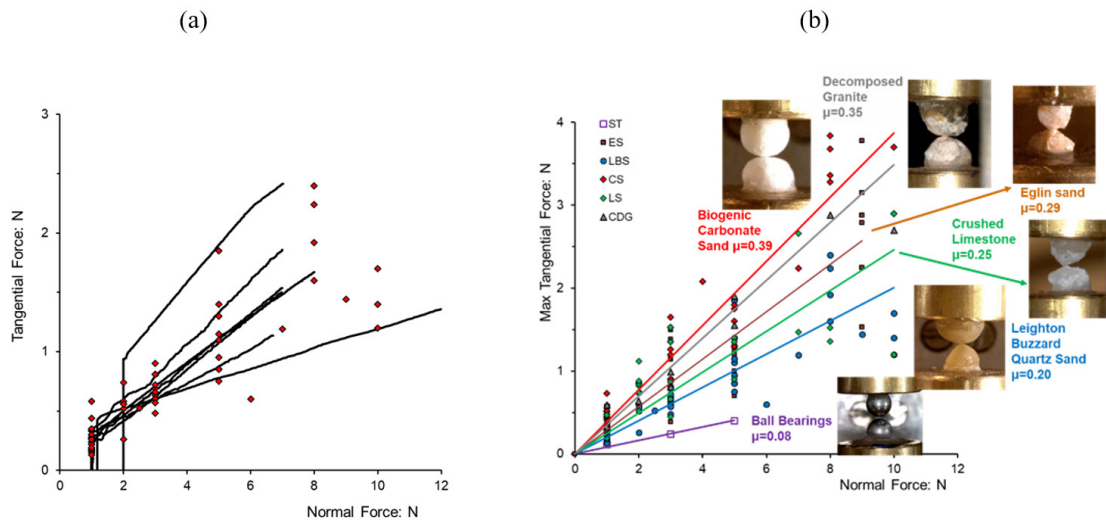


Figure 15. Frictional sliding failure envelopes: (a) Leighton Buzzard sand, data for sliding failure and “swipe” tests; (b) various sands (after Nardelli & Coop, 2019).

It is clear that shape plays little role in determining μ since the carbonate sand is one of the most rounded and most spherical but has the highest μ and Figure 16 reveals that it is the local roughness that is the controlling factor. The RMS or S_q roughnesses were measured over an area that was of the same order of magnitude as the likely particle contact area, subtracting the local shape so they would not be affected by roundness or angularity. For the sands and artificial materials this was done by white light interferometry but the much coarser roughnesses of the granite ballast were measured by a Z-stack 3D surface reconstruction using an optical microscope. The roughness of a sand will be related to its mineralogy, structure and geological history. The sedimentary Leighton Buzzard sand underwent erosion, transport and deposition and so the transportation/depositional processes in water give it its smooth surface and low μ as well as a rounded shape that will be shown below to affect strength. In contrast the decomposed granite has never undergone transport, having an origin through weathering, leading to rougher particles and hence higher μ , as well as greater angularity which reduces strength. The carbonate sand also had a sedimentary origin in water but the cellular structure of the coral probably gives rise to its greater roughness.

Wear of the particle contacts results in a change of μ . For the granite ballast 2 mm cycles were carried out between the nominal point and nominal flat contacts after an initial linear shear from the starting point at zero displacement to +1 mm. The linear shear μ values are given as cycle 1. There is a very rapid increase of μ for the first few full cycles after which the increase slowed, typically tending to stabilise after a few hundred cycles. The cycling gave rise to a large degree of grinding with a powder produced at the contact. However, even if the roughness, which started high, tended to reduce, μ increased (Figure 17). This occurred even when the ground powder was removed. The values of μ in Figure 17b were average

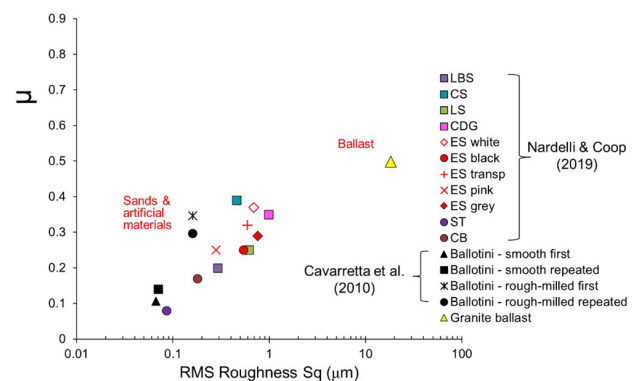


Figure 16. Influence of particle roughness on coefficient of inter-particle friction (modified from Wong & Coop, 2023, and Nardelli & Coop, 2019).

values taken in the middle section of the force-displacement curves, a selection of which are given in Figure 17a. As grinding continued the other effect is to excavate a cavity in the flat particle giving mechanical interlock and increased resistance as the corners of the travel are reached. On Figure 17b the mean values for tests at the three load levels used indicate no effect of the normal force N on μ even if for monotonic tests Nardelli & Coop (2019) seemed to observe a small μ reduction for high N on the quartz sand.

6. Particle strength

Following the localised damage that may occur at contacts (Figure 7), continued normal loading will lead eventually to catastrophic failure. This was examined using high speed imaging by Wang & Coop (2016) using the apparatus in Figure 4. The single particle strength data in

Figure 18a for particle size 1.18–2.36 mm were calculated as $0.9N/(d_2 \times d_3)$ where d_2 and d_3 are the intermediate and minimum diameters. Two predominant modes of failure were identified, a slow splitting from platen to platen and a violent explosive type, although there were variants on these two, notably failures through progressive chipping. The particles that failed explosively were the minority at about 34% but were stronger.

A local roundness at the particle-platen contact was defined as r_c/R_{ins} where r_c is the local radius of particle curvature at the contact and R_{ins} is the radius of the largest inscribed circle within the vertical particle outline. The minimum of the values for the two contacts was used, generally the top one. The effect on the strength is clear in Figure 19 for both the quartzitic Leighton Buzzard sand and the much weaker weathered particles of a

decomposed granite. The more angular particles tended to have lower strengths because the stress concentrations at the platen contacts promoted splitting while a more rounded shape allowed greater stored energy and a more even stress distribution so that failure was sudden and explosive. The overall sphericity of the particles had no effect on strength.

Loading the particles to failure between two points rather than platens ensured that a splitting mode of failure was achieved every time and gave the same strength as the splitting mode for platen loaded particles (Figure 20). Point load tests are therefore more consistent, but the question remains about which strength is more relevant to soil particles in a matrix of others.

In an attempt to examine strengths under conditions better resembling those in an assembly of grains, Todisco et al.

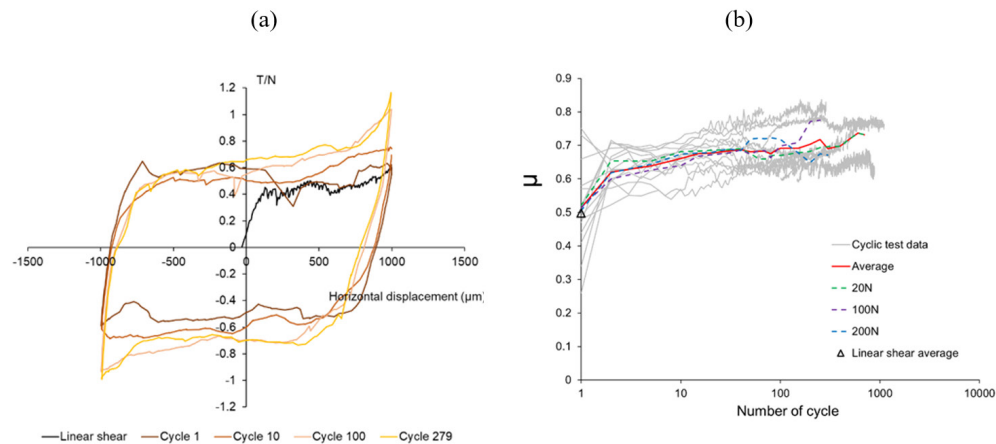


Figure 17. The development of inter-particle friction of granite ballast with wear (after Wong & Coop, 2023).

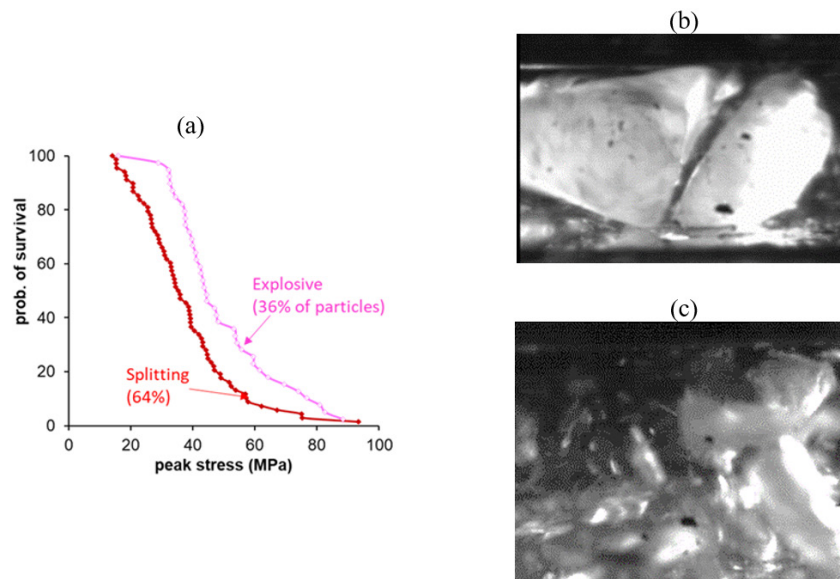


Figure 18. The interaction of mode of failure and strength for 1.18–2.36mm particles: (a) peak stress data; (b) example of a splitting mode; (c) example of an explosive mode (after Wang & Coop, 2016).

(2017) used the arrangement shown in the schematic of Figure 21, in which the crushed particle was between three support particles at both the base and top giving a coordination number CN of 6. Other tests used one contact particle at the top so $CN = 4$. The effect of coordination number is clear. The platen test strengths plotted between the $CN = 4$ and 6 data, but given the complexity of the loading of the platen loading test this is not surprising. A platen loading test is unusual in that anything other than a perfect sphere will generally have three contacts at the base of the particle and one at the top at the start of the test, so $CN = 4$. The nature of the contacts is also flat to curved surface rather than curved to curved as it would be in the assembly.

It is often assumed that particles fracture on diametric planes between the platen contacts, but close examination of fractured particles of the Leighton Buzzard sand (Figure 22) shows that actually quartz, which has no cleavage, fractures

on conchoidal surfaces. This image was taken by Zhao et al. (2015) who carried out single particle crushing tests in a micro X-ray CT. It is commonly assumed that larger sand particles are weaker by virtue of having a greater probability of containing larger flaws, but in Figure 22 there are practically no flaws visible and so this seems an unlikely explanation. Flaws would not indeed be expected in quartz that typically has its origin in a crystalline igneous rock. Cavarretta et al. (2017) demonstrated that a more consistent reason for the apparent size effect is that, as is also shown in Figure 19, a flatter relative curvature between the platen and particle gives higher strengths, and naturally that curvature depends on particle size.

The quartz particles of Leighton Buzzard sand therefore derive their high strength not only from the solid quartz mineralogy but also their rounded shape, while a decomposed granite has a much lower particle strength, as in Figure 19, which is mostly caused by its weak internal structure derived from its origin through weathering of a mixed mineralogy crystalline rock. Figure 23 shows the typical failure pattern of one particle. The particle is mostly of quartz and feldspar but also evident are faint lines that probably indicate pre-existing fissures relating to the stresses induced by weathering. Upon breakage, three types of fracture may be seen. Some run along

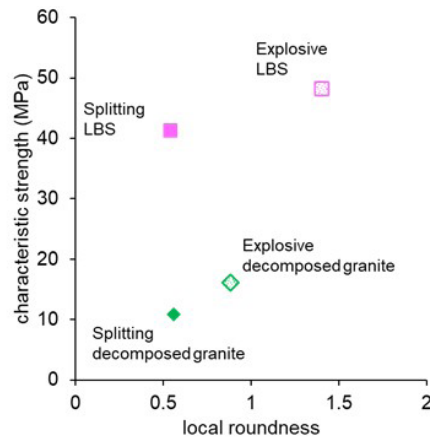


Figure 19. Failure strengths for 1.18–2.36 mm particles of Leighton Buzzard sand and decomposed granite (modified from Wang & Coop, 2016).

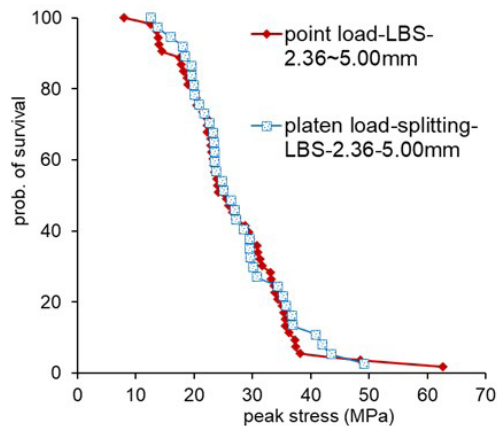


Figure 20. Failure strengths for 2.36–5.00 mm particles of Leighton Buzzard sand in platen loading and point loading tests (after Wang & Coop, 2018).

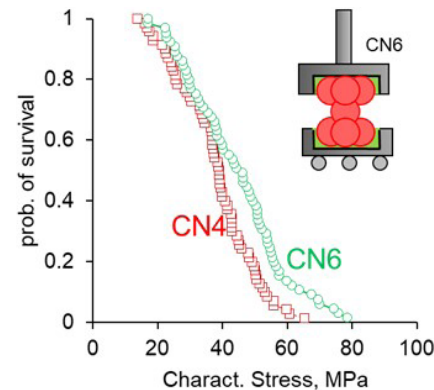


Figure 21. Multiple contact particle crushing tests for Leighton Buzzard sand (2–2.6 mm diameter). (Modified from Todisco et al., 2017).

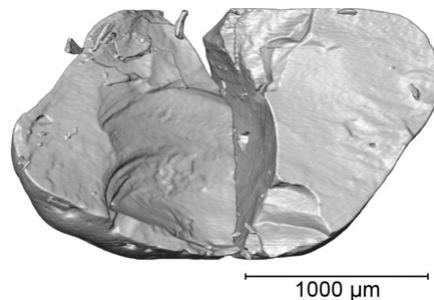


Figure 22. Fracture patterns of a typical Leighton Buzzard sand particle (after Zhao et al., 2015).

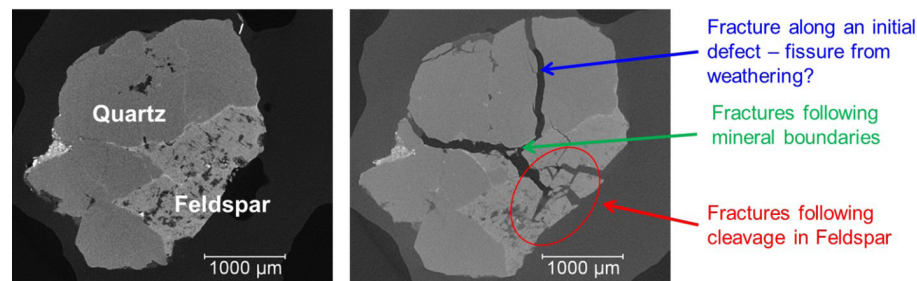


Figure 23. Fracture patterns in a highly decomposed granite particle (modified from Zhao et al., 2015).

these pre-existing planes of weakness in the quartz. In the feldspar the fractures tend to follow a grid intersecting at very roughly 90° which corresponds to the cleavage of the feldspar. Other fractures follow the mineral boundary between quartz and feldspar. The internal structure can therefore play a central role in the strength of sand particles.

7. Influence of fluid immersion

For the granite ballast the influence of the presence or absence of water on the normal contact stiffness was investigated by flooding the contact during normal loading. In Figure 24 there is clearly no effect, and this was also the case for tangential loading. The lack of any effect on the contact stiffnesses was reflected at the meso-scale by triaxial samples having the same small strain stiffness when saturated or dry (Altuhafi & Coop, 2023).

However, at sliding failure there was a substantial influence of water lubrication (Figure 25). While the water inundated contacts start with similar μ values to the dry, they tend to reduce rather than increase with cycling. One test was dried during shearing and one wetted, and the change of μ was immediate for either. The data points indicate where the interface was cleaned of the ground powder for some tests. Since this was done with water it also resulted in a temporary reduction in μ . Nardelli & Coop (2019) found no influence of water immersion for any of the sands that they tested. Possibly the difference of response for the ballast is related to its much greater roughness.

In contrast, immersion in oil does affect the inter-particle angle of friction ($=\tan^{-1}\mu$) for Leighton Buzzard sand (Figure 26).

While water can adversely affect the particle strength of some sands and rocks (e.g. Coop & Lee, 1995; Oldecop & Alonso, 2001) it typically does not affect the strength of quartz sands (Figure 27). However, immersion in glycerine did increase the particle strength considerably and this corresponded to greater stiffness and less particle breakage in oedometer tests.

8. Rate effects and creep

Since creep and rate effects influence the mechanics of sands at the meso-scale we should also expect to see them at

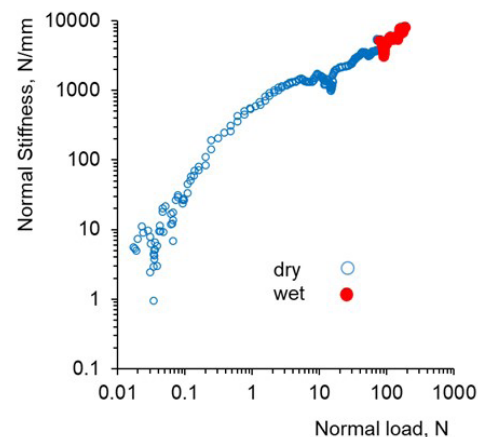


Figure 24. The effect of water inundation on the stiffness in normal loading of a granite ballast (after Altuhafi & Coop, 2023).

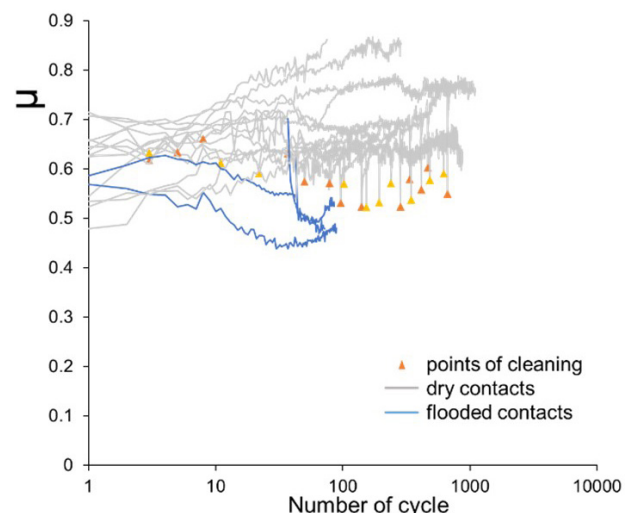


Figure 25. The effect of water inundation on μ during shear cycles on granite ballast (modified from Wong & Coop, 2020).

the micro-scale. Figure 28a shows creep data for a granite ballast contact; the creep stabilises after a couple of days.

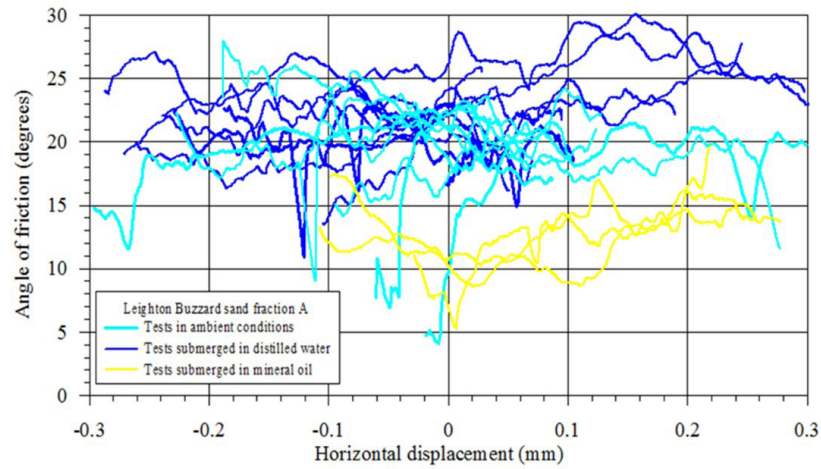


Figure 26. The influence of water and oil immersion on the inter-particle angle of friction between Leighton Buzzard sand particles (after Cavarretta et al., 2011).

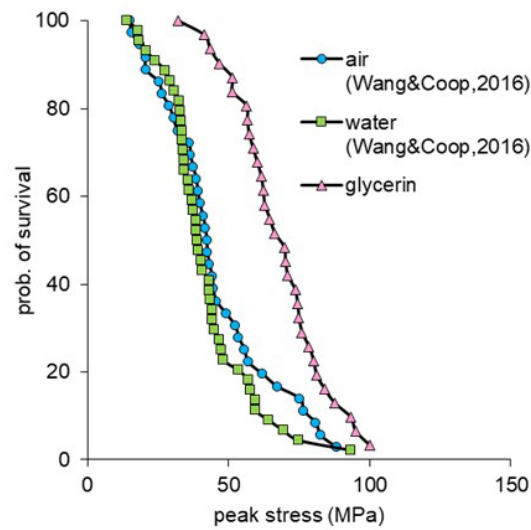


Figure 27. The effects of immersion in water and glycerine on strength of Leighton Buzzard sand particles (1.18–2.36 mm). (After Wang et al., 2023).

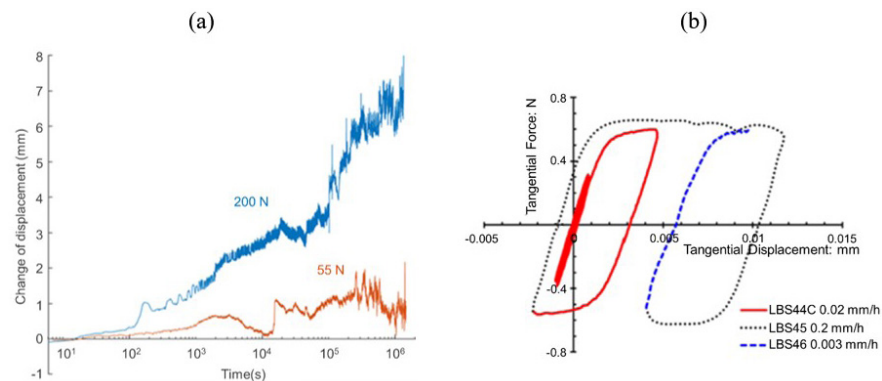


Figure 28. (a) creep under two normal loads of a point to flat granite ballast contact (after Wong & Coop, 2023); (b) influence of displacement rate on sliding shearing of Leighton Buzzard sand (after Nardelli & Coop, 2019).

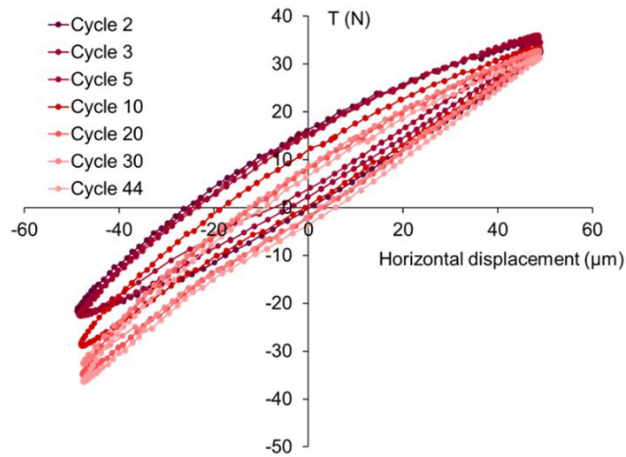


Figure 29. Pre-sliding tangential loading cycles of granite ballast (after Wong & Coop, 2023).

The amount of creep (or more correctly contact ageing) was dependent on the normal load and was exacerbated by the addition of a tangential load. Similar data were observed for a quartz sand by Michalowski et al. (2018) but at much smaller loads. While some authors have found rate of sliding effects in sand (Kasyap & Senetakis, 2019) over almost two orders of magnitude of rate of displacement there was no significant effect for the Leighton Buzzard sand (Figure 28b) or for the granite ballast (Wong & Coop, 2023).

9. Pre-failure cyclic loading

Typical data for pre-sliding cyclic tangential loading are shown in Figure 29 for the granite ballast. As the cycles continue the secant stiffness tends to increase but the loop area decrease, both tending to stabilise after around 20 cycles. The behaviour of the Leighton Buzzard sand was very similar (Nardelli & Coop, 2019). Similar behaviour was also seen in normal loading, although the stiffness tended to reduce slightly.

10. Conclusions

As computing power increases and the possibility to model soils with digital twins becomes more realistic, we will increasingly face the choice, at least for coarser graded soils, whether we continue with the rabbit hole of ever more complex continuum mechanics approaches or if we should adopt discrete ones. A correct digital twin on Avatar soil will need us to model accurately the particle shapes and contact fabric, which we can already do, and the key missing link is a thorough understanding of the contact mechanics up to and including particle damage and crushing. This requires us to jettison our beloved triaxial apparatus and develop new equipment for discrete tests on single particles.

Research on contact mechanics to date has highlighted the complexity of the response and the inadequacy of common

current models to capture that behaviour. Important challenges for the future will be how we model efficiently the plasticity and damage at the particle contacts that is clearly an important feature of the behaviour, since this will necessitate that the model retains a memory of what has previously happened at a particular location on a particle. Large-scale particle damage and the mechanisms by which real geological particles fail are also key areas for future investigation and modelling.

Developing similar apparatus to those shown here for silts may well be possible, but for clays it is unlikely, so it is interesting to ponder whether in twenty years we will see a division in approaches for elastic and plastic soils and whether we will have laboratories full of inter-particle apparatus.

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Declaration of interest

The author has no conflicts of interest to declare and there is no financial interest to report.

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

a_H	Radius of the contact area for Hertz
a^*	Apparent area of the contact between two rough surfaces
r_c	Local contact radius
CN	Coordination number
E	Young's modulus
E^*	Equivalent Young's modulus
G	Shear modulus
K_N	Normal stiffness
K_T	Tangential stiffness
K_{T0}	Elastic tangential stiffness
N	Normal load
R	Equivalent radius at contact
R_{ins}	Radius of the largest inscribed circle within particle outline
S_q	RMS roughness
T	Tangential load
T^*	Tangential load at last force reversal
α	Normalised roughness

δ	Mutual approach of contacting spheres
μ	Inter-particle coefficient of friction
ν	Poisson's ratios
σ	Combined roughness

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ARTICLES

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Soil testing at different scales: from micro-experiments to field tests in earthworks

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Article

Keywords

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Element and field tests
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Earthworks
Engineered barriers

Abstract

Examples of upscaling phenomena with experimental techniques are presented and discussed within the framework of compacted soils for hydraulic and environmental earthworks and engineered barriers for the energy sector. A series of laboratory experiments at different scales are presented and interpreted to focus on the need for experimental upscaling in compacted soils since distinctive behavioural features can only be detected at specific length scales. Permeability results on fine-grained soils and artificially prepared sand/bentonite mixture at different scales will be discussed with microstructural tests regarding 'dry' or 'wet side' compaction, element and mock-up tests on compacted soils in the laboratory, and field tests on a compacted trial embankment and demonstration test to explore anisotropic and heterogeneous features. The presented examples will help to motivate new experimental research subjects and promote experimental protocols at different scales ranging from mm-scale (micro), cm-scale element tests, dm-scale mock-ups and m-scale (trial/demonstration tests) to help understand and approach some fundamental questions observed at the application scale of compacted soils.

1. Introduction

The first question that can arise is why we need to observe and approach phenomena at different scales of compacted soils used in earthworks. A partial answer to this question is that the phenomena we detect often depend on the scale of experimental observation. Distinct behavioural features are observed at specific length scales, where soil preparation in the laboratory and field compaction follow different procedures (usually with larger aggregate sizes in the field) and impact the variability of soil properties. However, despite this limitation, where experimental results may display some difficulty in upscaling, we must provide answers to real applications on compacted soils with the available experimental results.

Within this context, experimental techniques in the laboratory and the field should assist in observing and understanding processes at the different scales demanded in earthworks (pore/grain level, phenomenological scale, field scale). Therefore, they will allow expanding our capability to predict and upscale the response of compacted soils in real-life engineering applications with predictive modelling. To achieve this objective, it is necessary to pay attention to the following aspects

- To select adequate techniques and improve experimental protocols to the appropriate length scale (microstructural tests, element and boundary value problem laboratory tests);

- To use larger-scale setups (dm-scale mock-ups, 1-g and N-g physical model tests, field tests) to mimic and better predict the response of field applications with realistic material properties (compacted soils with heterogeneous and anisotropic properties affected by the scale) and boundary conditions.

The present contribution will consider two examples of earthworks to focus on the need for experimental upscaling in compacted soils. On the one hand, earthworks for zoned earth dams and embankments for a canal in areas with collapsible soils. On the other hand, a case of experimental upscaling of an engineered gas-permeable seal within earthworks for the energy sector, where compaction variability in dry density has been assessed, and permeability tests have been carried out at different scales. Therefore, different phenomena have been approached in these examples at varying observation scales (from pore/grain microstructural level, element tests, mock-ups, and field experiments to progressively upscale to the application).

Figure 1 presents a schematic of a zoned earth dam to show an example of earthworks that rely on laboratory and field experiments and monitoring data during construction, with selected scale-dependent aspects of compacted soil behaviour and size effects of coarse aggregates. Regarding the latter aspect, much effort has been placed into the behaviour of coarse-grained geomaterials at the single particle scale using a new range of apparatus and testing techniques (e.g.,

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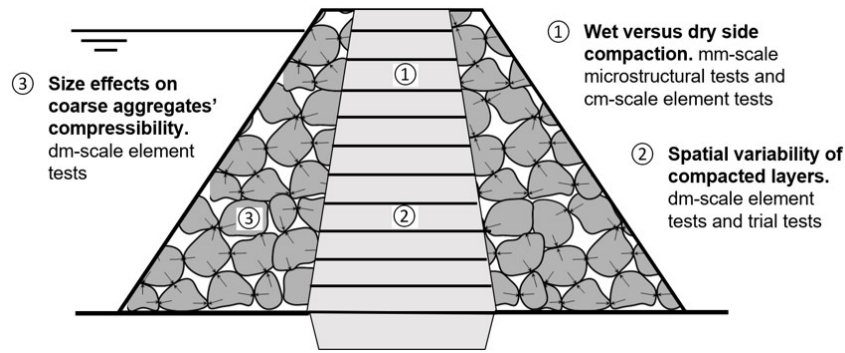


Figure 1. Selected aspects of compacted soil behaviour at different scales. Topics 1 and 2 are discussed in the paper.

Coop, 2024). Nevertheless, the large particle sizes in structures such as dams hinder laboratory testing. Predicting the field behaviour requires the development of discrete models that upscale (extrapolate) these effects from large-scale tests (e.g., Alonso et al., 2012; Alonso, 2014). Regarding earthworks for dams and embankments, the present contribution focuses on the compaction process when interpreted from a smaller-scale perspective (‘wet’ versus ‘dry side’ compaction) and the spatial variability in local dry density and anisotropy effects when viewed from a larger m-scale perspective (trial embankments).

2. Microstructural tests at mm-scale. ‘Dry side’ versus ‘wet side’ compaction

The state set on compaction (water content and dry density) substantially affects the hydro-mechanical behaviour of fine-grained soils used in zoned-earth dams and dikes. Besides the impact of the matric suction and maximum compaction stress of the compacted material (equivalent stress-based information to water content and dry density), the microstructure (connectivity of the larger pores and their sensitivity to loading, wetting and drying) also influences the behaviour (Romero et al., 2011; Casini et al., 2012; Romero, 2013; Alonso et al., 2013; Cordão Neto et al., 2017). An intuitive and straightforward way to differentiate this initial state is to refer to ‘dry side’ compaction with degrees of saturation S_r lower than the optimum reference (typically, the optimum water content and the maximum dry density correspond to S_r slightly above 85%) or to ‘wet side’ compaction with S_r higher than this reference. The ‘dry side’ compaction displays larger pore sizes due to fine-grain aggregation than the ‘wet side’, which impacts the properties dependent on the connectivity and the pore volume of these larger pores, such as water permeability, the water retention properties, the compressibility on loading (under unsaturated and saturated states) and the collapse on wetting. In addition to the initial state, the stress paths followed after the compaction (matric suction and net stress) influence the current pore size distribution (e.g., Yin & Vanapalli, 2022). Gonzalez-Blanco et al. (2023) proposed

a stress-based equation using a single constitutive stress to discern the microstructural void ratio e_m of silty clay (voids inside the aggregates to the volume of solids) after different stress paths based on several controlled-suction oedometer tests followed by the determination of the pore size distribution by mercury intrusion porosimetry MIP on freeze-dried samples.

Figure 2 presents an alternative to this proposal, showing the macrostructural void ratio e_M (voids between aggregates to the volume of solids; $e_M = e - e_m$, where e is the void ratio) as a function of the S_r at compaction, which allows a more practical separation between the ‘wet side’ and ‘dry side’ domains outlined above and their microstructural consequences. It is essential to highlight the experimental difficulty involved in separating these intra- and inter-aggregate pore domains, in which more emphasis is placed on selecting the pore size based on the phenomena observed (e.g., intra-aggregate pore size domain with dominant water absorption mechanism and not affected by mechanical actions). Different alternatives of intra- and inter-aggregate separations are discussed in Yuan et al. (2020).

The soil corresponds to silty clay ($58\% < 75 \mu\text{m}$ with a liquid limit of 31%, a plasticity index of 12%, and a density of solids $\rho_s = 2.70 \text{ Mg/m}^3$). As shown in the field emission scanning electron FESEM photomicrographs in the figure, the microstructure of a ‘dry side’ state develops aggregates with large and connected pores between these aggregations as opposed to a ‘wet side’ state. A pore size of $5 \mu\text{m}$ was used to separate the pore size domains using MIP on freeze-dried samples (macropore sizes $> 5 \mu\text{m}$ were the only ones affected by loading at different S_r).

The e_M results are fitted to Equation 1 with two parameters $\alpha = 0.024$ and $\beta = 0.723$ (two different void ratios were considered in the plot, $e = 0.40$ and 0.80 , to highlight its influence). Equation 1 is an updated expression that follows Romero et al. (2011) and Della Vecchia et al. (2013), which was already used by Romero et al. (2023) to interpret gas permeability.

$$\frac{e_m}{e} = \alpha + \beta S_r \leq 1; \quad \frac{e_M}{e} = 1 - (\alpha + \beta S_r) = 0.976 - 0.723 S_r \quad (1)$$

To learn the utility of Equation 1, the results of cm-scale and controlled-gradient saturated permeability tests of the silty clay at different compaction states are presented in Figure 3a. Even if the compacted silty clay becomes saturated, it essentially retains the microstructural properties set during

compaction. Consequently, the connected e_M of the ‘dry side’ compaction displays a permeability k one order of magnitude higher than the ‘wet side’ at the same $e = 0.571$ (corresponding to a dry density $\rho_d = 1.70 \text{ Mg/m}^3$), as shown in the figure. Nevertheless, when compacting to the standard Proctor

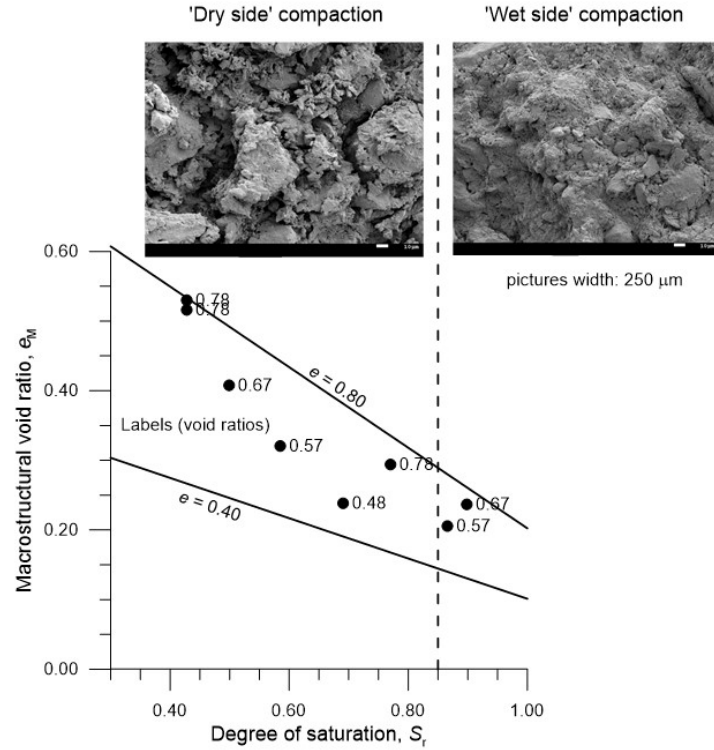


Figure 2. Macrostructural void ratio e_M as a function of S_r at compaction. Fitted curves for two different void ratios. Top: FESEM photomicrographs at two different hydraulic states (‘dry side’ and ‘wet side’ compactions) (the picture width is $250 \mu\text{m}$) (data processed from Mora Ortiz, 2016).

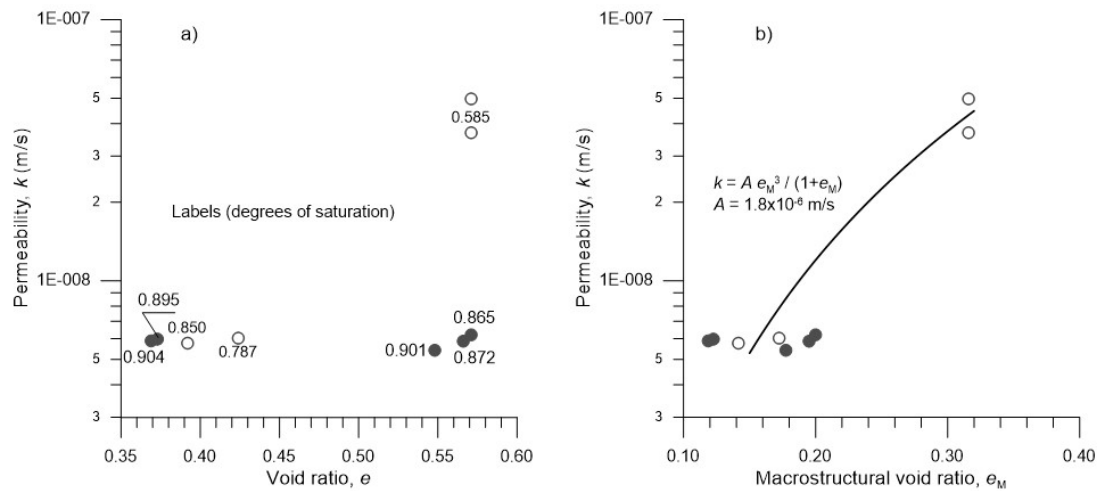


Figure 3. (a) Permeability k as a function of e (labels correspond to degrees of saturation at compaction, data processed from Mora Ortiz, 2016); (b) Permeability as a function of the macrostructural void ratio (Kozeny-Carman equation and as a function of e_M).

energy, the void ratio reduces to $e = 0.40$ ($\rho_d = 1.91 \text{ Mg/m}^3$), and the difference in k between ‘dry side’ and ‘wet side’ compactions vanishes. Figure 3b plots k as a function of e_M calculated from Equation 1, in which high e values of the ‘wet side’ compaction are shifted to much lower e_M values. An expression based on the Kozeny-Carman equation and as a function of e_M is proposed in Figure 3b. It is essential to highlight that these results and their interpretation are based on cm-scale permeability tests and mm-scale mercury intrusion porosimetry results on freeze-dried samples after compaction (Romero & Simms, 2008).

3. Microstructural tests at mm-scale. Drying after compaction

Another phenomenon with practical implications and interpreted by microstructural tests is the increase in collapse after a silty clay is compacted and undergoes drying. The natural material corresponds to lightly cemented low-density silty clay ($86\% < 75 \mu\text{m}$ with a liquid limit of 33%, plasticity index of 16% and $\rho_s = 2.70 \text{ Mg/m}^3$), prone to collapse upon wetting. One of the procedures envisaged to avoid damage to the canal embankments placed on these collapsible soils that may reach 10 m thickness was to replace the natural low-density soil with the same material but compacted to a higher dry density (Alonso et al., 2005). The geographical area of the canal is relatively arid. It has an annual mean rainfall in the range of 400-500 mm, which explains why it is essential to consider some desiccation of the compacted material. However, water losses from the canal may change this scenario dramatically after construction and increase the compacted material’s degree of saturation, as Alonso et al. (2005) explained.

Figure 4 presents the initial state ‘A’ of the natural material in the compaction plane (corresponding to $e = 1.06$ and $S_r = 0.33$), which was compacted at standard Proctor energy and optimum water content (state ‘B’ in the figure at $e = 0.44$ and $S_r = 0.91$). The modified Proctor state at optimum water content is also indicated in the figure ($e = 0.33$ and $S_r = 0.92$). State ‘C’ ($e = 0.41$ and $S_r = 0.37$) is reached after drying the soil to a total suction $s = 26 \text{ MPa}$ (i.e., relative humidity around $RH \approx 82\%$) measured with a WP4C dew point psychrometer from METER Group. An environmental scanning electron (ESEM) photomicrograph at the corresponding water content is shown in the figure at state ‘C’, where fine-grain aggregates and macropores between them are observed despite the high dry density achieved on compaction and subsequent shrinkage.

Figure 5 presents results under oedometer conditions of collapse on wetting at different vertical stresses of the various states considered (natural, compacted and dried after compaction). The maximum collapse (vertical strain under oedometer conditions) of the low-density natural material reached 10.6% at 200 kPa. The lightly cemented

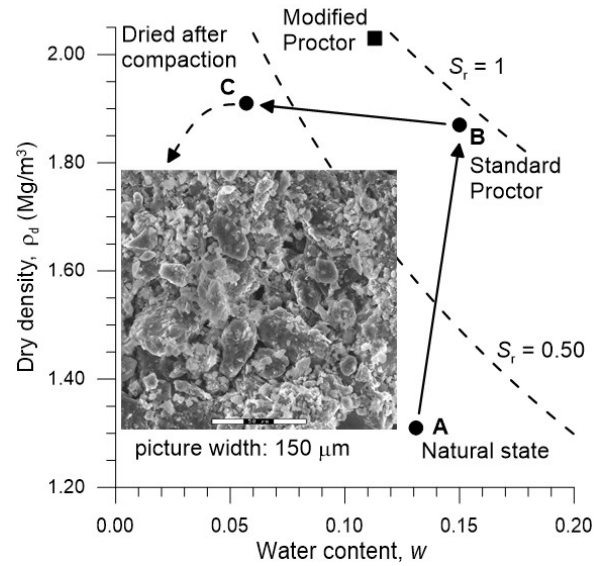


Figure 4. Natural ‘A’, compacted ‘B’ and dried after compaction ‘C’ states of a silty clay plotted in the Proctor plane. ESEM photomicrograph at state ‘C’ (the picture width is 150 μm).

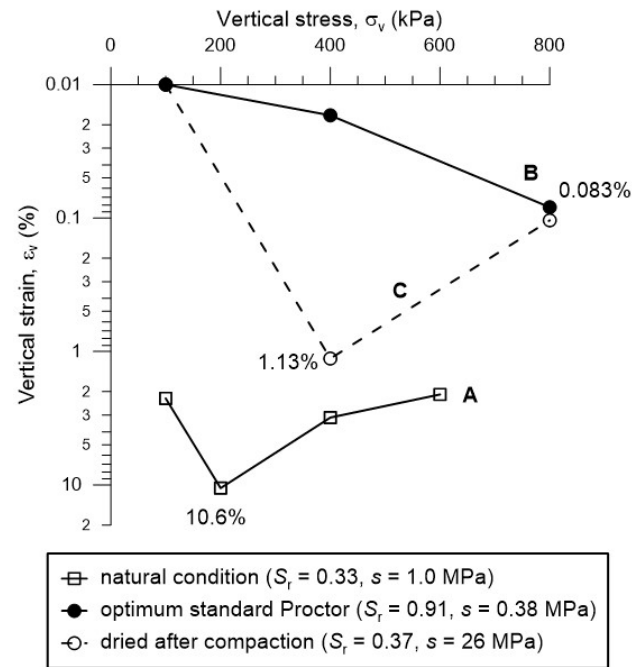


Figure 5. Collapse undergone on wetting under oedometer conditions: natural ‘A’, compacted ‘B’ and dried after compaction ‘C’ states of a silty clay. Legend: initial states before loading.

low-density silty clay with gypsum and carbonate types of cement is relatively rigid up to vertical stress of 200 kPa, and the macropores are preserved. Nevertheless, cementation is lost on wetting, and a significant collapse develops. At a vertical stress of 400 kPa, some of the cementation is

already lost by loading, and the material does not develop as much collapse when wetted. However, this framework changes drastically when compacting the material (the standard Proctor test is equivalent to a vertical stress of about 0.8 MPa, and the matric suction reaches $s = 0.38$ MPa measured with a high-range tensiometer) as the cementation is destroyed and the macropores are drastically reduced. The Barcelona Basic Model BBM (Alonso et al., 1990) predicts the development of the maximum collapse (plastic compression) at 800 kPa vertical stress when the loading-collapse LC curve is fully activated during wetting (maximum collapse 0.083%). Suppose the compacted material can dry by arid climate conditions (state 'C') above the maximum matric suction ever undergone. In that case, some irreversible shrinkage will occur by activating the suction-increase SI yield locus. Therefore, some strain hardening is expected to be generated during this irreversible shrinkage (BBM, Alonso et al., 1990). Nevertheless, the material is more prone to collapse despite the overall hardening, as shown in Figure 5, where a maximum collapse of 1.13% is reached at 400 kPa. Afterwards, increasing the vertical stress to 800 kPa reduces the collapse deformation since the material decreases the macroporosity and reaches higher degrees of saturation upon loading.

The explanation behind the enhancement of collapse on shrinkage is associated with the microstructural changes undergone during drying (enlargement of macroporosity between aggregates and fissuring). Figure 6 shows the evolution of the void ratio during suction increase from the compacted state 'B' to the dried state after compaction 'C'. The change in e is relatively small despite the significant suction change due to the silt shielding skeleton limiting this global shrinkage. As the figure shows, the microstructural void ratio is also affected and is associated with the aggregates' more significant and less constrained shrinkage. This shrinkage can be estimated using Equation 1 by calculating the variation of e_m/e with S_r and considering $e = 0.44$ (the initial value of the compacted state). Results of the microstructural void ratio determined by MIP on freeze-dried samples and reported by Romero (2013) for silty clays at $e \leq 0.44$ are also plotted, which also agree with the adopted trend. As shown in the figure, the difference between e and e_m (i.e., the macrostructural void ratio e_M) increases on drying (from $e_M = 0.14$ at 'B' to $e_M = 0.29$ at 'C') due to the opening of the macroporosity and fissuring. This increase in the macroporosity, also observed in the ESEM photomicrograph in Figure 4 between aggregates, can explain the enhancement of the collapsibility after a drying event. Alonso et al. (1999) also considered the macroscopic void ratio enlargement on drying on some materials when extending BBM to active soils during cyclic wetting and drying cycles. Although the change in e_M predicts a significant collapse if all the generated macroporosity vanishes on saturation, it does provide evidence of a phenomenon that can only be explained at this microstructural mm-scale.

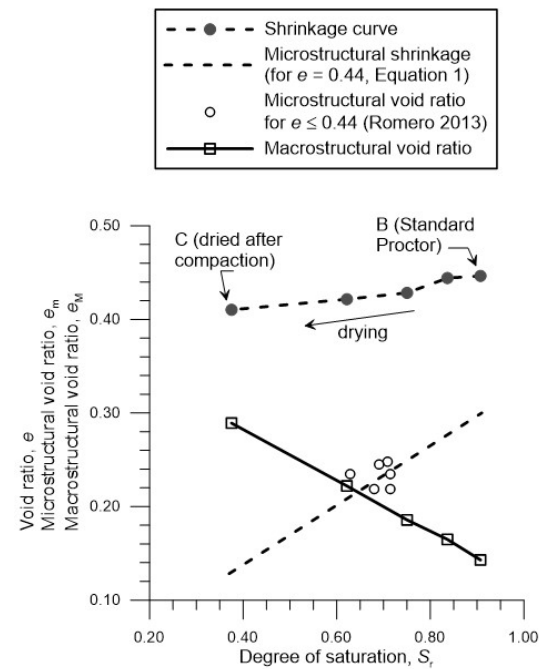


Figure 6. Evolution of e , e_m and e_M on drying after compaction. The evolution of the e_m is estimated from Equation 1 and results reported by Romero (2013).

4. Laboratory versus field dynamic compaction. Compaction-induced spatial variability in dry density

It is widely accepted that difficulties arise when characterising the hydro-mechanical behaviour of the compacted materials with laboratory tests and comparing the modelled response against field tests and monitoring data. Besides the uncertainty of S_r (or the matric suction), some uncertainties depend on the observation scale. It is generally found that permeability values measured in the laboratory are lower (sometimes by two orders of magnitude) than field measurements and that stiffness on loading is higher in the laboratory than in the field data. Pinyol & Alonso (2019) observed an example of the latter aspect when simulating the 90 m high Albagés zoned earth dam (Lleida, Spain). The authors modelled the dam construction using a numerical code dealing with unsaturated–saturated soils and the coupled hydro-mechanical phenomena (generation and dissipation of pore pressures and the corresponding mechanical response). The cm-scale element tests allowed a rigorous calibration of the model stiffness, which depends on the stress state (matric suction and net stress) and the preconsolidation stress of the compacted material, which also depends on matric suction. However, the authors consistently observed larger vertical field settlements than class A predictions using laboratory information along a vertical central section throughout the core. An updated model was also presented, considering

compaction data and field tests performed during construction (information came from plate loading/unloading tests on the embankments), which allowed for better modelling of the vertical displacement response. Despite the lower degree of saturation and the consequences of matric suction on stiffness, the dm-scale plate loading tests allowed averaging of properties over a larger volume where local inhomogeneities on field compaction are captured. Lower mean dry density was achieved under field conditions due to the consequences of compacting on the ‘wet side’.

To better understand this dynamic compaction-induced variability and its consequences on dry density and permeability anisotropy, Figure 7 presents the results of two compaction techniques in terms of e using tests at the dm scale (100-mm inner diameter PVC cylinder and 300 mm in height treated to minimise inner wall friction that microfocus X-ray CT verified). Sakaki et al. (2023) reported these tests on a sand/bentonite S/B mixture (80% by mass of SiO_2 sand and 20% Wyoming-type bentonite) at a water content of 11.0%. The Wyoming-type of bentonite presents 85% by mass of montmorillonite, and the mixture displays $\rho_s = 2.66 \text{ Mg/m}^3$. The authors used static compaction (controlled displacement conditions up to 4.9 MPa and target $e = 0.56$) and dynamic compaction considering controlled energy per volume via dropping a mass with a smaller diameter (24 mm) at a certain height (a single layer was used mimicking field emplacement and the total compaction energy per unit volume corresponded to 1800 kJ/m^3 and target $e = 0.59$). The compaction-induced variability was characterised by slicing. As observed in the figure, the dry density along the static procedure was relatively homogeneous (e varying

between 0.523 and 0.606), which was not the case with the dynamic compaction that led to a minimum void ratio of $e = 0.395$ at the top compaction surface and $e = 0.755$ close to the bottom boundary. A slight decrease in the void ratio is detected at the rigid bottom boundary with no displacement allowed (this bottom boundary condition is somehow different to that expected under field conditions).

A similar pattern using dynamic field compaction leading to higher dry density at the compaction surface was also observed in the large-scale ‘Gas permeable Seal Test’ GAST launched at Nagra’s Grimsel Test Site GTS (Nagra stands for ‘National Cooperative for the Disposal of Radioactive Waste’, Switzerland) (Nagra, 2024; Rüedi et al., 2012; Sakaki et al., 2023). This test placed in a tunnel in granitic rock demonstrated the effective functioning of an engineered gas-permeable seal concept during gas transport after long-term saturation for a deep geological repository for low and intermediate radioactive waste mainly produced by the energy/medicine/industry sectors (Romero et al., 2023). The cylindrical S/B seal with a diameter of 3.0 m and 8.0 m long consists of 28 horizontal layers (100 mm in height) of dynamically compacted S/B as above (water content of 11.0%) using a vibratory plate (central part) and rammer compaction (near-wall areas). The field compaction was controlled by two procedures: a) controlled mass in 0.10 m layer with known total volume, and b) by sampling method from the top 20 mm of each layer (ring cutter at compaction surface with diameter 50 mm and height 20 mm). Figure 8 shows the void ratio variation with seal height for the different layers

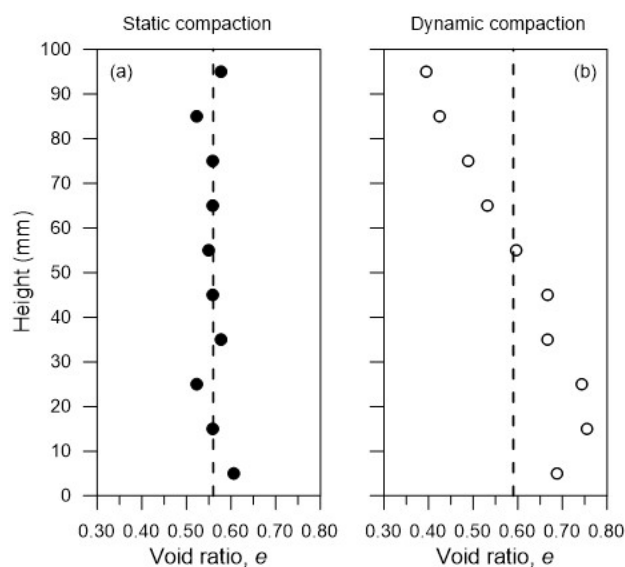


Figure 7. Evolution of e throughout the final compaction height (around 100 mm). (a) Static compaction; (b) dynamic compaction (data from Sakaki et al., 2023).

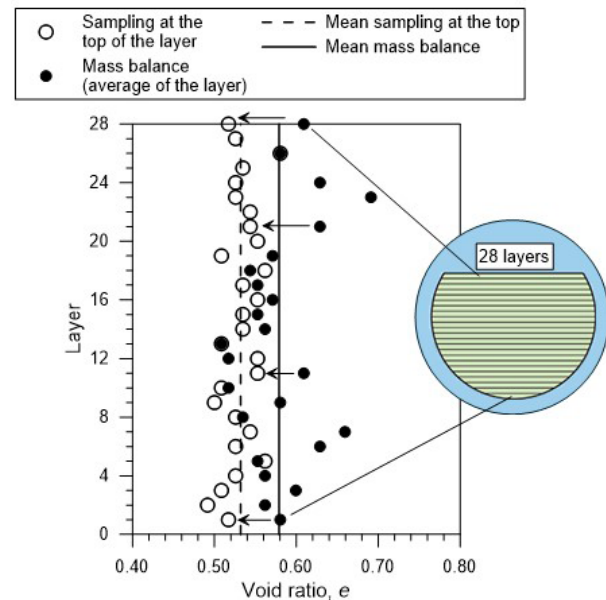


Figure 8. Evolution with height of e in each layer by mass balance method (layer average) and sampling (top 20 mm of the layer). The mean sampling results at the top of each layer displayed a lower e (data from Sakaki et al., 2023).

obtained by the mass balance method (average throughout the 8 m long layer) and ring cutter sample results (mean void ratio representative of the top compaction surface of each 8-m long layer). As observed, the samples retrieved from the top 20 mm usually reached lower void ratios (refer to the direction of the arrows in the figure) with a mean void ratio throughout the 2.8 m height of $e = 0.532$ (associated with $\rho_d = 1.74 \text{ Mg/m}^3$). The mean void ratio of the mass balance method throughout the 2.8 m height (representative of the layer average) is $e = 0.579$, which is associated with $\rho_d = 1.68 \text{ Mg/m}^3$ (layer average values display more considerable variability between $e = 0.509$ and 0.691 , corresponding to $\rho_d = 1.57$ and 1.76 Mg/m^3). The mean bottom dry density throughout the height is expected to be $\rho_d = 1.63 \text{ Mg/m}^3$. Therefore, there is a vertical zigzag variation of dry density in the field with lower void ratios at the top of each layer consistent with the single-layer compaction at the laboratory scale, which strongly impacts the permeability, as discussed in section 5.

Referring to the case of the Albagés zoned earth dam (Pinyol & Alonso, 2019), a distinctive aspect of the project was the execution of a trial embankment during the design phase. This innovative approach addressed the inherent shortcomings of laboratory small-scale tests of compacted samples in accurately reflecting geotechnical properties at full scale (performing vertical and horizontal field permeability tests and stiffness properties by plate loading/unloading tests). Moreover, the trial embankment facilitated an in-depth exploration of the compaction technique at a larger scale and different heights of the compaction layer. The 6-layer embankment was constructed with a lift thickness of 0.25 to 0.35 m (the first two layers 1 and 2 at 0.25 m, layers 3 and 4 at 0.30 m and final layers 5 and 6 at 0.35 m). Besides the variation in thickness, each lift was compacted with different wetting protocols and combinations of equipment, including sheepfoot rollers with two masses and vibratory compactors at two frequencies (the number of passes, passing over the same point twice in each pass, was between 4 and 10 for each lift).

Figure 9 summarises laboratory data in the Proctor plane (standard energy 600 kJ/m^3 , double standard energy and modified energy 2700 kJ/m^3) and field information. The dry densities and water contents at two different depths of each field compacted layer (at 0.05 m and 0.20 m from the top) are shown in the figure. The arrows indicate the deep-shallow evolution of the different lifts' dry density and water content. In general, the compaction zone close to the surface presents higher dry densities than the deeper ones (except for layer 6 and since in layer 2 no significant variation was detected), tending to the zigzag pattern in dry density described above. As observed, S_r varied between 0.6 and 0.8 (except for layer 3), which tends to the dry side due to the semiarid conditions of the site with an average rainfall of 400 mm/year (Lleida, Spain). This drier condition

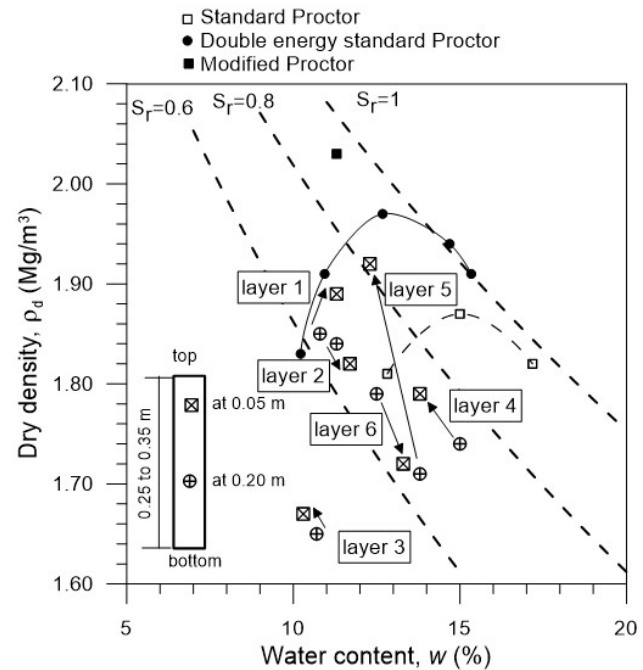


Figure 9. Compaction laboratory data (standard energy, double standard energy and modified energy) and field information of a trial embankment with six lifts of around 0.25–0.35 m thick. The arrows indicate the deep-shallow evolution of the different lifts' dry density and water content.

presents higher stiffness and hinders the achievement of high dry densities, particularly on layer 3.

5. Laboratory versus field compaction. Effects on permeability

The pattern mentioned above of zigzag variation of dry density in field compaction with a tendency towards lower e at the compaction surface has essential implications for permeability. The horizontal permeability (weighted arithmetic mean) reflects the higher permeability of the horizontal lifts at higher e (bottom of the layer). The high-density compaction surfaces condition the lower vertical permeability (weighted harmonic mean). This generates an anisotropy in the hydraulic response, whose interpretation depends on the experimental observation scale. The induced anisotropy, associated with a continuum approach, arises due to preferential vertical straining during 1-D static compaction (the usual way to reach a target dry density). This preparation technique affects the mechanical response (transversally isotropic fabric) and the permeability. Layering at a larger field compaction scale involves interfaces between lifts and low-density zones in the lift, which induces a layered structure with preferential pathways for water flow.

The first example to discuss involves the case of the Albagés zoned earth dam project (Pinyol & Alonso, 2019),

where permeability at the laboratory scale (natural and compacted samples from the field test) and field scale (trial embankment) are presented. Figure 10 summarises laboratory results at the cm-scale using controlled gradient tests under isotropic conditions (samples 50 mm in diameter and 100 mm high). Different test procedures were selected. First, field-compacted samples were retrieved from the trial embankment and then trimmed to the dimensions of a triaxial specimen at different isotropic stresses (150 and 450 kPa) and two orientations (these field-compacted samples are indicated as FCH and FCV for the horizontally and vertically oriented samples, respectively). Second, natural samples from the lightly cemented low-density silty clay with gypsum and carbonate types of cement were retrieved, trimmed to the triaxial dimensions, loaded at different isotropic stresses and finally saturated before determining the vertical permeability (NL and NH, at low and high confining stresses undergoing different collapse strains on saturation). The figure also includes field test results on the trial embankment using permeameters for vertical permeability (TEV) and Lefranc-type tests for horizontal permeability (TEH) once the embankment was completed. The figure also labels the degree of saturation before permeability determination. A second permeability determination was carried out in layers 2, 5 and 6 in the vertical permeability tests.

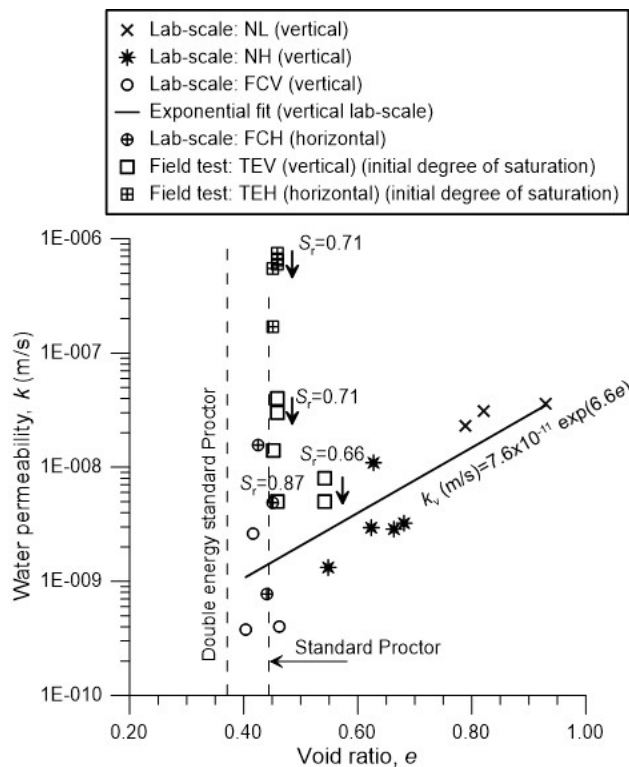


Figure 10. Permeability k at laboratory and field scales on field compacted samples at different orientations. Permeability of natural samples at different e .

The field tests capture the anisotropy induced by field compaction. The horizontal permeability (mean $k_h = 5.4 \times 10^{-7}$ m/s) was higher than the vertical permeability ($k_v = 1.7 \times 10^{-8}$ m/s) with an anisotropy ratio of around $k_h / k_v = 32$ due to preferential paths through potential interfaces and low-density zones in compaction layers (the zigzag pattern above). The permeability deduced from the field test is difficult to interpret since the compacted soil is not initially saturated. This implies that the actual infiltration gradients are different due to the incorporation of the matric suction potential (the vertical permeability slightly reduces on the second repetition of the test, as indicated by the arrows in the figure). On the other hand, the lack of saturation also modifies the permeability. The cm-scale tests in the laboratory for the fully saturated field compacted material indicated lower vertical permeability (mean $k_v = 1.1 \times 10^{-9}$ m/s; i.e., around one order of magnitude compared to field data) and lower horizontal permeability (mean $k_h = 7.1 \times 10^{-9}$ m/s) with a smaller anisotropy ratio $k_h / k_v = 7$. An expression for vertical permeability as a function of void ratio is included for lab-scale data. These results show the need to consider testing at different scales to better estimate the real application's conditions. While the tests at different scales provided reasonable control of e (references for the standard Proctor and double energy of standard Proctor are indicated in the figure), it was not the case for S_r . Differences are associated with the stress paths followed in the laboratory and field compactations, usually with distinct aggregate sizes and different procedures to transfer the compaction energy, which induce variability in dry density that depends on the scale and affects local pathways for water flow.

To better assess permeability on compacted soil at different scales with a well-controlled initial state (e and S_r), the results presented by Romero et al. (2023) associated with the field test 'GAs permeable Seal Test' GAST described in section 4 will be discussed. The study delves into process upscaling through different experimental techniques considering cm-scale element tests, innovative dm-scale mock-ups and m-scale demonstration tests (Nagra, 2024, GTS, Switzerland). The material is an artificially prepared sand/bentonite S/B mixture (80% SiO_2 sand and 20% Wyoming-type bentonite by mass described in section 4). Figure 11a shows the dm-scale mock-up (500 mm long and 300 mm in diameter). The mock-ups' semi-cylindrical shape facilitated the emplacement of dynamically compacted S/B layers with layer-parallel flow conditions (the hydraulic gradient was controlled at the lateral filter-to-filter boundaries), mimicking the field demonstrators' geometry. A mobile lid was designed to apply the necessary vertical stress and upper displacement to simulate constant volume conditions and mitigate any by-pass flow through steel/soil interfaces (a 10-mm thick granular bentonite layer was also placed at this interface). The S/B was emplaced in three layers of 75 mm thickness (with three controlled sub-layers) by dynamic compaction at standard Proctor energy of 600 kJ/m³ (Figure 11b). The initial conditions of

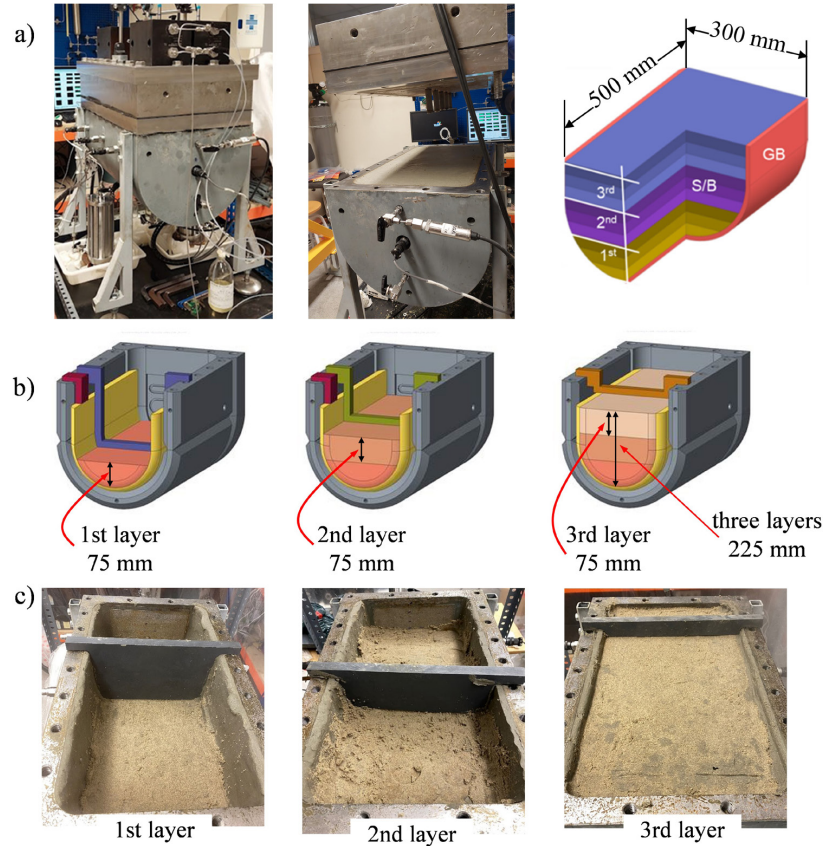


Figure 11. dm-scale mock-up developed at the Geotechnical Laboratory UPC to mimic field compaction of the GAST test and determine permeability parallel to dynamically compacted layers. (a) Photographs and schematics of the mock-up; (b) compaction layers; (c) 3D-printed tools to control compaction.

the S/B mixture were $w = (12.7 \pm 0.4)\%$ and $\rho_d = (1.66 \pm 0.01) \text{ Mg/m}^3$ (S_r around 0.56 with a maximum static fabrication stress of 1.0 MPa at $s = 0.2 \text{ MPa}$). The density of the lifts was carefully controlled using 3D-printed tools (Figure 11 c). The samples were prepared under high relative humidity to preserve their initial water content during the long-lasting, dynamic compaction process.

Figure 12 presents permeability results at the different scales, two orientations (horizontal flow parallel to compaction lifts and vertical flow) and varying e . The various scales of the laboratory tests corresponded to a) cm-scale oedometer tests (50 mm in diameter) for the vertical permeability after dynamic compaction; b) dm-scale infiltration tests in a cylindrical mould of 70 mm in diameter and 160 mm high in which the S/B was compacted in four layers (vertical flow orthogonal to compaction lifts); c) dm-scale mock-up tests described above with flow parallel to compaction layers; and d) m-scale tests for horizontal flow between piezometers installed in the GAST field test (injection flow rate and measured local pressures). The scale effects and anisotropic features are clearly shown in the figure. The induced anisotropy during laboratory compaction arises due to preferential vertical straining (transversally isotropic fabric), which affects the

flow. The dm-scale infiltration column tests were performed with flow parallel to this vertical straining, which induced lower water permeability values (the flow is conditioned by the high-density zones at the top of the compacted layers with the lowest water permeability). As observed in the figure, consistent vertical permeability in terms of e was observed in the cm-scale element and dm-scale infiltration column tests (an expression is given in the figure for k_v as a function of e). On the other hand, layering, a larger-scale concept involving compaction interfaces, has been a significant issue in the mock-up and the field demonstration tests where the low-density zones condition the flow pathways (usually at the bottom of the compacted layers). A slightly larger horizontal permeability is detected at the field m-scale. The figure shows that the anisotropy ratio at approximately equivalent e is small and typically between $k_h / k_v = 3$ and 5.

6. Summary and concluding remarks

Different examples of compacted soils used in earthworks for hydraulic, environmental and energy applications were presented to motivate experimental research and techniques at various scales (mm-scale (micro), cm-scale (element

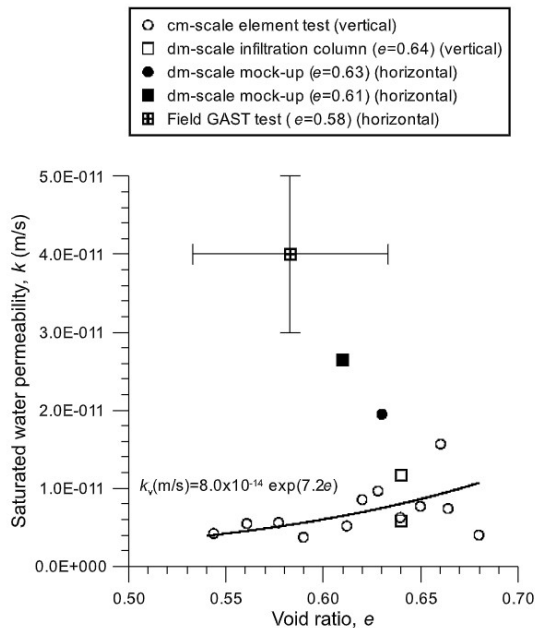


Figure 12. Saturated water permeability on S/B at different scales. cm-scale element tests, dm-scale infiltration column, and dm-scale mock-ups with flow parallel to compaction layers. GAST field test (water flow/piezometer data using injection flow rate and measured local pressures).

tests), dm-scale (mock-up tests), m-scale (demonstration/trial tests)). The paper showed the importance of selecting adequate techniques and improving experimental protocols to the appropriate length scale to capture different aspects of the behaviour of compacted soils (particular behavioural features were observed at specific length scales).

Although the experimental techniques cannot explain all the phenomena affecting compacted soils (mainly associated with the different preparation protocols followed in laboratory and field conditions), they highlight the critical role of the microstructure (aggregate size and inter-aggregate porosity) and the variability of soil properties at various scales. The contribution is helpful because it presents data on different compacted materials at multiple scales, ranging from mm- to m-scale, and they can help understand some fundamental issues observed at a larger scale.

Concerning the different experimental scales, we can indicate the following.

- Microstructural (mm-scale) and element tests (cm-scale). They help understand processes where the pore size and the aggregate/grain size distribution play an essential role (macropore connectivity between aggregates). Microstructural techniques are also helpful in validating and improving conceptual models of compacted soils. cm-scale element tests are convenient for first parameter estimation (permeability, water retention properties, volume changes on wetting, ...) and for capturing processes

under controlled initial state and boundary conditions (particle mobilisation during flow, temperature effects and chemically induced processes);

- High-quality mock-up tests (dm-scale). They help study the role of compaction layers, interfaces, and soil anisotropy under controlled initial state and boundary conditions. They are valuable in supporting larger-scale field tests. More realistic soil properties (permeability, compressibility) than element tests;
- Field/demonstration tests (m-scale). They are convenient for capturing larger-scale heterogeneous properties and assessing the impact of the compaction layers and interfaces on soil response (permeability and stiffness properties of compacted soils). Although the initial state and the boundary conditions are not so controlled. More realistic soil properties (permeability, compressibility) accounting for compaction layers, anisotropic features, and heterogeneity to predict the response of field applications;
- Model. Determine upscaled properties by considering heterogeneous zones and realistic boundary conditions (back-analysed parameters from field tests and monitoring data) to predict better the response of field applications with realistic boundary conditions and material properties (heterogeneous and anisotropic properties). Using the knowledge gained at different experimental scales is essential to help develop constitutive and numerical models to better upscale behavioural features of compacted soils.

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Declaration of interest

The author has no conflicts of interest to declare. There is no financial interest to report.

Data availability

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

List of symbols and abbreviations

e	Void ratio: $e = e_M + e_m$
e_m	Microstructural void ratio (voids inside the aggregates to the volume of solids)
e_M	Macrostructural void ratio (voids between aggregates to the volume of solids)
k	Permeability
k_h	Horizontal permeability
k_v	Vertical permeability
s	Matric suction for tensiometer data and total suction for psychrometer data
RH	Relative humidity of the air
S_r	Degree of saturation
α, β	Parameters of Equation 1
ρ_d	Dry density
ρ_s	Density of solids

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Micro to macro investigation of clays advising their constitutive modelling - part I

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Article

Keywords

Microstructure
Constitutive modeling
Soil microstructure analysis

Abstract

This keynote lecture discusses the results of a long lasting experimental research, devoted to the investigation of clay microstructure and its evolution upon loading. Micro-scale analyses, involving scanning electron microscopy, image processing, mercury intrusion porosimetry and swelling paths to test the clay bonding, are presented on clays subjected to different loading paths, with the purpose of providing experimental evidence of the processes at the micro-scale which underlie the clay response at the macro-scale. Data from the literature on clays of different classes, either soft or stiff, are compared to original results on two stiff clays, Pappadai and Lucera clay, both in their natural state and after reconstitution in the laboratory. The results presented herein allow building a conceptual model of the evolution of clay microstructure upon different loading paths, providing microstructural insights into the macro-behaviour described by constitutive laws and advising their mathematical formalization in the framework of either continuum mechanics or micro-mechanics. For editorial purposes, the research results are presented in two parts. The first part, presented in this paper, concerns the results for reconstituted clays, whereas a second part, concerning the corresponding natural clays, is discussed in a second companion paper.

1. Introduction: background and research perspectives

This keynote lecture reports on research concerned with the correspondence between the engineering behaviour of clays, at the scale of the representative element volume (REV – macro-behaviour), and the multi-scale and multi-physics processes controlling such behaviour. The main objective is to provide experimental evidence of the micro-scale processes which determine the response of consolidated clays, to support the mathematical formalization of their constitutive laws, either in the framework of continuum mechanics, or in that of micro-mechanics (Figure 1).

For natural clays, far more than for clean coarse soils, the engineering response depends not only on the grain, or particle, size distribution and mineralogy, but also on physical and chemical processes taking place in the clay history since deposition, which impact its microstructure. This has been defined (e.g., Lambe & Whitman, 1969) as a combination of fabric (i.e., the geometric arrangement of the soil particles and grains) and bonding (i.e., the inter-particle forces), where clay bonding is not necessarily of a mechanical nature (Cotecchia & Chandler, 2000). Hence, in this research, the

interpretation of the micro-scale processes determining the clay macro-behaviour has been always based upon, on one side, the direct investigation of the microstructural features, according to the scientific literature in clay micro-morphology and clay physics and, on the other, the knowledge of the clay history since deposition, i.e., the geological history for natural clays. The results of such interpretations can then be used to characterize the correspondence between classes of clays, of given composition, background history and achieved microstructural features, and classes of macro-behaviour.

Relating classes of macro-behaviour to classes of clays may support the use, in the engineering practice, of existing constitutive laws developed in the framework of continuum mechanics and elasto-plasticity (Figure 1, research line a-i). This is the case if, for constitutive laws (e.g., Schofield & Wroth, 1968; Roscoe & Burland, 1968; Gens & Potts, 1988; Gens & Nova, 1993; Rouainia & Wood, 2000; Kavvasdas & Amorosi, 2000; Baudet & Stallebrass, 2004) resulting from macro-behaviour studies (to date calibrated only based on macro-scale data and optimization strategies, e.g., Borja et al., 1997; Zhang et al., 2009; Knabe et al., 2013), the set of parameter values required to predict the class of macro-behaviour relating to clay class is provided.

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Also, the knowledge of the physical background of a given class of macro-response could support further development of the constitutive laws, e.g., implementing additional microstructure constitutive variables in the hardening law (Figure 1, research line a-ii). On the other hand, the identified relations between classes of clays, macro-response and micro-scale processes, may support the integration of micro-mechanics in the constitutive modelling of clays (Figure 1, research line b).

Background of the presented research work is the geotechnical studies of the effects of composition, deposition and consolidation conditions on the clay response (e.g. Skempton & Northey, 1952; Bjerrum, 1967; Schmertmann, 1969; Pusch, 1970; Skempton, 1970), which, since the early fifties, have shown the differences in macro-response of natural and reconstituted clays. The research insight into these differences (Burland, 1990; Leroueil & Vaughan, 1990; Hight et al., 1992; Smith et al., 1992; Cotecchia & Chandler, 1997, 2000; Cafaro & Cotecchia, 2001; Gasparre et al., 2007) has given evidence to the important influence on clay macro-response of non-mechanical processes taking place in the geological history of natural clays (e.g., thixotropy, diagenesis, weathering). Accordingly, the comparison between the macro-behaviour of natural and reconstituted clays has proved a useful

method to investigate the mechanical effects of differences in microstructure, once these are characterized through micro-scale studies of the natural clay and of the same clay when reconstituted.

Background of the presented research work is also the direct investigations of clay microstructure developed since the availability of scanning electron microscopy, SEM. In the late seventies, pioneer SEM applications to the investigation of the effects of loading on clay microstructure, i.e., post-mortem investigations (Tovey, 1973; Sfondrini, 1975; Tavenas et al., 1979; Sides & Barden, 1971; Mitchell, 1976; Smart & Tovey, 1981; Cotecchia et al., 1982; Delage & Lefebvre, 1984), already provided evidence of the microstructural changes in the background of given clay stress-strain responses. Since then, SEM, later combined with Mercury Intrusion Porosimetry, MIP (Diamond, 1970; Romero & Simms, 2008), have been used in experimental studies of the dependence of clay macro-behaviour on clay microstructure (e.g., Delage & Lefebvre, 1984; Griffiths & Joshi, 1990; Locat, 1995; Lapierre et al., 1990; Cotecchia & Chandler, 1997, 1998; Delage, 2010; Hattab & Fleureau, 2010; Hicher et al., 2000; Cetin, 2004; Monroy et al., 2010; Hattab et al., 2013; Cotecchia et al., 2016, 2019; Mitaritonna et al., 2014; Guglielmi et al., 2018, 2023, 2024; Jia et al., 2020).

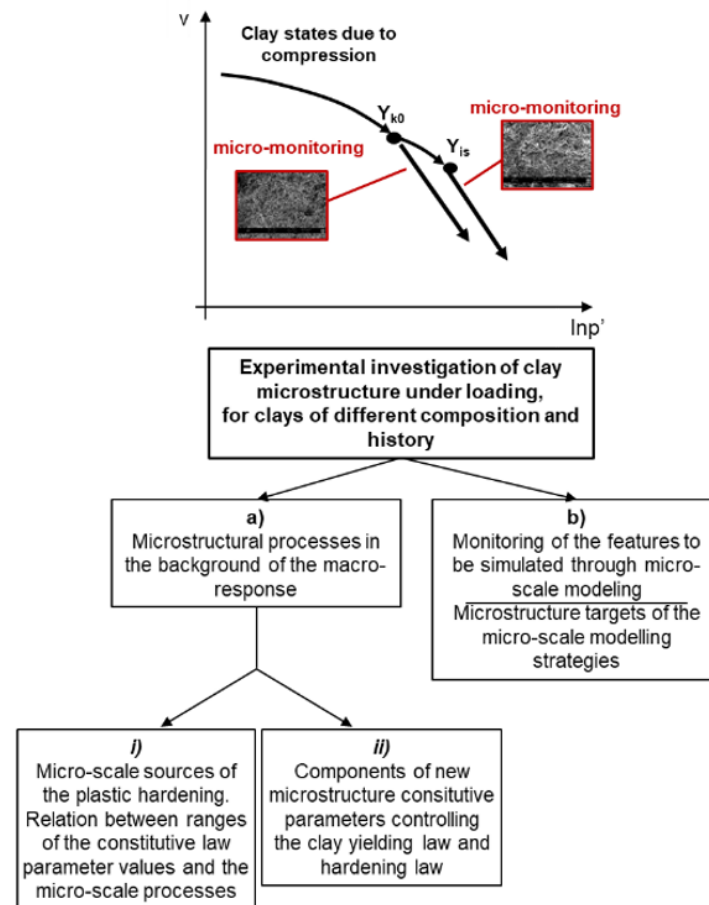


Figure 1. Research lines (a) and (b) of reference in the lecture.

Nonetheless, the exploration of micro-mechanical modelling of clays has become a main research issue only in the last decade, prompted by the parallel intense development of micro-mechanical modeling of coarse soil behaviour. The latter has indeed resulted from long-lasting research (e.g., Oda, 1972, 1993; Cundall & Strack, 1979; Oda et al., 1985; Wan et al., 2005; La Ragione & Jenkins, 2007; Li & Li, 2009), boosted recently thanks to new techniques revealing grain-scale processes in these geomaterials (Desrues et al., 2010; Hall et al., 2010; Andò et al., 2013; Viggiani et al., 2015; Guida et al., 2018; Nardelli & Coop, 2019).

It must be acknowledged, though, that the electro-chemo-mechanical processes taking place at colloidal scale in clays, subject of sciences such as clay mineralogy, crystallography, colloid behaviour and micro-morphology (e.g., Gouy, 1910; Chapman, 1913; Derjaguin & Landau, 1941; Verwey & Overbeek, 1948; Sides & Barden, 1971; Collins & McGown, 1974; Gay & Berne, 1981; O'Brien & Slatt, 1990; Gupta et al., 2011; Israelachvili, 2011), make the simulation of clay macro-behaviour through the modelling of micro-scale processes a challenge, much more complex than for coarse soils. Such modelling needs to be strongly interdisciplinary and involve systemic experimental investigation of the micro-scale processes for different classes of clays. In any case, the micro to macro investigation of clays, making use of SEM and MIP to assess the effects of loading on clay microstructure, are still too few to date.

The limited number of post-mortem micro-analyses of clays is due to the great effort required by such studies. These require the cryo-lifofilization of small clay specimens to be performed according to an appropriate protocol (Gillott, 1970; Delage & Pellerin, 1984; Delage & Lefebvre, 1984; Penumadu & Dean, 2000; Cuisinier & Laloui, 2004; Mitchell & Soga, 2005; Sasanian & Newson, 2013; Romero & Simms, 2008; Guglielmi et al., 2024) not to disturb the clay microstructure; doubts about its effects on clay slurry remain (Deirieh et al., 2018). Furthermore, relating micro-processes to clay macro-behaviour through post-mortem micro-analyses requires a great number of tests, since tests must stop at different stages of loading. In addition, a successful SEM application requires collaboration between geotechnical researchers, microscopy analysts and mineralogists. Given so, the use of alternative less-invasive technologically advanced techniques to achieve continuous knowledge of the clay fabric changes all the way through loading paths, such as X-ray Computed Tomography, Small Angle X-ray scattering (Birmipilis et al., 2019, 2022a, b) has been attempted in micro-macro investigation of clays. However, to date, 'in-operando' techniques attain very coarse spatial resolution in clays, which makes them not satisfactory for clay fabric analyses.

The mathematical modelling of clay micro-scale processes has been so far attempted through the use of the Derjaguin-Landau-Verwey-Overbeek, DLVO, model to predict non-contact forces between clay particles. Due to

the limitations of the employed theoretical assumptions, such attempts have addressed mono-mineral clay particles within simple, strictly defined scenarios (e.g., Anandarajah, 1994, 2000; Yao & Anandarajah, 2003; Frenkel & Smit, 2013; Ebrahimi et al., 2012, 2014, 2016; Liu et al., 2015; Sjoblom, 2016). Other modelling strategies entail the application of the distinct element method, DEM, to clay micro-mechanics, simplifying the inter-particle forces through equivalent mechanical elements, to be calibrated based upon either phenomenological assumptions or empirical observations (Pedrotti & Tarantino, 2018; Pagano et al., 2020). Although Molecular Dynamics, MD (Alder & Wainwright, 1959; Frenkel & Smit, 2013), is generally used for intra-particle phenomena, recent studies have explored the implementation of the DLVO theory to calibrate potential functions within molecular dynamics deployed between particles (simulated as platelets), even endorsing the implementation of the Gay-Berne potential (Ebrahimi et al., 2014; Bandera et al., 2019, 2021). Whichever micro-scale modelling strategy, though, still requires a deeper experimental insight into the clay fabric and bonding evolutions under loading to be taken as the target of the model predictions (Figure 1, research line b). To date, for multi-mineral clays, either reconstituted or natural, such insight may be still pursued solely through post-mortem micro-analyses.

In order to progress in this direction, the research work here of reference has entailed post-mortem micro-analyses of clay specimens of varying composition, either reconstituted in the laboratory or natural, both at early stages of consolidation and after compression under different constant stress ratios, η . For editorial purposes, the research results have been split in two parts. The first part, presented in this paper, concerns the results for reconstituted clays, whereas the second part, concerning the corresponding natural clays is discussed in a second companion paper. The research methodology, adopted for both the studies, on the natural and the reconstituted clays, is presented in the following section of this paper.

The investigation results provide evidence of how the interaction forces between the clay components determine different clay states (in the compression plane) and fabrics for varying composition, deposition environment (laboratory or field), constant stress ratio (η) compression and other processes taking place at the micro-scale under burial. The micro-scale investigations have made use of SEM and field emission SEM (FESEM; Gillott, 1973; Tovey & Wong, 1973), MIP (Diamond, 1970) and swelling tests (to assess the clay bonding strength; Burland, 1990). To quantify the fabric orientation, SEM and FESEM micrographs have been subjected to image processing (e.g., Martinez-Nistal et al., 1999; Hattab & Fleureau, 2010).

The micro-analysis results are intended to support advances for both research lines in Figure 1. For line a-i, they can support the use, in engineering design (e.g., Manzano et al., 2023), of constitutive laws in the framework of elasto-plasticity, since the knowledge of the micro-processes generating a prediction

achieved through a given set of parameter values, provides the engineer with a guidance to the selection of the appropriate parameter values. Furthermore, according to line a-ii, the results provide indications about microstructure constitutive parameters (either scalar or tensorial) useful to represent clay microstructure in new hardening laws (e.g., for sands: Oda et al., 1980; Oda, 1993). For clay micro-scale modelling (Figure 1, line b), the research results outline the main micro-scale processes impacting the clay macro-response, which could guide the design of such modeling.

2. Research methodology

The experimental data discussed in both companion papers (for either reconstituted or natural clays) resulted either from tests performed by the authors, or from previous published studies of other authors. In any case, the clay composition is always characterized through mineralogical and chemical analyses and geotechnical index tests (liquid and plastic limits, w_L and w_p , grading fractions, e.g., clay fraction CF). The clay macro-state is always characterized in terms of void ratio, e , and degree of saturation, S_r . Furthermore, the clay history is characterized providing: i) the original deposition conditions, ii) the preconsolidation pressure, iii) subsequent unloading/loading sequences, iv) ageing and diagenesis for the natural clays. The clay class of the specific clay under study is then identified, accordingly.

For each clay of reference, a set of one-dimensional compression tests has been carried out, together with micro-analyses to assess the clay fabric and bonding at different stages of compression. For some of the clays, one-dimensional swelling has also been performed to characterize the clay bonding strength through the swell sensitivity C_s^*/C_s (Schmertmann, 1969), since the strengthening of bonding which may develop in the natural clay tends to restrain the swell capacity. Also, different η compressions have been performed to investigate the influence of η on the clay microstructure (degree of fabric orientation and degree of mechanical anisotropy).

For most clay prototypes, the macro-tests and the micro-analyses have been carried out on both the natural and the reconstituted clay, since the comparison between the macro-behaviour and the microstructural features of the natural and the reconstituted clay provide evidence of the micro-structure effects on the clay macro-response. The reconstituted clay data are discussed in the following, while those for the natural clays are compared with the following in the companion paper. In both papers the constitutive modelling implications of the micro-scale findings are explored.

2.1 Micro-scale investigation techniques

Different techniques have been combined to investigate the clay microstructure post-mortem. The specimens have been unloaded in undrained conditions at the end of the test

and cube-shaped specimens of $\approx 1 \text{ cm}^3$ volume have been cut from their central part. Before microstructural testing, the small specimens were subjected to freeze-drying. This is routinely used for sample dehydration, to minimize the clay disturbance compared to air, or oven drying (Gillott, 1970; Delage & Pellerin, 1984; Delage & Lefebvre, 1984; Penumadu & Dean, 2000; Cuisinier & Laloui, 2004; Mitchell & Soga, 2005; Sasanian & Newson, 2013; Romero & Simms, 2008).

MIP data provide the clay pore size density function, PSD. SEM (after specimen gold coating), or FESEM (carbon coating) micrographs have been acquired for vertical fractures of the freeze-dried specimens, obtained by fracturing the soil before coating.

Fabric orientation, which may represent an internal source of anisotropy of both mechanical and hydraulic properties, has been assessed not only qualitatively (e.g., Delage & Lefebvre, 1984; Lima et al., 2008; Cotecchia et al., 2016), but also quantitatively, using image processing. This has been performed through a digital operator-independent technique (Martinez-Nistal et al., 1999; Mitaritonna et al., 2014; Cotecchia et al., 2019), successfully applied to clays in the past (e.g., Pisa clay and Pappadai clay; Veniale et al., 1993, 1995; Cotecchia & Chandler 1997, 1998) and extensively illustrated by Mitaritonna et al. (2014) and Cotecchia et al. (2019). The procedure is based on the thinning of the elongated bright regions of the micrograph, which represent the edges of oriented particle aggregates, as exemplified by Cotecchia et al. (2019)¹ and Guglielmi et al. (2024)². The resulting field of vectors is processed to derive a histogram of orientations (rosette) and a scalar statistical expression of the dispersion of the vectors with respect to the mean direction, L . L has a value of 1 for a completely iso-oriented fabric and it decreases for 'low oriented' ($0.15 < L < 0.21$) and 'randomly oriented' ($L < 0.15$) fabric (Martinez-Nistal et al., 1999).

3. The micro-scale REV

Cotecchia et al. (2019) discuss post-mortem microstructural analyses of Pappadai clay, in its natural and reconstituted state, with the aim of characterizing the micro-scale sources of the clay macro-behaviour in compression. The authors discuss the clay fabric features, through qualitative and quantitative investigations of the SEM micrographs, and provide elements characterizing the bonding of the clay when natural, through: 1) its inspection in SEM using energy-dispersive X-ray spectroscopy, EDS; 2) the measurement of C_s^*/C_{si} ; 3) the comparison of the overconsolidation ratio due to geological loading processes, $OCR (= \sigma_p'/\sigma_{v0}')$, where σ_p' is the preconsolidation pressure and σ_{v0}' the *in situ* vertical effective stress), and the yield stress ratio of the clay in compression, $YSR = \sigma_y'/\sigma_{v0}'$. The

¹ See Figure 3 in Cotecchia et al. (2019).

² See Figure 2 in Guglielmi et al. (2024).

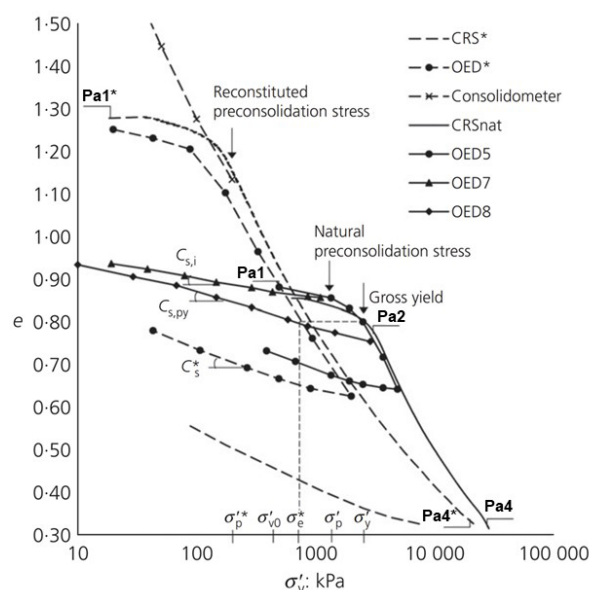


Figure 2. 1D behaviour of Pappadai clay in the $[e; \sigma'_{v,}]$ plane (modified after Cotecchia et al., 2019).

monitoring of C_s^*/C_{si} under loading provides indications of the bonding evolution with compression. Figure 2 shows the results of 1D compression and swelling tests on natural and reconstituted Pappadai clay and the clay states for which the authors discuss the micro-scale data.

The state of the natural clay (Pa1 in the figure) resulted from its geological overconsolidation (Cotecchia & Chandler, 1995; OCR = 3). When subjected to 1D compression, the clay exhibits gross yield (onset of major decay in stiffness; Hight et al., 1992), at σ'_y about twice σ'_p ($YSR \approx 2 \cdot OCR$), as a result of diagenesis under burial, which has increased the strength of the clay bonding. Through EDS, Cotecchia (1996) shows that the clay bonding increased its strength during diagenesis, partly as an effect of the development of an amorphous calcite film, binding the natural clay particles.

The reconstituted Pappadai clay (parameters*), prepared according to Burland (1990), exhibits a swelling index compatible with $C_s^*/C_{si} = 2.5$, which confirms the existence of an additional bonding in the natural clay, characterized through EDS as said above. It allows the natural fabric to achieve higher void ratio than the reconstituted along compression paths, providing the clay with a stress sensitivity, $S_\sigma = \sigma'_y/\sigma_e^*$ (where σ_e^* is the pressure on the ICL for the same void ratio of the natural clay at gross yield) of 3.5 (Figure 2).

The size of the minimum clay volume including a representative distribution of all the bonding and fabric features recurring in the clay microstructure, clay Micro-REV, has been investigated by Cotecchia et al. (2019) for both the natural and reconstituted Pappadai clay characterized above. To this aim, the authors investigate, in the SEM, the recurrence across the specimens of both the fabric and the bonding features, the latter using EDS.

The reconstituted clay 1D pre-compressed to $\sigma'_v = 200$ kPa and then swelled to Pa1* in Figure 2, is found to embody fabric features like those exemplified in Figure 3a, showing a vertical fracture of about $10^4 \mu\text{m}^2$ size area (cubic volume 10^{-3} mm^3), investigated at a 'medium scale' of 10^3 magnification (Collins & McGown, 1974; O'Brien & Slatt, 1990). Such fabric exhibits a repetitive pattern, formed of densely packed domains in face to face contact forming stacks (Figures 3b and 3e), confining either macro-pores (diameter above $1 \mu\text{m}$; Matsuo & Kamon, 1977; Guglielmi et al., 2018), or aggregates of randomly oriented particles/domains, in edge to face contact. The image processing of several medium scale micrographs of Pa1* delivers repetitive L values, in the narrow range 0.23–0.27 (e.g., Figure 3c), which suggests a repetitive medium-good orientation at this scale (Martinez-Nistal et al., 1999). A repetitive porosimetry is expected to correspond to such a fabric pattern.

For the natural clay at state Pa1 ($e = 0.88 - \sigma'_v = 414$ kPa; Figure 2), both the qualitative analysis and the image processing of several medium scale micrographs result in a repetitive fabric too. This is formed by a dense packing of stacks, locally burying either randomly oriented domains (e.g., bookhouse, Figure 3e), or macropores, or micro-fossils (Cotecchia et al., 2019³). The direction histograms deliver repetitive values of L in the range 0.24–0.37, indicative of a good orientation fabric. At higher magnifications (i.e., 10^4 – 10^5 ; clay portions of about 10^{-6} mm^3 volume; 'large scale'), in either the reconstituted specimen Pa1*, or the natural Pa1, the fabric is highly variable, from a complete preferred orientation, c.p.o., to a randomly oriented fabric, as exemplified in the figures reported by Cotecchia et al. (2019)⁴. Accordingly, for both clays, the image processing at large scale results in variable indices of orientation, $0.15 < L < 0.28$. Similarly to both Pa1* and Pa1, at high pressure, e.g. samples Pa4* and Pa4 in Figure 2, the medium scale fabric is still not uniform and, at large scale, the image processing detects highly variable L values.

The authors find that such variability in local fabric generally applies to natural and reconstituted clays 1D compressed to medium-high pressures (Guglielmi et al., 2024), providing evidence of the crucial need for a characterization of the micro-scale features within a micro-REV of the clay, if wishing to assess the micro-scale sources of the clay macro-behaviour. The micro-REV structure must include, in a repetitive way, the different local fabric and bonding features, to fulfil the role of internal variable controlling the clay macro-behaviour. For Pappadai clay, either natural or reconstituted, Cotecchia et al. (2019) show that the micro-REV size is larger than 10^{-6} mm^3 , corresponding to clay fractures of about 10^{-3} mm^3 size, examined at the medium-scale. Mitaritonna et al. (2014) are the first

³ See Figure 5 in Cotecchia et al. (2019).

⁴ See Figure 6 in Cotecchia et al. (2019).

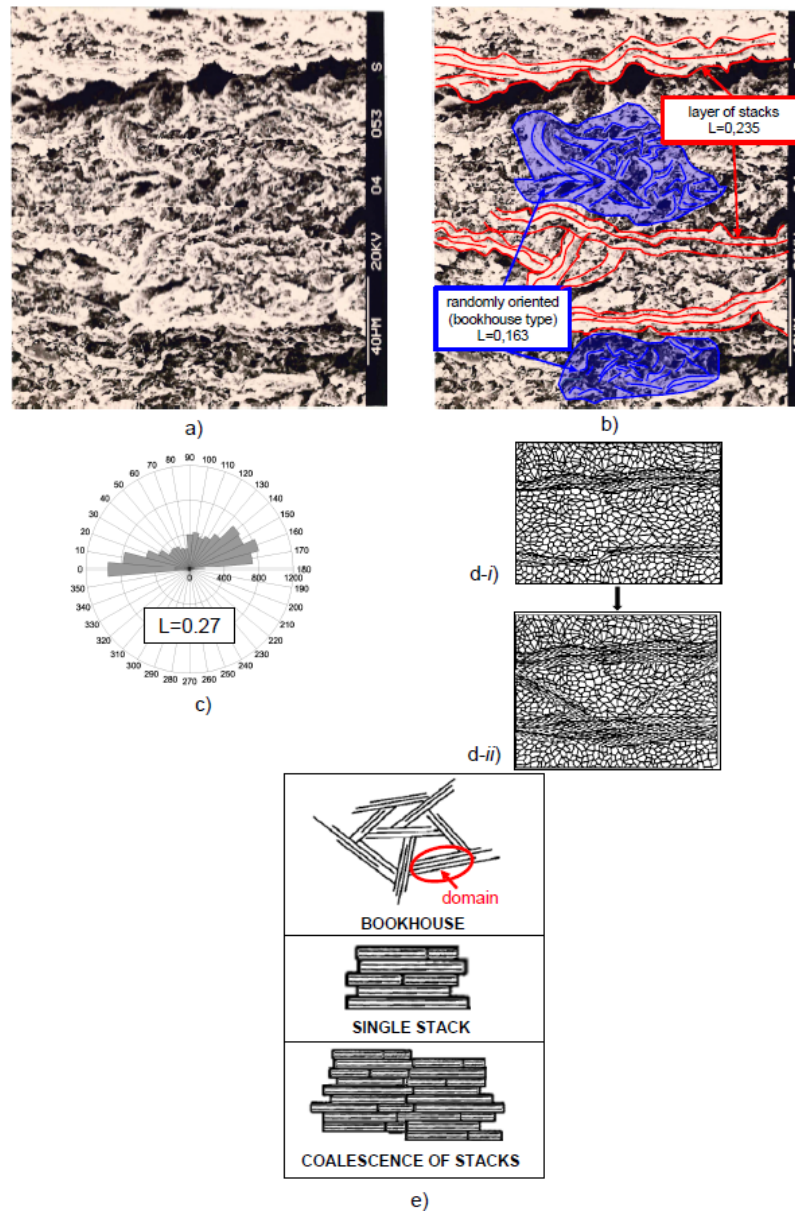


Figure 3. (a) Micro-REV fabric of reconstituted Pappadai clay (Pa1* in Figures 2 and 5), with (b) examples of different local fabric arrangements and local L values; (c) corresponding direction histogram and index of fabric orientation (modified after Cotecchia et al., 2019 and Guglielmi et al., 2024); (d-i) 1D compressed clay fabric scheme, after Sfondrini (1975), and (d-ii) its evolution in 1D compression; (e) fabric components (sketches from Sides & Barden, 1971, modified).

to provide evidence of how the clay macro-behaviour relates to the medium-scale clay fabric features, i.e., the micro-REV fabric, as further explored in the following sections.

All the results discussed above suggest a possible strategy to define the microstructure constitutive parameters, which may be either scalar or tensorial functions indicative of the influence of fabric and bonding on the clay hardening law, in macro-behaviour constitutive laws (Figure 1, research line a). These functions could formalize the influence of the degree of orientation of the micro-REV fabric, L , for

medium magnification SEM micrographs, or the influence of the micro-REV porosimetry, on the macro-response. The microstructure constitutive parameters should embody also the current bonding strength, although the quantitative characterization of bonding through micro-scale observations is not pursued yet through micro-scale investigations. Further research is still necessary for the acquisition of a database of micro-scale observations sufficient for the characterization of the microstructure parameters recalled above. To date, such a database is achievable only through post-mortem testing. The analyses of post-mortem test data

reported in the following are intended to contribute to the reaching of those objectives.

It is worth highlighting that the micro-REV scale is smaller than the scale of the single particle interactions. Hence, after the very early stage of initial deposition, the set of micro-features which is expected to impact most of the clay macro-behaviour appears to be dominated not only by the electrostatic, electromagnetic and chemical inter-particle forces, but also by the mechanical interactions of the particle aggregates (either the stacks or the flocculated aggregates; Figure 3e) which recur in the micro-REV. In the following, it will be shown how these aggregates evolve and how, at the same time, the overall micro-REV fabric changes under different η compressions.

4. Clays under study

4.1 Composition

The index properties of the clays of reference in the paper are plotted in the plasticity and activity charts in Figure 4; their mineralogical composition is also reported. Their behaviour when reconstituted in the laboratory is discussed in the paper, whereas details about their natural state and history will be provided in the companion paper, where their micro to macro behaviour will be also discussed.

4.2 Clay states and macro-behaviour in compression

Figure 5 reports data resulting from laboratory 1D compression on the reconstituted clays in Figure 4, normalized for composition using the void index $I_v = (e - e_{100}^*) / (e_{100}^* - e_{1000}^*)$ (Burland, 1990).

While the post-mortem micro-analyses for the clay states in Figure 5 allow for insight into the evolution of reconstituted clay microstructure in 1D compression, evidence of the microstructure variations activated by changes in η has been acquired through post-mortem micro-analyses of reconstituted Lucera and Gulf of Guinea clays subjected to different constant η compressions.

Mitaritonna et al. (2014) report results of such micro-analyses for reconstituted Lucera clay. This had been 1D normally compressed from slurry to $\sigma'_v = 100$ kPa ($I_v = 0$) in the consolidometer, before further constant η compression in the stress path (Figure 6), where, for this clay, 1D compression corresponds to $\eta = 0.6$. As shown in Figure 6, η was set to range from 0 to 0.8 in the stress path testing, during which the evolution of clay stiffness anisotropy, in terms of ratio G_{hh}/G_{vv} , was measured by means of bender element tests, as discussed by the authors.

In particular, the specimens were subjected to $\eta = 0, 0.3, 0.6$ in Tests 1, 2, 3 respectively, and to $\eta = 0.8$ in both Tests 4 and 5 (Figure 6). The state paths followed by the

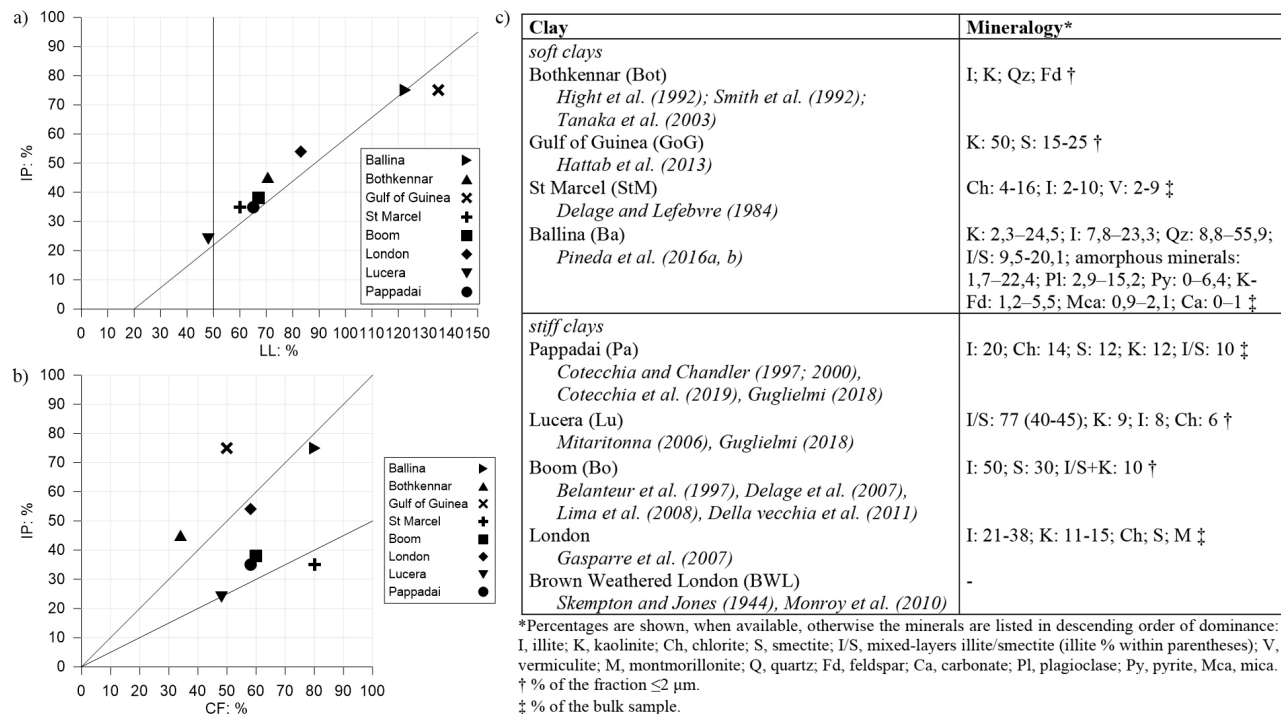


Figure 4. (a) Plasticity chart, (b) activity chart and (c) mineralogy of the clays analysed in the lecture (after Guglielmi et al., 2024; data after Bishop et al., 1965; Burland, 1990; Cotecchia & Chandler, 2000; Delage & Lefebvre, 1984; Hattab & Fleureau, 2010; Smith, 1992; Pineda et al., 2016a).

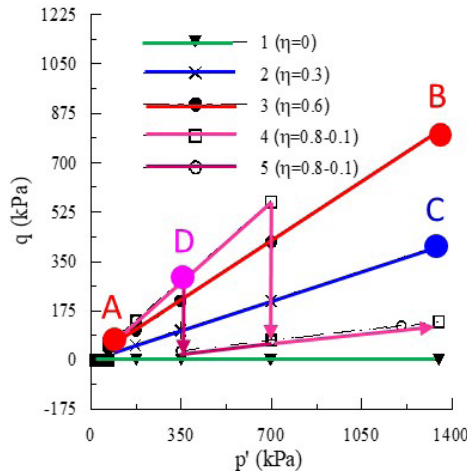


Figure 6. Results of constant- η compression tests in the q - p' plane (adapted after Mitaritonna et al., 2014).

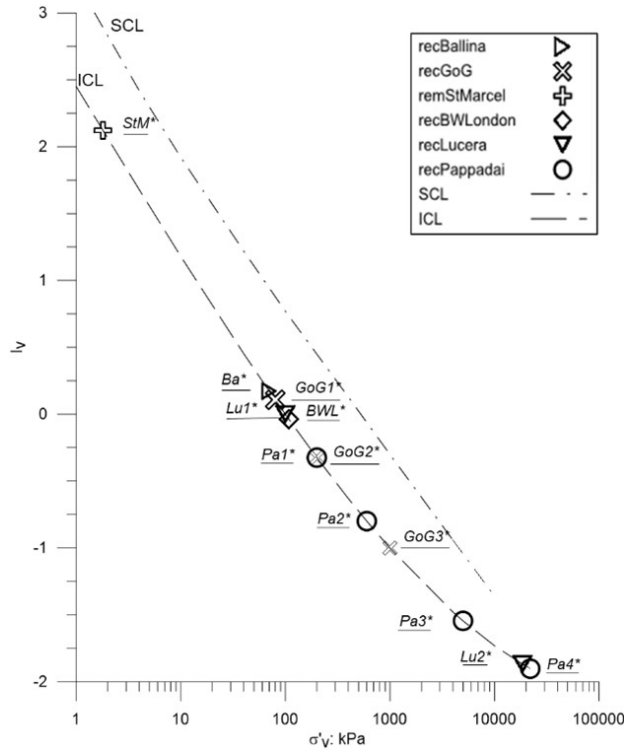


Figure 5. Reconstituted clays in Figure 4: 1D normally consolidated states subjected to micro-analyses (data after Bishop et al., 1965; Burland, 1990; Cotecchia & Chandler, 1997 and 1998; Cotecchia et al., 2019; Delage & Lefebvre, 1984; Hattab et al., 2013; Smith, 1992; Monroy et al., 2010; Pineda et al., 2016a, b; Cotecchia et al., 2019; Guglielmi et al., 2024). ICL: intrinsic compression line; SCL: sedimentation compression line (Burland, 1990).

specimens in the compression plane: specific volume, v , versus $\log p'$, are reported in Mitaritonna et al. (2014)⁵. The roughly parallel normal consolidation lines, NCLs, followed by the

clay state in different η compressions confirm the shift to the right of the NCL for decreasing η . Also, the data show the shift of the clay state from one η NCL to the other while undergoing a variation in η (Tests 4 and 5). The reconstituted clay fabric was investigated before and after stress path testing (states shown in Figure 6), by means of SEM analyses and digital image processing; the corresponding microscale data are recalled in the following section.

5. Reconstituted clay microstructure at different stages of 1D compression

Figure 7 shows SEM micrographs taken from Delage & Lefebvre (1984), Pineda et al. (2016b) and Hattab et al. (2013) on vertical fractures of the clay specimens of $I_v > 0$ in Figure 5; these micrographs are analysed herein to provide knowledge on the microstructure of such clays, with the purpose to build a conceptual model. The SEM micrographs in Figures 7a and b refer to state StM* of St Marcel clay, the least plastic of the soft clays of reference (Figure 4). The micrographs of the higher plasticity clays (Figure 4), Ballina and Gulf of Guinea clay, at states Ba* and GoG1* respectively in Figure 5, are shown in Figures 7c, d and e. The void ratios of all the specimens investigated in Figure 7 are high and very close (see the void ratios indicated in the figure), but the I_v value of StM* is highest (Figure 5), due to the much lower liquid limit of this clay of high content in rock flour.

For all the three clay specimens, of $I_v > 0$, the fabric is found to be formed of flocculated aggregates of particles and domains (e.g., card-house to bookhouse aggregation; first sketch in Figure 3e). The randomness in particle orientation holds from the large to the medium scale (recognizable at the different magnifications, Figure 7). In the higher activity clay specimens Ba* and GoG1*, a spatial variability in the density of packing of the flocculated particles and domains is recognized, since densely flocculated aggregates are recognizable, within which the intra-aggregate porosity is formed by hardly detectable pores, much smaller than the pores between the aggregates, which appear to be connected by domains acting as bridges. Such inter-aggregate pores are about 1 μm in size (Figure 7). Such variability in density of the flocculated packing across the whole fabric is more evident in the higher activity Ba* and GoG1* specimens, than in the low activity StM* specimen.

It follows that the micro-REV fabric of all three specimens at $I_v > 0$ is classifiable as random (low average orientation; Delage & Lefebvre, 1984; Pineda et al., 2016b; Hattab et al., 2013), as typical for card-house to bookhouse fabric classes (Sides and Barden 1971). Especially for the higher plasticity clays (Ba* and GoG*), the fabric appears to match the honeycomb sub-class (Sides & Barden, 1971; Collins & McGown, 1974), sketched in Figure 7f modified after Griffiths & Joshi (1990). Honeycomb fabric features had been already recognised for a high-water content reconstituted kaolin ($e \approx 2^*e_{wL}$) at an early stage of compression by

⁵ See Figure 4 in Mitaritonna et al. (2014).

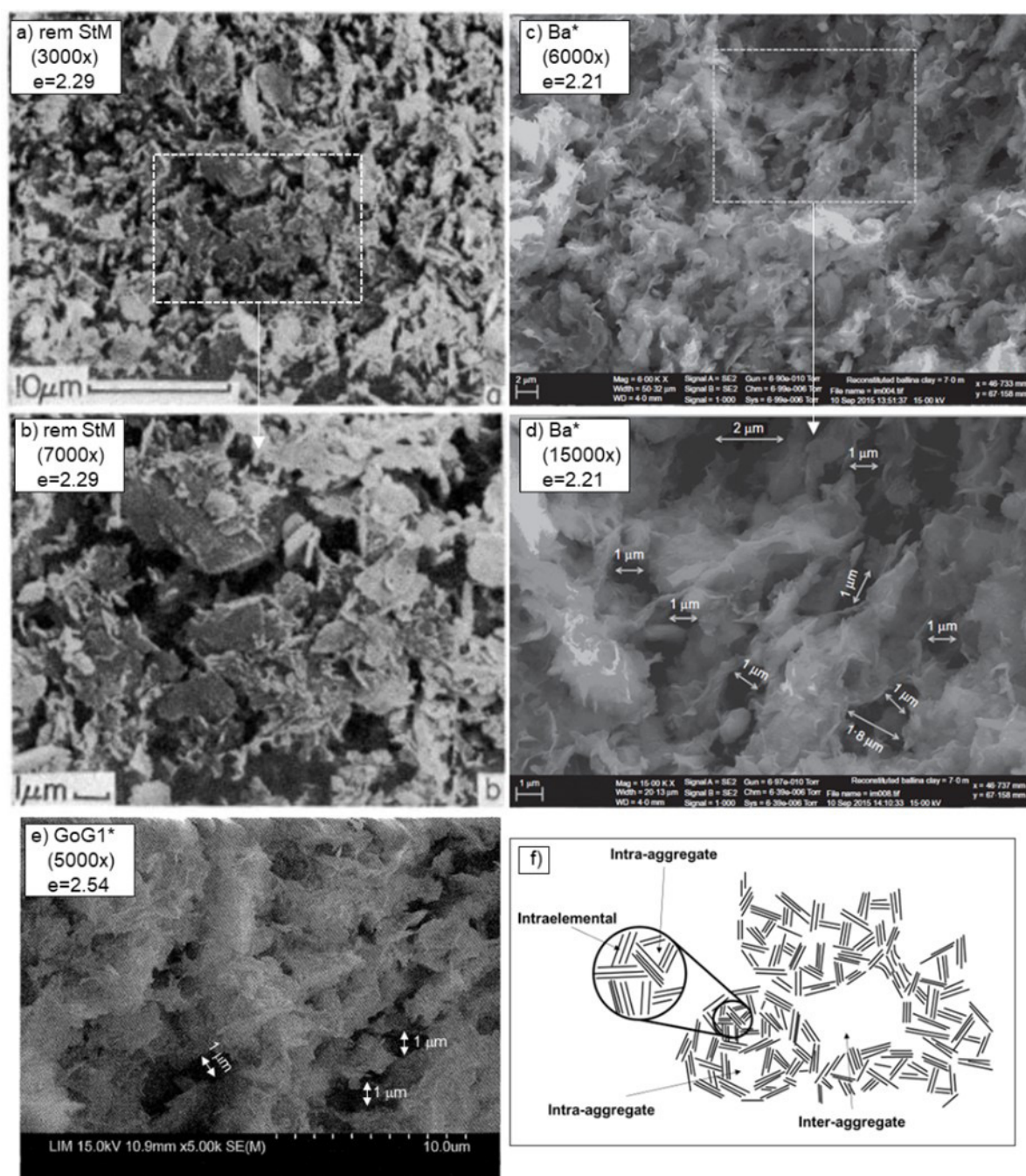


Figure 7. SEM micrographs corresponding to the clays states in Figure 5: StM* - a, b; Ba* - c, d; GoG1* - e. Sketch of honeycomb fabric (f) modified after Griffiths & Joshi (1990). Sources of the micrographs: Delage & Lefebvre (1984); Pineda et al. (2016b); Hattab et al. (2013).

Cotecchia et al. (1982) (see Guglielmi et al., 2024), and for an illitic clay slurry at the liquid limit by Griffiths & Joshi (1990), who distinguished: the smallest pores within either the domains, or the most tightly aggregated floccules, as intra-elemental (Figure 7f); the relatively higher pores within the flocculated aggregates as inter-group (intra-aggregate in the following); the larger pores confined by the aggregates and the bridges as inter-assemblage (inter-aggregate in the following).

The porosity features detected through the SEM investigation are confirmed by the MIP data in Figure 8 (data from Guglielmi, 2018). Both the high activity Ba* and GoG1* specimens include a bimodal pore size distribution, PSD, with a large dominant pore size (DPS) for the inter-aggregate porosity, around 900 nm for Ba* and 1250 nm for GoG1*, and a DPS around 60 nm for the intra-aggregate porosity. For StM*, instead, the PSD curve (replotted from Delage & Lefebvre 1984) is characterized by a single DPS around 444

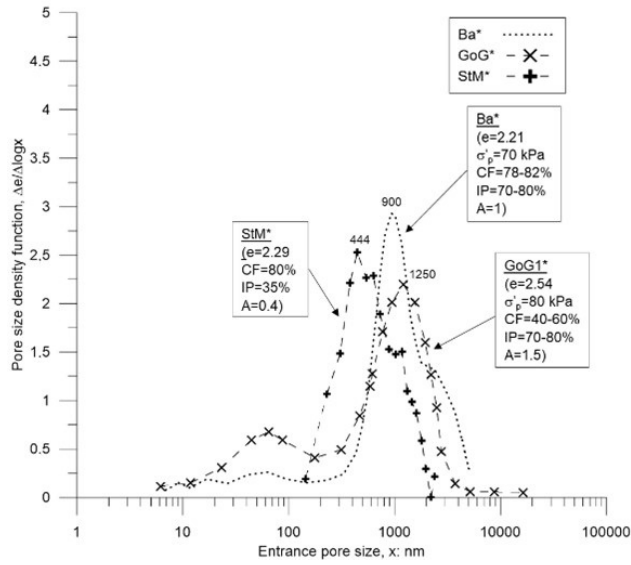


Figure 8. Comparison of PSD curves for StM*, GoG1* and Ba* in Figure 5 (data from Delage & Lefebvre, 1984; Hattab et al., 2013; Pineda et al., 2016b; modified after Guglielmi et al., 2024).

nm and by significant pore volume distributed across the pore sizes, both below and above the DPS (the latter representing inter-aggregate pores). Therefore, the PSD curves in Figure 8 suggest an influence of the activity index A on the PSD of the flocculated clay, with a possible shift from a mono-modal to a bi-modal PSD with increasing A . However, Yu et al. (2016) recognized a bimodal PSD for a kaolin slurry (i.e., a low activity clay) reconstituted at $2w_L$ and consolidated to $\sigma'_v = 50$ kPa. Hence, the influence of composition and boundary conditions on the tendency of low plasticity clays to acquire either a mono-modal, or a bi-modal PSD at early stages of compression needs further investigation.

The previous micro-scale data have been compared with those obtained for the reconstituted specimens 1D compressed to higher pressures in Figure 5. The results of SEM analyses carried out on reconstituted Lucera clay (of activity close to that of StM*), Lu1*, consolidated to $e = 0.92$ and $I_v \approx 0$ (Figure 5), are shown in Figure 9. The micrographs reveal typical features of a compressed bookhouse fabric (Sides & Barden, 1971). The image processing indicates, for both the micro-REV micrographs

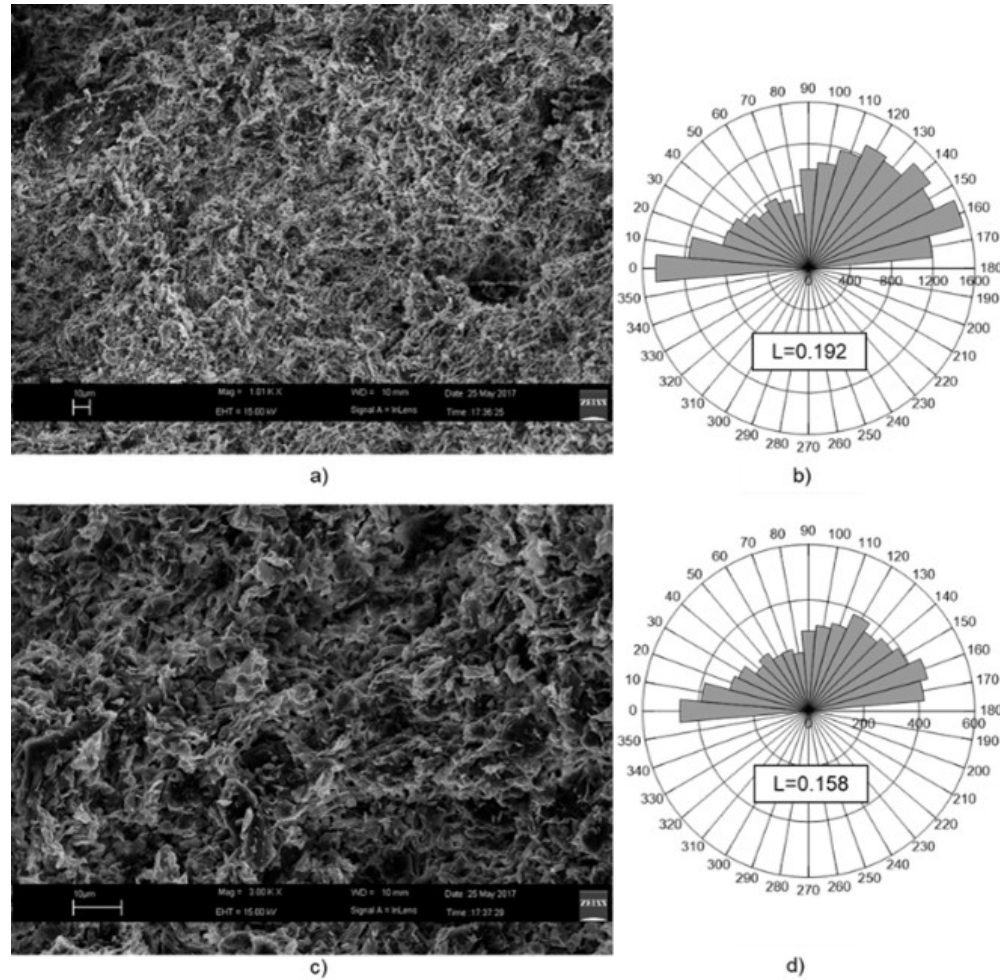


Figure 9. (a, c) FESEM micro-REV micrographs of Lu1* in Figure 5 and (b, d) corresponding direction histograms and indices of fabric orientation (modified after Guglielmi et al., 2024).

in Figure 9, a low orientation degree, since the index of fabric orientation does not reach $L = 0.21$. Hence, the data indicate that the 1D compression of the reconstituted clay to $I_v \approx 0$ increases the fabric orientation, but it does not yet generate a highly oriented fabric by this stage of compression. However, Cotecchia et al. (2019, 2020) found that for reconstituted Pappadai clay, of slightly higher plasticity and activity than Lucera clay (Figure 4), compression to I_v slightly beyond 0, as for Pa1* in Figures 2 and 5, results in a well-oriented micro-REV fabric, characterized by an index of fabric orientation $L = 0.23 - 0.27$ (e.g., Figure 3c). In this case, the fabric matches a repetitive alternation of long sub-horizontal stacks, of highly oriented particles (Figure 3e), and aggregates of randomly oriented particles/domains in edge to face contact (see §3 and Cotecchia et al., 2019, 2020 for other micrographs). On the whole, the 1D compression of the clay beyond $I_v = 0$ (e.g. from Lu1* to Pa1*) seems to determine a fabric evolution exemplified in Figure 3d: sub-horizontal stacks form across the random fabric, as originally sketched by Sfondrini (1975) (Figure 3d-i); thereafter, additional stacks form and the first ones thicken, as sketched in Figure 3d-ii. The image processing provides quantitative evidence that the degree of orientation of the micro-REV fabric of reconstituted clays is initially very low, but increases through 1D compression and can become high ($L \geq 0.21$) early beyond $I_v = 0$ ($\sigma'_v > 100$ kPa). However, the degree of orientation of the clay fabric is far from uniform across the micro-REV of $L \geq 0.21$; conversely, as observed at large scale (§3), it is very high within the stacks and low in the flocculated fabric preserved between the stacks.

Figure 10 shows the PSD curves of Pa1* and Lu1* (Figure 5) and the PSD function obtained by Monroy et al. (2010) for reconstituted brown weathered London clay (Figure 4), BWL* in Figure 5, whose fabric is characterized by a

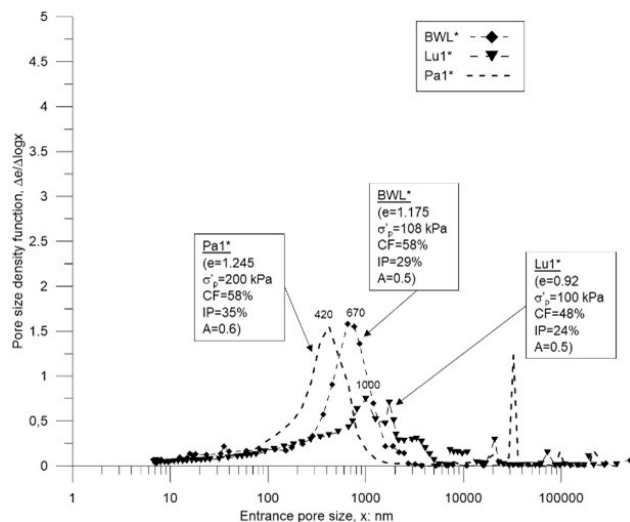


Figure 10. Comparison of PSD curves for Pa1*, Lu1* and BWL*, after compression in consolidometer (data for London clay after Monroy et al., 2010; modified after Guglielmi et al., 2024).

recognizable degree of orientation as well (Monroy et al., 2010). Lu1* is characterized by a less regular PSD than both Pa1* and BWL*, with a larger DPS ≈ 1000 nm, probably as a result of both its slightly coarser composition and the less oriented fabric, achieved by $I_v \approx 0$. The other two specimens, of similar CF, A and high degree of orientation, embody a markedly mono-modal PSD, with a DPS $< 1 \mu\text{m}$, in the range of the micro-porosity (Matsuo & Kamon, 1977), and a limited macro-porosity, distributed over a wide range of entrance pore sizes (data from Guglielmi, 2018; Guglielmi et al., 2018). The slightly higher value of DPS of BWL* may be due to its lower compression with respect to Pa1*. On the whole, with 1D compression the increase in the degree of fabric orientation is found to be accompanied by the onset of a more regular and monomodal pore size distribution, as well as a reduction in the DPS (Figure 10). The DPS tends to correspond to the porosity present within the stacks, as recognized by Guglielmi et al. (2024) through the micrograph inspections. Hence, these results enhance the previous knowledge about the changes in PSD of clays under 1D compression (Delage & Lefebvre, 1984; Lapierre et al., 1990; Tanaka & Locat, 1999; Hattab & Fleureau, 2010; Hattab et al., 2013), indicating that both the inter-aggregate and the intra-aggregate porosity change with compression.

While all the micro-scale data discussed above refer to reconstituted clay specimens at the end of compression in the consolidometer, further fabric evolution in 1D compression has been investigated through micro-analyses on clay specimens of GoG*, Pa* and Lu* compressed to higher pressures in the oedometer (GoG1*, GoG2* and GoG3*; Lu1* and Lu2*; Pa1*, Pa2*, Pa3* and Pa4*, Figure 5).

The PSD (Figure 11a) and SEM data for both GoG2* ($\sigma'_v = 200$ kPa) and GoG3* ($\sigma'_v = 1000$ kPa) have been reported by Hattab et al. (2013). These confirm that 1D compression reduces the inter-aggregate DPS both in size and frequency, while the medium scale orientation of the particles increases (Hattab et al., 2013). An increase in fabric orientation is observed also by Hicher et al. (2000) based upon SEM micrographs of both kaolinite and bentonite slurry compressed to 1 MPa and ≈ 6 MPa, respectively.

1D compression of Lu*, from Lu1* at $I_v \approx 0$, to very high pressure, Lu2*, is found to change the initial irregular PSD curve of Lu1* into a neat monomodal curve (Figure 11b), with an intra-aggregate DPS which undergoes a major decrease with compression to high pressure. Therefore, the data confirm that compression first causes the collapse of the inter-aggregate porosity, but further on it causes the reduction in size of the intra-aggregate DPS. Since the DPS occurs within the stacks (Guglielmi et al., 2024), 1D compression to high pressures is found to cause volumetric straining of the stacks.

Figure 11c shows that the compression of Pa* from Pa1* to Pa2* and Pa3* at high pressures (Figure 5), also causes the shift of the DPS towards lower pore sizes, i.e., reduction of the intra-aggregate pore sizes.

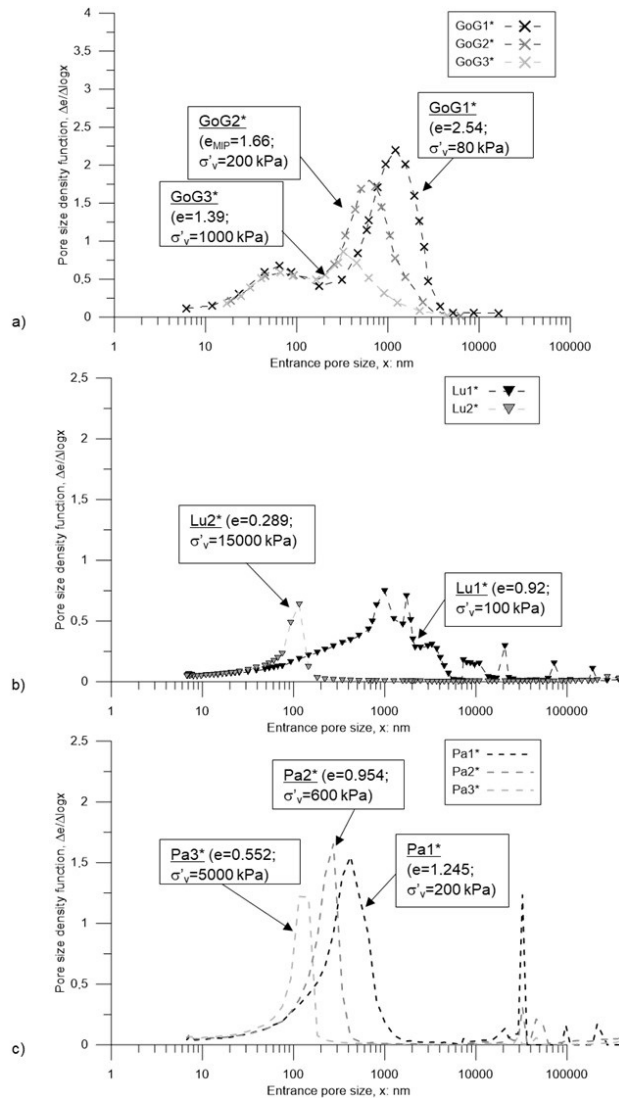


Figure 11. Evolution in 1D compression of PSD curves of reconstituted clays: Gulf of Guinea (a), Lucera (b) and Pappadai (c) (states in Figure 5; data from Guglielmi, 2018; modified after Guglielmi et al., 2024).

Furthermore, Cotecchia et al. (2019) also show that Pa* clay fabric compressed from Pa1* to Pa4* at $\sigma'_v = 22$ MPa (Figure 5), is characterized by an index of micro-REV fabric orientation $L = 0.24$ (Figure 12), which is about that measured for specimen Pa1*. Hence, 1D compression is found to determine the onset of a high degree of micro-REV fabric orientation quite early beyond $I_v = 0$, but its continuation up to high pressure does not determine any significant further increase in such orientation degree, which is found to remain constant.

This observation is consistent with the recognition that the particle orientation does not become ubiquitous through 1D compression, since layers of bookhouse fabric are found preserved even at very large pressures, as shown in Figure 12, sketched in Figure 3d-ii and discussed by Guglielmi et al. (2024). All the changes in

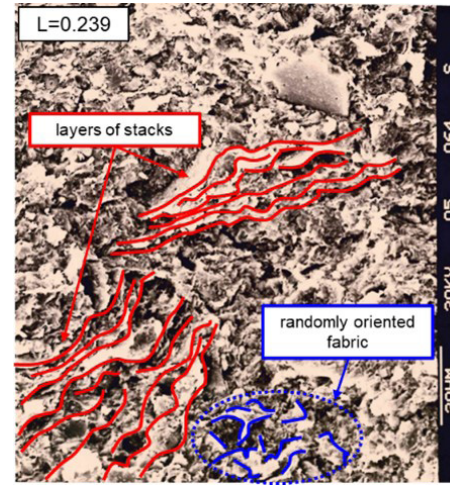


Figure 12. Compressed reconstituted Pappadai clay, Pa4* (Figures 2 and 5): medium magnification micrograph (with examples of different local fabric arrangements) and index of fabric orientation (modified after Cotecchia et al., 2019).

reconstituted clay fabric and PSD characterized through the data presented above are synthesized in the framework shown in Figure 13.

6. Compressions at different stress ratios, η

Figure 14 shows SEM micrographs resulting from the micro-scale investigation programme performed on reconstituted Lucera clay and quoted in §4.2, which has been extensively presented by Mitaritonna et al. (2014). The micrographs refer to clay specimens subjected to the different constant η compressions shown in Figure 6 and, for each of them, the figure shows also the direction histograms resulting from image processing and the corresponding index of fabric orientation L .

The SEM micrographs are representative of many others acquired in the investigation of the medium scale fabric features of the clay, i.e. of the micro-REV clay fabric, after loading to the stress states A, B, C and D in Figure 6. In particular: A refers to the state of the reconstituted clay 1D consolidated ($\eta = 0.6$) to $\sigma'_v = 100$ kPa in the consolidometer; B to the state achieved by the same clay when, after 1D consolidation to state A, is further 1D compressed (constant $\eta = 0.6$) up to $p' = 1400$ kPa; C to the state of the clay, initially consolidated to state A, and thereafter compressed at constant $\eta = 0.3$ up to $p' = 1400$ kPa; D to the state of the clay initially consolidated to state A, and thereafter compressed at constant $\eta = 0.8$ up to $p' = 350$ kPa.

The SEM data for both specimens A and B (Figures 14 a and b) confirm that the micro-REV fabric achieved in 1D compression to $I_v \geq 0$ is formed of sub-horizontal stacks of domains inter-layered with trusses of domains (bookhouse-like), as sketched in Figure 13; hence this fabric is not uniform at the large scale. Also, the direction histograms corresponding to states A and B confirm that the index of fabric orientation, L , becomes

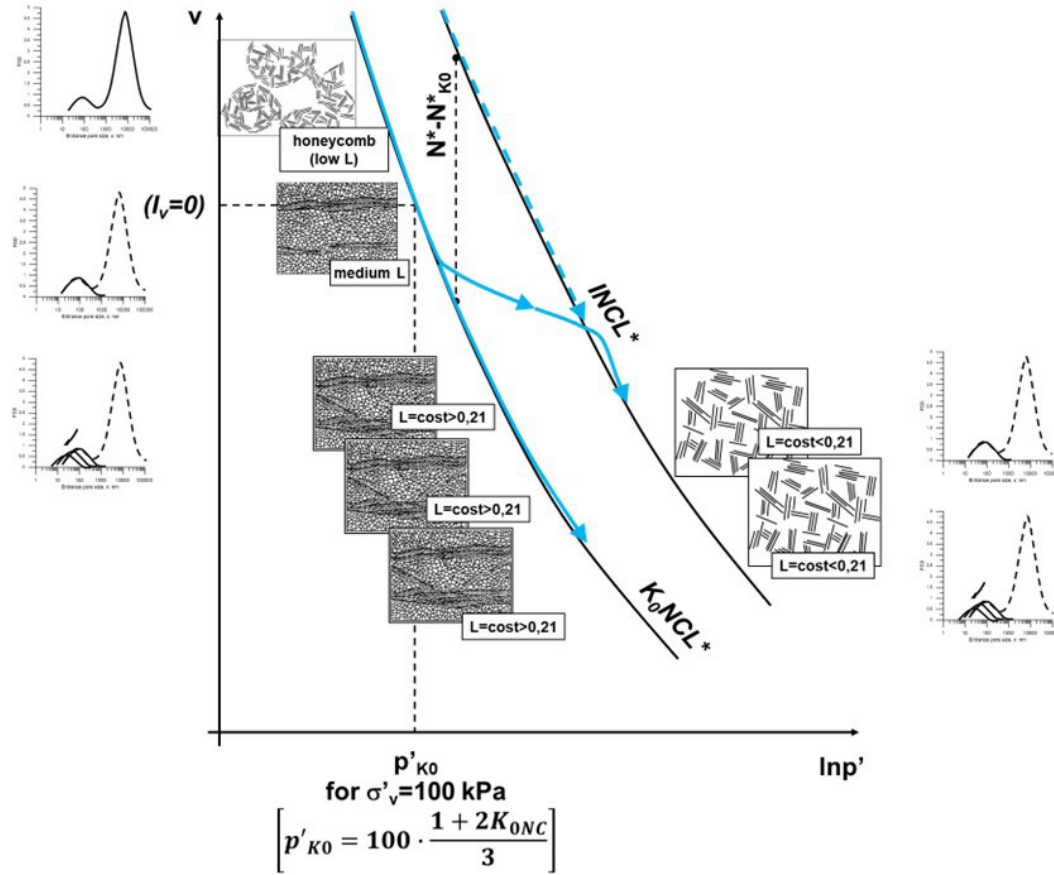


Figure 13. Schematic framework of the evolution in microstructure of reconstituted clay subjected to 1D and isotropic compression.

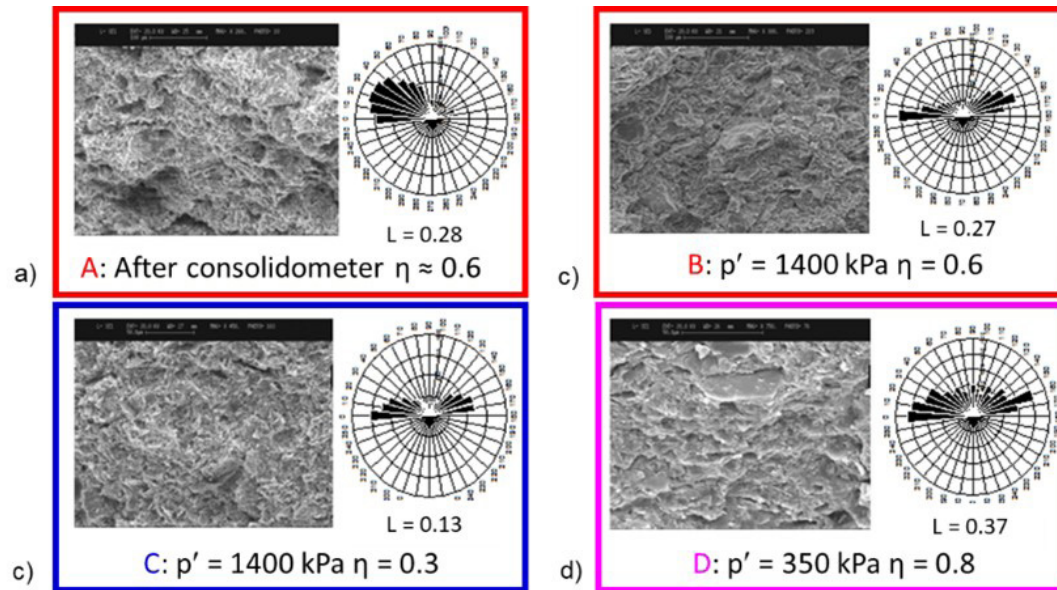


Figure 14. Micro-REV fabric of reconstituted Lucera clay after different η compression tests and corresponding direction histograms and indices of fabric orientation (Mitaritonna et al., 2014).

high in 1D compression by $I_v = 0$, or slightly beyond (in this case about 0.28), and that further yielding of the clay in 1D

compression to high pressure does not increase such degree of orientation. The micro-scale observations achieved for the clay

fabric after different constant η compressions beyond gross yield, instead, show that the fabric features change with η .

Figure 14c shows that the initially oriented fabric, corresponding to state A in Figure 6 (Figure 14a), experiences a severe rearrangement into a far less oriented one when brought to yield and further normally compressed at $\eta = 0.3$ up to $p' = 1400$ kPa, as demonstrated by the drop in L to ≈ 0.13 for state C in Figure 6 (Figure 14c). Such a drop in L makes the fabric in C be classified as random (Martinez-Nistal et al., 1999). Instead, Figure 14d shows that the oriented fabric at state A experiences an accelerated increase in fabric orientation when further compressed under $\eta = 0.8$, so that the index of fabric orientation L attains a very high value, 0.37, at $p' = 350$ kPa (state D in Figure 6).

It is worth highlighting that the data from Mitaritonna et al. (2014) portray the change in fabric orientation that can be achieved by a medium density reconstituted clay (1D compressed to $p' \approx 70$ kPa), through a three times increase of p' imposed at a constant η different from the initial one. Consistent results have been achieved by Hattab & Fleureau (2010) for initially 1D consolidated reconstituted kaolinite, which is found to achieve a more random fabric through isotropic compression to a p' over 12 times the initial one. The behaviour of reconstituted kaolinite under isotropic compression was also investigated by Hicher et al. (2000), who found that the increase of p' of about 3 times in isotropic conditions generates a rather isotropic particle orientation for an initially oriented fabric. Sivakumar et al. (2002), instead, claim that kaolin specimens consolidated isotropically from slurry to $p' = 500$ kPa retain evidence of an anisotropic microstructure; however, this may be due to the far larger pressures which are required to generate a change in fabric degree of orientation by changing η . The different micro-REV fabrics found to be achieved by reconstituted clays under different constant η compressions are sketched, for η_{KO} and η_{θ} , in Figure 13. Further micro-scale investigations are still required to assess the reconstituted fabric changes over small p' increases after the change in η .

Data are available also about the PSD changes taking place in the clay when moving from a constant η NCL to another one. Hattab et al. (2013) report microstructural analyses of reconstituted GoG clay after isotropic compression starting from state GoG1* (Figure 5). Figure 15 (data from Guglielmi, 2018) shows the PSD changes occurring with isotropic compression to GoG*_{iso}, by comparison with the PSD changes achieved through 1D compression to GoG3* (Figure 5). For the two specimens, compressed from the same initial state, the volumetric strain ϵ_{vol} induced by further compression is calculated by Hattab et al. (2013) and reported in the figure. A mild bimodal PSD is still preserved in the 1D compressed state GoG3*. Conversely, despite the volumetric strain in either the isotropic ($\epsilon_{vol} = 30\%$) or the 1D ($\epsilon_{vol} = 32\%$) compression being almost the same, a larger change in PSD is observed in the isotropic case, for which the inter-aggregate porosity is reduced, and the PSD changed

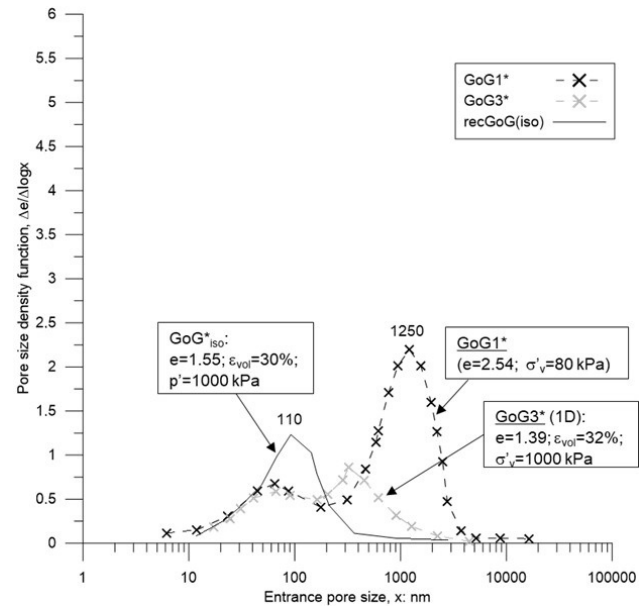


Figure 15. Comparison of PSD curves after isotropic (GoG*_{iso}) and 1D compression (GoG1* and GoG3*) for reconstituted GoG clay (states GoG1* and GoG3* in Figure 5; data after Hattab et al., 2013); modified after Guglielmi et al. (2024).

from bimodal to monomodal. This PSD change is achieved despite micrographs of the isotropically compressed specimen revealing that fabric orientation is still locally preserved.

7. Summary of the findings

From the above data the following findings can then be extracted:

- Medium-low to high activity clay particles, within a slurry settling at high I_v (> 0 ; liquidity index $LI > 1$) in 1D conditions, tend to aggregate according to a repetitive card-house to bookhouse mode (Figure 3e), with a separation between distinct aggregates, which results in a honeycomb fabric (Figure 7f), increasing the higher the clay activity index is. Hence, medium to high plasticity reconstituted clays achieve a low orientation micro-REV fabric, of honeycomb type (Figure 7f) at early stages of compression, with $L \ll 0.2$ and significant inter-aggregate porosity (major DPS $\approx 1 \mu\text{m}$). However, the PSD is bimodal for high plasticity clays, since the most active particles (e.g., the smectite fraction) allow for the presence of a far lower DPS within the aggregates, whereas this is not the case for the less active clays.
- When the clays are consolidated under 1D compression towards $I_v \approx 0$ ($\sigma'_v = 100$ kPa), a progressive reduction of the inter-aggregate porosity of the honeycomb fabric and a reduction of the macro-porosity for either the overall card-house, or bookhouse, fabric of the less active clay occurs; in both cases the larger

pores, of size about $1\mu\text{m}$, are reduced in size and the clay PSD heads towards a monomodal PSD, with DPS in the range of the hundreds of nm. Such a process is associated with an increase in the degree of orientation of the micro-REV fabric. This appears to result from the piling up of face-to-face contact particles, or domains (Figure 3e), within stacks, which tend to acquire a sub-horizontal orientation. For the originally honeycomb fabric, it can be observed that there are inter-aggregate bridges which first contribute to the collapse of the inter-aggregate porosity and which tend to adhere to the stacks. However, such a more orderly fabric is still of low average orientation by $I_v \approx 0$ (Figure 13).

- With increasing σ'_v , below $I_v \approx 0$ the micro-REV fabric achieves an increasing orientation degree at a rate dependent on the plasticity index. Further 1D compression generates further reorientation of particles, which coalesce with the stacks or form new ones, as sketched in Figure 3d-ii and Figure 13. Part of such an orientation process contributes to the thickening of the sub-horizontal stacks.
- Despite such an orientation process, original bookhouse fabric portions are preserved. Hence, the orientation of the particles under 1D compression is not at all ubiquitous, also for $I_v < 0$ (Figure 13). Evidently, the inter-particle forces within the preserved bookhouse portions, which are largely of either electrostatic, electromagnetic, or chemical nature, resist the mechanical action, so that these aggregates act as a whole mechanical fraction, undergoing straining (volumetric and shear), but keep the random fabric.
- At the same time, the achieved monomodal PSD of the clay undergoes an evolution characterized by a reduction in both the frequency and the size of the DPS, due to a reduction in porosity of the stacks and of the random fabric aggregates which are preserved (see sketch in Figure 13).
- Possibly further orientation of particles may occur with further 1D compression to high pressures, and the additional oriented particles are likely to adhere to and thicken the stacks (Figure 3d-ii). However, the micro-REV fabric orientation degree, L , is found to reach a threshold value and not increase above it. Such an important finding may be justified if a different deformability between the stacks and the flocculated fabric portions, preserved between the stacks, is accounted for. Indeed, the reduction in size of the intra-aggregate pores, occurring with 1D compression, may be expected to be higher within the stacks than within the bookhouse fabric portions, due to the higher inter-particle forces in the bookhouse fabric. Consequently, the straining of the stacks compensates their thickening due to further particle orientation and the proportion of

the micro-REV fabric having a very high degree of orientation is not found to increase when inspecting the medium magnification micrographs (i.e., the ratio between the size of the stacks and the size of the bookhouse fabric portions in the medium scale fabric analyses remains reasonably constant).

- No micro-scale observations are available for the fabric changes generated by lower η compression (e.g., isotropic) of a reconstituted clay fabric 1D consolidated to $I_v > 0$. However, MIP data for high I_v clays compressed isotropically have shown that the reduction in η leads to a faster collapse of the inter-aggregate porosity with respect to higher η compressions. Indeed, a η decrease affects the inter-aggregate voids sooner.
- With regard to the fabric changes caused by different constant η compressions of a medium dense 1D consolidated clay, Mitaritonna et al. (2014) show that, starting from $I_v \approx 0$, the change in η results in significant variations in fabric orientation if the clay is compressed to medium-high pressures.

8. Preliminary conclusions about the modelling implications of the micro-scale data

The results discussed above represent a database useful for the progress of the research line b in Figure 1, aimed at the modelling of the micro-scale processes in clays for the prediction of their macro-response. In particular, since the findings summarized in §7 refer to reconstituted clays, they represent fabric settings which should be achieved through the micro-scale modelling for medium to high plasticity clays of stable bonding, as is the case for the clays sedimenting either in the laboratory consolidometer, or *in situ* in a high energy deposition environment. Furthermore, the clays should achieve initial consistency under initial 1D compression. Figure 13 reports the framework of the micro-scale findings of reference for such micro-scale modelling.

The micro-scale findings in §7 represent also a database useful for the research line a in Figure 1, aiming at relating the clay macro-behaviour predicted through constitutive laws formalized in the framework of continuum mechanics, to the micro-scale processes. In the following, some of the indications provided by the summarized findings are briefly discussed.

First, the fabric changes taking place in the clay when undergoing constant η compression (illustrated in Figure 13) provide reasons for the occurrence of the normal consolidation line of reconstituted clays under isotropic compression ($\eta = 0$), INCL^* , to the right of that followed in 1D compression, K_0NCL^* . In other words, the micro-scale observations justify why the intercept of the INCL^* , N^* , is higher than that of the K_0NCL^* , $N_{K_0}^*$. It can be recognized that the less oriented fabric developing through isotropic gross yielding, within which

particles and domains are largely in edge to face contact, is consistent with a less deformable material under increasing p' than that embodying a more oriented fabric, such as that of the clay gross yielding in 1D compression. As a matter of fact, the edge to face inter-particle forces of the less oriented fabric developing with isotropic yielding, resists more the external mechanical actions (more limited straining) than the face to face forces within the stacks present in the 1D compressed clay. It can be argued that the smaller the difference in fabric between the clay subjected to isotropic gross yielding and that subjected to 1D gross yielding, the smaller is the difference $[N^* - N_{K0}^*]$.

Mitaritonna et al. (2014) provide evidence that the high degree of orientation that the reconstituted clay achieves through 1D compression gives rise to directional properties at the macro-scale. They report measurements of the elastic stiffness anisotropy ratio, G_{hh}/G_{hv} , of reconstituted Lucera clay achieved by means of T-shape horizontal bender element tests (Dyvik & Madshus, 1985; Pennington et al., 1997; Mitaritonna et al., 2010), performed all the way through the tests shown in Figure 6. The values of G_{hh}/G_{hv} measured during the radial stress paths in Figure 6 are plotted versus p' in Figure 16. It can be seen that in 1D compression, at $\eta = 0.6$, a constant stiffness anisotropy ratio $G_{hh}/G_{hv} = 1.12-1.13$ (Figure 16) is found to correspond to the constant degree of fabric anisotropy assessed through the micro-scale investigations (Figures 14a, b). Hence, the steady orientation degree, L , measured during the $\eta = 0.6$ compression is consistent with the constant value of the anisotropy degree $G_{hh}/G_{hv} = 1.11$ recorded at the macro-scale. Such finding confirms that the micro-REV fabric features, rather than the large-scale fabric features, control the clay macro-behaviour and that the elastic anisotropy of the clay is generated by the high degree of orientation of the micro-REV fabric, irrespective of the local poorly oriented fabric aggregates present in the highly oriented medium scale fabric. Furthermore, Mitaritonna et al. (2014) find that the elastic stiffness ratios either decrease, or increase, depending on the imposed stress ratio η (Figure 16), and reach different constant G_{hh}/G_{hv} values at p' about four times the initial one ($p' = 70$ kPa).

These observations indicate that the clay has to be compressed well beyond gross-yield in order to acquire constant directional properties which, evidently, depend on the achieved constant degree of micro-REV fabric orientation, L , which in turn depends on η . In particular, compression paths at η lower than the initial $\eta = 0.6$ ($\eta = 0$, or 0.3), induce a permanent reduction in the degree of elastic anisotropy, leading to $G_{hh}/G_{hv} \approx 1.05$ for $\eta = 0.3$ and about 1.01 for $\eta = 0$. The isotropic loading to high pressures appears almost to erase the initial directional properties of the clay. In contrast, the elastic anisotropy ratio increases up to 1.22 in the clay compressed at $\eta = 0.8$. Also, the data show that a change in degree of shear stiffness anisotropy requires significant plastic straining, as a small amount of

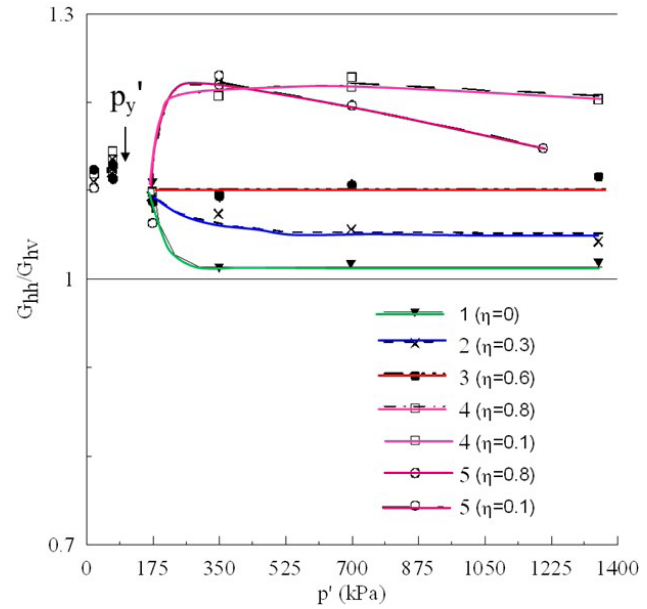


Figure 16. Anisotropy ratio G_{hh}/G_{hv} plotted against mean effective stress for different η compression tests (modified after Mitaritonna et al., 2014).

irreversible deformation under the new η is not sufficient to modify the previously acquired directional property to a significant extent.

Mitaritonna et al. (2014) report, for reconstituted Lucera clay, the plot of $G_{hh}/G_{hv} - \eta$,⁶ along with the corresponding L values. The corresponding plot relating the degree of micro-REV fabric orientation, L , achieved through the constant η compressions is shown in Figure 17a. Accordingly, Figure 17b formalizes a monotonic increase in G_{hh}/G_{hv} with L , evidence of the dependency of the elastic cross-anisotropy of the clay on the micro-REV degree of orientation. Therefore, the research results show that the directional elasto-plastic coupling made evident by the plot in Figure 16 is due to the micro-REV fabric changes taking place in the clay with gross yielding, as shown by the relation between G_{hh}/G_{hv} and the microfabric constitutive parameter $L(\epsilon_{ij}^p)$ in Figure 17b.

Since the data validate the dependency of the variation in elastic anisotropy ratio on the variation in fabric orientation at the micro-REV scale (Figure 17b), the observation that the elastic anisotropy ratio varies slowly after the change in compression stress ratio η , shown in Figure 16, in turn suggests that the variation in the degree of fabric orientation, L , takes place slowly after the change in η . It may be envisaged a chaotic fabric change taking place soon after η variation, followed by a progressive fabric evolution heading towards a new constant fabric orientation, L (Figure 17b). At the same time, the MIP data discussed in §6 suggest that the change in η impacts sooner the PSD (faster for isotropic

⁶ See Figure 14 in Mitaritonna et al. (2014).

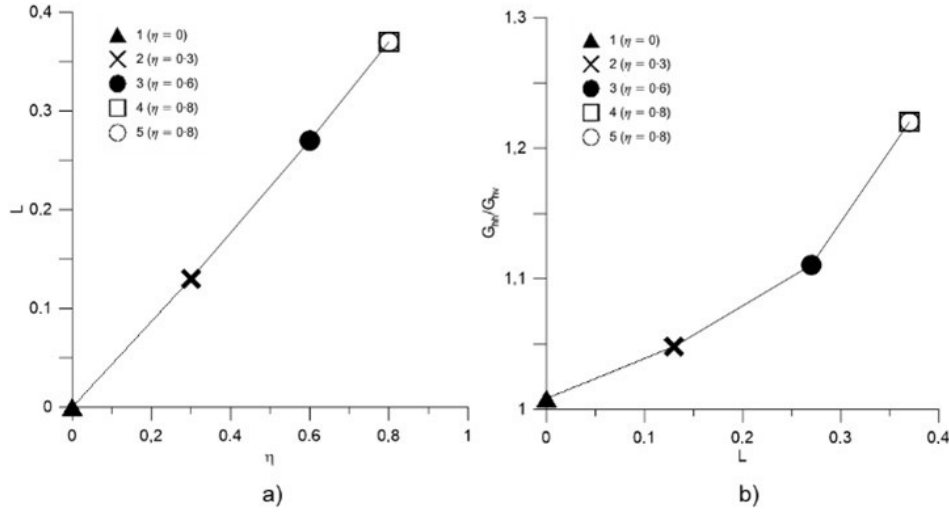


Figure 17. Anisotropy ratio G_{hh}/G_{hv} against: stress ratio η (a) and micro-REV orientation index L (b).

compression versus 1D compression) than the micro-REV fabric orientation, L . The findings concerning the micro-REV fabric orientation degree and the PSD features for each constant η compression of the clay at $I_v < 0$, i.e.: $L(\eta)$ reached in constant η compression after a significant amount of plastic straining at constant η , and the monomodal PSD with reducing DPS, are schematized in Figure 13.

The findings sketched in Figure 13 can be of reference in the selection of new microfabric constitutive variables for clays of stable bonding, like the reconstituted clays, to be implemented in advanced constitutive laws accounting for the micro-scale processes which determine the clay macro-behaviour (research line a-ii in Figure 1; e.g., Rollo & Amorosi, 2022). In particular, useful microfabric constitutive parameters appear to be: the micro-REV fabric orientation index, $L(\eta)$, or $L(\varepsilon_{ij}^p)$ if accounting for the transient stages of evolution of L ; the corresponding angle of orientation $\theta(\varepsilon_{ij}^p)$; the PSD function and the DPS (ε_{ij}^p) function.

Meanwhile, still concerning the research line a-i in Figure 1, the steady degree of fabric orientation, L , and the monomodal PSD characterizing the micro-REV fabric of reconstituted clays during plastic straining under constant η compression (Figure 13), justify the success of the normalization for volume (through $p_e^*(e)$) of the shear response of reconstituted clay specimens all compressed along the same radial stress paths (constant η ; e.g., Figure 6). Such a response is exemplified in Figure 18. The micro-scale data show that, when the clay undergoes constant η compression from $I_v \approx 0$ to $I_v \ll 0$, its micro-REV fabric reaches soon a stage in which the only fabric feature undergoing major changes with constant η compression is the DPS of a monomodal PSD, since L and θ remain constant. This occurs for any η (Figure 13). Therefore, the reduction in clay void ratio, e , with no significant change in L is the possible reason why e is the main internal variable of the plastic hardening of

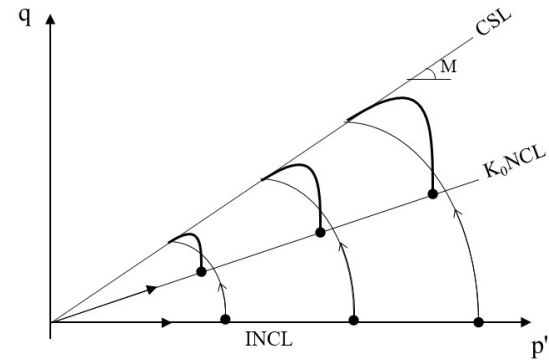


Figure 18. Schematic representation of undrained shear paths after different η compressions.

reconstituted clays (Rendulic, 1936; Roscoe et al., 1958; Henkel, 1956) and isotropic volumetric hardening applies when yielding takes place under constant η (e.g., the Cam Clay models; Schofield & Wroth (1968); Roscoe & Burland (1968); Figure 18).

However, since the steady degree of micro-REV fabric orientation reached at constant η varies with η , the differences in clay stiffness and stiffness anisotropy among the specimens compressed at different constant η make their stress-strain response to shearing vary with η . It may be envisaged that the higher is the variation of L with changing η , the less the shear response of the clay complies with the Rendulic principle⁷. Accordingly, it is likely that the Rendulic principle, i.e., the successful normalization by $p_e^*(=e^{[(N^*-I)/\lambda^*]})$ of the stress paths of specimens sheared after different η compressions, suits the shearing behaviour only of a family of specimens, including

⁷ For instance, the reader is referred to the normalization by p_e^* of shear paths of either isotropically or 1D consolidated specimens of reconstituted Pappadai clay, as shown by Cotecchia et al. (2011) in their Figure 20.

1D compressed specimens and specimens isotropically compressed, for which compression has not been such as to determine significant variation in fabric orientation, L .

However, reaching a common critical state at the end of shearing of specimens compressed under different η is justified by the major fabric changes caused by shearing to large strains, which causes very high degrees of fabric orientation (function $L(\eta)$; see L values for high η), irrespective of the original consolidation fabric (Guglielmi et al., in press).

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

Federica Cotecchia: conceptualization, methodology, validation, writing - reviewing and editing. Simona Guglielmi: conceptualization, investigation, data curation, validation, writing - reviewing and editing. Francesco Cafaro: investigation, validation, reviewing. Antonio Gens: supervision, validation, reviewing & 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

DPS	dominant pore size
G_{hh}/G_{hv}	elastic stiffness anisotropy ratio
I_v	void index
e	void ratio
e_{wL}	void ratio at liquid limit
w	water content
A	clay activity
C_s	swelling index
C_s^*/C_s	swell sensitivity
CF	clay fraction
ICL	intrinsic compression line
L	index of fabric orientation
w_L	liquid limit
MIP	mercury intrusion porosimetry
OCR	overconsolidation ratio σ'_p / σ'_v
w_p	plastic limit
PSD	pore size density function
S_σ	stress sensitivity
S_r	degree of saturation
SCL	sedimentation compression line
SEM	scanning electron microscopy

YSR	yield stress ratio σ'_y / σ'_v
η	stress ratio
σ_e^*	equivalent vertical effective stress on the ICL
σ_p'	vertical (geological) preconsolidation pressure
σ_v'	vertical effective stress
σ_y'	vertical effective stress at yield

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Micro to macro investigation of clays advising their constitutive modelling – part II

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Article

Keywords

Microstructure
Constitutive modelling
Soil microstructure analysis

Abstract

This paper reports the second part of the keynote lecture, whose part I has been already published in this journal, presenting extensive experimental research on the investigation of clay microstructure and its evolution upon loading. Whether the first part focused on the micro to macro behaviour of different reconstituted clays, this part instead concerns the microscale features of the corresponding natural clays, their changes under different loading paths and the ensuing constitutive modelling implications. The experimental investigation is carried out according to the methodology outlined in the part I-paper, hence micro-scale analyses are presented on natural clays subjected to macro-scale mechanical testing, with the purpose to provide experimental evidence of the processes at the micro-scale which underlie the clay response at the macro-scale. As for the reconstituted clays in the part I-paper, original results on stiff Pappadai and Lucera clay, this time in their natural state, are compared to literature results on clays of different classes, either soft or stiff. The results presented in this paper, together with those discussed in the part I, allow for a conceptual modelling of the microstructure evolution under compression of natural versus reconstituted multi-mineral clays, providing microstructural insights into the macro-behaviour described by constitutive laws and advising their mathematical formalization in the framework of either continuum mechanics or micro-mechanics.

1. Introduction

This paper reports the second part, II, of the keynote lecture whose part I has been published in this journal by Cotecchia et al. (2024) focused on the micro to macro behaviour of reconstituted clays. This paper, instead, is concerned with the microscale features of different natural clays, their changes under compression and the corresponding constitutive modelling implications, all investigated through the same methodology outlined in the part I-paper. Hence, the reader is referred to the part I-paper for the research objectives and methods of the whole research work, presented in both the part I-paper and this part II-paper.

Among natural clays, the microstructural features vary not only due to differences in composition and stress history, as outlined for reconstituted clays in the part I-paper. Natural clays' microstructural features depend also on their original natural deposition conditions and on many possible chemo-hydro-mechanical processes taking place over the geological history (e.g. creep, thixotropy, calcite deposition from water supersaturated with calcium carbonate, diagenesis,

weathering, drying-wetting cycles, etc.; e.g. O'Brien & Slatt (1990), Mitchell & Soga (2005), Cotecchia & Chandler (1995, 2000), Cafaro & Cotecchia (2001)).

According to the methodological approach described in the part I-paper (see Figure 1 in the part I-paper), the microstructures of different natural clays and their variations under compression are discussed in the following, in order to shed light on the micro-scale processes that are the source of variable macro-scale responses among natural clays. The natural clays of reference are those whose reconstituted version has been the subject of the part I-paper.

2. Clays under study

2.1 Composition, deposition conditions and processes impacting the microstructure

Table 1 reports the index properties (discussed in the companion paper, see Figure 4 of the part I-paper) and in situ states of the natural clays under study. For some of these, the

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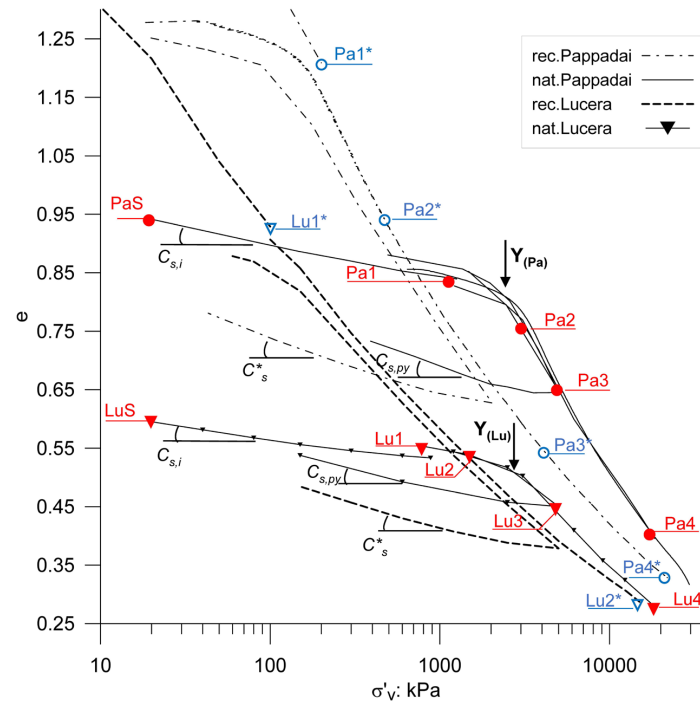


Figure 1. 1D behaviour of Lucera and Pappadai clay in the $[e; \sigma'_v]$ plane. Y = gross yield. States of microstructural analysis discussed in the paper are highlighted for both the natural and reconstituted clay [modified after Guglielmi et al. (2022)].

yield stress ratio, YSR (Burland, 1990), values exceed the geological overconsolidation ratio, OCR , due to geological processes which occurred under burial and affected the clay microstructure (change from sedimentation to post-sedimentation structure according to Cotecchia & Chandler (2000)). The table reports also the clay stress sensitivity¹, S_σ (Cotecchia & Chandler, 2000), and the ‘strength sensitivity’ S_t (Terzaghi, 1944; Skempton & Northey, 1952; Cotecchia & Chandler, 2000; here computed as ratio of the clay undrained strength for $YSR = 1$, c_{uNC} , to that of the reconstituted clay² at the same void ratio, c_{uNC}^* , $S_t = c_{uNC} / c_{uNC}^*$).

The deposition environment of the clays and the processes experienced after deposition are synthesized in the following.

2.1.1 Soft clays

St Marcel, Ballina, Bothkennar and Gulf of Guinea clays are soft clays; in particular, both St Marcel and Ballina clay are slightly to medium quick clays [S_t in the range 10-20; Mitchell & Soga (2005)].

St Marcel clay is a Champlain Sea clay from Quebec, whose sensitivity was induced by the leaching action generated by brackish to fresh water, which replaced the original saline

pore water (Quigley, 1980; Delage & Lefebvre, 1984). As typical for Champlain clays, a non-negligible proportion of its clay fraction is not made of phyllosilicates, being composed of primary minerals, such as quartz, feldspar, amphibole, mica and chlorite (Quigley, 1980). These minerals are ground down to very small sizes by glaciers to form rock flour (Quigley, 1980; Locat et al., 1984; Delage, 2010). The clay mineralogy (Table 1) confers a low activity index (part I-paper). Scanning Electron Microscopy, SEM, and Mercury Intrusion Porosimetry, MIP, investigations were carried out by Delage & Lefebvre (1984) on St Marcel clay block samples from 5.2 m depth, soon after sampling and 1D compression.

Ballina clay originated, in the Holocene, in the Pimplico clay deposit (east Australian coastline), in a low-energy estuarine environment (Bishop & Fityus, 2006; Pineda et al., 2016b). Compared to St Marcel clay, Ballina clay is of far higher plasticity and activity index (see Table 1) due to a higher content of illite and mixed illite/smectite layers. Pineda et al. (2016b) carried out microstructural analyses on a natural undisturbed sample (Sherbrooke sampler) from 7m depth (Table 1).

Bothkennar clay (Scotland) is a mainly illitic Holocene medium sensitive clay (Table 1), of high plasticity and activity (Table 1; Cotecchia et al. (2024) – part I). It deposited in a sheltered environment of brackish water, allowing the setting of a flocculated fabric (Hight et al., 1992). In addition, post-

¹i.e. the ratio of the natural clay gross yield pressure in compression to the equivalent pressure on the NCL of the reconstituted clay.

²Reconstituted clay states will be denoted with * in the following.

Table 1. Properties of the clays discussed in the paper [modified after Guglielmi et al. (2022)].

Clay	z (m)	w_0 (%)	e_0 (-)	σ'_{v0} (kPa)	CF (%)	LL (%)	PI (%)	A (-)	mineralogy*	YSR	S_p, S_e
<i>soft clays</i>											
BOTHKENNAR (Bot) Hight et al. (1992), Smith et al. (1992), Tanaka et al. (2003)	17	70	1.61	≈ 100	28-40	55-86	40-50	1.3	I; K; Qz; Fd †	1.4-1.6	4-7
GULF OF GUINEA (GoG) Hattab et al. (2013)	10	130-170	3.28	36.5	40-60	110-160	70-80	1.5	K: 50; S: 15-25 †	1.3-1.4	6-8
ST MARCEL (StM) Delage & Lefebvre (1984)	5.2	80	2.3	<10	80	60	35	0.4	Ch: 4-16; I: 2-10; V: 2-9 ‡	-	19
BALLINA (Ba) Pineda et al. (2016a, b)	7.4-7.9	110-120	2.95-2.97	<50	78-82	115-130	70-80	1	K: 2,3-24,5; I: 7,8-23,3; Qz: 8,8-55,9; I/S: 9,5-20,1; amorphous minerals: 1,7-22,4; Pl: 2,9-15,2; Py: 0-6,4; K-Fd: 1,2-5,5; Mca: 0,9-2,1; Ca: 0-1 ‡	<2	10-20
<i>stiff clays</i>											
PAPPADAI (Pa) Cotecchia & Chandler (1997, 2000), Cotecchia et al. (2019), Guglielmi (2018)	25	30	0.88	414	58	65	35	0.6	I: 20; Ch: 14; S: 12; K: 12; I/S: 10 ‡	6	3.5
LUCERA (Lu) Mitaritonna (2006), Guglielmi (2018)	40	18	0.55	≈ 800	48	48	24	0.5	I/S: 77 (40-45); K: 9; I: 8; Ch: 6 †	3.7	2
BOOM (Bo) Belanteur et al. (1997), Delage et al. (2007), Lima et al. (2008), Della Vecchia et al. (2011)	223	22.6-24	0.58-0.61	2250	50-60	67	38	0.7	I: 50; S: 30; I/S+K: 10 †	2.3-2.4	2(?)
LONDON Gasparre et al. (2007)	1-45	21-29	0.6-0.8		41-63	59-78	33-62	0.7-0.8	I: 21-38; K: 11-15; Ch; S; M ‡	3-20	1.3-2.9
BROWN WEATHERED LONDON (BWL) Skempton & Jones (1944), Monroy et al. (2010)	4-6	-	1.175		58	83	54	0.93	-	-	-

Keys: z , sample depth; w_0 , initial water content; e_0 , initial void ratio; σ'_{v0} , in situ vertical effective stress; CF , clay fraction; LL , liquid limit; PI , plasticity index; A , activity index; YSR, Yield Stress Ratio; S_p , strength sensitivity, or S_e , stress sensitivity (in italics). *Percentages are shown, when available, otherwise the minerals are listed in descending order of dominance: I, illite; K, kaolinite; Ch, chlorite; S, smectite; I/S, mixed-layers illite/smectite (illite % within parentheses); V, vermiculite; M, montmorillonite; Qz, quartz; Fd, feldspar; Ca, carbonate; Pl, plagioclase; Py, pyrite, Mca, mica. † % of the fraction $\leq 2 \mu\text{m}$. ‡ % of the bulk sample.

depositional bonding, ageing and leaching have impacted its microstructure (Paul et al., 1992; Hight et al., 1992; Tanaka, 2000; Tanaka et al., 2003). Tanaka et al. (2003) carried out MIP tests on the undisturbed clay sampled at 17 m depth (Japanese fixed piston sampler).

Gulf of Guinea clay (GoG) is a deep-water marine clay, sampled by means of a 10 m long sampler below the sea floor of the Gulf of Guinea, under 700 m sea water column, in an oil-producing area (Hattab et al., 2013). The clay is of very

high plasticity and has the highest activity index among the soft clays of reference (part I-paper), due to its considerable smectite content. At the microscale, it embodies a physico-chemical bonding generated during the sedimentation and consolidation process in the seabed (De Gennaro et al., 2005; Hattab & Favre, 2010; Hattab et al., 2013). Hattab et al. (2013) analysed the microstructure of the undisturbed clay and after both 1D and isotropic compression to $\sigma'_v = 1000 \text{ kPa}$ and $p' = 900 \text{ kPa}$, respectively.

2.1.2 Stiff clays

Both Pappadai and Lucera clay are stiff clays part of the Pleistocene Subapennine Blue clays (Del Prete & Valentini, 1971; Cotecchia & Chandler, 1995), illitic and of medium-high plasticity and low activity (see Table 1). The deposition environment of Pappadai clay was a protected still water sea, where the clay overlaid a calcarenite bedrock (Ciaranfi et al., 1971) in a chemically reducing environment, allowing for flocculation and lamination (O'Brien & Slatt, 1990; Cotecchia & Chandler, 1995). The deposition of Lucera clay, instead, occurred in a marine environment of higher energy than for Pappadai clay, in a peripheral margin of the Apennine foredeep (Gallicchio et al., 2003; Santaloia et al., 2004).

Both clays were buried under more than one hundred metres of sediments, overconsolidated due to erosion and subjected to diagenesis, which strengthened the clay bonding, as confirmed by mineralogical evidence (Cotecchia & Chandler, 1995) and chemical micro-probing in the SEM (Cotecchia & Chandler, 1997; Guglielmi et al., 2018). Microstructural analyses have been performed on both clays, at different levels of compression (Table 1).

For comparison, the microstructural features of stiff Boom clay (Lima et al., 2008), a mainly illitic marine Tertiary clay, of high plasticity and medium-low activity (Table 1), block sampled at more than 200 m depth (HADES underground laboratory, Mol - Belgium), will be also discussed. For all these stiff clays, S_e is 2-3 (Table 1).

2.2 Clay states and compression macro-behaviour

Figure 1 shows the results of 1D compression and swelling tests on natural and reconstituted Pappadai and Lucera clay specimens.

Both stiff clays exhibit a stiff response to swelling, with high swell sensitivity C_s^*/C_{st} (2.5 for Pappadai clay and 2 for Lucera clay), as already observed by Cotecchia & Chandler (1997) and Cafaro & Cotecchia (2001), which is indicative of a significant strength of the natural bonding developed through diagenesis during ageing under burial (Cotecchia & Chandler, 1997). Such bonding provides both clays with $YSR > OCR$ (e.g. for Pappadai clay, $YSR = 6$ twice the OCR) and $S_e > 1$, although much lower than for soft clays. The compression post-gross yield (Figure 1) is seen to cause a rather immediate C_s^*/C_s decrease, indicative of a rather fast weakening of bonding upon gross yielding, and a progressive reduction in S_e . The micro-analyses for states Pa1*, Pa4*, Pa1 and Pa4 in Figure 1 have been already referred to in the companion paper (see §3 in the part I-paper) and in Cotecchia et al. (2019) and will be recalled later.

Figure 2 illustrates compression data already plotted in Figure 1 along with data resulting from laboratory 1D compression tests on either natural, or reconstituted samples of several of the other clays in Table 1. The compression data are normalized for composition using the void index $I_v = (e -$

$e_{100}^*)/(e_{100}^* - e_{1000}^*)$ (Burland, 1990), where e_{100}^* and e_{1000}^* are the void ratios on the intrinsic compression line, ICL, for σ'_v equal to 100 kPa and 1000 kPa, respectively. The states corresponding to the microstructural analyses (SEM and/or MIP) are indicated with large symbols (full symbols correspond to natural samples and empty symbols to the reconstituted ones).

The natural samples of soft St Marcel, Ballina, Bothkennar and GoG clay, normally consolidated to lightly overconsolidated in situ, exhibit $YSR < 2$ (Table 1; Delage & Lefebvre, 1984; Pineda et al., 2016a; Hight et al., 1992; Smith et al., 1992; Hattab et al., 2013) and S_e higher than for stiff overconsolidated Pappadai, Lucera and Boom clay. Evidently, the bonding they achieved in the geological history, although much weaker than for the stiff diagenized clays, has allowed them to keep far more open fabrics than the reconstituted clay since early stages of compression. However, the large compressibility of these clays post-gross yield [e.g. Locat & Lefebvre (1985), Leroueil & Vaughan (1990), Delage (2010)] indicates a fragile weakening of such bonding upon gross yielding. It is worth noting that for Bothkennar clay, the 1D compression curve (Bot b) in Figure 2 refers to a sample (Smith et al., 1992) different from that subjected to microstructural analysis [Bot a, Tanaka et al. (2003)].

While the post-mortem micro-analyses on the specimens indicated in Figures 1 and 2 allow for insight into the evolution of microstructure solely due to 1D compression, the microstructural changes activated under different constant $q/p' = \eta$ compressions have been investigated for both the natural Lucera and Pappadai clays. Figure 3 reports the comparison, in the $[v; \log p']$ plane, between the 1D and the isotropic compression curve of the two clays.

Pre-gross yield, the straight-line gradient κ ($\kappa = 0.015$ for Lucera clay and $\kappa = 0.02$ for Pappadai clay) is lower than the corresponding κ^* . For both stiff clays, κ^*/κ , as for C_s^*/C_s , reduces to unity post-gross-yield, confirming that bonding weakens also with gross yield in isotropic compression (Cotecchia et al., 2016, 2019, 2020). In isotropic compression, Lucera clay (Figure 3a) and Pappadai clay (Figure 3b) exhibit λ values of 0.141 (Guglielmi, 2018) and 0.254 (Cotecchia & Chandler, 1997) respectively, both close to the λ value in 1D compression (as expected according to critical state soil mechanics) and exceeding λ^* (0.139 for reconstituted Lucera clay and 0.204 for reconstituted Pappadai clay). Figure 3 indicates the states for which post-mortem micro-analyses have been conducted.

3. Natural clay microstructure under compression

3.1 Soft clays under 1D compression

Figure 4 shows the SEM micrographs for states StM1 and Ba (Figure 2) of the highly sensitive St Marcel and Ballina clay (Table 1), which were reported by Delage

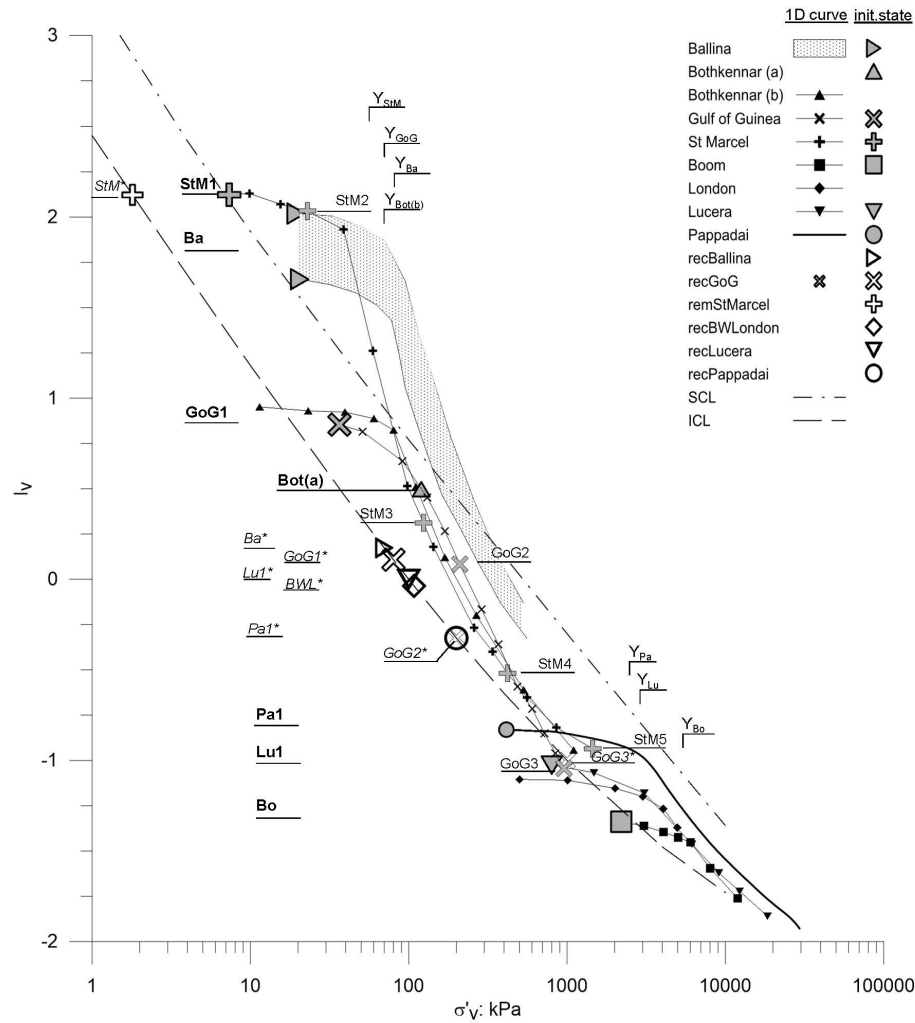


Figure 2. One-dimensional compression of natural and reconstituted clays normalized for composition using the void index, I_v [data after Bishop et al. (1965), Burland (1990), Cotecchia & Chandler (2000), Delage & Lefebvre (1984), Hattab et al. (2013), Smith (1992), Pineda et al. (2016a, b)]. Y = gross-yield; ICL = Intrinsic Compression Line; SCL = Sedimentation Compression Line. Larger symbols (empty for reconstituted, full grey for natural) represent states of samples on which microstructural analyses were carried out; the largest symbols correspond to the sample initial states [modified after Guglielmi et al. (2022)].

& Lefebvre (1984) and Pineda et al. (2016a), respectively. The micrographs in Figure 4a and 4c correspond to the micro-REV scale (Cotecchia et al., 2019), and those in Figure 4b and 4d to larger scales. At the micro-REV scale, aggregates of random orientation particles appear to be connected by bridges formed of domains and to confine inter-aggregate pores for both the less plastic St Marcel clay and the more plastic Ballina clay. A similar fabric was observed for the reconstituted more plastic clays, Ballina and GoG (ss sketched in Figure 7f in the part I-paper).

Despite the higher void ratio of Ballina clay with respect to St Marcel clay, both clays appear to contain a significant inter-aggregate porosity (Figure 4a, 4c), but Ballina clay includes also an important intra-aggregate porosity (Figure 4d), formed of pores smaller than for the intra-aggregate porosity of St Marcel clay (Figure 4b). Such evidence is consistent

with the more elastic nature and lower activity of the clay fraction of St Marcel clay. The more active CF of Ballina clay is largely made up of small active clay particles which can form aggregates that include small pores. Similar fabric features are recognizable in the micrographs of the slightly less sensitive natural GoG clay samples (state GoG1 in Figure 2) described by Hattab et al. (2013). Overall, the fabric features observed through SEM in these natural soft clays are qualitatively similar to those observed for the lightly consolidated reconstituted clays, discussed in part I-paper (see Figure 7 in the part I-paper).

Figure 5 compares the MIP curves of the clay samples StM1, Ba, GoG1 and Bot(a) (Figure 2), which are all either normally consolidated clays, or slightly overconsolidated. A bimodal porosity is observed for Ba, GoG and Bot(a), with inter-aggregate dominant pore size, DPS, about 1000

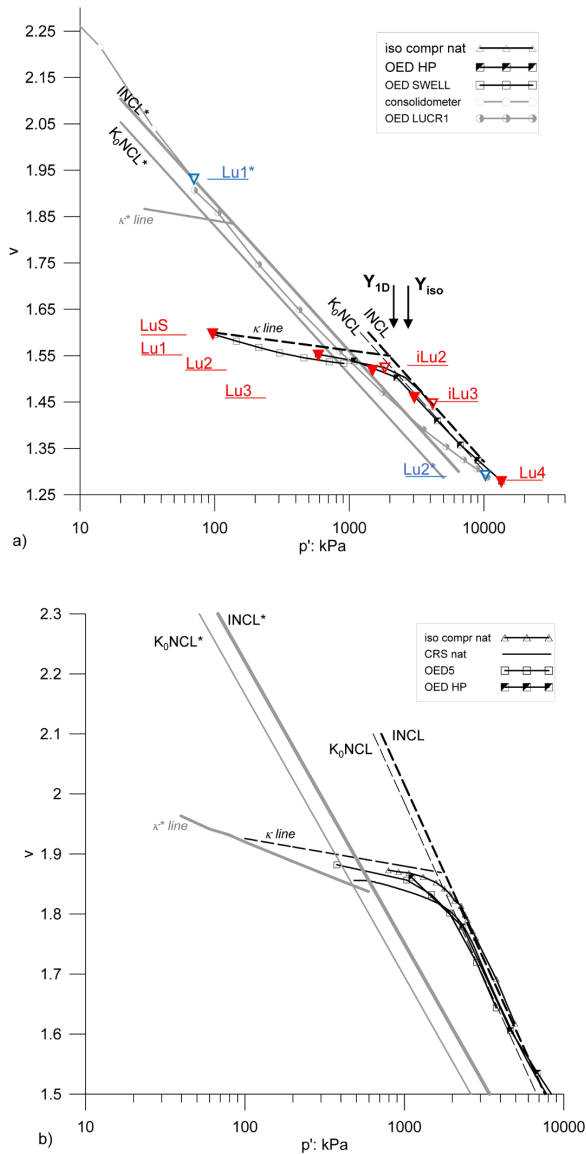


Figure 3. Isotropic and 1D behaviour of Lucera (a) and Pappadai (b) clay. Y = gross yield; INCL = Isotropic Normal Compression Line; $K_0NCL = K_0$ Normal Compression Line. States of microstructural analysis discussed in the paper are highlighted for both the natural (in red: Lu1 for the initial state; Lu2, Lu3 and Lu4 for the clay compressed 1D; LuS for the clay swelled 1D; iLu2, iLu3 and iLu4 for the clay compressed isotropically) and the reconstituted (in blue: Lu1* after consolidometer and Lu2* compressed 1D) clay.

nm for both Ba and GoG1, and of 1700 nm for Bot(a). Also, a smaller frequency DPS is found to correspond to intra-aggregate pore volume, about 63 nm for GoG1, 100 nm for Bot(a) and 50-70 nm for Ba.

Hence, the MIP data are consistent with the SEM pore observations [sizes of inter-aggregate pores indicated in Figure 4d for Ba; Pineda et al. (2016a)] and confirm that, in addition to the large inter-aggregate pores, significant porosity is made of small intra-aggregate pores within the

higher plasticity clays. This intra-aggregate porosity is much less for lower plasticity clays (e.g. St Marcel).

It follows that both SEM and MIP investigations show that the fabric features of soft natural clays, characterized by an activity from medium-low to high (Table 1) and originally deposited in a marine environment, are similar to those of the corresponding reconstituted clays (part I-paper), consolidated in the laboratory from slurries³. Such fabric features comply with the fabric sketch by Griffiths & Joshi (1990) (as sketched in Figure 7f in the part I-paper), which appears therefore suited to represent the fabric of such clays when at $I_v > 0$, whether natural or reconstituted (Figure 2). The fabric is flocculated and inter-aggregate macro-porosity is dominant, but also micro-porosity can occur within the aggregates. The latter corresponds to a secondary DPS whose size decreases with increasing activity index.

These findings then suggest that the difference in response to compression of natural and reconstituted clays of $I_v > 0$ (Figure 2; Burland, 1990; Cotecchia & Chandler, 2000) is more the result of a difference in their bonding features, than in their fabric features. Such difference in bonding allows the difference between the normal consolidation line of the reconstituted clay (ICL, Figure 2) and that of the natural clay, which lies to the right of the ICL, consistent with $S_\sigma > 1$.

Natural St Marcel clay was subjected to several 1D compression tests, stopping at σ'_v of 23, 124, 421 and 1452 kPa, i.e. states StM2, StM3, StM4, StM5 in Figure 2, respectively. Both SEM and MIP tests were carried out on each specimen after unloading to 4 kPa (Delage & Lefebvre, 1984). As the St Marcel clay gross-yield pressure is equal to 54 kPa (Y_{StM} , Figure 2), the MIP data for the first specimen, StM2 ($\sigma'_v = 23$ kPa), allow to check whether microstructural changes take place during compression pre-gross yield, when compared with the MIP data of the undisturbed specimen StM1 (Figure 6). As commented by Delage & Lefebvre (1984), neither the micrographs, nor the PSD curves in Figure 6 give evidence of any significant change in clay microstructure with respect to the initial one, suggesting that pre-gross yield compression does not cause significant modifications in microstructure, even for highly sensitive clays.

When compressed beyond gross-yield to state StM3 ($\sigma'_v = 124$ kPa; Figure 2), St Marcel clay experiences a reduction of the inter-aggregate porosity and an onset of fabric anisotropy [large pores and aggregates are stretched along the horizontal plane; Figure 7a, 7b; Delage & Lefebvre (1984)]. The initial undisturbed fabric arrangement (Figure 4a, b) is no longer recognizable at states StM4 ($\sigma'_v = 421$ kPa; Figure 7c, 7d) and StM5 ($\sigma'_v = 1452$ kPa; Figure 2), since the particles are densely packed and horizontally oriented (Figure 7e, 7f). The corresponding PSD curves are plotted in Figure 8a and show how post-gross yield compression determines a mono-modal PSD, with a single DPS, whose value and frequency

³Made using either tap water, or distilled water.

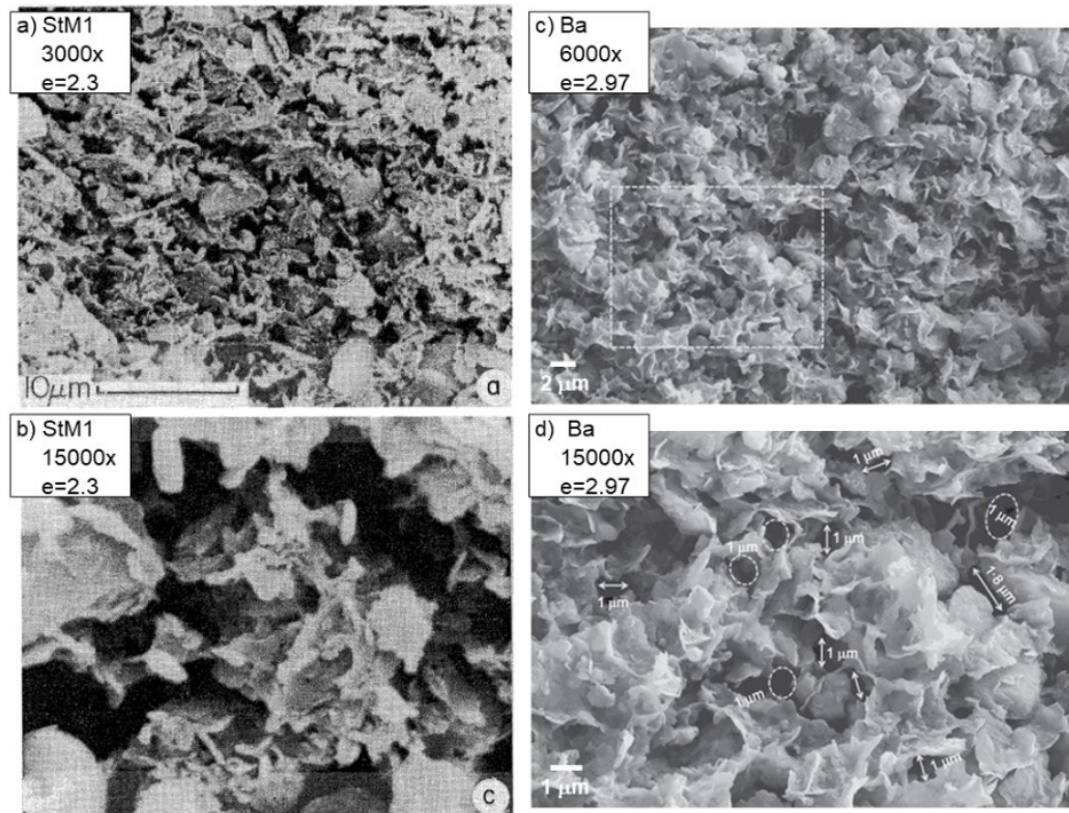


Figure 4. SEM micrographs of natural St Marcel clay (a, b; state StM1 in Figure 2) and natural Ballina clay (c, d; state Ba in Figure 2). **Sources:** Delage & Lefebvre (1984), Pineda et al. (2016b); modified after Guglielmi et al. (2022).

reduces due to the collapse of the inter-aggregate porosity. However, the intra-aggregate pore sizes are not significantly modified by compression up to $\sigma'_v = 1452$ kPa for this clay (Delage & Lefebvre, 1984).

Similar microstructural changes are observed for natural GoG clay, compressed from GoG1 state to GoG2, $\sigma'_v = 200$ kPa, and GoG3, $\sigma'_v = 1000$ kPa (Figure 2). Figure 8b shows a transition from a prevailing inter-aggregate porosity to a prevailing intra-aggregate porosity, with a complete loss of bimodality post-gross yield [at GoG3; Hattab et al. (2013)], even by a σ'_v increase smaller than that required for a similar process with reconstituted clay GoG* (see Figure 11a in the part I-paper).

Overall, the data suggest that 1D compression of clays, starting at $I_v > 0$, causes a transition from a large inter-aggregate porosity to a prevailing small intra-aggregate porosity, accompanied by an increase in fabric orientation. Both processes start only beyond gross-yield for natural soft clays, i.e. when bonding weakens. For the reconstituted clays, the fabric degree of orientation has been shown to reach a maximum constant value by I_v values little below 0 (part I-paper). Data are not available to check the reach, through 1D compression, of a constant fabric orientation degree, for the soft natural clays here of reference.

3.2 Stiff clays under 1D compression

According to the mineralogy (Table 1) and deposition conditions of natural Lucera clay (Lu1 in Figures 1 and 2) and Pappadai clay (Pa1), the original fabric of both these clays was most probably either honeycomb, or bookhouse (part I-paper), the same as that recognized for the soft clays discussed above. In their geological history, such fabric has been 1D compressed to significant pressures and unloaded (due to erosion – current $e_0 - \sigma'_{v0}$ states reported in Table 1); furthermore, diagenesis has impacted the clay microstructure under burial. For Pappadai clay, Cotecchia & Chandler (1995, 1997) report that diagenesis determined the growth of an amorphous calcite film binding the clay particles. Medium magnification micrographs of the current micro-REV fabric of both Lu1 and Pa1 (Figures 1 and 2) are shown in Figure 9.

Both Pa1 and Lu1 are found to embody a fabric (Figure 9a and 9c) formed of thick stacks of particles/domains, which are mostly sub-horizontal and densely packed. These bury local aggregates of randomly oriented domains. Further evidence of the fabric features of natural Pappadai and Lucera clay, detected through micrographs of magnification varying from medium to high, are provided by Cotecchia et al. (2019) [see Figure 5 in Cotecchia et al. (2019)] and Guglielmi (2018), respectively. According to such studies, the local

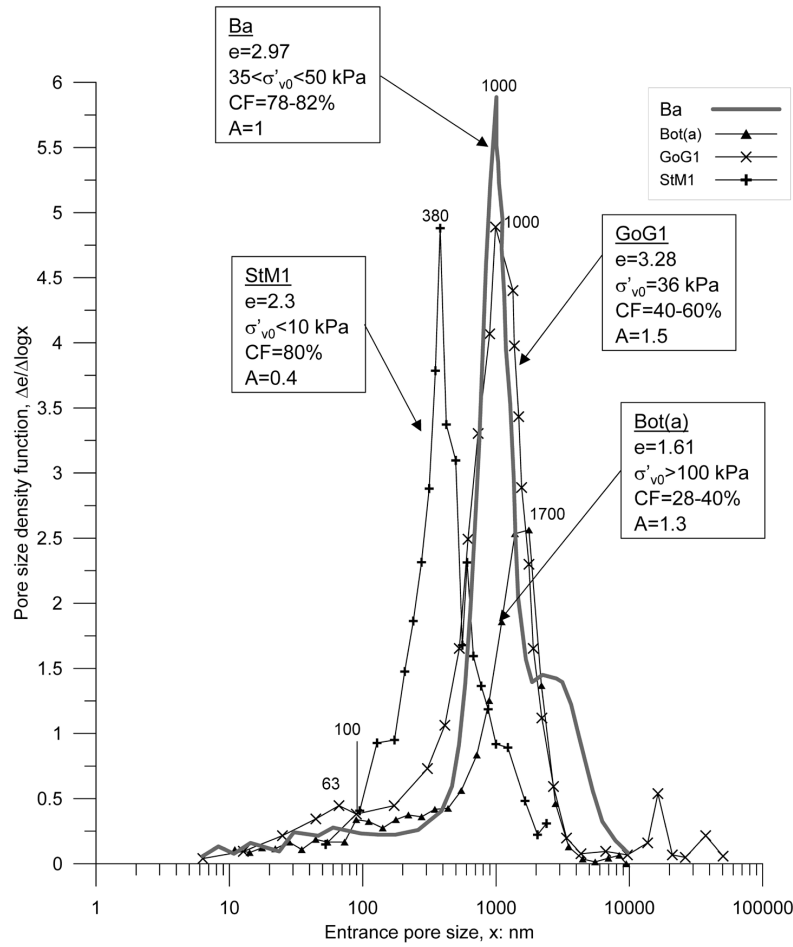


Figure 5. Comparison of pore size density, PSD, curves for soft St Marcel, Gulf of Guinea, Bothkennar and Ballina clay at undisturbed state (states StM1, GoG1, Bot(a) and Ba in Figure 2). Source: Delage & Lefebvre (1984), Hattab et al. (2013), Tanaka et al. (2003), Pineda et al. (2016b); modified after Guglielmi et al. (2022).

fabric of these stiff clays is highly variable, since the degree of local fabric orientation varies from very high to very low, the latter applying to relic portions of primary bookhouse fabric which did not undergo any significant increase in orientation during the geological pre-compression (Sfondrini, 1975). Nonetheless, the average degree of orientation index, L (see part I-paper), characterizing the micro-REV fabric observed through medium magnification micrographs, is high, corresponding to $L = 0.24-0.37$ for Pappadai clay and $0.22-0.29$ for Lucera clay (Figure 9b and 9d). Thus, both these stiff clays are found to embody a fabric qualitatively similar to that of reconstituted Pappadai clay 1D compressed to $I_v < 0$ (part I-paper).

The PSD curves of both stiff clays, Pa1 and Lu1, are shown in Figure 10, compared with that of Boom clay (Bo in Figure 2), reported by Lima et al. (2008). As for the PSD curves of reconstituted clay samples at $I_v < 0$ (Figure 2), i.e. BWL* and Pa1*, all the PSD curves in Figure 10 are monomodal and indicative of a pore volume mostly distributed in the micro-porosity range (pore size $< 1 \mu\text{m}$). For both

the high plasticity Boom and Pappadai clay (Table 1), the DPS (respectively 90 nm and 220 nm) is lower than for the lower plasticity and more silty Lucera clay (around 300 nm). The DPS of Boom clay is the lowest due to the far larger σ'_p and lower I_v (Figure 2).

Integrating the previous findings about the in-situ fabric of soft clays with those concerning the fabric of stiff natural clays, it appears that 1D compression of medium to high plasticity clays in the field, caused by the increase in burial depth, at early stages of compression ($I_v > 0$) allows for an either honeycomb or bookhouse fabric of bi-modal PSD. Further compression due to increase in burial depth brings about an increase in micro-REV fabric orientation, a loss of inter-aggregate porosity and a reduction in size of the monomodal DPS. The DPS of the natural stiff clays here of reference, of $I_v \ll 0$, is up to an order of magnitude smaller than that of the micro-porosity DPS of the soft clays of $I_v > 0$, and is of much lower frequency too.

Examples of pores of different size in stiff clays are shown in Figures 9 and 11. Where a relic macro-porosity,

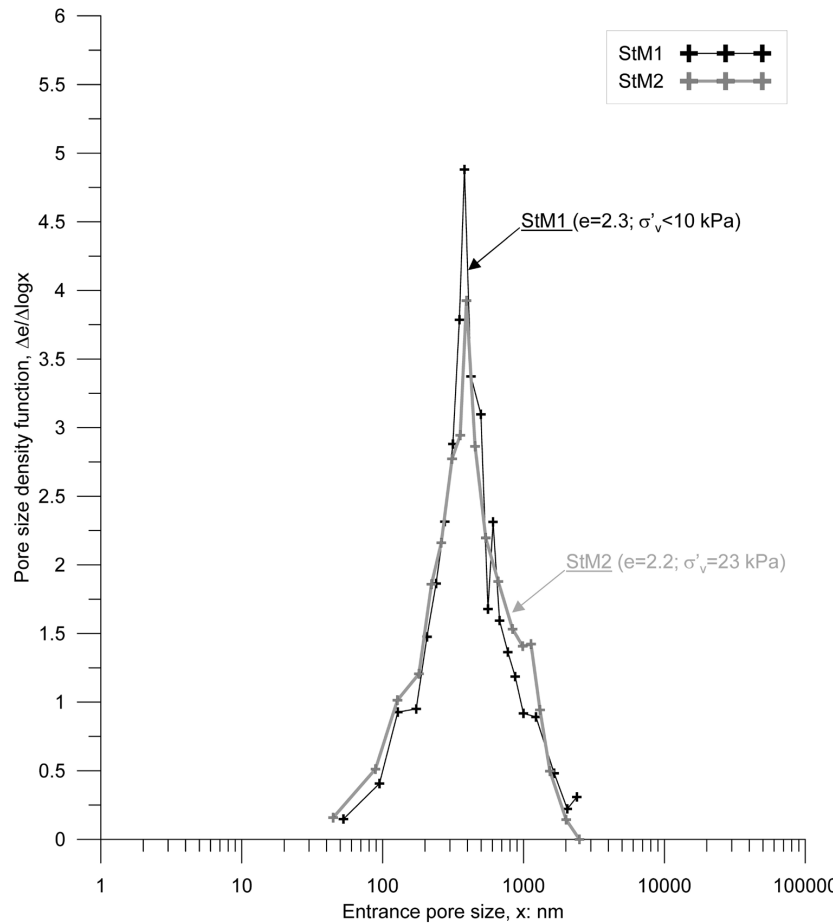


Figure 6. Comparison of PSD curves for pre-gross-yield states of St Marcel clay [states StM1 and StM2 in Figure 2, replotted and adapted after Delage & Lefebvre (1984)].

covering a wide range of entrance pore sizes, occurs here and there locally, most of the micro-pores of size about the DPS occur inside the densely packed stacks of particles.

The microstructure changes of both Pappadai and Lucera clay, induced by swelling from the undisturbed state to very low pressures ($\sigma'_v = 20$ kPa), have been investigated, along with their microstructure changes under compression, either before or beyond gross-yield (Figures 1 and 3). For the swelled natural Pappadai clay (PaS, Figure 1), SEM micrographs of medium magnification (e.g. Figure 12) show a very similar fabric to that of Pa1 (Figure 9a). Although long thick stacks of oriented particles are recognizable (Figure 12c), preferred orientation is not ubiquitous. The corresponding L values (Figure 12b, 12d) demonstrate that monotonic swelling does not change the original fabric orientation degree ($L = 0.24$ - 0.37). The PSD curve of the swelled specimen (Figure 13a) is still monomodal, with only a slight shift to the right of the DPS, for a slight increase of the intra-aggregate pores. Local widening of inter-aggregate pores is also observed, probably due to the opening of fissures.

Similarly, the PSD curve of natural Lucera clay, when either swelled to state LuS, or compressed pre-gross-yield, to $\sigma'_v = 1.4$ MPa (Lu2; Figure 1), are very similar to that of the undisturbed specimen (state Lu1), as shown in Figure 13b. Hence, the fabric and pore size distribution of either soft or stiff clays undergo no significant changes under monotonic swelling and compression pre-gross yield.

Figure 14a-14d shows examples of micrographs of natural Pappadai clay compressed beyond gross-yield, to states Pa3 and Pa4 ($\sigma'_v = 5$ MPa and $\sigma'_v = 18.5$ MPa; Figure 1), with the corresponding direction histograms and indices of fabric orientation. Even when the clay is compressed to very high pressures (state Pa4), the micro-REV fabric (Figure 14c) appears formed by thick layers of stacks interbedding randomly oriented fabric portions, which do not acquire an oriented fabric. Indices of average fabric orientation in the range $0.24 < L < 0.345$ are obtained, confirming that compression to high pressures does not determine any significant increase in the degree of micro-REV fabric orientation, with respect to the undisturbed fabric of the stiff natural clay occurring in the field, Pa1 ($L = 0.24$ - 0.37).

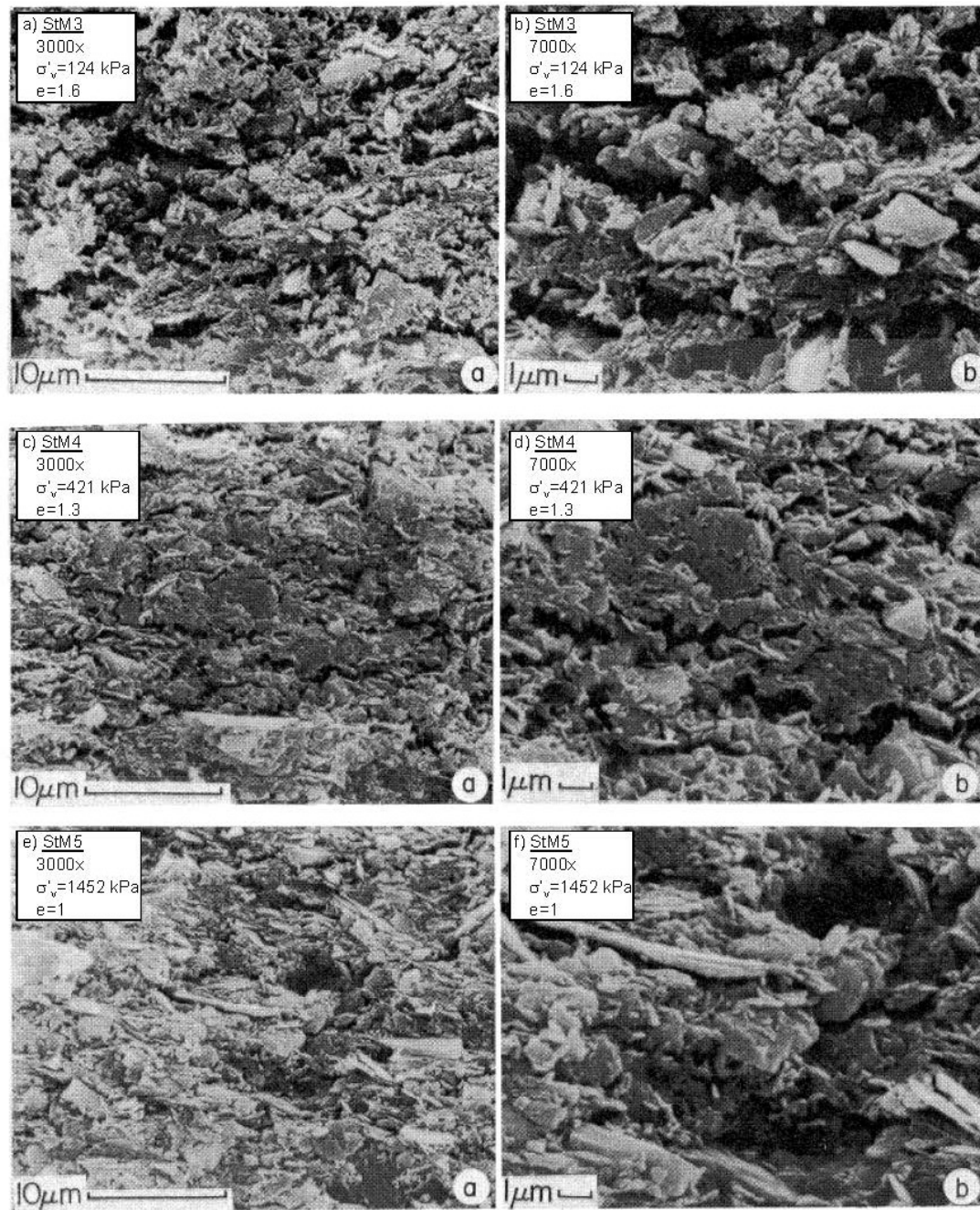


Figure 7. Evolution in 1D compression of the fabric of natural soft St Marcel clay, analyzed by means of SEM at different magnification along vertical planes at states: StM3 (a, b); StM4 (c, d); StM5 (e, f) in Figure 2 [source: Delage & Lefebvre (1984); modified after Guglielmi et al. (2022)].

For natural Lucera clay compressed to state Lu3 ($\sigma'_v = 5$ MPa; Figure 1), indices of micro-REV fabric orientation in the range 0.289-0.335 are obtained from image processing (an example is given in Figure 14e, 14f), indicating, again, that 1D compression to high pressure may determine only a minor increase in orientation, with respect to in situ clay fabric Lu1 ($L = 0.22$ -0.24).

The evolution in PSD of the stiff clays during 1D loading beyond gross-yield is shown in Figure 15. The PSD curves of Pappadai clay (Figure 15a) are indicative of a translation of the DPS towards smaller pore sizes and a reduction of the DPS frequency, both starting soon after gross-yield (Pa2, $\sigma'_v = 3$ MPa; Figure 1) and continuing all way through compression to high pressures. The same evolution is observed

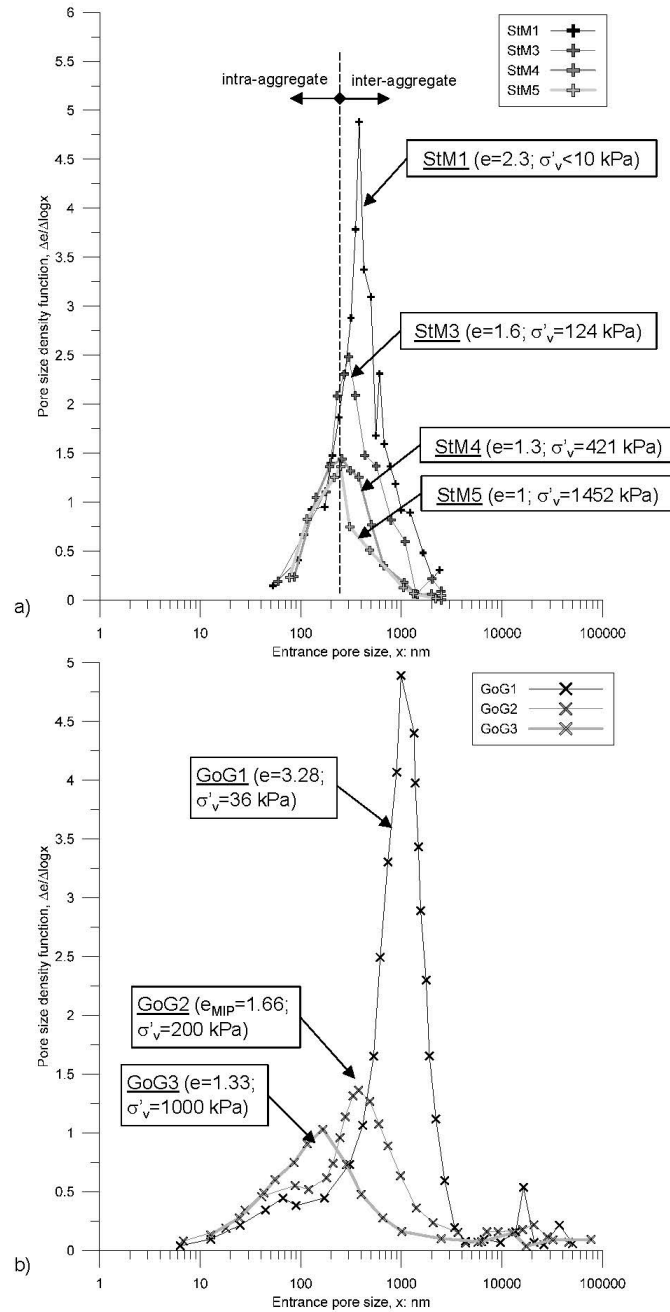


Figure 8. Evolution in 1D compression of PSD curves of a) natural St Marcel clay [states in Figure 2; replotted and adapted after Delage & Lefebvre (1984)] and b) natural GoG clay [states in Figure 2; after Hattab et al. (2013)]. Note that e_{MIP} is the maximum void ratio measured in the MIP test, reported when the specimen void ratio is not available [modified after Guglielmi et al. (2022)].

for Lucera clay (Figure 15b) and for the PSD of stiff Kyoto clay [1D compressed to ≈ 10 MPa; Tanaka et al. (2003)]. Such reduction in size of the intra-aggregate pores suggests that 1D compression determines the vertical straining of either the stacks, or the relic random fabric portions of the stiff clays. Such straining is such to keep constant the index of average fabric orientation, L , all way through 1D compression from the in-situ state of the stiff clay to high pressure, as

also observed for reconstituted clays 1D compressed to high pressures (part I-paper).

3.3 Different η compressions

For natural GoG clay isotropically compressed from the undisturbed state GoG1 (Figure 2) to $p' = 1000$ kPa, Hattab et al. (2013) observed a loss of inter-aggregate pores

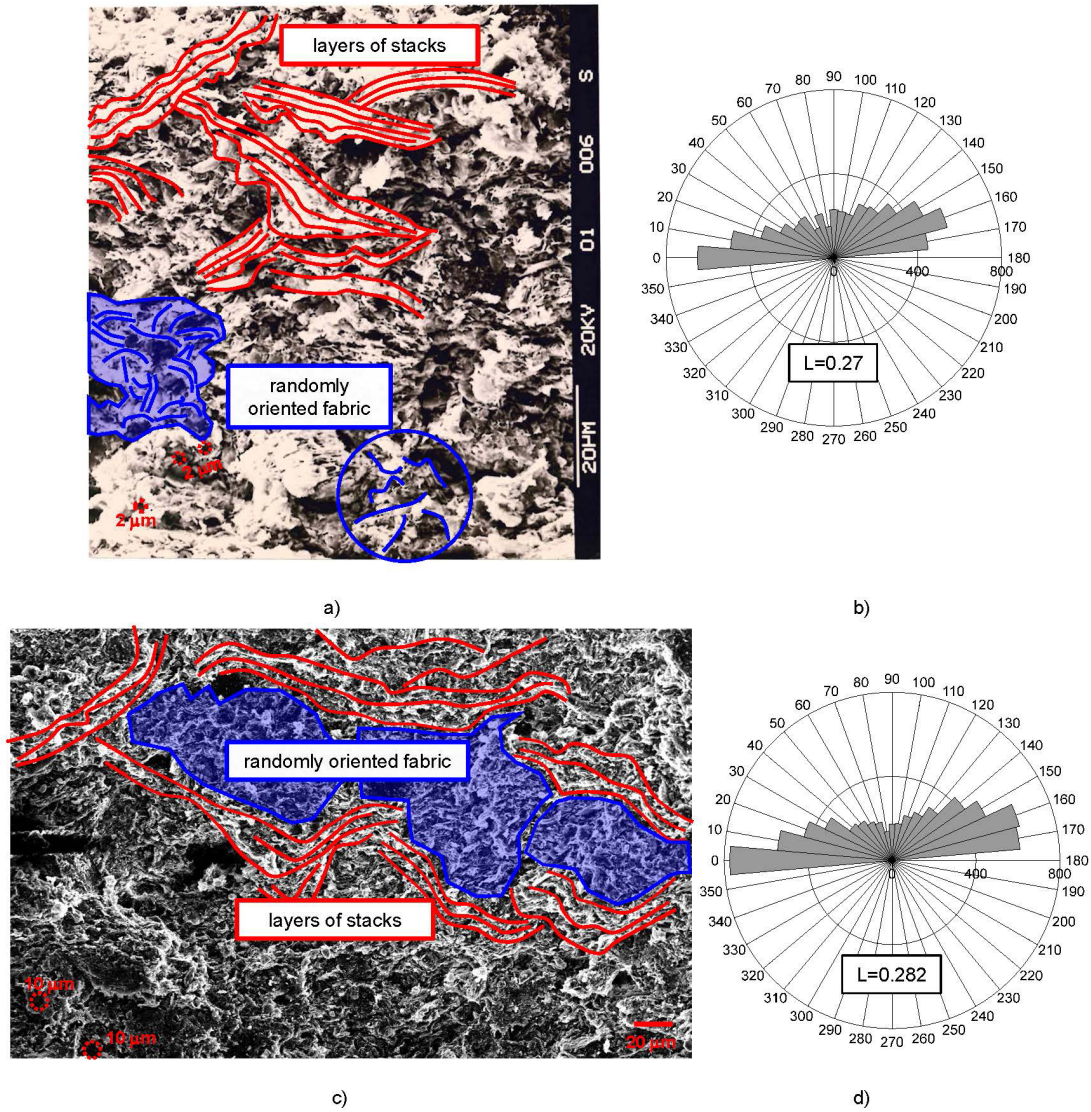


Figure 9. Micro-REV fabric and corresponding direction histograms of: natural Pappadai clay (a, b; state Pa1 in Figures 1, 2) [modified after Cotecchia et al. (2019)]; natural Lucera clay (c, d; state Lu1 in Figures 1, 2).

and bimodality [see Figure 19b in Guglielmi et al. (2022)] and a random orientation fabric. As for reconstituted clays (part I-paper), the change in PSD appears faster in isotropic than in 1D compression.

Pre-gross-yield, isotropic compression of natural Lucera clay ($p'_{y, is} \approx 2700$ kPa; Y_{iso} in Figure 1), from Lu1 to iLu2 ($p' = 1640$ kPa; Figure 3), is found to induce a very limited change in PSD, the same as pre-gross yield 1D compression (Figure 16). In contrast, isotropic compression beyond gross yield to iLu3 ($p' = 4200$ kPa $\approx 1.5 \cdot p'_{y, is}$; Figure 3) determines a reduction in DPS of natural Lucera clay to 220 nm (Figure 16), again at a faster rate than in 1D compression (state Lu3, $\sigma'_v = 5$ MPa; Figure 3: both iLu3 and Lu3 underwent the same volumetric strain of 7% since Lu1), as found already for both soft or reconstituted clays.

The micro-REV fabric orientation for state iLu3 (Figure 17) suggests that isotropic compression to $p' \approx 1.5 \cdot p'_{y, is}$ is not enough to induce a considerable change in L , which is found still high (range 0.25-0.32; Figure 17). Small particle aggregations defining concentric layouts are observed in both iLu2 and iLu3 (Figure 17b), which might represent just the start of a reduction in fabric orientation degree.

4. Modeling implications of the micro-rev changes: preliminary conclusions

The data discussed so far, in both this part II-paper and previous part I-paper, allow for a conceptual modelling of the microstructure evolution under compression of natural versus reconstituted multi-mineral clays, of low-medium to high

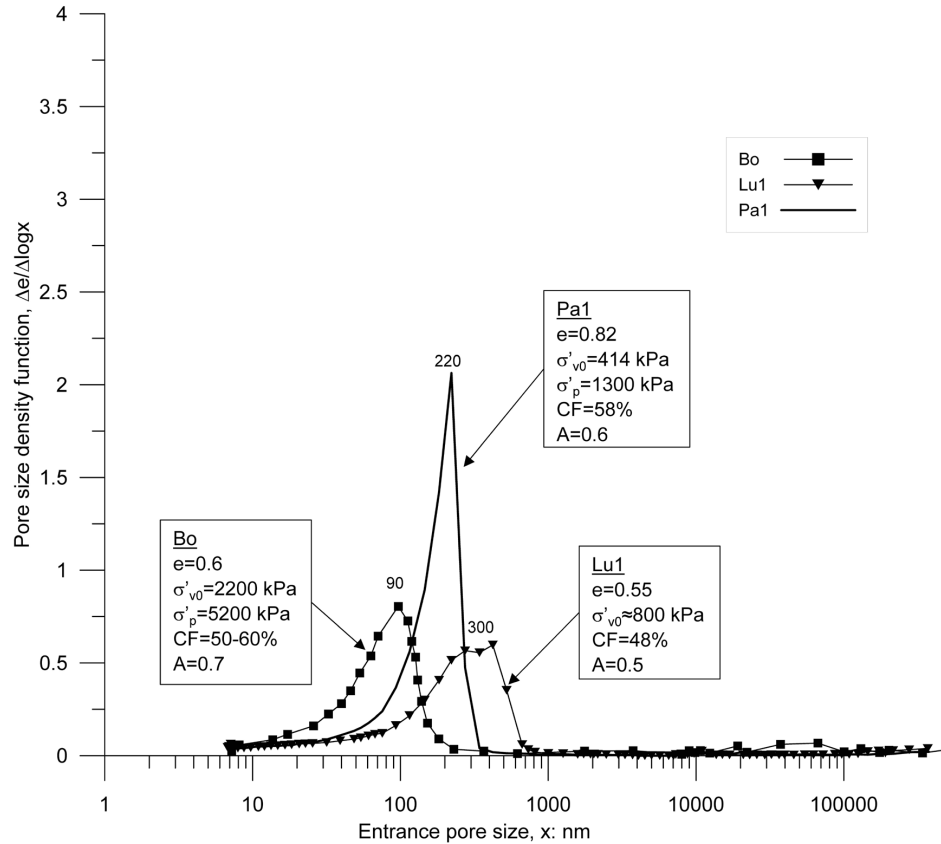


Figure 10. Comparison of PSD curves of stiff Pappadai, Lucera and Boom clay at undisturbed state [Pa1, Lu1, Bo in Figure 2; data for Boom clay after Lima et al. (2008); modified after Guglielmi et al. (2022)].

activity (Table 1). Figure 18 shows such conceptual model, in the $I_v - \sigma'_v$ plane, with reference to 1D compression. Based on the post-mortem micro-analyses discussed above and in the part I-paper, the model portrays a fabric of low orientation degree, $L \ll 0.21$, when the clays are consolidated to $I_v > 0$, whether in the field, or in the laboratory (from slurry). Such fabric is classifiable as either honeycomb or bookhouse [see Figure 7f in the part I-paper; Sides & Barden (1971); Griffiths & Joshi (1990)]. Inter-aggregate macropores and intra-aggregate micro-pores characterize the honeycomb fabric PSD, which is markedly bimodal. In contrast, macropores are more evenly distributed in the bookhouse fabric (e.g. St Marcel clay), whose PSD is already closer to mono-modal at the early stages of compression. Therefore, the micro-scale data provide evidence that natural and reconstituted clays of low-medium to high activity achieve fabrics of low degree of orientation whether they are consolidated from slurry in the laboratory⁴, or in either marine or periglacial environments over geological time.

Upon 1D compression of these initially high I_v clays, either natural or reconstituted, the model shows a major loss of macropores (§3.1; Figures 5 and 8) and the onset

of a monomodal PSD for any clay reaching I_v about 0. At the same time, as observed for reconstituted clays (see Figure 13 in the part I-paper), a diffuse increase in fabric orientation degree starts, since particles and domains tend to coalesce into stacks, in the same way for 1D compression in the laboratory of reconstituted clays, or in the field, for natural clays under burial. The single DPS, recognizable for all clays by $I_v < 0$, occur mostly within the stacks and evolves towards smaller values, reaching values typical of microporosity (Figure 18).

As shown by the model, the development of stacks that takes place through 1D compression towards $I_v < 0$ occurs for: i) the reconstituted clay 1D compressed in the laboratory from a slurry of initial $I_v \gg 0$; ii) the soft natural clay of high I_v in the field, 1D compressed in the laboratory post-gross yield; ii) the natural clay 1D compressed under burial in the field, during the geological history. The enrichment in stacks with 1D compression brings about an increase in micro-REV fabric orientation degree, such that L becomes higher than 0.21 for all the cited classes of clays. However, the maximum micro-REV fabric orientation degree is reached already by I_v values a little below 0 (depending on clay composition). During further 1D compression to high pressures, the clay micro-REV fabric keeps L_{const} , relics of bookhouse fabric

⁴Using either distilled or tap water.

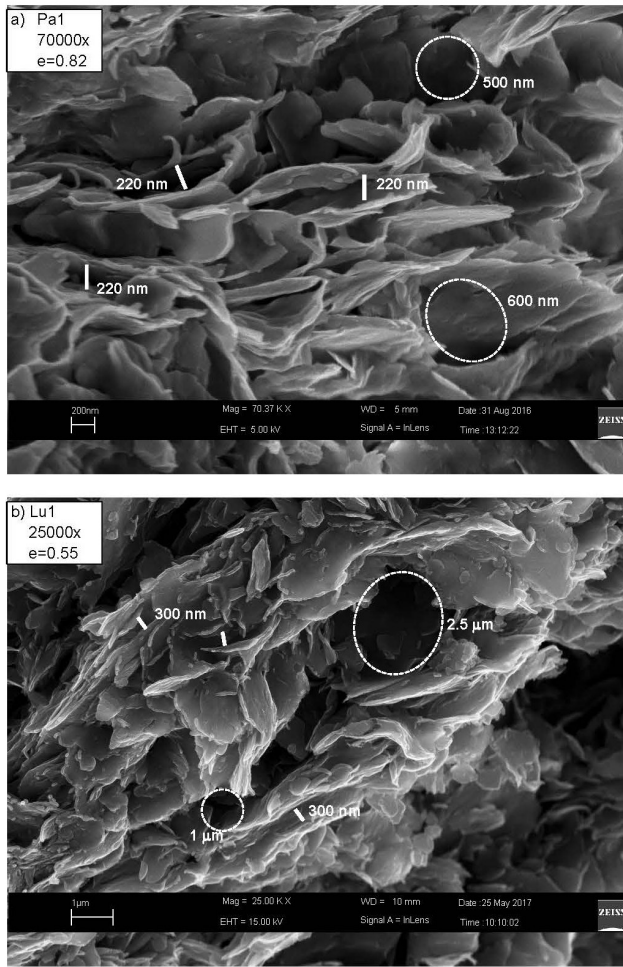


Figure 11. High magnification SEM micrographs of natural Pappadai clay (a; Pa1 in Figure 1) and natural Lucera clay (b; Lu1 in Figure 1); modified after Guglielmi et al. (2022).

are preserved between the stacks (Figure 18) and particle orientation does not become ubiquitous (complete preferred orientation, CPO, is achieved within the stacks and low orientation fabric is interbedded between the stacks).

As anticipated in part I-paper, the reduction in clay void ratio while micro-REV fabric orientation has become steady, L_{const} , results from a combination of micro-scale processes taking place with 1D compression at $I_v < 0$. These are: a) the rotation and coalescence of particles and domains with the sub-horizontal stacks, contributing to their thickening; b) further reduction of the DPS due to vertical straining of both the stacks and the relic bookhouse fabric portions. The stacks, though, are more compressible than the preserved random fabric portions (part I-paper) and contribute most to the overall clay straining. At the same time, though, they are thickened thanks to further orientation of particles, so that, on the whole, the ratio between the CPO fabric portions and the flocculated ones does not increase, and L remains constant. Such finding represents a major target of micro-

scale modelling of clays (see Figure 1, line b in the part I-paper), either reconstituted or natural. At the same time, this finding provides indications for the hardening function of constitutive laws, formalized in the framework of elastoplasticity and accounting for fabric as internal variable within a microstructure parameter of the constitutive law (see Figure 1, line a-ii in the part I-paper). It is proposed that, for the hardening in 1D compression, $L(I_v)$ (Figure 18) might be used as function representing the evolving micro-REV fabric orientation (e.g. Figure 17b in the part I-paper). Both the hardening function and the elastic stiffness anisotropy of the clay under 1D compression could be related to $L(I_v)$.

The micro to macro-scale investigation results discussed in the previous sections show also that, despite the similarities in fabric features between the natural and the reconstituted clay at the same void ratio (Figure 18), the corresponding microstructures are of different strength. Hence, such difference is evidently due to a difference in bonding between the two types of clays, and bonding can be considered the main source of the differences between the gross yield states in 1D compression of the natural and reconstituted clays in Figure 18.

It is worth recalling that, in the literature, gross yield corresponds to the effective stress state, $Y(\sigma'_{ij})$, after which a major immediate drop in stiffness occurs along a given stress path, and for clays, this has been assumed to correspond to the onset of major microstructural change (Hight et al., 1992; Smith et al., 1992; Cotecchia & Chandler, 1997, 2000). Direct evidence of such change has been seldom provided (Cotecchia et al., 2016, 2019; Guglielmi et al., 2022) and both parts, I and II, of this keynote lecture contribute to the provision of such evidence. Furthermore, for both natural and reconstituted clays, gross yield has been generally found to occur near the state boundary surface in the v - q - p' space, SBS [e.g. Leroueil et al. (1979), Graham et al. (1988), Cotecchia & Chandler (2000), Smith et al. (1992)], as shown for some clay examples in Figure 19. The model in Figure 18 suggests that the difference in bonding between the natural and the reconstituted clay is the main source of the differences between the gross yield surface of the natural clay, $F(\sigma'_{ij})$, and that of the reconstituted clay, $F^*(\sigma'_{ij})$, for the same hardening stage. In the same way, bonding is the source of the difference between the natural clay state boundary surface, SBS, and the reconstituted clay SBS* (e.g. Figure 19), in the v - q - p' space. It follows that the microstructure constitutive function controlling the clay gross yielding and hardening law (see Figure 1, line a-ii, in the part I-paper) should include the clay micro-REV fabric orientation degree, the porosimetry, and the clay bonding, all evolving with compression.

The data presented in the previous sections provide also direct evidence of the insignificant changes in clay microstructure that occur during monotonic loading paths pre-gross yield, at least for either reconstituted, or intact unfissured clays of the type in Table 1. For example, the data show that along monotonic 1D swelling, or 1D compression,

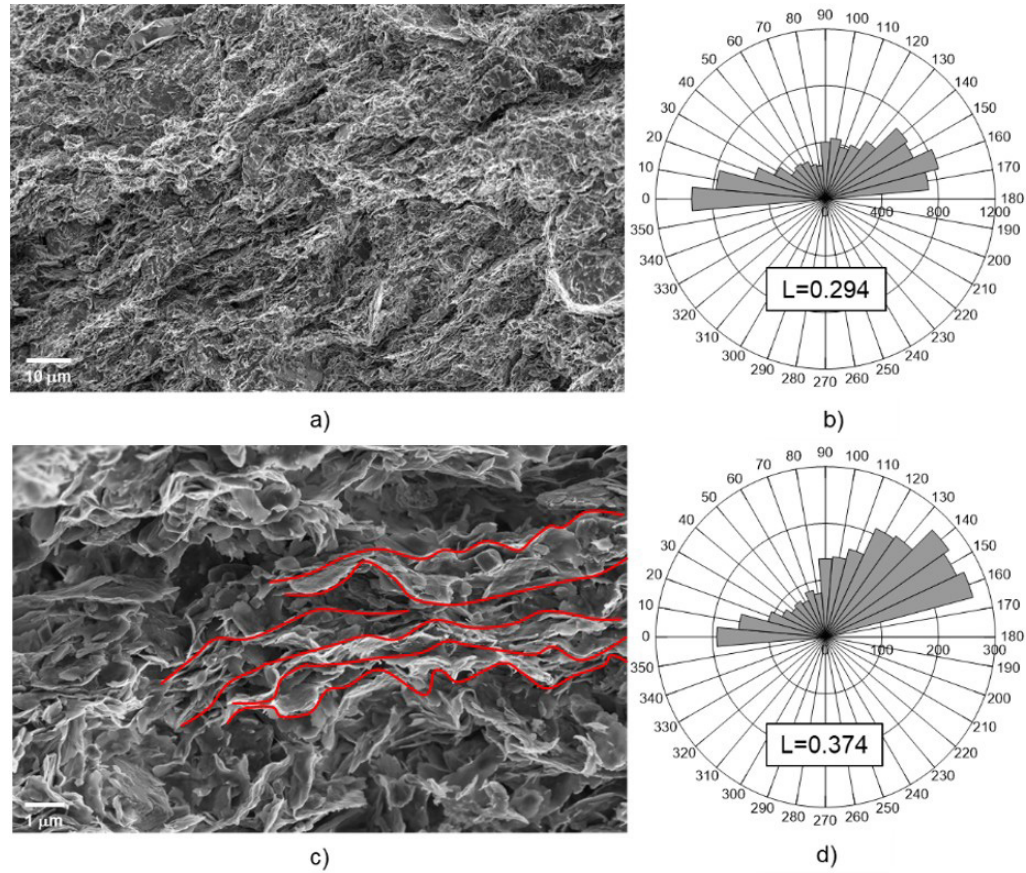


Figure 12. SEM micrographs along vertical fractures and corresponding direction histograms of natural Pappadai clay 1D swelled to $\sigma'_v = 20$ kPa (state PaS in Figure 1) at the micro-REV scale (a, b) and at higher magnification (c, d), with example of stack [modified after Guglielmi et al. (2022)].

or isotropic compression pre-gross yield, no changes of either the bonding strength (reflected by C_s^*/C_s), or the PSD, or the orientation degree of the micro-REV fabric (L), are observed for either the natural or the reconstituted clay. Such observations confirm previous speculations about the source of gross yielding and should inform the modeling of the hardening plasticity inside the SBS [see Figure 1, line b in the part I-paper; e.g. Kavvas & Amorosi (2000), Rouainia & Muir Wood (2000), Baudet & Stallebrass (2004)], suggesting that this should not be related to microstructural changes in monotonic loading paths. This is not the case, however, for fissured clays, as shown by Vitone & Cotecchia (2011) and Vitone et al. (2019). Also, cyclic loading pre-gross yield may be capable of weakening the clay bonding [e.g. Massaro (2004)] and affecting the clay fabric and PSD below the SBS, also for un-fissured natural clays.

Given the similarity in the type of fabric for the natural and the reconstituted clay of given void ratio and the negligible change in microstructure taking place with monotonic loading pre-gross yield, if the natural bonding is isotropic and amplifies the gross yield stresses through a scalar factor common to all the loading paths (i.e. the onset

of the natural bonding degradation, determining gross yield, is not influenced by the stress ratio and occurs when it is reached a given ratio $\sigma'_{ijY} / \sigma'_{ije}$ common to all the loading paths), the SBS and the SBS* are homotetic. This is the case of natural Pappadai clay, whose gross yield curve is compared with that of the reconstituted clay in Figure 19a, and of several other firm to stiff clays [e.g. Coop & Cotecchia (1995), Cotecchia & Chandler (2000), Cafaro & Cotecchia (2001), Cotecchia et al. (2007), Guglielmi (2018)]. In this case, the scalar stress sensitivity, $S_\sigma = p'_{K0Y} / (p'_{K0Y})^*$, may be considered the effect on gross yielding of the difference in bonding between the natural and the reconstituted clay.

If the bonding is isotropic, but the onset of bonding degradation occurs at ratios $\sigma'_{ijY} / \sigma'_{ije}$ which vary depending on the loading paths, i.e. bonding degradation is sensitive to the stress ratio η , the SBS and SBS* are not homotetic. This is the case for several highly sensitive soft clays, e.g. Saint Alban and Bothkennar clay, whose SBS and SBS* are shown in Figures 19b and 19c. For such a class of clays, S_σ is found to represent the gross yield amplification ratio only for loading paths characterized by $\eta \geq \eta_{K0}$, as will be discussed later.

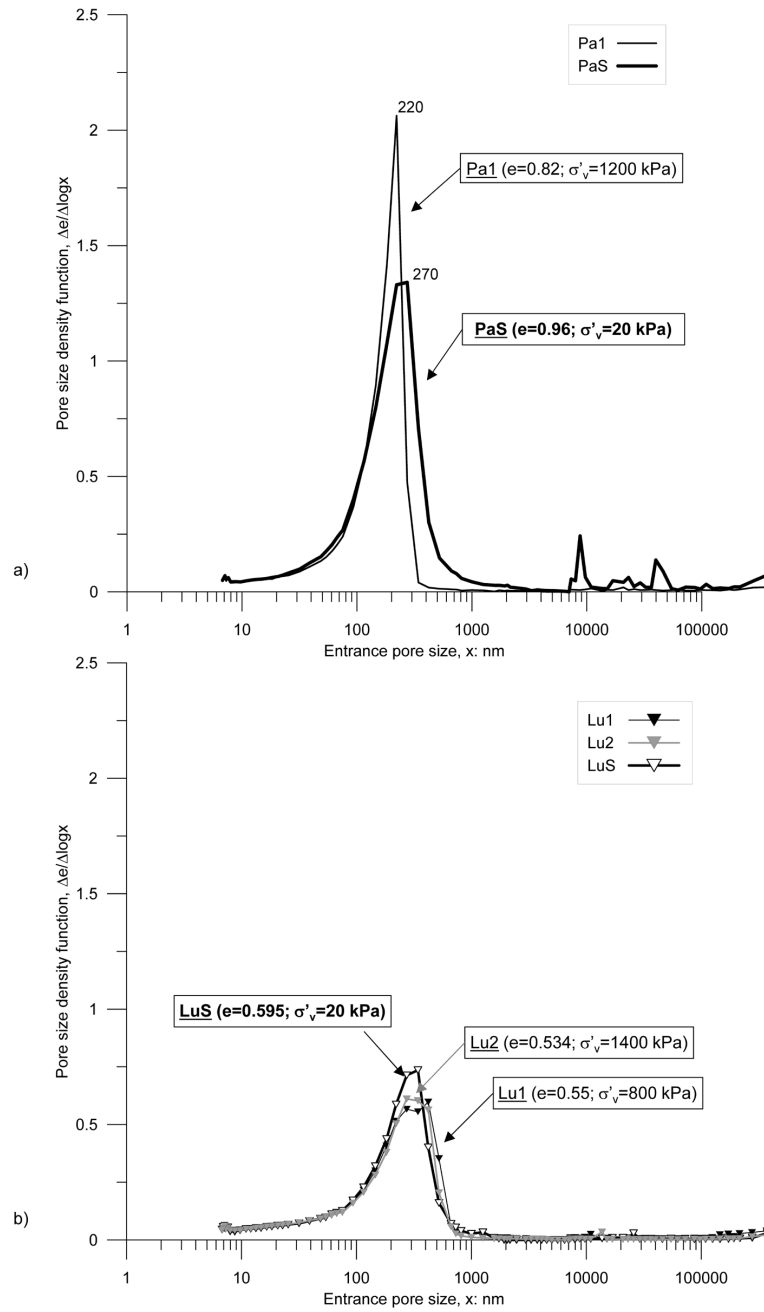


Figure 13. Comparison of PSD curves for pre-gross-yield states of a) natural Pappadai clay and b) natural Lucera clay [modified after Guglielmi et al. (2022)].

Regarding the rate of bonding decay, the data discussed in the previous sections provide evidence of a rather fast decay of the bonding strength over gross yielding, since the swell sensitivity of the natural clay, C_s^*/C_s , drops to 1 (Figure 1) soon after gross yield. Correspondingly, S_σ starts decreasing, and the fabric starts changing too (Figure 18). However, despite the fast drop of C_s^*/C_s , S_σ decreases slowly and the function $S_\sigma(\varepsilon_{vol}^p)$ represents the effect of current bonding on the clay 1D compressibility. $S_\sigma(\varepsilon_{vol}^p)$ keeps being higher than 1 up to high pressures. This observation highlights that the

combination of fabric and bonding acquired by the natural clay differs from that of the reconstituted clay at any stage of compression, despite the qualitative similarity of the natural and reconstituted clay fabric for any given void ratio. Hence, natural and reconstituted clays must be considered different materials at the micro to the macro-scale, at any stage of loading, despite their equal composition and fabric features.

Since the data discussed so far in this part II-paper have shown that, irrespective of the bonding features, the fabric changes occurring in natural clays under compression, either

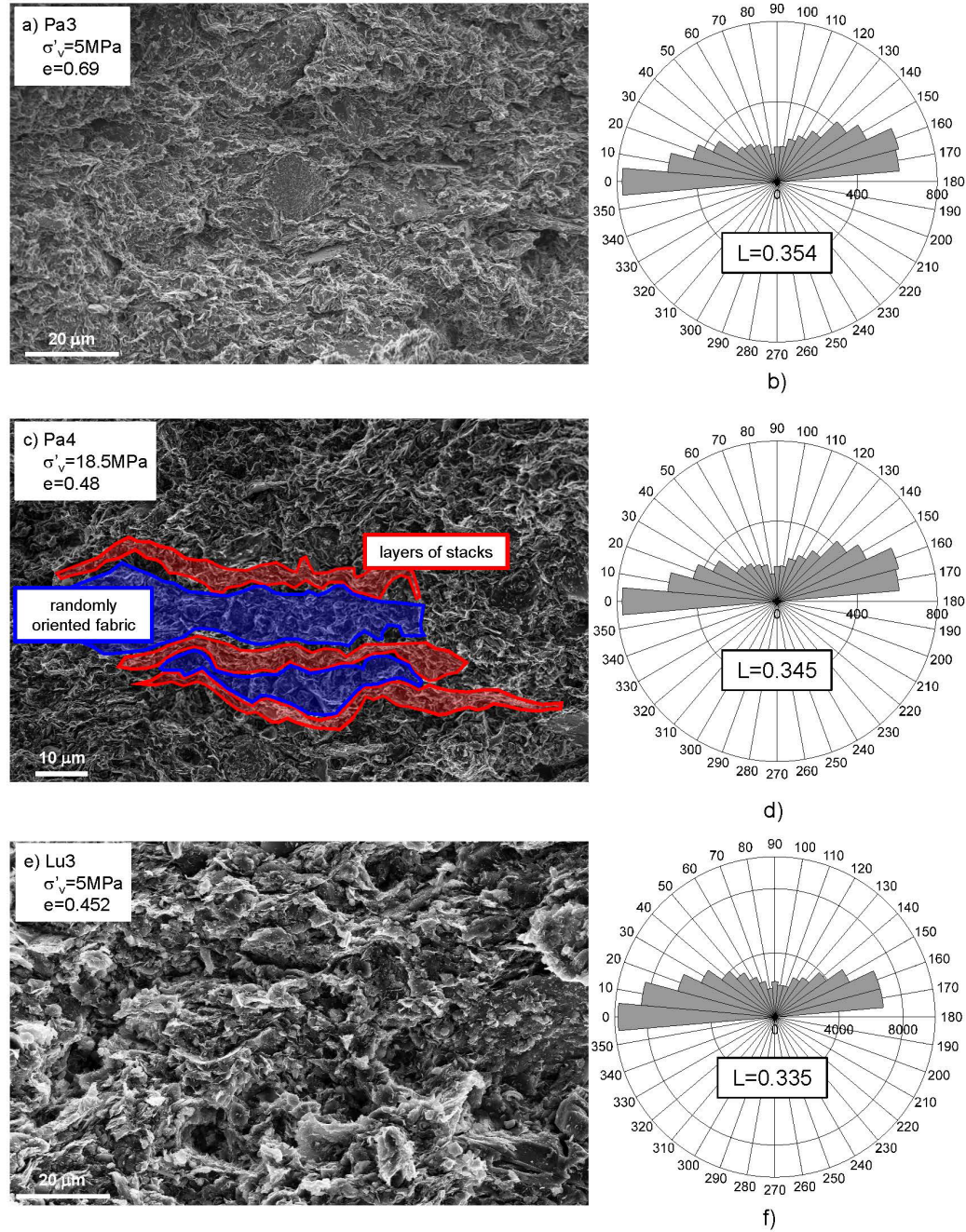


Figure 14. Micro-REV fabric and corresponding direction histograms of: natural Pappadai clay compressed to state Pa3 (Figure 1) (a, b) and to state Pa4 (Figure 1) (c, d); natural Lucera clay compressed to state Lu3 (Figure 1) (e, f); modified after Guglielmi et al. (2022).

1D or isotropic, are similar to those observed for reconstituted clays, in Figure 20a the same framework used in part I-paper to schematize such changes for reconstituted clays is used to represent the micro-REV fabric changes in natural clays, under either 1D or isotropic compression (in this case along the K_0 NCL and the INCL, whereas in Figure 13 of part I-paper these changes occur along K_0 NCL* and INCL*, respectively). Accounting for such framework and for the changes in bonding strength discussed above for the natural clays, indications about

the reasons for the differences in yield surface and plastic strain hardening of natural stiff ($I_v < 0$) versus soft clays ($I_v > 0$) can be commented. It is convenient to start with comparing the gross yield hardening of the natural stiff clay with that of the corresponding reconstituted clay when both are at $I_v < 0$. The analysis will again refer first to the strain hardening in 1D compression and, secondly, to that in isotropic compression, concluding with more general considerations referring also to the other possible loading paths.

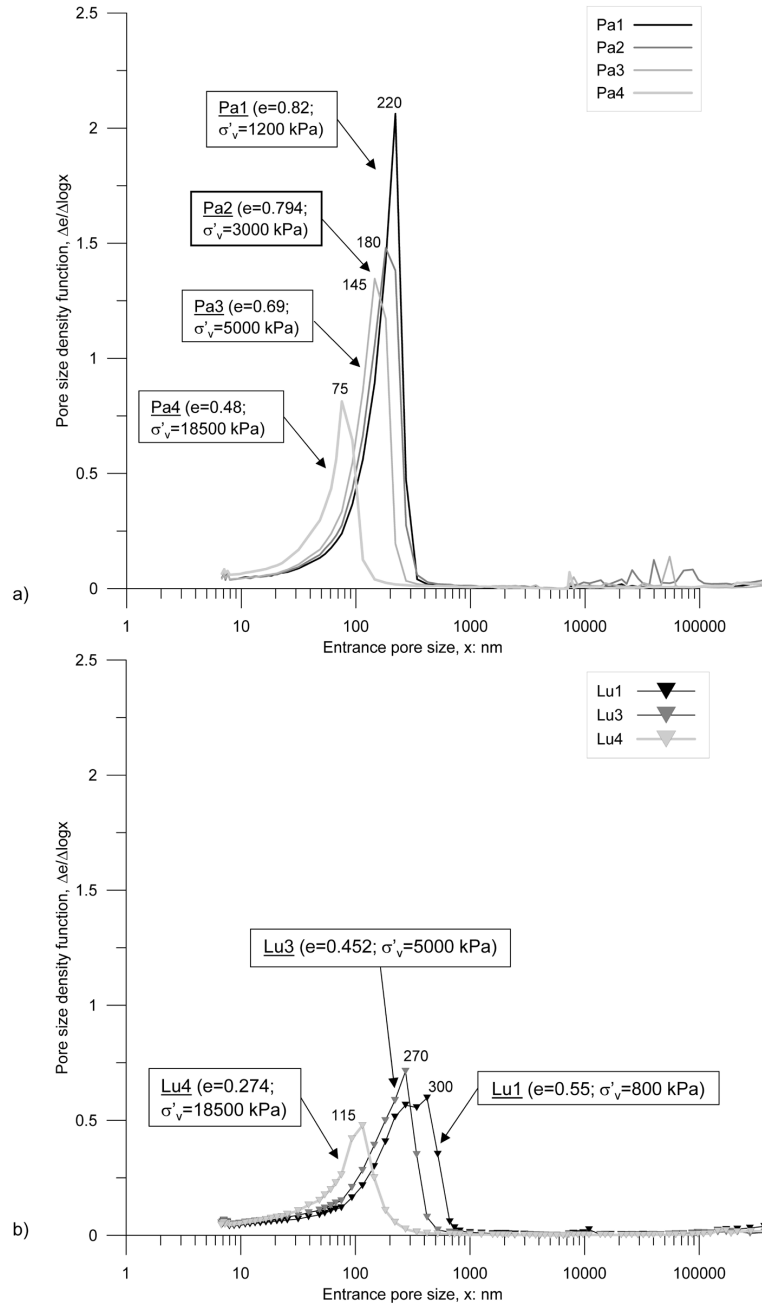


Figure 15. Evolution in 1D compression of PSD curves of natural Pappadai (a) and natural Lucera clay (b); modified after Guglielmi et al. (2022).

The data have shown that, for $I_v < 0$, the orientation degree acquired by both the natural and the reconstituted micro-REV fabric is already high, $L > 0.21$. Furthermore, since the degree of fabric orientation does not increase with 1D compression post-gross yield of the natural clay, the only evolving fabric feature which influences the hardening in post-gross yield 1D compression is the DPS reduction and the corresponding reducing porosity (Figure 18). It follows that the natural fabric changes in 1D compression are a source of an isotropic gross yield hardening, relating to plastic volumetric straining, as already observed for reconstituted

clays. This is made evident by macro-scale test data such as those acquired through shear testing of specimens 1D compressed to states either about gross yield, or post-gross yield, like those shown in Figure 20b and 20c, respectively for stiff Montemesola clay (Cotecchia et al., 2007) and Vallericca clay (Amorosi, 1996; Amorosi & Rampello, 1998). For these clays the undrained shear paths of specimens compressed to different post-gross yield pressures are similar. However, the hardening function holding for reconstituted clays, $p^*_e(\epsilon_{vol}^p)$, does not normalize these stress paths, due to the concurrent weakening of bonding, reflected in the scalar function

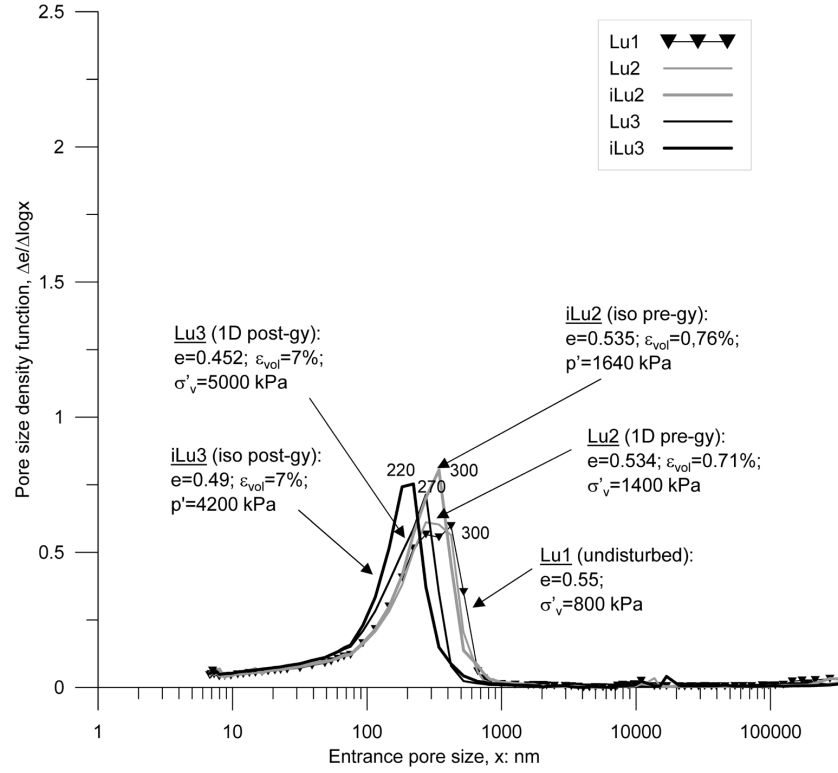


Figure 16. Comparison of PSD curves for specimens of natural Lucera clay 1D and isotropically compressed to states pre- and post-gross-yield to approximately the same void ratios (states in Figure 1); modified after Guglielmi et al. (2022).

$S_\sigma(\varepsilon_{vol}^p)$. Such weakening determines a negative hardening contribution, which combines with the positive hardening determined by the volumetric plastic straining for natural clays, but not for reconstituted clays.

As suggested by Cotecchia & Chandler (2000), for this class of clays, the scalar function $S_\sigma(\varepsilon_{vol}^p)p_e^*(\varepsilon_{vol}^p)$ may be suitable as isotropic volumetric hardening function to predict the response of the 1D compressed natural stiff clay. Such suitability is confirmed by the normalization of the results of shear tests on stiff clay specimens 1D compressed to different p'_{K0y} , shown in Figure 21. Both $S_\sigma(\varepsilon_{vol}^p)$ and $L(\eta_{K0})$ may become components of the microstructure constitutive parameter of the clay constitutive law, the first to represent the clay bonding, and the second for the clay fabric under 1D compression.

For such class of clays, which are only mildly sensitive ($S_\sigma \leq 3$), the natural bonding is rather stable and the onset of bonding degradation has been found not to be sensitive to η and, as such, to be the source of an amplification of the gross yield surface (scalar ratio $\sigma'_{ijV}/\sigma'_{ije} = S_\sigma$, or $p'_{K0y}/p^*_{K0y} = S_\sigma$). Therefore, the SBS and the SBS* are homothetic, as shown in Figure 19a for Pappadai clay. Both state boundary surfaces are seen to have an arch shape, probably partly due to the high degree of micro-REV fabric anisotropy. Nonetheless, $p'_{yi} > p'_{yK0}$ and $N-N_{K0} > 0$, as for reconstituted clays [see Cotecchia et al. (2007); also, Lucera clay data in Figure 3

and Figure 20a]. Evidently, the reduction of η , does not cause a weakening of the clay bonding faster than for $\eta \geq \eta_{K0}$, so that $S_\sigma = p'_{K0y}/p^*_{K0y} = p'_{yi}/p^*_{yi}$.

For this type of clays, isotropic compression post-gross yield determines a weakening of bonding equivalent to that occurring in 1D compression, as shown by the drop in swell sensitivity, κ^*/κ , recorded post isotropic gross yielding, as shown in Figure 3. With regard to fabric, the isotropic compression post-gross yield seems to cause a reduction of the macro-porosity faster than for 1D compression (Figure 16) and the onset of a less oriented fabric, but the fabric transition seems as slow as for reconstituted clays (see part I-paper). Therefore, the micro-scale data suggest that, for either 1D or isotropic compression (varying η), the microstructure of the stiff clays undergoes similar evolutions to those observed for reconstituted clays, except for the weakening of bonding which is experienced by the natural clay only and appears to be the same in isotropic and 1D compression (i.e. not dependent on η), being represented by the scalar function $S_\sigma(\varepsilon_{vol}^p)$. Such similarity is consistent with the parallelism of the INCL and the K_0 NCL of the stiff natural clays, making $S_\sigma = p'_{K0y}/p^*_{K0y} = p'_{yi}/p^*_{yi}$ at any void ratio.

It follows that the fabric changes, similar for the natural and the reconstituted stiff clays undergoing the same constant η compression, result in constant L values which reduce with reducing η (see Figure 17 in the part I-paper). Consequently,

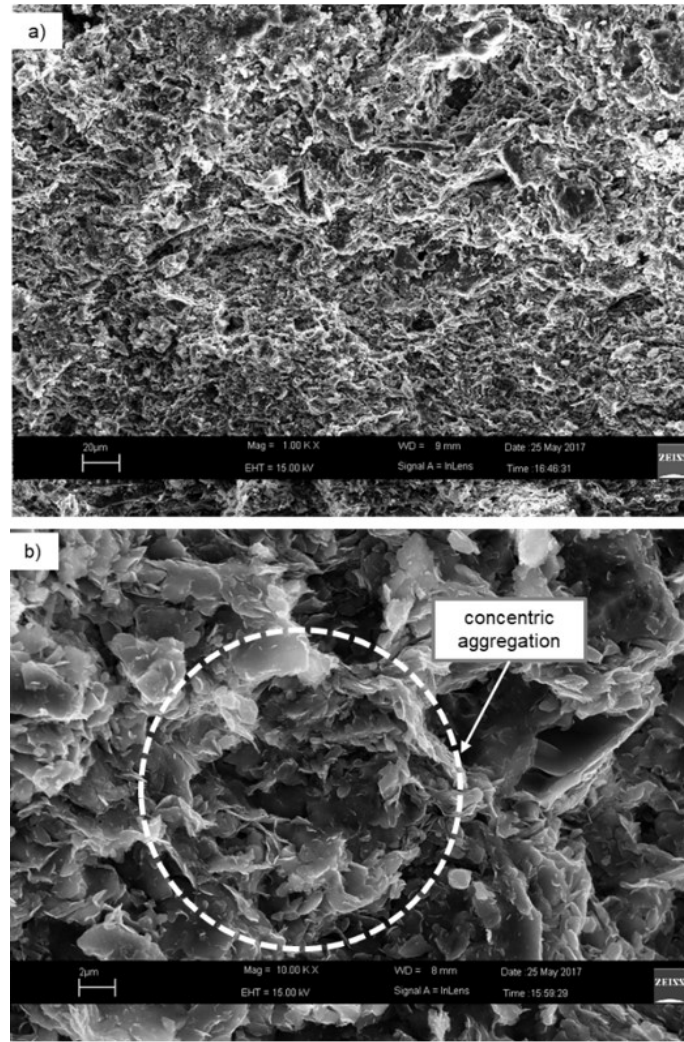


Figure 17. Fabric of natural Lucera clay isotropically compressed to post-gross-yield state iLu3 (Figure 3) at the micro-REV scale (a); example of concentric aggregation of particles (b); modified after Guglielmi et al. (2022).

the function controlling the isotropic hardening of the clay depends on η , as sketched in Figure 20a (as well as in Figure 13 of part I-paper), for both natural and reconstituted clays. However, the response of the natural clay depends also on bonding weakening, which determines a negative hardening contribution. This is exemplified in Figure 19a, where, along with the gross yield curve of the undisturbed Pappadai clay, the shear stress paths of natural clay samples compressed isotropically post-gross yield are also shown; it can be seen that these are not normalized by the sole hardening function $p_e^*(\epsilon_{vol}^p) (= e^{[(N^*-v)/\lambda^*]})$. Cotecchia & Chandler (2000) show that the isotropic volumetric hardening function $S_\sigma(\epsilon_{vol}^p)p_e^*(\epsilon_{vol}^p)$ suits, in first approximation, the normalization of the gross yield states of the undisturbed clay and those of the samples compressed post-gross yield, either isotropically, or 1D, as shown in Figures 21 and 22. However, the figures show that while the function $S_\sigma(\epsilon_{vol}^p)p_e^*(\epsilon_{vol}^p)$ predicts the influence on the clay response of the evolution in bonding strength and

PSD, it is not sufficient to predict the effects of the variation in fabric anisotropy determined by the variation in η applied during compression. The same limitation applies to the use of isotropic volumetric hardening with reconstituted clays (see Figure 18 in the part I-paper). For both the reconstituted and the natural clay, the positive hardening function $p_e^*(\epsilon_{vol}^p)$ does not represent successfully the effects of fabric on the hardening law, since it does not take into account the effects on clay stiffness of the different fabric orientation degrees, acquired by the clay for different η compressions. The different L values achieved through different η compressions cause, for example for η_{K0} and η_0 , the normalized undrained shear paths of the 1D consolidated clay to be much steeper than those of the isotropically consolidated specimens.

Differently from what is observed for stiff clays, gross yielding of soft sensitive clays [e.g. Bothkennar clay, Pisa clay, Saint Alban clay, Gulf of Guinea clay, etc.; e.g. Smith et al. (1992), Leroueil et al. (1979), Le (2008), Callisto

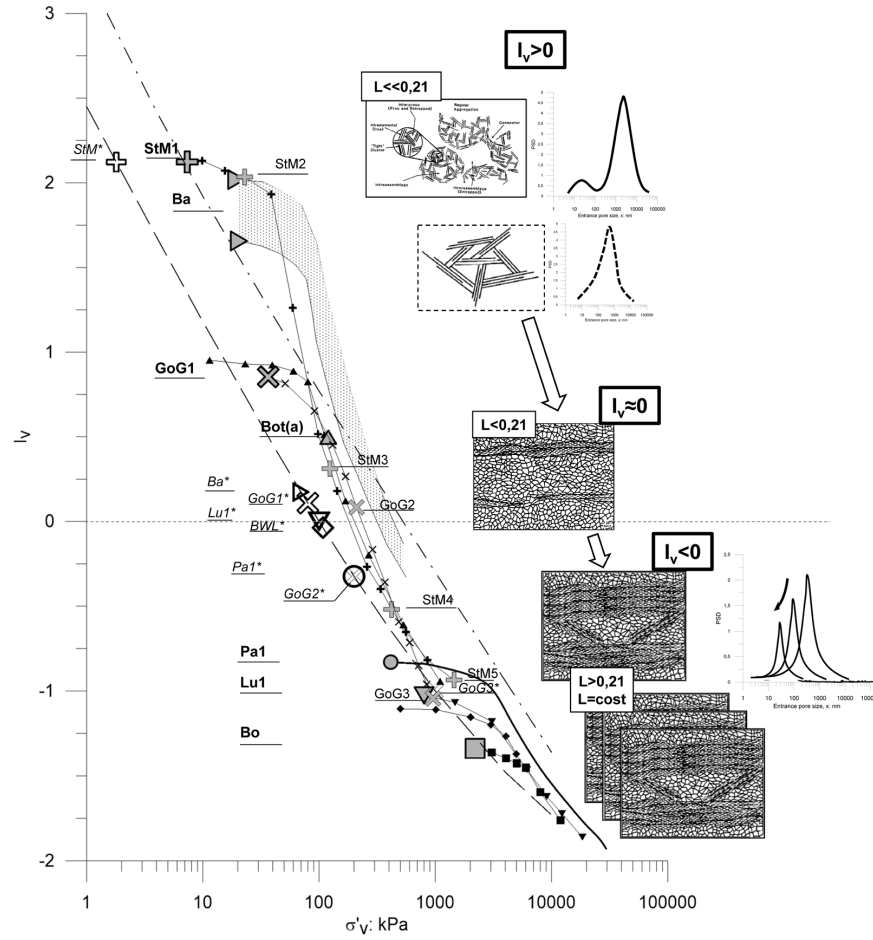


Figure 18. Conceptual model of the fabric features of multi-mineral clays under 1D compression [modified after Guglielmi et al. (2022)].

& Calabresi (1998)] is found to be dominated by a fragile weakening of the clay bonding, which is also sensitive to η . Hence, the ratio $\sigma'_{ijY} / \sigma'_{ije}$ is found to reduce severely with reducing η . In other words, the weakening of the clay bonding occurs much earlier in isotropic compression than in anisotropic compression. Consequently, the clay undergoes important volumetric straining soon after the η reduction to 0, as shown in Figure 23 for Saint Alban and Bothkennar clays. In this case, the drop in stiffness typically characterizing gross yielding, is much smoother in isotropic yielding than in 1D yielding and $p'_{YK0} > p'_{YI}$, $N-N_{K0} < 0$ and $S_\sigma = p'_{YK0}/p'_{eK0} \neq p'_{YI}/p'_{ei}$. Consequently, the shape of the lower portion of the SBS differs from that of the SBS*, as shown for Saint Alban and for Bothkennar clay in Figure 19b and 19c. Nonetheless, while bonding degrades, the clay compressibility in isotropic compression is lower than that exhibited by the clay in a more fragile gross yielding in 1D compression. Consequently, the state path of the clay undergoing isotropic compression crosses that of the clay in 1D compression post-gross yield in e - p' plane (Figure 23) and, once bonding is largely weakened, the INCL becomes

parallel to the K_0 NCL and shifts to the right, acquiring the pattern typical of reconstituted clays. Such behaviour may be envisaged to be due to the increase in orientation that the original random fabric acquires with 1D compression post-gross yield, with respect to the lower orientation fabric of the isotropically compressed clay, in accordance with the framework shown in Figure 20a. Therefore, at relatively high pressure $N_{K0} < N$ also for this type of clay.

Once bonding has been weakened and the degree of fabric orientation has evolved through the different η compressions, the role and limitations of the hardening function $S_\sigma(\epsilon_{vol}^p)p_e^*(\epsilon_{vol}^p)$, discussed above for stiff clays, applies also to soft clays. Nevertheless, the processes taking place at the micro-scale and their macro effects at gross yielding for the natural soft clay differ from those occurring in stiff clays. On the whole, the data suggest that the soft clay response at gross yielding is dominated by the fragile bonding decay, which masks the effects of the L variations between the fabric pre- and post-gross yield. Thereafter, the clay response heads towards that of the reconstituted clay, with a corresponding variation in L controlled by η , although

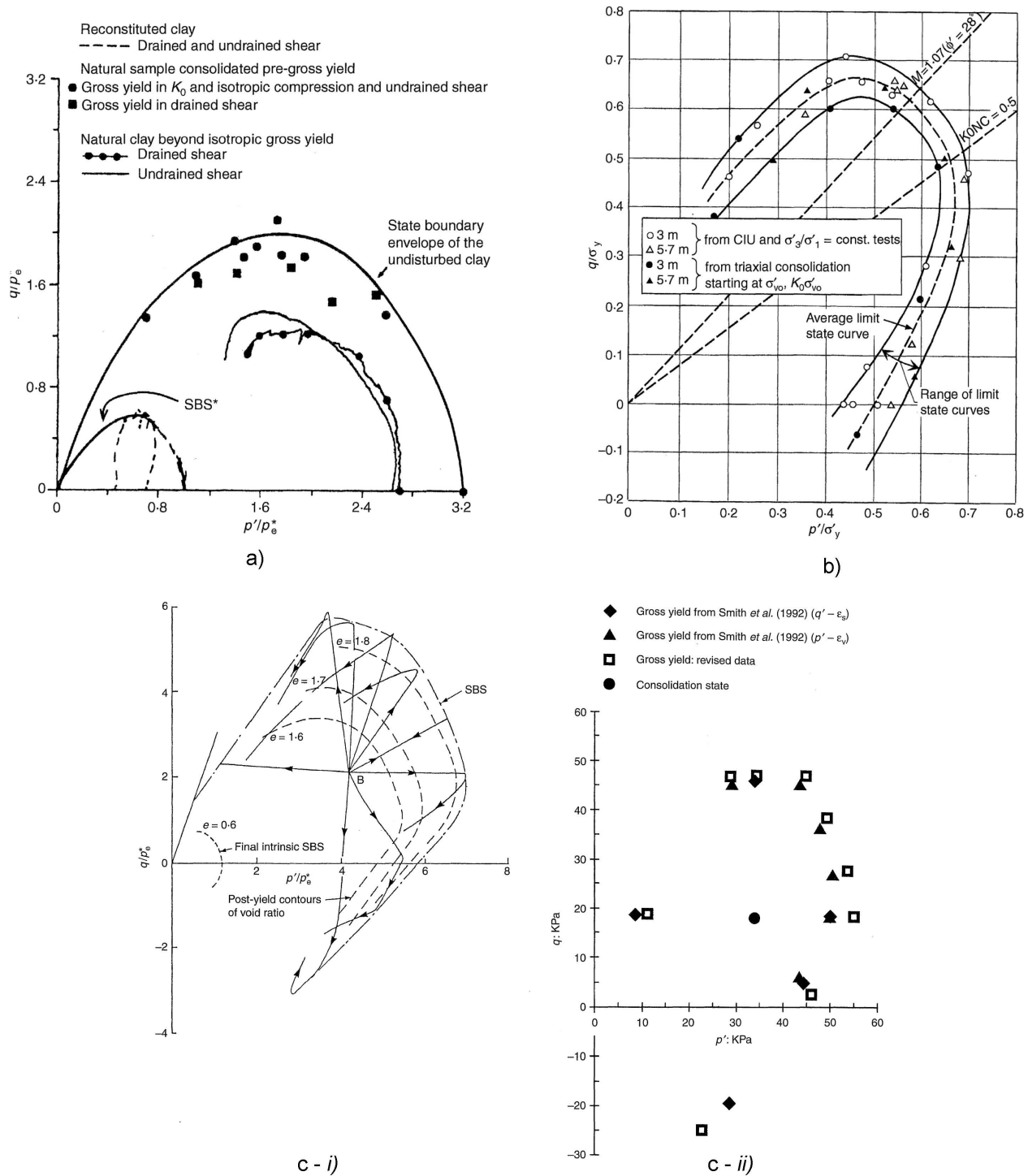


Figure 19. a) Pappadai clay data in the plane of the effective stresses normalized with respect to specific volume by means of p_e^* : stress paths and state boundary surface of the reconstituted clay, SBS*, gross yield states and state boundary envelope of the natural clay consolidated pre-gross-yield and shear paths of the natural clay consolidated beyond isotropic gross-yield [after Cotecchia & Chandler (1997)]; b) Saint Alban clay data in the plane of the effective stresses normalized with respect to specific volume by means of p_y : gross yield data for samples from two depths [from Leroueil *et al.* (1979)]; c) Bothkennar clay data: i): SBS* and results of stress path tests on the natural clay in the plane of the effective stresses normalized by p_e^* [from Smith *et al.* (1992)]; ii) gross yield states along the stress paths in (i) plotted in the effective stress plane [modified after Cotecchia & Chandler (2000)].

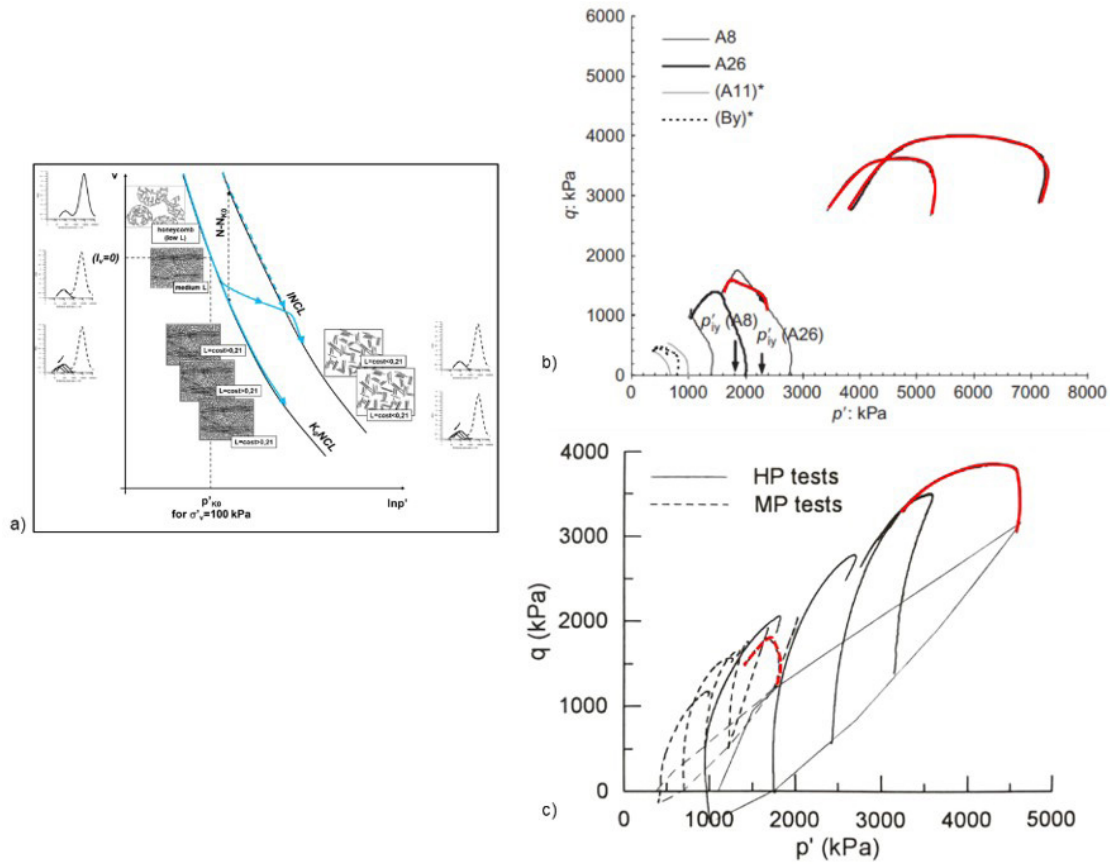


Figure 20. a) Schematic framework of the evolution in micro-fabric applying to either reconstituted (see Figure 13 of part I-paper), or natural clays subjected to compression; b) Montemesola clay: undrained shear paths of natural and reconstituted specimens consolidated before and beyond gross-yield, either isotropically, or anisotropically (these latter are highlighted in red for specimens compressed either to or beyond gross yield); c) Vallericca clay: undrained shear paths of 1D consolidated specimens; the shear paths of specimens sheared after 1D consolidation either to or beyond gross yield are highlighted in red; modified after Amorosi & Rampello (1998).

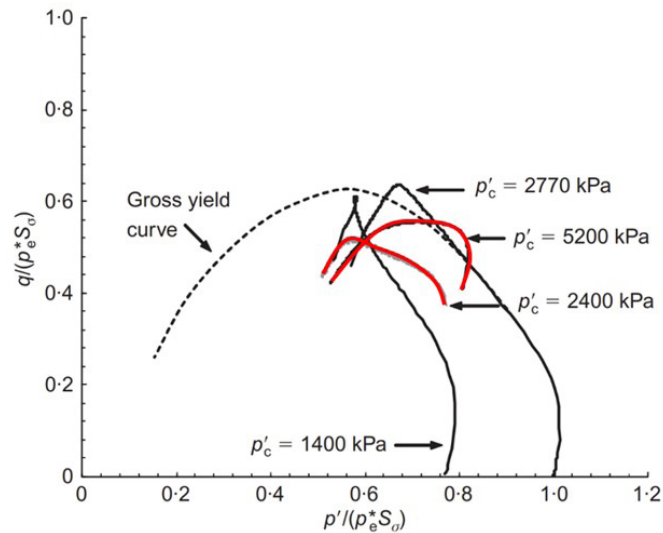


Figure 21. Undrained shear paths of specimens from sample A8 of natural stiff Montemesola clay normalised for volume and structure [modified after Cotecchia et al. (2007)].

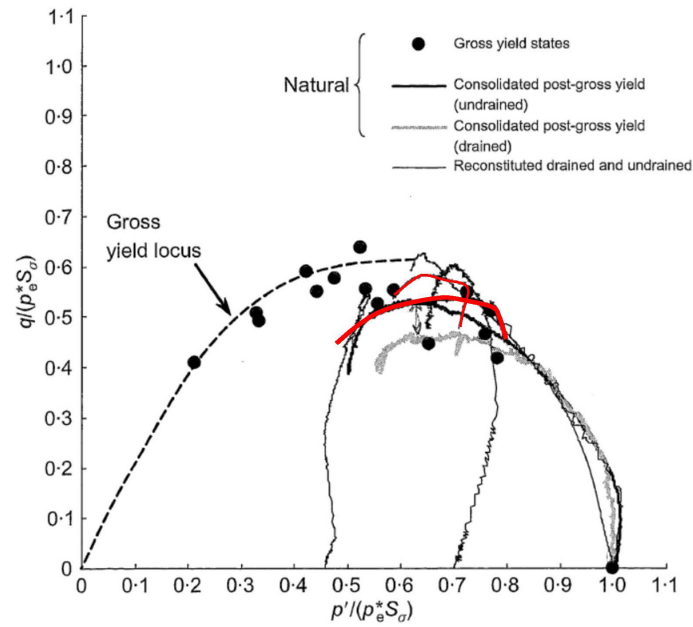


Figure 22. Pappadai clay behaviour normalized for both volume and structure: gross yield data of the natural clay, post-gross yield stress paths of the natural clay (undrained and drained), and stress paths of the reconstituted clay [modified after Cotecchia et al. (2007)]. In red, the shear paths of 1D compressed specimens.

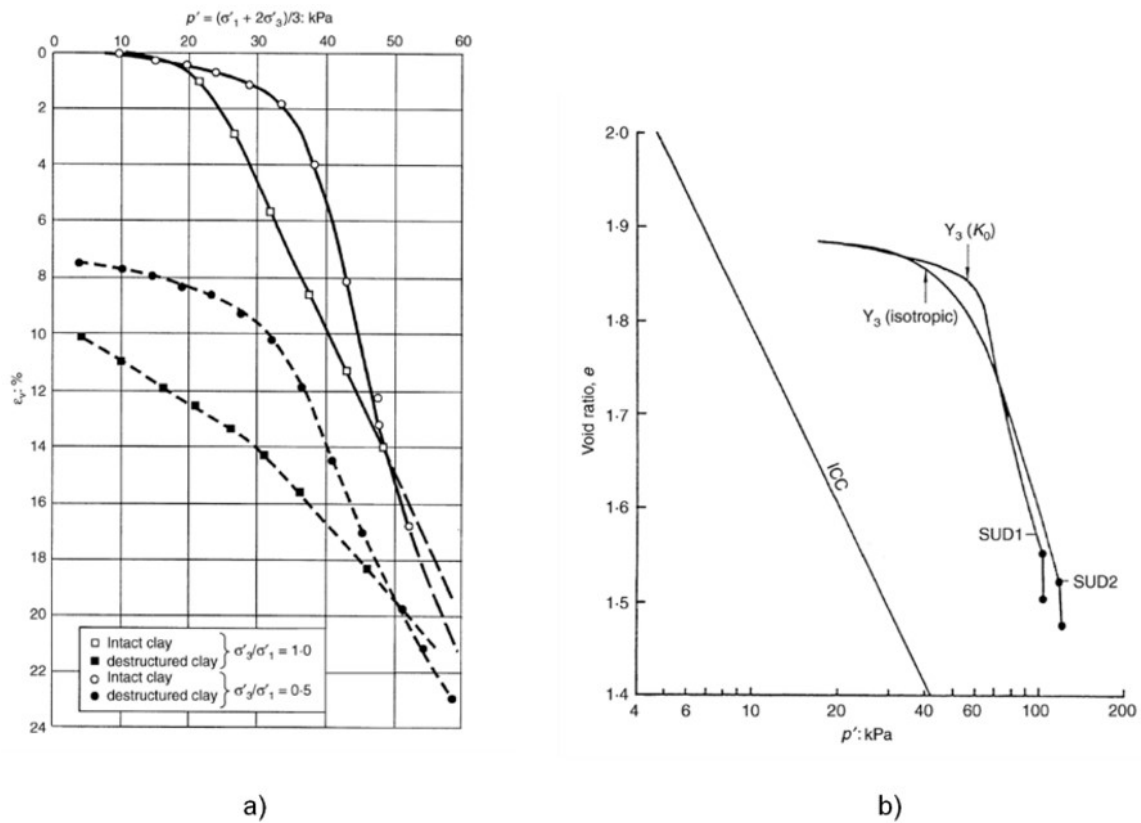


Figure 23. 1D and isotropic compression curves: a) Saint Alban clay (Leroueil et al., 1979); b) Bothkennar clay [ICC = Intrinsic Compression Line; Smith (1992)].

bonding is still different from that of the reconstituted and is reflected by $S_o(\varepsilon_{vol}^p)$.

When the undisturbed sensitive not oriented fabric of the soft clay is subjected to isotropic compression and gross yielding, again its response is dominated by the bonding decay, although it differs from that taking place in 1D compression, being less abrupt. It is evident, then, how this stage of behaviour of soft clays requires a function representative of the η - dependent bonding decay, different from $S_o(\varepsilon_{vol}^p)$.

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

Federica Cotecchia: conceptualization, methodology, validation, writing – reviewing and editing. Simona Guglielmi: conceptualization, investigation, data curation, validation, writing – reviewing & editing. Francesco Cafaro: investigation, validation, reviewing. Antonio Gens: supervision, validation, reviewing & editing.

Data availability

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

List of symbols and abbreviations

e_0	initial void ratio
w_0	initial water content
A	activity index
CF	clay fraction
I_v	void index
LL	liquid limit
OCR	overconsolidation ratio
PI	plasticity index
S_t	strength sensitivity
S_o	stress sensitivity
YSR	yield stress ratio
σ'_{v0}	in situ vertical effective stress

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In-situ and laboratory characterisation of stiff and dense geomaterials for driven pile analysis and design

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Article

Keywords

Laboratory testing
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Dense sand
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Abstract

Integrated field and laboratory characterisation of geomaterial behaviour is critical to foundation analysis and design for a wide range of offshore and onshore infrastructure. Challenges include the need for high-quality sampling, addressing natural and induced micro-to-macro structures, and applying soil and stress states that represent both *in-situ* and *in-service* conditions. This paper draws on the Authors' recent research with stiff glacial till, dense marine sand and low-to-medium density chalk, and focuses particularly on these geomaterials' mechanical behaviour, from small strains to failure, their anisotropy and response to cyclic loading. It considers a range of *in-situ* techniques as well as highly instrumented monotonic and cyclic stress-path triaxial experiments and hollow cylinder apparatus tests. The outcomes are shown to have important implications for the analysis of large driven piles under monotonic-and-cyclic, axial-and-lateral loading, and the development of practical design methods. Also highlighted are the needs for approaches that integrate field observations, advanced sampling and laboratory testing, numerical and theoretical modelling.

1. Introduction

The potential capabilities and practical values of advanced laboratory testing have been demonstrated through ISSMGE TC101's International Symposia (IS) from Hokkaido 1994 to Glasgow 2019. Jardine (2014, 2020, 2023a) highlighted their application with large piles driven to support key offshore and onshore infrastructure, noting that geomaterial behaviour under *in-situ* and *in-service* conditions can be affected markedly by pile installation effects as well as operational and extreme loading. Representative numerical analyses rely critically on experiments that match these conditions faithfully to provide the fundamental physical basis for constitutive model development and calibration (Zdravković et al., 2021).

The worldwide drive for renewable offshore wind energy has prompted large-scale research projects to tackle offshore wind foundation design challenges; those involving advanced soil characterisation include the PISA (Byrne et al., 2017), PICASO (Byrne et al., 2020a), WAS-XL (Page et al., 2021) ALPACA and ALPACA Plus (Jardine et al., 2023b) projects. High-quality pile test databases have also been collated by Yang et al. (2017) and Lehan et al. (2020, 2022).

Establishing the behaviour of dense marine sands, stiff glacial tills, Eocene to Jurassic stiff clays and chalks has been particularly significant in developing large concentrations of

offshore windfarms around the UK and northern European coastlines where such strata are often encountered. This requires: (i) high quality sampling (Hight & Jardine 1993); (ii) addressing both natural geomaterials' properties and potential alterations induced by pile driving and soil-pile interaction; and (iii) applying appropriate effective stress states and perturbing stress paths (Jardine, 2014). This paper draws on contributions made to the PISA, ALPACA and ALPACA Plus Joint Industry Projects (JIPs) by Liu (2018), Ushev (2018) and Vinck (2021). The experiments conducted on the three formations considered below have contributed to making significant improvements in offshore wind turbine foundation design.

- **Cowden clay**, stiff, high Yield Stress Ratio (YSR, or apparent OCR) sandy glacial till sampled at the PISA test site, NE England;
- **Dunkirk sand**, fine marine predominately silica sand from the PISA site in Northern France, consisting of dense to very dense normally consolidated hydraulic fill (average relative density $D_r \approx 100\%$) and Flandrian sand (average $D_r \approx 75\%$);
- **St Nicholas at Wade (SNW) chalk**, low-to-medium density B2/B3 grade Margate and Seaford chalk from the Upper Cretaceous sampled at the ALPACA site in Kent, SE England.

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An overview of the ground conditions at these sites is presented, before considering three topics for each geomaterial:

- Mechanical behaviour from very small to large strains, emphasising non-linearity, anisotropy as well as rate-and-time dependency, interpreted within suitable frameworks;
- Synthesis of *in-situ* and laboratory measurements;
- The geomaterials' response to drained or undrained cyclic loading.

Mention is made briefly of how soil-steel interface behaviour is affected by stress level, time (ageing), displacement, surface roughness, pore water chemistry and other conditions. Attention is also drawn to how field and laboratory testing can be applied in analyses of impact driven piles subjected to general loading.

2. Geotechnical overview

2.1 Site profiles

Cowden site encounters Bolders Bank formation glacial tills, whose deposition involved cycles of ice loading and horizontal shearing and/or glaciotectionic passive pressures (Davies et al., 2011). The till comprises principally stony, sandy and silty clay units that under-drain into two sand layers (Powell & Butcher, 2003; Ushev, 2018; Zdravković et al., 2020a; Ushev & Jardine, 2022a). While a water table is encountered at 1.0 m depth, pore-pressures become under-drained at depth. The till's index properties are summarised in Table 1 and Figure 1. Figure 2a presents typical profiles of corrected Cone Penetration Test (CPTu) resistances (q_t); the occasional spikes indicate hard stone inclusions. Triple-barrel, wireline, Geobore-S rotary coring and block sampling campaigns retrieved a sufficient stock of high-quality intact specimens, although the till's stony nature and 8% gravel content limited the core recovery rate to $\approx 42\%$.

Dunkirk conditions consist of mainly normally consolidated marine Flandrian sands down to the Ypresian Eocene marine clay found at 30 m (Brucy et al., 1991; Chow, 1997; Jardine et al., 2006; Zdravković et al., 2020a). Three metres of hydraulic sand fill of the same origin were placed over the Flandrian sands in the 1970s without compaction or surcharging. Particle size analyses indicate uniform

fine-to-medium sand; see Figure 1. The sub-angular particles comprise $\approx 85\%$ silica, plus calcium carbonate (CaCO_3) shell fragments and other minerals that leave the soil slightly alkaline (Chow, 1997). Table 1 summarises index properties, after Kuwano (1999) and Aghakouchak (2015). The site's q_t profiles are illustrated in Figure 2b. Groundwater was encountered at 5.4 m in 2014, 1.4 m deeper than that found by Chow (1997) with hydrostatic conditions below. While rotary coring was conducted for Brucy et al.'s (1991) investigation, coring for PISA was unsuccessful due to disturbance and drilling fluid contamination. The laboratory testing therefore had to rely on reconstituted bulk samples.

St Nicholas at Wade (SNW) The chalk site was characterised by Diambra et al. (2014) and Ciavaglia et al. (2017) prior to Buckley's (2018) and Buckley et al. (2018) investigations for the Innovate UK project and the exhaustive ALPACA and ALPACA Plus studies (Vinck, 2021; Vinck et al., 2022; Jardine et al., 2023c). Pure (98.6% CaCO_3) white Margate Chalk was found down to the Barrois' Sponge Bed at 5.2 m Above Ordnance Datum (AOD) above the Seaford white Chalk. The chalk classifies as CIRIA Grade B3/B2 (structured, very weak to weak, low-to-medium density) over the depths of interest, with discontinuity apertures < 3 mm and fractures spaced at 60 to 600 mm. Predominantly vertical linear micro-fissures were identified at all depths with ≈ 10 to 25 mm spacings. The ALPACA team located the (fresh) water table ≈ 0.9 m AOD, with $\pm$

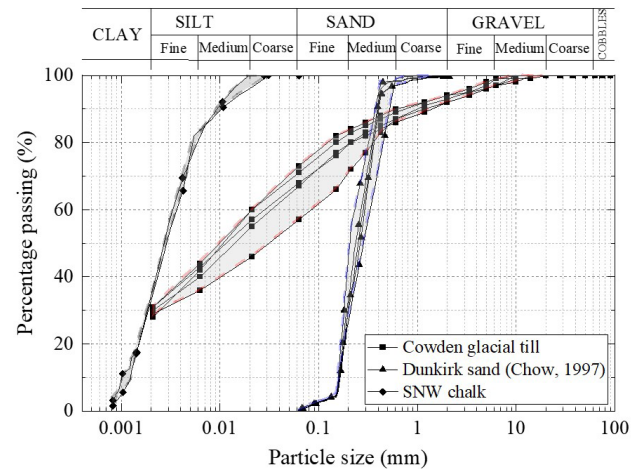


Figure 1. Particle size distributions of Cowden till, Dunkirk sand and SNW chalk over the depths of interest.

Table 1. Typical index properties of the three geomaterials.

Material	D_{50} (mm)	G_s	w_c (%)	w_p (%)	w_L (%)	ρ_{bulk} (g/cm ³)
Cowden glacial till	0.01-0.03	2.707-2.723	16.1-20.2	14.0-23.0	32.0-45.0	1.95-2.25
Dunkirk sand	0.21-0.28	2.65-2.66	5-7 (above WT) 22.7 (Below WT)	NA	NA	1.74 (above WT) 2.03 (below WT)
SNW chalk	0.003	2.71	27.9-33.0	22.3-23.9	30.2-32.0	1.85-1.98

Note: 'NA': Not Applicable.

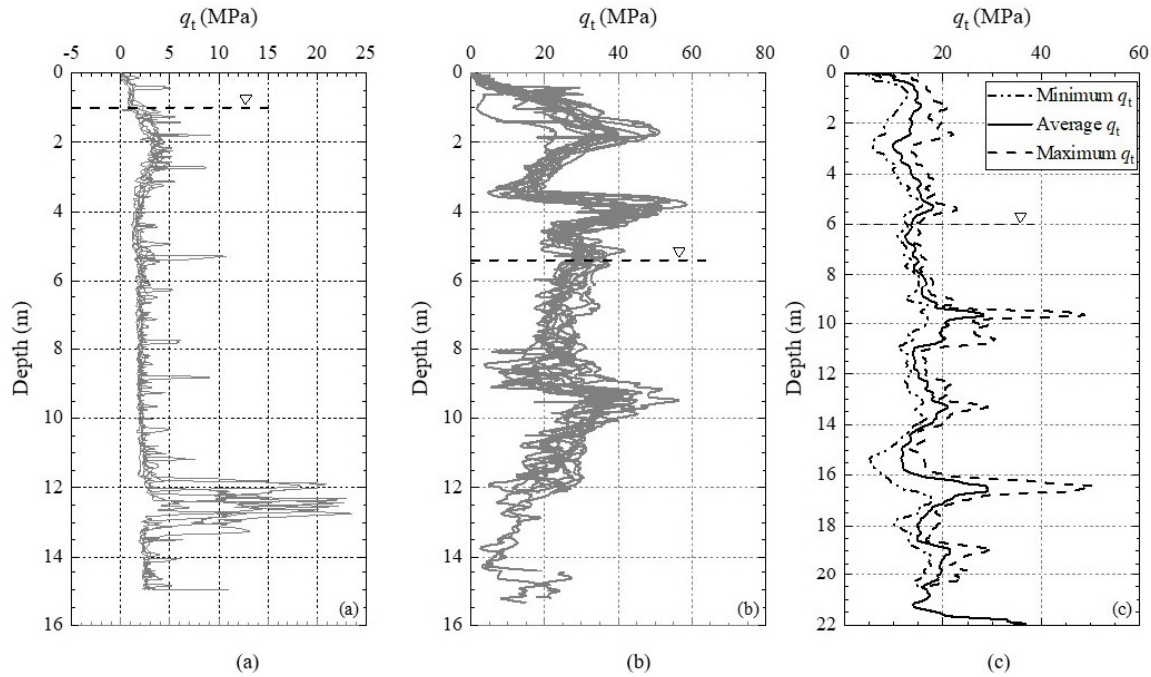


Figure 2. Corrected cone resistances (q_t) profiles for (a) Cowden, (b) Dunkirk and (c) St Nicholas at Wade (SNW) sites.

0.25 m variations. The chalk grains classify as fine silts (D_{50} 3–4 μm) with 1.43 Mg/m^3 to 1.53 Mg/m^3 Intact Dry Densities (IDD) and a 0.91 average liquidity index; see Table 1.

Typical CPTu profiles are presented in Figure 2c, showing ‘spikes’ in thin flint bands. Significant chalk de-structuration beneath and around CPT tips leads to excess pore pressures up to 10 MPa at u_f (face) piezocone positions (Buckley, 2018) and ≈ 4 MPa at u_s (shoulder) locations. Friction sleeve resistances indicate the chalk’s partial de-structuration towards the soft ‘putty’ found around driven pile shafts. CPTu dissipation tests generally showed t_{50} times < 10 s from which Vinck et al. (2022) interpreted *in-situ* consolidation coefficients. Geobore-S rotary coring retrieved samples to ≈ 16 m depth, although the chalk’s fractured and brittle nature limited the solid core recovery rate to 51% and a rock quality designation of 32%. Large block samples were taken from a 4 m deep excavated pit.

2.2 Test equipment

The Imperial College London laboratory testing employed fully instrumented automated Bishop-Wesley type triaxial cells for 38 mm and 100 mm diameter specimens. Liu et al. (2022a) detail the equipment and their stress and strain sensors’ resolutions and precisions. The LVDTs employed to measure local strains offered resolutions $\approx 0.1 \mu\text{m}$. Mid-height sensors tracked local pore water pressures in the Cowden tests and lubricated top and base platens were deployed in most tests

(Vinck et al., 2019). Liu (2018) and Liu et al. (2020) provide full details of the Hollow Cylinder Apparatus (HCA) and Bishop ring shear apparatuses employed in parallel testing campaigns; supplementary experiments were undertaken at other research and commercial laboratories.

3. Cowden till

The PISA Cowden campaign first established profiles of index properties and oedometer test outcomes. Comparisons between the 1-D compression behaviours of natural and reconstituted specimens indicated a dense and insensitive natural fabric. Initial triaxial tests also provided information on the till’s initial linear elastic response through to large-strain critical states that fed into finite element modelling. Liu (2018) and Ushev (2018) extended the scope to investigate the till’s anisotropy, strain rate and pressure dependent monotonic shearing behaviour, as well its undrained cyclic loading response, while Ushev & Jardine (2022a) explored the till’s large-displacement soil-soil and soil-steel interface shearing characteristics.

3.1 Index and mechanical property profiles

The Cowden till comprises predominantly low plasticity (plastic limit ≈ 14 –23%) clay over the depth of interest; the 1.95 Mg/m^3 – 2.25 Mg/m^3 bulk densities reflect its gravel content. Figure 3a plots the 1-D compression yield stresses, σ_{vy} , and Yield Stress Ratio (YSR, or apparent $OCR = \sigma_{vy}/\sigma_{v0}$) profiles from stage-loaded and constant rate of compression

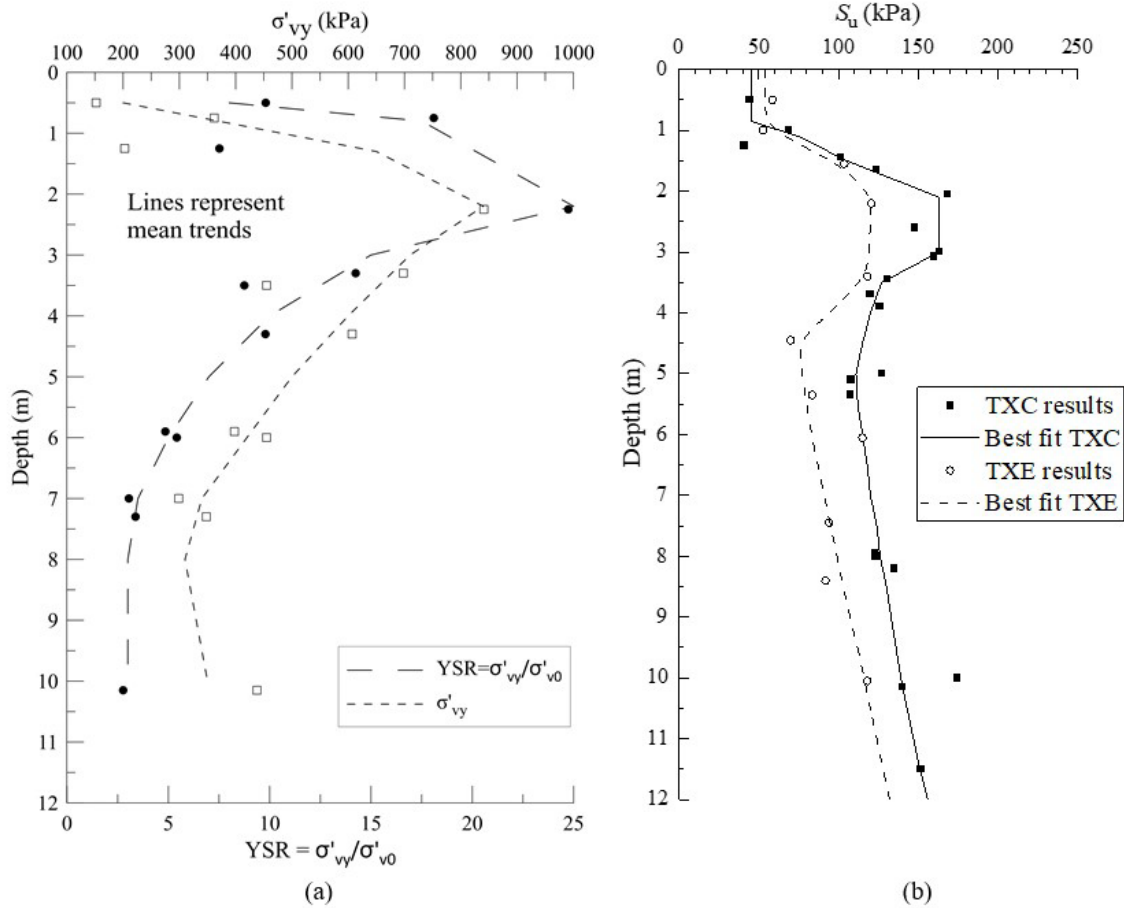


Figure 3. Cowden site profiles of (a) 1-D yield stress σ'_{vy} and YSR and (b) S_u from triaxial compression (TXC) and extension (TXE) tests; all tests performed on high-quality intact specimens (modified from Liu et al., 2020; and Ushev & Jardine, 2022a, b).

oedometer tests. The high YSRs noted over the upper 4 m reflect complex glacial and post-glacial processes. Although Powell & Butcher (2003) interpreted K_0 values exceeding 2.5 from various empirical procedures, Jardine (1985) argued that glacial lodgement till deposition leads to lower K_0 values that are more compatible with the networks of open vertical fissures observed especially within the desiccated and more stony upper sections. Therefore, the laboratory triaxial tests and analyses limited K_0 to 1.5 within the upper 7 m and unity below this depth.

The peak undrained triaxial compression and extension strengths (S_u) of natural specimens re-consolidated to *in-situ* stresses are reported in Figure 3b. The profiles exhibit similar shapes that, like the oedometer σ'_{vy} profile in Figure 3a, differ markedly from the patterns expected in gravitationally consolidated waterborne sediments after monotonic geostatic loading and unloading. The compression profile indicates an average CPT $N_{kt} \approx 18$, while the average S_u^{TXC}/S_u^{TXE} ratio is 1.25. Liu et al. (2020) argue that the latter ratio does not reflect the till's anisotropy because the extension strengths are affected by necking and different Lode angles (or b values) applied in compression and extension. Liu et al. (2020,

2023a) report HCA experiments that investigated the till's anisotropy more rigorously.

3.2 Triaxial monotonic shearing

3.2.1 Stiffness

The triaxial tests addressed behaviour from strains of 10^{-6} up to ultimate failure with deviatoric strains $\epsilon_s > 25\%$. The natural Cowden till only exhibited elastic behaviour under small stress and strain increments, no greater than a few kPa and 0.002% respectively, that remained within the till's Y_1 kinematic yield surface (Jardine, 1992). Undrained vertical Young's moduli established by regression analyses of these initial portions led to the E_{sec}^u profile in Figure 4a along with secant stiffness profiles for axial (or ϵ_s) levels of 0.001%, 0.01%, 0.1% and 1%, which indicate nearly parabolic trends with depth. Analysis confirmed that E^u scales with p' raised to a partial power, as expressed in Equation 1, where p_{ref} is atmospheric pressure. The exponent β can be taken as 0.5 and the a values are largely constant with depth for each strain level;

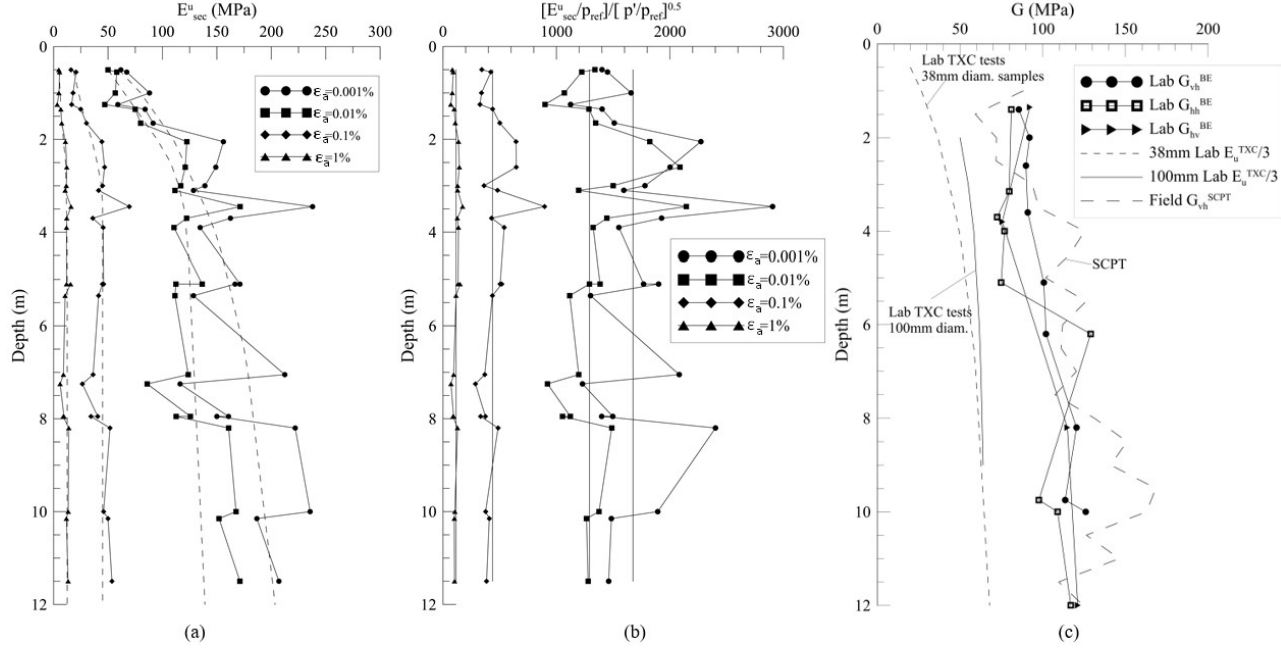


Figure 4. Cowden stiffness profiles of: (a) triaxial compression secant stiffness E^u_{sec} at axial strains of 0.001%, 0.01%, 0.1%, and 1%; (b) normalised triaxial secant stiffness $[E^u_{sec}/p_{ref}]/[p'/p_{ref}]^{0.5}$ at four axial strain levels, applying Equation 1 with $\beta = 0.5$; and (c) elastic shear stiffnesses from laboratory and field geophysical measurements, with $E^u_{TXC}/3$ profile from triaxial compression tests (modified from Ushev & Jardine, 2022a).

see Figure 4b. Similar traces were observed for the interpreted triaxial tangent Young's moduli (E^u_{tan}) (Ushev, 2018).

$$E^u = \alpha p_{ref} \left(\frac{p'}{p_{ref}} \right)^\beta \quad (1)$$

Figure 5 shows the degradation of normalised secant stiffness $[E^u_{sec}/p_{ref}]/[p'/p_{ref}]^{0.5}$ with strain, with degradation being steeper in extension than in compression.

As discussed in greater detail later, Cowden till shows cross-anisotropic stiffnesses. Figure 4c plots laboratory bender element G_{vh} , G_{hv} and G_{hh} profiles for intact specimens along with in-situ seismic CPT (SCPT) G_{vh} data from Zdravković et al. (2020a). The laboratory and SCPT G_{vh} traces plot relatively close together, although the field values are (on average) slightly higher. Also shown in Figure 4c is the equivalent isotropic octahedral shear moduli G_{oct} taken as $E^u_{TXC}/3$ which falls far below the bender element G_{vh} and G_{hh} trends because of the till's anisotropy.

3.2.2 Large-strain yielding and rate-dependent behaviour

Cowden till exhibits ductile, strain hardening, triaxial compression behaviour. Stable failure states were reached after imposing large strains, with the specimens' barrelling without any evident shear banding. These points were interpreted as critical states with $M^{TXC} = q/p'$

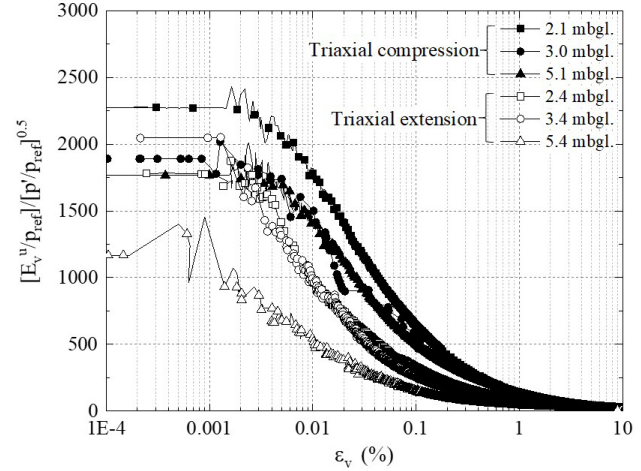


Figure 5. Typical degradation trends for normalised vertical secant undrained Young's moduli from anisotropically consolidated triaxial compression and extension tests on intact Cowden till from upper layers (modified from Liu et al., 2020).

around 1.0, equivalent to $\phi'_{cs} = 25.4^\circ$, $c' = 0$ and applied in constitutive modelling of the till. Figure 6 shows typical undrained effective stress paths from compression (TXC) and extension (TXE) tests in q - p' ($q = (\sigma'_v - \sigma'_h)$, $p' = (\sigma'_v + 2\sigma'_h)/3$) coordinates. Also indicated are large-scale 'Y₃ yield points' interpreted as the points (at $q \approx 100$ kPa) where the effective stress paths rotated from their initial negative $dp'/$

dq ratios (that reflect the till's elastic stiffness anisotropy) to follow paths with positive, dilative, dp'/dq directions. While the TXC tests progressed to stable critical states, all TXE specimens developed necking failures which lead to apparently low peak shear strengths and post-peak softening. A notional Hvorslev surface is shown that runs from the final TXC critical state point back to the no-tension cut-off and encompasses the large strain undrained effective stress paths. Similar critical states and Hvorslev surfaces were assumed to apply in extension.

Noting that rapid lateral pile loading tests at Cowden delivered markedly higher peak loads and stiffness than slower stage-loaded experiments (Byrne et al., 2020b), Ushev (2018) investigated the till's rate-dependency. Triaxial tests run at axial strain rates of 5%/day, 50%/day and 500%/day showed S_u and secant stiffness increasing by $\approx 6\%$ and 10% respectively per tenfold rate change. Compatible rate trends and reversible isotach behaviour were observed in other tests that alternated between 5% and 500%/day shearing rates.

3.3 Anisotropic stiffness and shear strength characteristics

Differences observed in the till's triaxial compression and extension shear strengths and the triaxial and field stiffness profiles in Figures 3 and 4 prompted Liu et al. (2020) to investigate the till's stiffness anisotropy. Suites of triaxial probing tests on samples from 4 depths applied drained vertical ($d\sigma_v'$), horizontal ($d\sigma_h'$) and undrained vertical (dq) stress increments (of typically 2 kPa) at rates of 0.1-0.2 kPa/hour

established the till's stiffness anisotropy. A suite of undrained HCA ' α_{dr} -controlled' experiments that achieved five final σ_f axis orientations with respect to the vertical (α_f values) spaced equally between 0 and 90° . HCA simple shear (HCASS) and HCA triaxial compression tests (HCA TXC) performed on block sample specimens from 2.9 m established in this way the till's anisotropy up to shear failure. The α_{dr} -controlled and HCASS tests were controlled to keep the intermediate principal stress parameter (b) close to 0.5.

3.3.1 Cross-anisotropic elastic stiffnesses

Treating the till's linear elastic behaviour as cross-anisotropic and strain-rate independent allows dynamic and static measurements to be considered as compatible and inter-comparable. The cross-anisotropic stiffness matrix has five independent parameters, E_v' , E_h' , ν_{hv}' (or ν_{vh}), G_{vh} , G_{hh} (or ν_{hh}') that can be determined experimentally (see for example Kuwano & Jardine, 2002). Three independent parameters, E_v^u , E_h^u and G_{vh} apply under undrained conditions that can be established from the drained set (see Brosse et al., 2017) or the combined drained and undrained probing approach proposed by Nishimura (2014) that places emphasis on the higher quality axial strain measurements.

The profiles with depth of G_{hh}/G_{vh} , E_h'/E_v' and E_h^u/E_v^u derived from Liu et al.'s (2020) triaxial and HCA tests plotted in Figure 7 show a generally consistent trend for a stiffer response under horizontal than vertical loading, which is

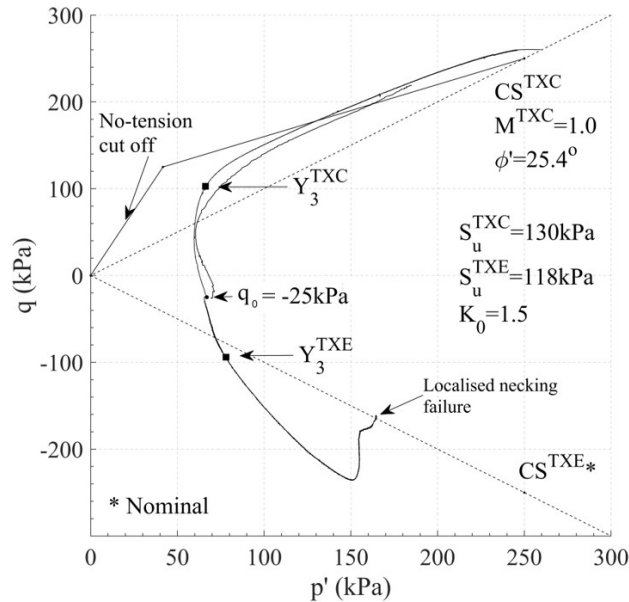


Figure 6. Typical undrained triaxial compression and extension effective stress paths for Cowden till samples from 3.0 to 3.5 m depth (Axial shear strain rate: 5%/day) (modified from Ushev & Jardine, 2022b).

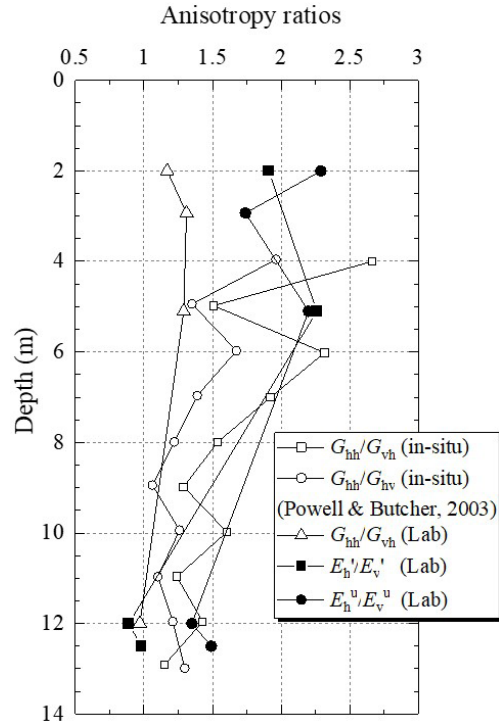


Figure 7. Anisotropic stiffness ratio profiles for Cowden site (modified from Liu et al., 2020).

important to recognise when analysing lateral pile loading. Also shown are G_{hh}/G_{vh} and G_{hh}/G_{hv} ratios from in-situ down-hole and cross-hole shear velocity tests by Powell & Butcher (2003); stiffness anisotropy clearly diminishes with depth.

3.3.2 Non-linear stiffness characteristics over full strain range

Following Brosse et al. (2017), the α_{ds} -controlled Cowden HCA tests allowed undrained vertical and horizontal Young's moduli and torsional shear stiffness to be tracked over a wide strain range. The till's marked elastic-plastic stiffness degradation under HCA conditions is demonstrated in Figure 8 by plotting E_v^u and E_h^u against vertical strain ε_z and average horizontal strain $(\varepsilon_r + \varepsilon_\theta)/2$, respectively. Despite showing initial scatter, the moduli exhibit consistent degradation trends. Narrow spreads apply to each component, suggesting that the cross-anisotropic stiffness components are relatively insensitive to the shearing path followed. The two HCA triaxial tests, which employed pre-shearing $b = 1.0$ and ultimate $b_u = 0$, showed overlapping E_v^u trends that fall slightly below those of most $b = 0.5$ tests. Overall, the E_h^u curves plot above the E_v^u data trends, confirming that stiffness anisotropy persists as strains grow.

When taken to failure, under nominally plane strain conditions, in HCA tests, the highest shear strengths developed when σ_f acted horizontally ($b = 0.5$, $\alpha_f = 90^\circ$) and the lowest when σ_f acted vertically; the shear strengths specimens applying at intermediate α_f values spanned these limits, confirming that any S_u anisotropy inferred from comparing triaxial compression and extension strengths (see Figure 3b) would be misleading.

3.4 Response to undrained cyclic loading

Cyclic loading, which can be critical for offshore structures, was addressed in the PISA field pile testing. Ushev (2018) and Ushev & Jardine (2022b) supported this work by investigating Cowden till's response to undrained triaxial cyclic loading, considering multiple nominally identical samples from 3.4 mbgl re-consolidated to in-situ stresses $q_0 = -25$ kPa, $p_0' = 67$ kPa. Mean (q_{mean}) and cyclic (q_{cyc}) deviatoric stresses, in the -25 to 200 kPa and 10 to 150 kPa respective ranges, were applied to populate the q_{mean} - q_{cyc} interactive loading plane and investigate how the combinations affected the evolving trends for effective stress, cyclic strain and stiffness, damping ratios and any cyclic failures.

3.4.1 Number of cycles to failure and cyclic interaction diagram

The influences of the mean q_{mean} and cyclic q_{cyc} deviatoric stresses were considered in both (i) the conventional total stress $q_{cyc}/(2S_u)$ and $q_{mean}/(2S_u)$ interactive loading space and (ii) alternative 'effective-stress' normalised $q_{cyc}/p'_{in-situ}$ and

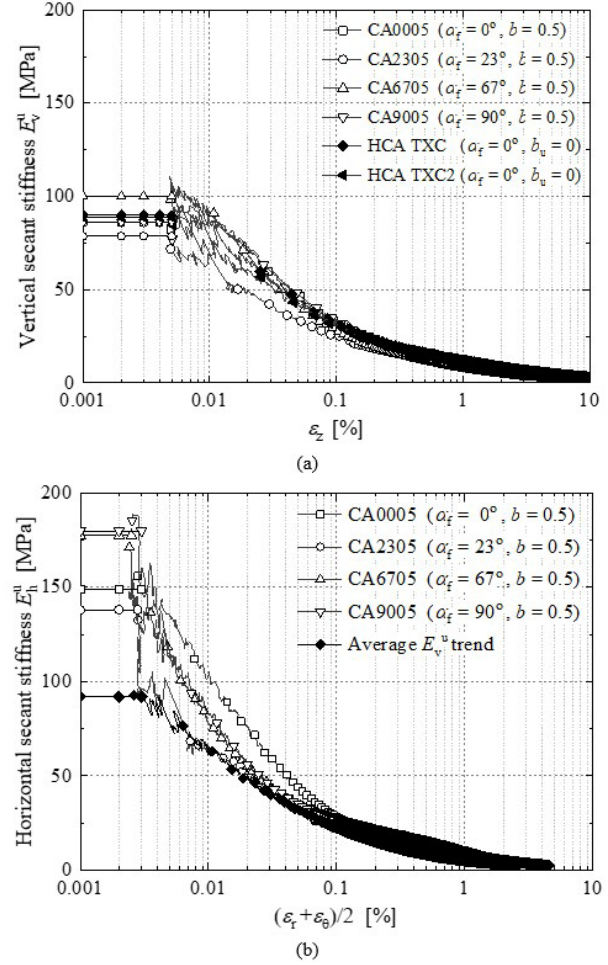


Figure 8. Cowden stiffness degradation from HCA tests: (a) undrained vertical secant Young's moduli E_v^u against vertical (axial) strain; (b) undrained horizontal secant Young's moduli E_h^u against mean horizontal strain (modified from Liu et al., 2020).

$q_{mean}/p'_{in-situ}$ space, as shown in Figure 9. Five of the higher amplitude cyclic tests failed after the indicated number of cycles (N_f). All other specimens remained un-failed when loading halted after 1500 to 3500 cycles. Interpreted contours are shown of the conditions under which failure can be expected at $N_f = 10$, 100 or 1000. A stable zone is indicated where cyclic failure is not expected within 1000 cycles, and regions where failures occurred by either (i) large cyclic and average strains that led to "abrupt failure" or (ii) mean strains accumulating and samples "creeping towards failure".

3.4.2 Evolution of effective stress, cyclic strain and stiffness

Cowden till's cyclic effective stress path and overall cyclic loading behaviour varied systematically with the normalised loading parameters. Typical 'stable' and 'unstable' responses are illustrated in Figure 10 for specimens cycled

under $q_{cyc} = 25$ kPa and a single test involving $q_{cyc} = 150$ kPa from ‘in-situ’ $q_{mean} = -25$ kPa. Also shown are the no-tension lines, the Hvorslev surfaces and M lines identified from the static tests. Figure 10a illustrates the stable outcomes of experiments involving $q_{cyc} = 25$ kPa ($q_{cyc}/(2S_u) = 0.1$). Shifts can be seen from an initially “dilative” drift trend to one indicating slightly “contractive” behaviour. In contrast, the higher amplitude $q_{cyc} = 100$ and 150 kPa ($q_{cyc}/(2S_u) = 0.4$ and 0.6) experiments conducted from $q_{mean} = -25, 50$ and 125 kPa all initially violated the ‘slow shearing’ Hvorslev surface

and subsequently progressed to fail within 1000 cycles. One example is given in Figure 10b. The final stages of ‘unstable’ experiments could involve axial strain rates up to 200 times higher than the reference monotonic experiments, explaining why they were able to sustain transient conditions that could not be sustained under slow monotonic loading. The specimens’ p' values fell more significantly on unloading than they rose during loading, leading to an overall contractive trend with the paths heading leftward towards the origin, developing irregular loops and broad “butterfly-wing” shapes.

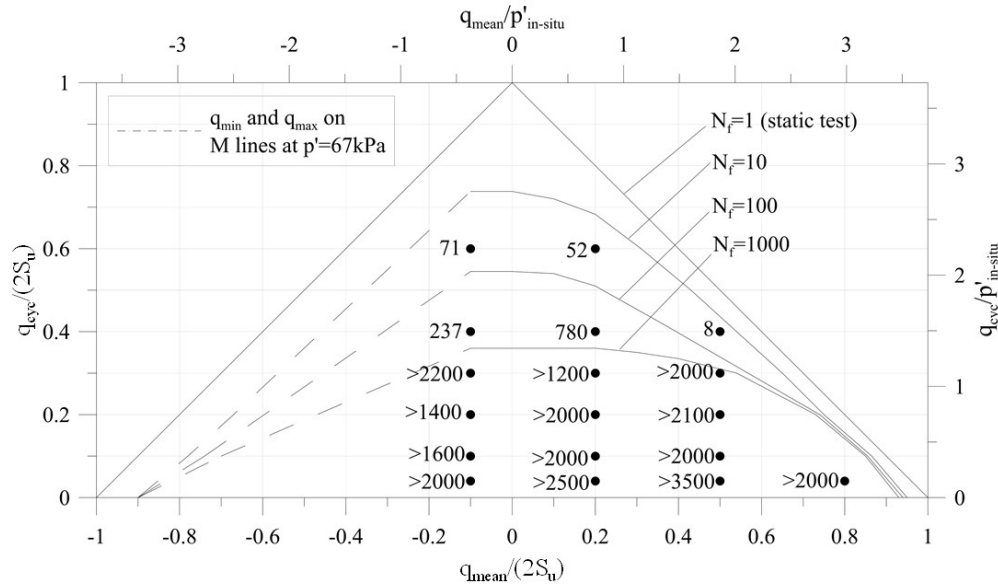


Figure 9. Contours of number of cycles to failure for intact Cowden till in normalised stress space (modified from Ushev & Jardine, 2022b).

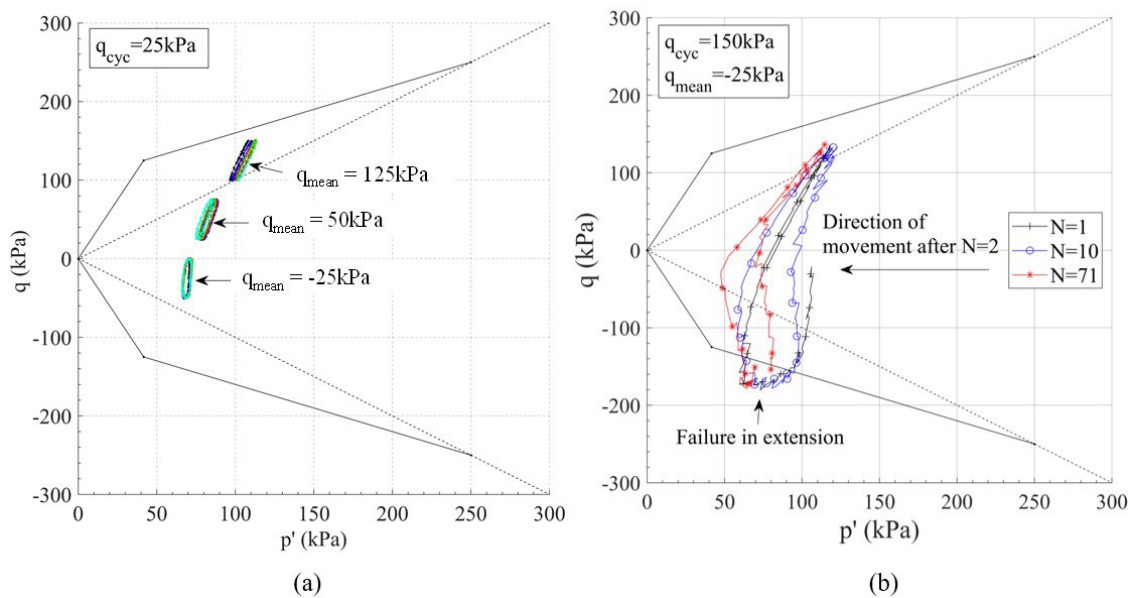


Figure 10. Effective stress paths of Cowden till under undrained cyclic triaxial shearing: (a) typical stable response under low amplitude cycling; (b) unstable cycling response under high q_{cyc} level of 150 kPa (Ushev & Jardine, 2022b; Liu et al., 2023a).

Typical patterns of permanent axial strain (ϵ_{mean}) accumulation with N are plotted in Figure 11a on logarithmic scales. The strains developed under the lowest (10 kPa) cyclic amplitudes tended to stabilise after 100 to 1000 cycles to rates comparable to the slow residual creep rates ($< 0.005\%/day$) applying before cycling started. Empirical power law relationships were found that relate the strain development of stable and metastable tests to their normalised cyclic loading parameters. Figure 11b presents the cyclic peak-to-trough secant stiffness trends for tests performed at in-situ q_{mean} . The tests that manifested stable effective stress loops showed stiffness either growing slightly with N or falling modestly before stabilising; unstable cycling response led to cyclic stiffness falling sharply with N .

3.4.3 Cyclic kinematic yield surfaces

The cyclic responses observed in the stable and metastable tests can be related to two kinematic yield surfaces that, once engaged, move with the current

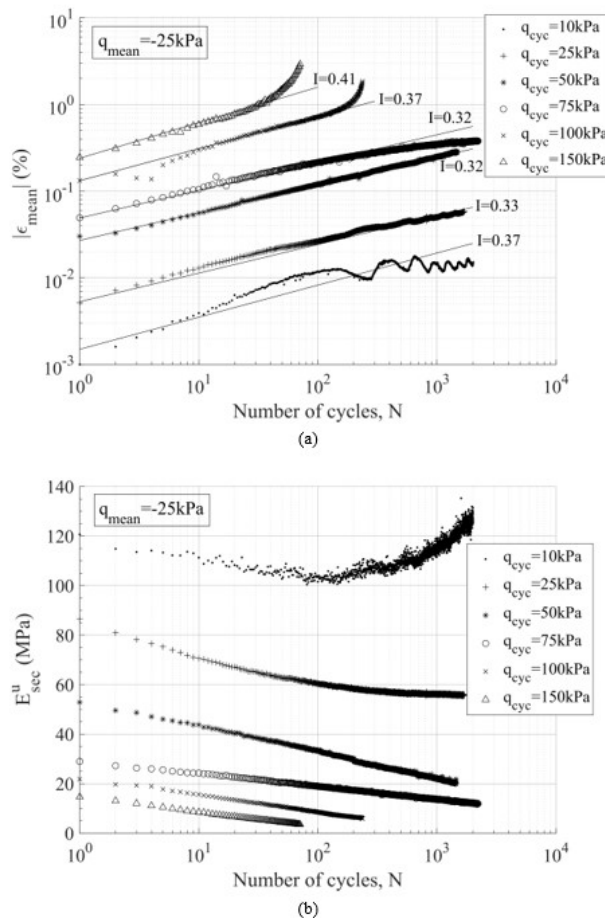


Figure 11. Evolution trends for (a) mean (permanent) axial strain and (b) cyclic secant stiffness against number of cycles for intact Cowden till specimen cycled from in-situ mean $q_{mean} = -25$ kPa (Ushev & Jardine, 2022b, Liu et al., 2023a).

effective stress point; Jardine (1992). The first elastic, Y_1 , region is approximately elliptical in shape and has a maximum height of only about ± 2.5 kPa in-situ for the cyclic series' specimens. The second Y_2 kinematic yield surface (KYS), defined here as the threshold limit to: (i) stable effective stress conditions that involve less than 5% variation in p' over 2000 ± 500 cycles and (ii) practically constant cyclic stiffness. Figure 12 marks the approximate boundaries of the Y_1 (red circles) and Y_2 (black ellipses) surfaces that surround each q_{mean} point. The Y_2 surfaces' vertical dimensions diminished as the pre-loading effective conditions approached the Hvorslev surface, even though p' increased as the samples moved along this undrained loading path. While cyclic loading within the initial Y_2 kinematic yield surface does not appear deleterious, paths that engage Y_2 on each loading and unloading stages dissipate far more energy and provoke marked stiffness degradation and strain accumulation.

4. Dunkirk sand

The Dunkirk PISA triaxial testing focused initially on calibrating the Taborda et al. (2014) state parameter-based bounding surface plasticity model employed to design and model lateral pile tests. The research scope extended later to consider stiffness anisotropy and long-term drained cyclic loading behaviour, again aiming to support cyclic pile tests. Liu (2018) and Liu et al. (2019a, b) report on the sand's interface shearing behaviour.

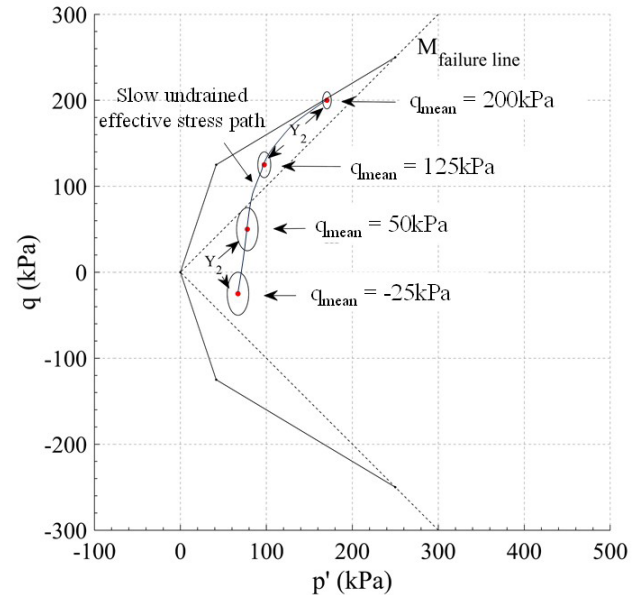


Figure 12. Y_2 kinematic yield surfaces for Cowden till interpreted from cyclic undrained triaxial tests at four mean deviatoric stress (q_{mean}) points along a slow compression test path to failure (modified from Ushev & Jardine, 2022a, b).

4.1 Site stiffness profiles

Extensive field SCPT and laboratory bender element and small-stress probing tests led to the stiffness profiles plotted in Figure 13. The 2016 PISA SCPT tests indicated higher G_{vh} profiles than those reported by Chow (1997) at a location 100 m distant, potentially reflecting local ground variability and/or the effects of ageing over ≈ 20 years under the hydraulic fill (Zdravković et al., 2020a). Drained triaxial tests indicated consistently higher vertical (E_v') than horizontal (E_h') moduli, reflecting the applied $K_0 = 0.4$, although the G_{vh} and G_{hh} components showed less difference. The *in-situ* SCPT G_{vh} profiles plotted distinctly above laboratory bender element (BE) values, especially above the water table, which was ascribed to long-term ageing, partial saturation *in-situ* and seasonal water table fluctuations. Checking for, and addressing, such discrepancies is crucial to successful practical modelling of laterally loaded monopiles.

4.2 Full strain mechanical behaviour under monotonic loading

4.2.1 Linear elastic stiffness anisotropy

Liu (2018) undertook stress-path probing tests on fully instrumented 100 mm diameter specimens that were consolidated and swelled along five constant $K (= \sigma_h'/\sigma_v')$ stress ratio paths.

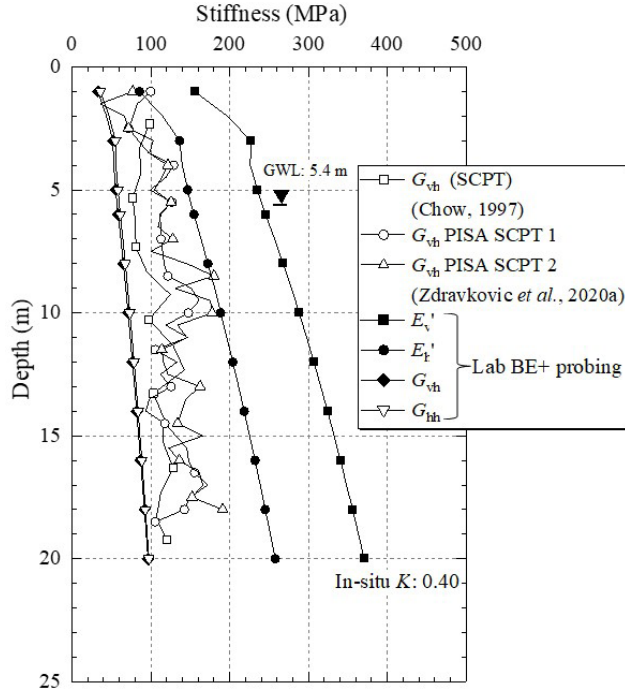


Figure 13. Dunkirk stiffness profiles from *in-situ* SCPT, laboratory bender element (BE) and triaxial small-stress probing tests (Liu, 2018).

The following functions were fitted to correlate the cross-anisotropic stiffnesses with the principal stresses σ_v' and σ_h' .

$$E_v' = f(e)C_v \left(\frac{\sigma_v'}{p_{ref}} \right)^{a_v} \quad (2)$$

$$E_h' = f(e)C_h \left(\frac{\sigma_h'}{p_{ref}} \right)^{b_h} \quad (3)$$

$$G_{vh} = f(e)C_{vh} \left(\frac{\sigma_v'}{p_{ref}} \right)^{a_{vh}} \left(\frac{\sigma_h'}{p_{ref}} \right)^{b_{vh}} \quad (4)$$

$$G_{hh} = f(e)C_{hh} \left(\frac{\sigma_v'}{p_{ref}} \right)^{a_{hh}} \left(\frac{\sigma_h'}{p_{ref}} \right)^{b_{hh}} \quad (5)$$

where C_v , C_h , C_{vh} , C_{hh} , a_v , b_h , a_{vh} , b_{vh} , a_{hh} and b_{hh} are material coefficients and $f(e)$ is taken as $(2.17-e)^2/(1+e)$.

Table 2 summarises the cross-anisotropic stiffness coefficients determined from Liu's (2018) study and Kuwano's (1999) earlier Dunkirk sand probing tests. C_h/C_v and C_{hh}/C_{vh} were invariably greater than unity, indicating inherent anisotropy induced by grain orientation during water (or air) pluviation. Although the simple treatment generally works well, the experiments indicated coefficients that varied moderately with K with C_v values from $K = 0.5$ tests around 15% above those from $K = 1$ experiments; see Figure 14.

4.2.2 Non-linear stiffness and yielding

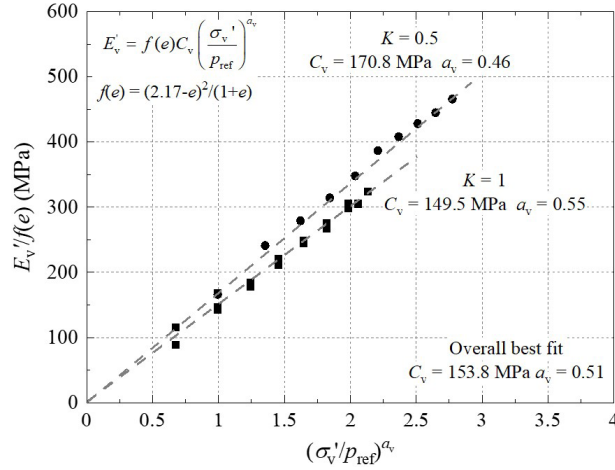
Vinck (2016) and Liu (2018) investigated Dunkirk sand's behaviour from its linear elastic range through to critical state conditions in over sixty locally instrumented tests, varying D_r , effective stress level (p_0') and OCR as well as platen configurations. Figure 15 illustrates the effects of OCR on $E_{v,tan}'/f(e)$ through samples tested at $p_0' = 50$ and 150 kPa after unloading to a range of OCR s. While over-consolidation affected the initial $E_{v,tan}'$ values by only 5%, it extended the initial linear tangent stiffness plateau through changes in the granular contacts and micro-structure that were accompanied by minimal volume changes and so cannot be captured by, for example, applying a simple $f(e)$ function.

Rowe & Barden (1964), Bishop & Green (1965) and Koseki (2021) identified how end friction affects large-strain triaxial behaviour and shear strength of sands. However, its impact on small-strain behaviour was only recently investigated experimentally (with peat) by Muraro & Jommi (2019) and numerically by Cui et al. (2021), considering stiff, high OCR , London clay. Liu et al. (2022a) explored the impact of platen roughness on the pre-failure stiffness of siliceous sand and Figure 16 demonstrates how end constraint affects Dunkirk sand's stiffness and degradation trends even

Table 2. Cross-anisotropy stiffness coefficients of Dunkirk sand.

Sand batch	D_R (%)	Method	K	Stiffness parameters				
Batch 1 (5.85-11.9 m bgl.)	57-65	AP	1, 0.53, 0.35	C_v : 225 0.55	a_v : C_h : 228 0.50	b_h : C_{vh} : 70 0.20	a_{vh} : b_{hh} : 105 0.46	C_h/C_v : 1.01 C_{hh}/C_{vh} : 1.5
Batch 3 (0-0.8 m bgl.)	73-92	WP	1, 0.5	C_v : 154 0.51	a_v : C_h : 166 0.50	b_h : C_{vh} : 49 0.19	a_{vh} : b_{hh} : 63 0.42	C_h/C_v : 1.08 C_{hh}/C_{vh} : 1.29

Notes: (1) Batch 1 data from Kuwano (1999); (2) Sample preparation method: AP = air pluviation; WP = water pluviation; (3) parameters C_v , C_h , C_{vh} and C_{hh} in MPa; (4) All tests performed with lubricated platens.

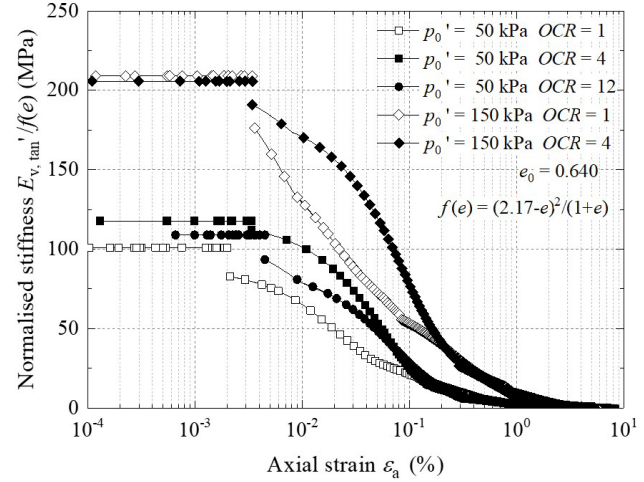
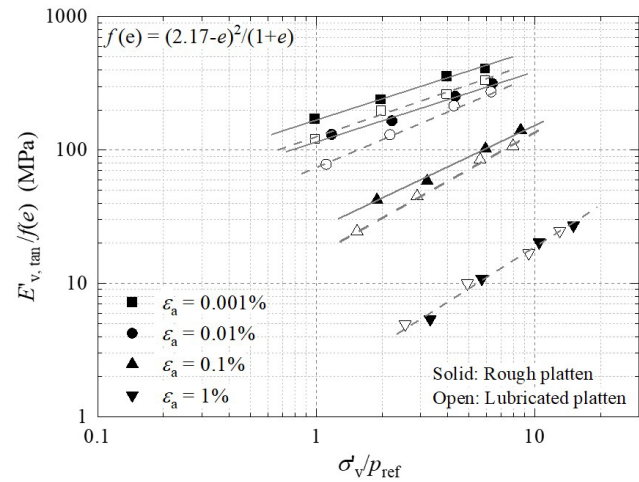
**Figure 14.** Correlations between $E_v'/f(e)$ and σ_v'/p_{ref} and the stiffness coefficients for the $K = 1$ and 0.5 small-stress probing tests on Dunkirk sand (Liu, 2018).

when specimens had 2:1 initial height-to-diameter ratios and strains were gauged locally over the central ≈ 50 mm height. Figure 16 plots $E_{v,tan}'/f(e)$ against σ_v'/p_{ref} for medium dense specimens, considering axial strains of 0.001%, 0.01%, 0.1% and 1%. Medium density specimens tested at relatively low stress levels were most heavily affected by sample end constraints at low to modest strain levels.

4.2.3 Large-strain and critical state behaviour

End constraint also influences medium-to-large strain behaviour. Full end constraint leads to misleadingly high peak stress ratios and shear strengths, at apparently smaller strains, and ultimate stress ratios that did not tend to converge to a unique M with medium dense specimens. Deformation was more uniform in tests employing enlarged lubricated platens, although bulging about the vertical axis was observed with these specimens, but without visible bifurcation even with very dense specimens.

Liu (2018) interpreted Dunkirk sand's large strain and ultimate shearing behaviour within the state parameter critical state framework, providing the basis for bounding surface plasticity (Taborda et al., 2014) and Nor-Sand (Jefferies & Been, 2015) modelling. Taborda et al. (2020), Castillo-

**Figure 15.** Effects of effective stress level and history on the pre-failure small strain stiffnesses of Dunkirk sand (Liu, 2018).**Figure 16.** Effects of platen conditions on the pre-failure small strain stiffnesses of medium dense ($D_R \approx 53\%$) Dunkirk sand specimens (Liu, 2018; Liu et al., 2022a).

Fuentes et al. (2023) and Selby et al. (2023) demonstrated these models' application in the simulations of the PISA Dunkirk pile tests.

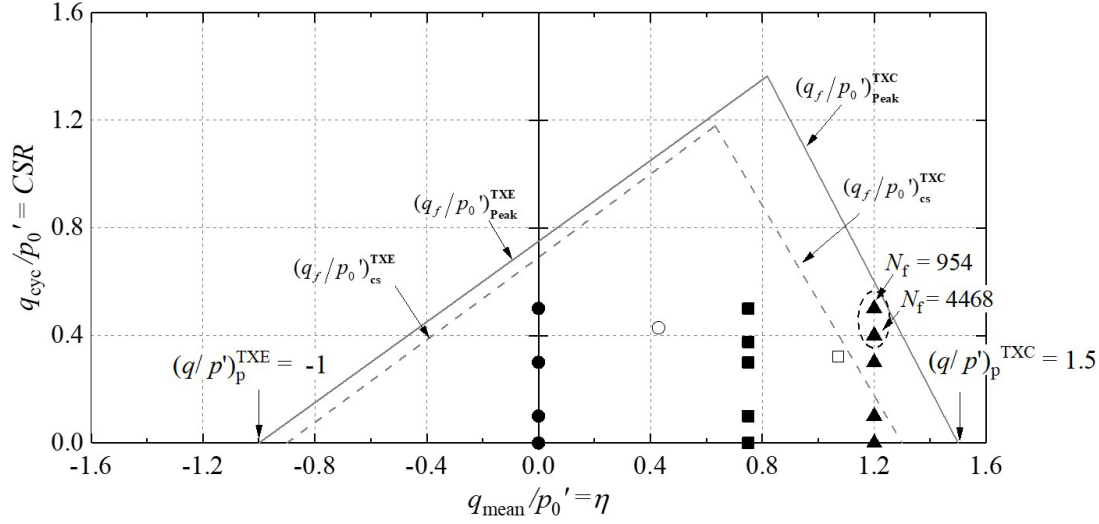


Figure 17. Normalised $q_{cyc}/p'_0 - q_{mean}/p'_0$ ($CSR-\eta$) stress states with reference to the peak and critical state envelopes of Dunkirk sand (Note $(q/p')^{T_{XC}} = 1.5$, $M_{cs}^{T_{XC}} = 1.31$, $(q/p')^{T_{XE}} = -1$ and $M_{cs}^{T_{XE}} = -0.9$) (Liu, 2018).

4.3 Long-term drained cyclic loading behaviour

Aghakouchak (2015) and Aghakouchak et al. (2015) conducted cyclic triaxial and hollow cylinder (HCA) tests to model axial cyclic pile loading and showed how to cater for the ‘near-shaft’ conditions, stress and loading histories that govern field behaviour. Lateral cyclic pile loading engages larger volumes in the passive, active and side shear reaction regions located around the pile circumference. Predominantly drained conditions applied in the PISA Dunkirk tests; conditions are expected to be substantially drained under long-term cyclic loading around even larger monopiles.

Liu’s (2018) cyclic triaxial testing programme for Dunkirk therefore focussed on long-term drained constant σ'_r experiments, aiming to produce a benchmark dataset against which cyclic constitutive models could be validated, calibrated, and applied in boundary value problems. The tests applied the cyclic and mean deviatoric stress combinations (q_{cyc}, q_{mean}) as defined in $q_{cyc}/p'_0 - q_{mean}/p'_0$ (or $CSR-\eta$) space in Figure 17. Specimens were consolidated to, and allowed to creep at, initial (q_{mean}, p'_0) states before constant amplitude cycling to target conditions with $N > 10^4$. All but two specimens survived cycling and were subsequently sheared monotonically to failure to assess how cycling had modified their behaviour. Figure 18 illustrates the $q-\epsilon_a$ trends shown for a typical test on a very dense ($D_R = 92\%$) specimen that started from $(q_{mean}, p'_0) = (100, 133.3)$ (kPa) with $q_{cyc} = 40$ kPa ($CSR = 0.3$). The local axial strains recorded with high resolution sensors manifested the accumulation trends plotted logarithmically in Figure 19, considering each of the 10^4 cycles. Also annotated are power law equations that fitted the experimental trends. Liu (2018) further related the fitting parameters to the $q_{cyc}/p'_0 - q_{mean}/p'_0$ loading conditions and showed how to

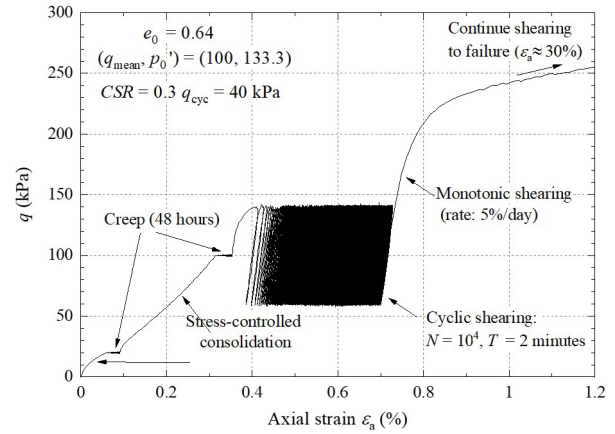


Figure 18. Deviatoric stress strain ($q-\epsilon_a$) trends for a typical long-term drained cyclic triaxial test involving two sets of stress-controlled consolidation and creep stages, a cyclic shearing stage and post-cyclic monotonic shearing to failure (Liu, 2018).

interpret ratcheting, cyclic stiffness, damping and other cycling characteristics from the high-resolution cycle-by-cycle stress-strain data.

Post-cyclic shear stiffness trends are considered in Figure 20, plotting normalised secant Young’s moduli $E_{v,sec}'$ against axial strain for specimens cycled from the same $(q_{mean}, p'_0) = (100, 133.3)$ (kPa) mean conditions but with different amplitudes (q_{cyc}). Applying $f(e)$ accounted for void ratio changes under cycling, showing how cycling led to higher initial stiffness, longer linear plateaux and more gradual stiffness degradation. These changes grew markedly with the prior cyclic stress ratio, reflecting micro-fabric optimisation under cycling. The significantly better organised internal particle force chains and contact

distributions also relocated and expanded the sand's Y_1 and Y_2 kinematic yield surfaces.

5. St Nicholas at Wade (SNW) chalk

Extensive laboratory and *in-situ* characterisation were conducted for the ALPACA piling JIP. Noting that pile driving alters chalk properties significantly, the testing considered the behaviour of both intact chalk and the de-structured 'putty' material that formed adjacent to the pile shafts during driving and controlled their axial behaviour. The programme encompassed monotonic and undrained cyclic triaxial loading, supported by geological logging, index, unconfined compression strength (UCS), oedometer, direct simple shear (DSS), Brazilian tension (BT) and interface shear tests (Vinck, 2021). A metallurgical corrosion study was also undertaken by Vinck et al. (2023b) to provide fundamental insights into the piles' highly age-dependent axial shaft capacity trends (Buckley et al., 2018, 2020; Jardine et al., 2023b, c; Vinck et al., 2023a).

5.1 Elastic stiffness profiles

The chalk's elastic stiffnesses were examined by laboratory and field testing and Figure 21 considers the shear stiffness profiles measured at the smallest strains offered by various techniques. As emphasised by Clayton et al. (2003), the chalk's meso-to-macro fabric plays a dominant role. The mean SCPT G_{vh} and cross-hole G_{hv} trends amount to around 2/3 of the shallow triaxial tests' BE G_{vh} measurements and converge more closely at depth, reflecting the reducing occurrence of open fissures, which are naturally excluded from laboratory specimens. The direct simple shear (DSS) G_{vh} maxima and the pressuremeter G_{hh} values fall far below those interpreted from shear wave velocities, due to their low resolution and the chalk's brittle response to the

non-uniform straining imposed by these tests. Triaxial BE measurements made on the same samples indicated $G_{hh}/G_{vh} \approx 0.5$ in the shallow layers, which gradually rose to unity at depth; the field seismic data show a similar, but more muted, trend. Also plotted in Figure 21 is the E_h'/E_v' profile developed from four suites of triaxial probing tests by Vinck (2021). Horizontal loading from *in-situ* stresses provokes a far softer response than vertical compression, which is important when analysing lateral pile loading. This aspect of the chalk's marked anisotropy contributed to undrained and drained triaxial compression paths following similarly inclined effective stress paths (Vinck et al., 2022).

5.2 Chalk's response to monotonic triaxial loading

5.2.1 Pressure-dependent yielding and failure

The SNW chalk's cemented open micro-structure (Alvarez-Borges et al., 2020) leads to significant sensitivity, brittleness and pressure-dependent behaviour. Figure 22 shows example loading curves up to 0.5% axial strain from locally instrumented triaxial and UCS tests conducted from *in-situ* p_0' and 300 kPa + p_0' stresses. Very stiff and nearly linear pre-failure behaviour is evident, as well as a notably brittle post-peak response that involved tensile fracturing and led to low strengths at large strains. High pressure triaxial tests that explored conditions from *in-situ* p_0' (≈ 63 kPa) up to $p_0' = 12.8$ MPa identified a gradual transition (shown in Figure 23) from brittle to ductile behaviour under increasing p' levels. The chalk's highly curved failure envelope (Figure 24) reaches an ultimate $(q/p')_{ult} = 1.25$, equivalent to critical state $\phi' \approx 31^\circ$, that matches (as shown later) that of fully de-structured chalk.

Figure 25 depicts how the initial Y_1 and Y_3 yield loci of unfractured chalk may be represented with near elliptical

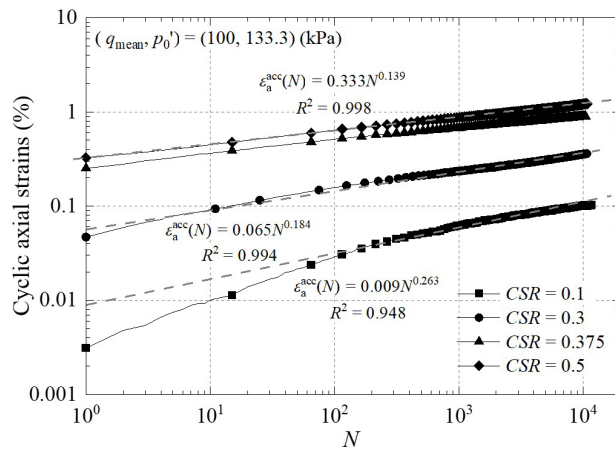


Figure 19. Accumulated cyclic axial strains for very dense ($D_R = 92\%$) triaxial Dunkirk sand specimens cycled from $(q_{mean}, p_0') = (100, 133.3)$ (kPa) under fully drained conditions (Liu, 2018).

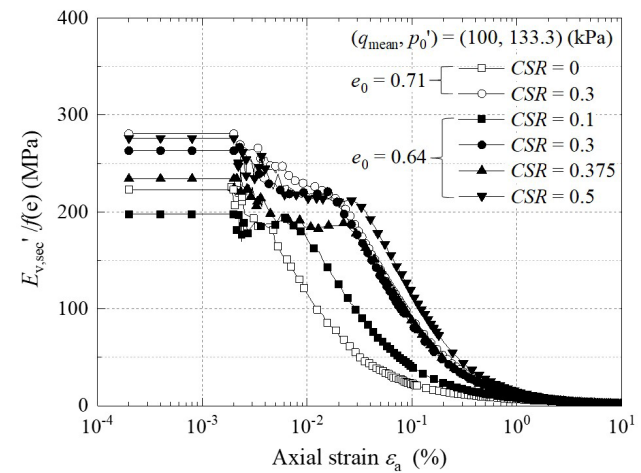


Figure 20. Effects of cyclic loading history on the post-cyclic monotonic stiffness degradation trends for Dunkirk sand (Liu, 2018).

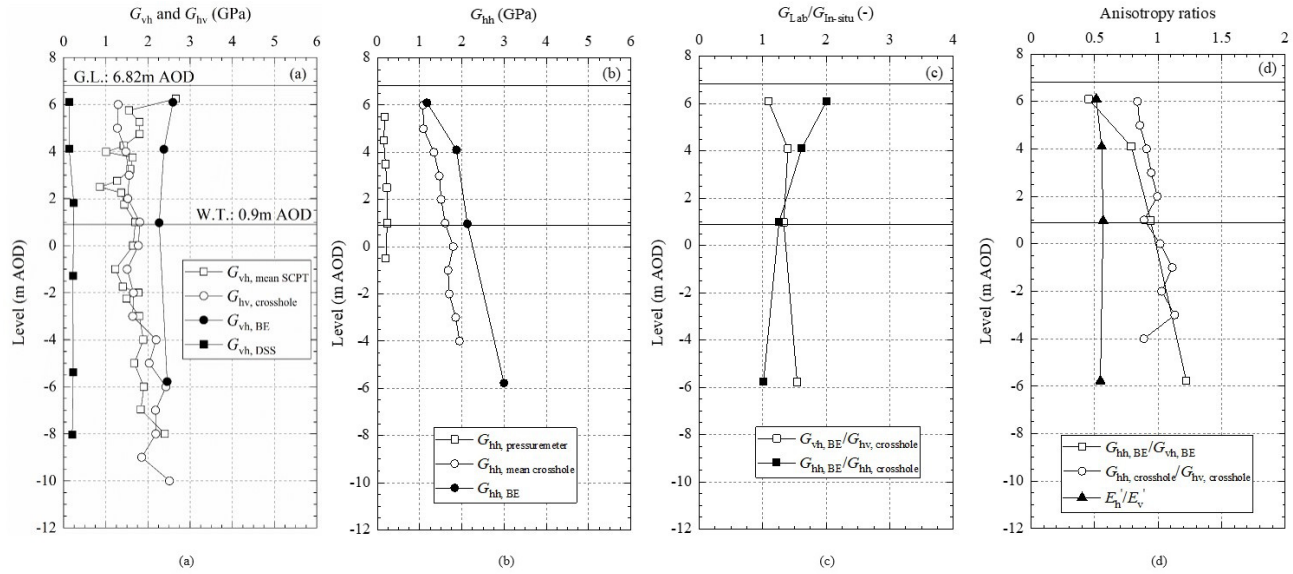


Figure 21. SNW profiles and ratios of maximum stiffnesses: (a) laboratory and in-situ measurements of G_{vh} and G_{hv} ; (b) G_{hh} profiles; (c) ratios between laboratory and in-situ stiffnesses; and (d) profiles of stiffness anisotropy ratios (modified from Vinck et al., 2022).

surfaces in triaxial q - p' space. Intact (cemented) chalk displays linear behaviour over a far larger region of q - p' space than soils such as the unbonded Cowden till or Dunkirk sand. Laboratory elastic vertical Young's moduli $E_{v,max}$ are comparatively insensitive to applied p_0' , with modest increases up to the peak (7.7 GPa) found with $p_0' = 650$ kPa before falling to a 4 MPa plateau as bonds break under increasing p_0' (Liu et al., 2023b). However, the chalk's systems of micro-to-macro fissures, which gradually close under normal stress, lead to the maximum mass chalk stiffnesses invoked by pile loading being four times lower than the laboratory values (Jardine et al., 2023c).

Chalk's propensity to tensile fracture leads to BT and DSS strengths 90% and 50% lower respectively than those from UCS or triaxial compression tests, with many implications for the modelling of geotechnical problems that involve multi-directional loading.

5.2.2 Fully de-structured chalk's response to undrained monotonic shearing

Laboratory dynamic compaction, applied at in-situ water content, was employed to de-structure low-to-medium density chalk in an analogous way to pile driving, producing uniform chalk putty with 9 ± 3 kPa fall-cone shear strengths and liquid and plastic limits of 30.6% and 24.2% respectively. Instrumented, consolidated, triaxial specimens were formed with (pre-shearing) void ratios and effective stress states matching those noted in the de-structured annuli identified around driven pile shafts several months after driving (Buckley et al., 2018).

The de-structured chalk's undrained triaxial compression and extension behaviour is demonstrated first in Figure 26 by

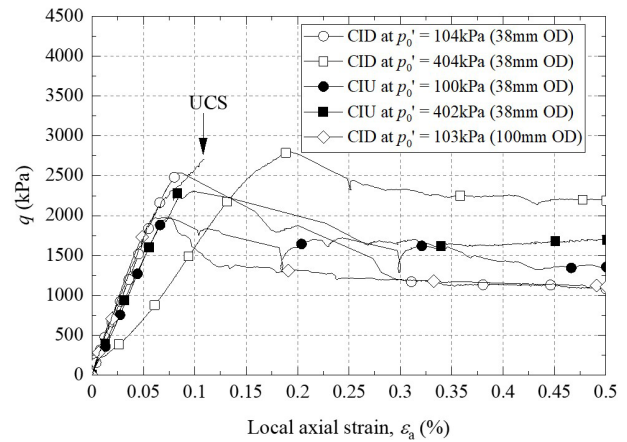


Figure 22. Example stress-strain curves up to 0.5% axial strain from locally instrumented triaxial and unconfined compression (UCS) tests on intact SNW chalk from in-situ p_0' and 300 kPa + p_0' (Vinck et al., 2022).

considering the q - p' effective stress paths. These are initially nearly vertical, suggesting that the re-consolidated and (mildly aged) putty's initial stiffness response was largely isotropic. The effective stress paths rotated to follow leftward (contractive) stages after mobilising modest 'peak' resistances (at $\epsilon_a < 0.2\%$) and showed strain softening as shearing continued up to phase transformation (PT) points at which their paths rotated abruptly and climbed towards ultimate (critical state) conditions. Continued straining led to markedly higher ultimate strengths as the specimens attempted to continue dilating. Extension tests indicated similar, yet not fully symmetric, paths and shear strengths.

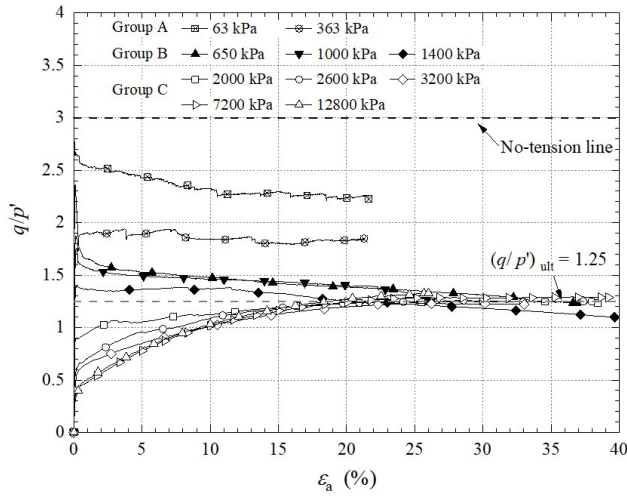


Figure 23. Full-strain triaxial compression behaviour of intact SNW chalk under a wide range of pressures (Liu et al., 2023b).

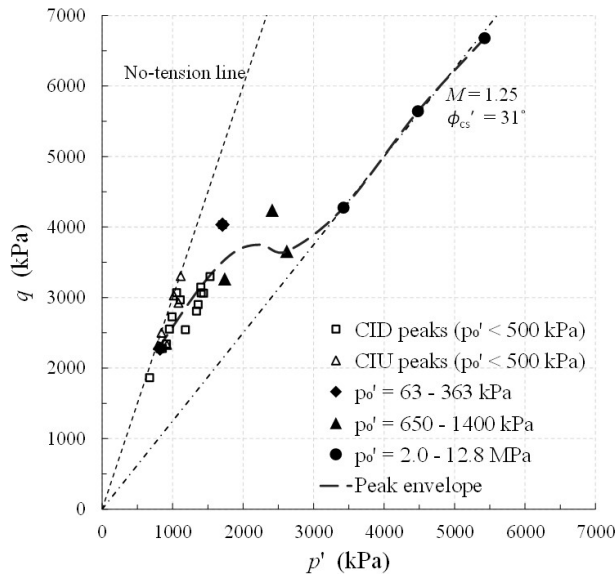


Figure 24. Peak and ultimate shear strength envelope for intact SNW chalk (Liu et al., 2023b).

The ultimate critical states gave $\phi'_{cs} \approx 31^\circ$, matching the high-pressure intact tests in Figure 24.

5.3 Response to undrained cyclic triaxial loading

5.3.1 Intact chalk's failure patterns

Undrained cycling affects intact chalk differently to saturated sedimentary sands or clays (Ahmadi-Naghadeh et al., 2022). Figure 27 presents a typical unstable response observed with a specimen loaded from $p'_0 = 42$ kPa, $q_{mean} = 1225$ kPa

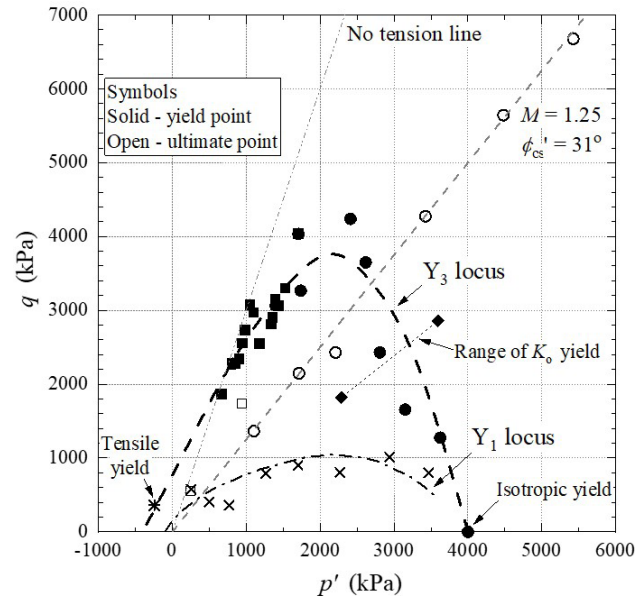


Figure 25. Yield stresses and interpreted pre-failure Y_1 and large-scale Y_3 loci for intact SNW chalk (Liu et al., 2023b).

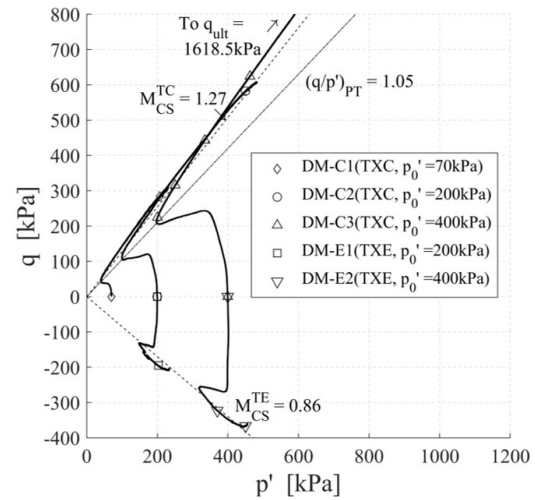


Figure 26. Effective stress paths of de-structured chalk under undrained triaxial compression and extension (modified from Liu et al., 2022b).

by high-level cycling with $q_{cyc} = 950$ kPa and $q_{max}/(2S_u) = 0.91$. The axial strain and pore pressure records show little or no sign of impending instability; little change was seen in either the damping ratio (which remained at around 4%) or the cyclic stiffness, until degradation set in over the last 30 cycles and brittle failure occurred abruptly after 181 cycles. In contrast, specimens that sustained large number ($> 4,000$) of cycles developed minimal straining and excess pore pressure accumulation and manifested nearly visco-elastic behaviour.

The experimental outcomes allow cyclic interaction diagrams to be established in $q_{cyc}/(2S_u) - q_{mean}/(2S_u)$ space, as in Figure 28. A tentative fan of linear N_f contours indicates the

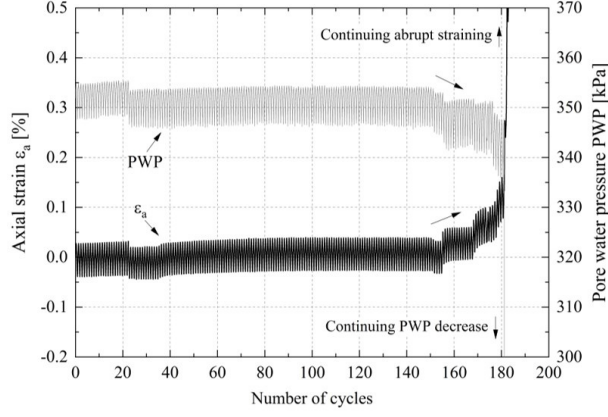


Figure 27. Typical unstable cycling response of intact chalk: Variation of axial strain and pore water pressure against number of cycles (Test conditions: $p'_0 = 42$ kPa, $q_{mean} = 1225$ kPa, $q_{cyc} = 950$ kPa and $q_{max}/(2S_u) = 0.91$) (Ahmadi-Naghadeh et al., 2022).

main trends applying between 1 and 3000 cycles. The region below the 3000-cycle contour in Figure 28 signifies the stable region within which undrained cycling improved stiffness without any loss of undrained shear strength.

Overall, the intact chalk's behaviour appears more like that of crystalline rocks or metals than sedimentary soils. Repeated loading prompts progressive wear and shearing between grains, forming microcracks that may coalesce into macrocracks (Cerfontaine & Collin, 2018). The outcomes can be interpreted as 'S-N' curves that plot the maximum cyclic load ($q_{max} = q_{mean} + q_{cyc}$) against $\log N$, with contours interpolated to show how the $q_{max}/(2S_u)$ curves fall as q_{cyc}/q_{mean} rises, expressed as:

$$\frac{q_{max}}{2S_u} = f(N) \cdot \frac{1 + q_{cyc}/q_{mean}}{f(N) + q_{cyc}/q_{mean}} \quad (6)$$

in which $f(N)$ represents for each N_f line its projected intercept on the vertical axis of the interactive $q_{cyc}/(2S_u)$ - $q_{mean}/(2S_u)$ diagram and is expressed as:

$$f(N) = 0.35 + \frac{1}{1.54 + 0.37 \times [\log_{10}(N_f)]^{2.75}} \quad (7)$$

A lower limit of $f(N) = 0.35$ is incorporated in this expression which corresponds to a fatigue limit (or fatigue strength) of $q_{max}/(2S_u) = 0.52$, below which specimens could sustain cycles indefinitely under the most critical $q_{cyc}/q_{mean} = 1$ one-way loading condition. The above two equations lead further to a three-dimensional representation of intact chalk's undrained cyclic failure characteristics in a $q_{max}/(2S_u)$ - q_{cyc}/q_{mean} - $\log_{10}(N)$ space, as shown in Figure 29.

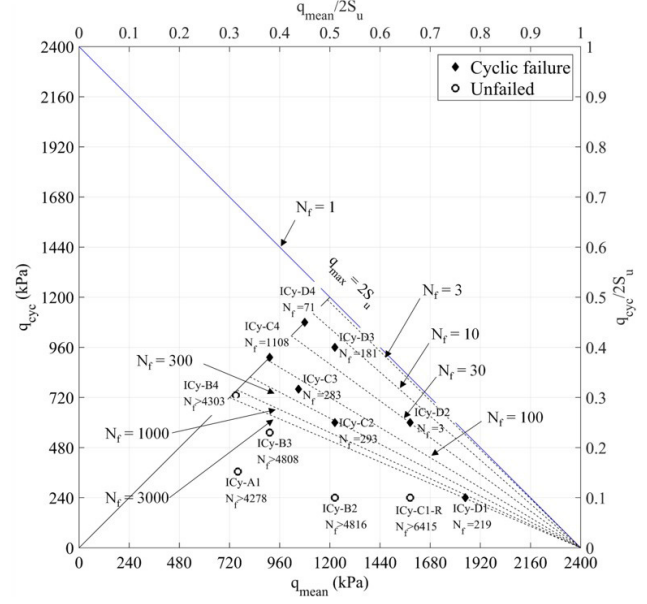


Figure 28. Cyclic interaction diagram for intact SNW chalk tested from $p'_0 = 42$ kPa, showing the interpreted contours of number of cycles to failure for specified q_{cyc} - q_{mean} conditions (Ahmadi-Naghadeh et al., 2022).

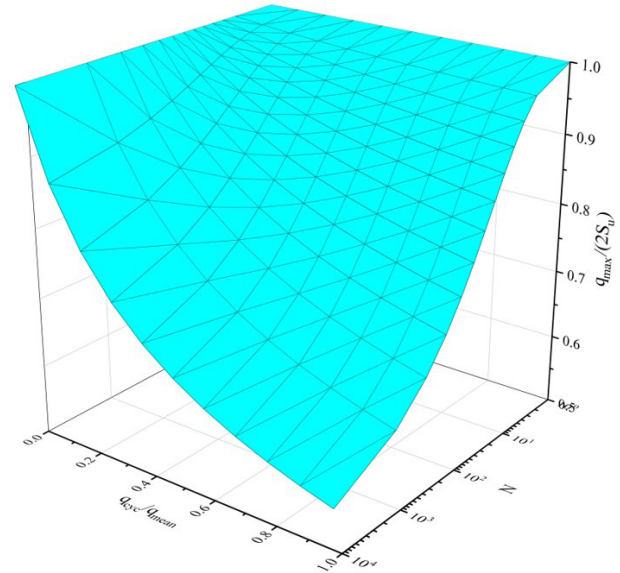


Figure 29. Three-dimensional representation of the undrained cyclic failure characteristics of intact SNW chalk in $q_{max}/(2S_u)$ - q_{cyc}/q_{mean} - $\log_{10}(N)$ space (Ahmadi-Naghadeh et al., 2022).

5.3.2 Cyclic failure characteristics of de-structured chalk

Liu et al. (2022b) show that fully de-structured, re-consolidated, chalk putty responds to undrained cyclic loading similarly to silts and silty sands, and markedly differently to the intact chalk. As illustrated in Figure 30,

de-structured specimens respond to high-level cycling by generating excess pore pressures that shift their effective stress paths leftwards. Both contractive and dilative phases occur within individual cycles. Cyclic phase transformation lines with (q/p') gradients of ≈ 0.54 and 0.38 can be identified in compression and extension respectively, following Mao & Fahey (2003), that fall well below the monotonic phase transformation lines. The transient cyclic effective stress paths can also cross the monotonic critical state gradients M .

The full suite of one- and two-way cyclic tests is set-out in an interactive loading diagram in Figure 31, which adopts both $q_{cyc}/(2S_u) - q_{mean}/(2S_u)$ and $q_{cyc}/p_0' - q_{mean}/p_0'$ axes. A tentative family of contours is shown for number of cycles to failure (N_f); the lower-levels, high N_f contours show less curvature and tighter spacings than those representing high-level cycling. As with all soils, chalk putty is more susceptible to high-level two-way (compression and extension, with $q_{mean} \leq q_{cyc}$) loading than one-way compression cycling.

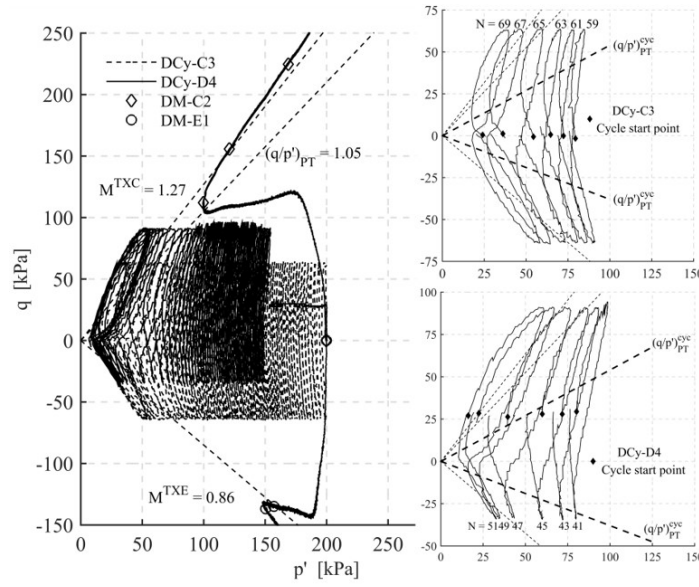


Figure 30. Effective stress paths for typical unstable tests on de-structured chalk and the identified cyclic phase transformation lines (Liu et al., 2022b).

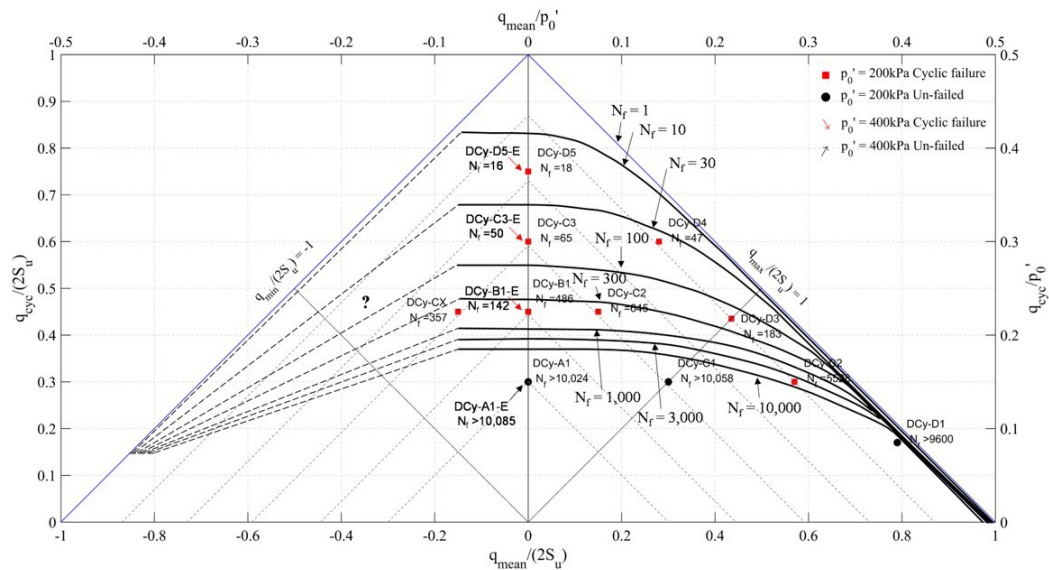


Figure 31. Cyclic interaction diagram for de-structured chalk expressed in normalised $q_{cyc}/(2S_u) - q_{mean}/(2S_u)$ and $q_{cyc}/p_0' - q_{mean}/p_0'$ stress space for $p_0' = 200$ kPa and $p_0' = 400$ kPa test series (Liu et al., 2022b).

Significant axial strain development and stiffness losses occur in unstable tests that accelerate markedly as cyclic failure approaches. Liu et al. (2022b) report on these features and the specimens' damping ratios. Cycle-by-cycle effective stress changes were tracked in the experiments, enabling the development of a laboratory-based global prediction method for axial shaft capacity under cyclic loading (Jardine, 2020; Buckley et al., 2023). All tests showed $\Delta p'/p_0'$ decreasing continuously with N , although it is possible that cycling at lower levels than those applied would identify conditions under which no reduction occurs. Liu et al. (2022b) show that the mean effective stress drifts of tests run with $(q_{cyc} + q_{mean})/p_0' < 0.4$ can be related to the number of cycles applied by the power-law Equation 8.

$$\frac{\Delta p'}{p_0'} = A \times \left(B + \frac{q_{cyc}}{p_0'} \right) \times N^C \quad (8)$$

Parameters A , B and C define the rates of p' degradation and the maximum cyclic stress ratio that could lead to beneficial, null, or deleterious cycling effects. Higher level cyclic tests that failed within just a few cycles show more complex trends. Figure 32 reports the outcomes of cyclic tests on samples consolidated to $p_0' = 200$ kPa and shows how Equation 8, applied with $A = -0.05$ and $B = -0.12$ and $C = 3.48 \times q_{cyc}/p_0'$ fits the experimental trends well.

6. Applications of the research

The Cowden, Dunkirk and SNW characterisation programmes were vital to interpreting the high-value, large-scale, instrumented pile experiments run at the three sites. They also led to new site investigation and practical pile design approaches that are being applied widely in

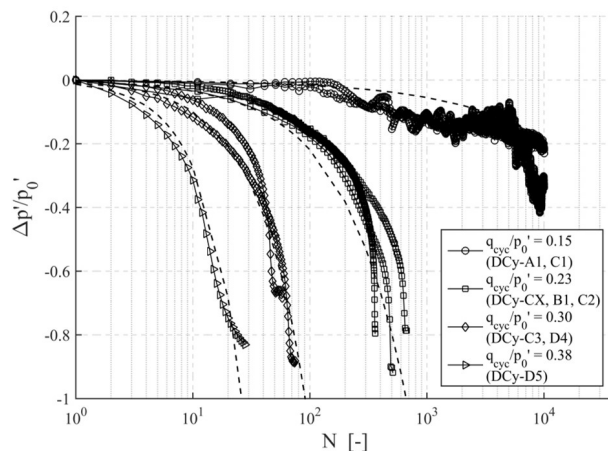


Figure 32. Mean effective stress trends during cycling and fitted trends (in dashed lines) for ‘putty’ tests with $p_0' = 200$ kPa and mostly $(q_{cyc} + q_{mean})/p_0' < 0.4$ (Liu et al., 2022b).

major offshore wind projects. As noted below, advances have flowed in four main areas.

6.1 Pile behaviour under monotonic axial loading

Jardine et al. (2005) and Jardine (2014) showed that advanced finite element (FE) modelling based on the glacial tests by Jardine et al. (1984) and sand tests by Kuwano (1999) offered good matches to full-scale field observations made at North Sea clay till sites and tests to failure on 19 m long, 457 mm diameter piles driven at Dunkirk. The most significant new findings to emerge regarding axial load-displacement behaviour, which is central to jacket supported structures, therefore concern the chalk, about which little information was available previously. Wen et al. (2023a, b) employed the laboratory testing to calibrate new 1D load transfer and advanced 2D/3D FE models that provide generally good fits to the ALPACA and ALPACA Plus field tests on piles with diameters up to 1.8 m and embedded lengths up to 18 m.

6.2 Pile behaviour under cyclic axial loading

Buckley et al. (2023) and Liu et al. (2023c) report on how the laboratory chalk putty cyclic testing contributed to a simplified yet accurate global prediction method for axial cyclic pile design in chalk, which is now available for use by industry. Jardine (2020) and Jardine et al. (2012) have reported previously on the use of undrained cyclic tests on sands and clays to predict pile response to axial loading, showing how the laboratory experiments by Aghakouchak et al. (2015) on Dunkirk sand enabled accurate predictions of the cyclic axial pile tests reported by Jardine & Standing (2012). The Cowden till cyclic testing data is also now available to aid back-analysis of earlier cyclic axial pile tests (see Ove Arup and Partners, 1988) as well as monotonic axial tests reported by Karlsrud et al. (2014).

6.3 Pile behaviour under monotonic lateral loading

Modelling of field lateral pile loading response is critical to the efficient design of monopile supported offshore wind turbines. Taborda et al. (2020) and Zdravković et al. (2020b) describe how the site characterisation described above was central to advanced 3D FE analyses that aided the design and interpretation of the PISA lateral loading tests on instrumented piles with diameters up to 2 m driven at Dunkirk and Cowden. The resulting PISA design approach (Byrne et al., 2020c) has offered a step change in design practice that has been applied internationally. Pedone et al. (2023) describe how still more advanced 3D FE analyses developed from the chalk testing captured accurately the lateral loading behaviour shown by ALPACA tests in the fractured, highly brittle SNW chalk. These experiments and

analyses are now being applied in the design of major new windfarms at offshore chalk sites.

6.4 Pile behaviour under cyclic lateral loading

It is essential to consider lateral cyclic loading cases when designing monopile foundations. The PISA cyclic pile tests are applied widely as benchmarks against which alternative modelling strategies are routinely calibrated and tested. The Authors' recently published cyclic laboratory test programmes offer a basis for re-analyses of the Cowden and Dunkirk cyclic PISA pile tests, as well as the recent PICASO tests (Byrne et al., 2020a) at Cowden. The cyclic laboratory programmes on chalk will also pave the way for well-calibrated lateral cyclic analyses at chalk sites.

7. Summary and conclusions

This paper summarises recent research into the stiff glacial till, dense marine sand, and low-to-medium density chalk encountered at the PISA, ALPACA and ALPACA Plus JIPs instrumented pile test sites. Coordinated field and laboratory studies investigated the materials' stiffness patterns, full strain mechanical behaviour, anisotropy and response to cyclic loading. Their outcomes are being employed widely to aid the modelling of field behaviour under axial-and-lateral, monotonic-and-cyclic loading. By enabling more efficient driven pile foundation design, the research is contributing significantly to more efficient offshore wind energy generation and CO₂ emission reductions. The large volume of new findings supports four general conclusions.

1. Monotonic laboratory testing to failure showed how the geomaterials behave over their full strain range, including the distinct properties of the fully de-structured chalk putty. Combining these data with *in-situ* measurements enables representative modelling of the pile test outcomes and shows the way forward for their practical application;
2. Laboratory and field measured elastic stiffnesses may show significant divergence. While broadly similar profiles applied at Cowden, triaxial tests on reconstituted Dunkirk sand underpredicted the site's field stiffnesses, which had grown after 20 years of ageing under hydraulic fill. In contrast, the chalk's mass field stiffness fell far below the measurements made from laboratory tests on intact samples because they did not capture the important effects of partially open meso-to-macro fractures;
3. All three geomaterials displayed elastic stiffness anisotropy. Vertical stiffnesses exceeded horizontal moduli in the normally consolidated dense Dunkirk sand and in the high YSR intact SNW chalk, while the opposite applied in high YSR Cowden glacial till;
4. The cyclic programmes showed a wide range of behaviours and provide vital new evidence to aid

the development of suitable predictive approaches. While the intact chalk's response was distinctly different to that of the till and sand, resembling that of crystalline rocks or metals, the chalk putty's cyclic behaviour was closer to that shown by the glacial till and marine sand.

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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

Tingfa Liu: conceptualization, investigation, data curation, methodology, formal analysis, writing – original draft. Ken Vinck: conceptualization, investigation, data

curation, methodology, formal analysis, writing – review & editing. Emil R. Ushev: conceptualization, investigation, data curation, methodology, formal analysis, writing – review & editing. Richard J. Jardine: supervision, methodology, project administration, funding acquisition, writing – review & editing.

Data availability

Some data that support the findings of this study are available from the corresponding author upon reasonable request.

List of symbols and abbreviations

a_v, a_{vh}, a_{hh}	Cross-anisotropy stiffness parameters; see Equations 2-5
b	Intermediate principal stress factor ($= (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$)
b_h, b_{vh}, b_{hh}	Cross-anisotropy stiffness parameters; see Equations 2-5
c'	Soil cohesion
e	Void ratio
e_0	Specimen initial void ratio
p'	Mean effective stress
$p'_{in-situ}$	<i>In-situ</i> mean effective stress
p'_0	Initial mean effective stress
p_{ref}	Reference atmospheric pressure ($= 101.3$ kPa)
q	Deviatoric stress
q_c, q_t	Measured cone resistance and cone resistance corrected for pore pressure respectively
q_{cyc}	Cyclic deviatoric stress amplitude ($= (q_{peak} - q_{trough})/2$)
q_f	Deviatoric stress at failure
q_{mean}	Mean q applied in stress cycle
q_{max}	Maximum q applied in stress cycle ($= q_{mean} + q_{cyc}$)
q_{trough}	Minimum q applied in stress cycle ($= q_{mean} - q_{cyc}$)
q_{PT}	Deviatoric stress at the phase transformation state
w_c	Water content
w_L	Liquid limit
w_p	Plasticity limit
ALPACA	Axial-Lateral Pile Analysis for Chalk Applying multi-scale field and laboratory testing
AOD	Above ordnance datum
BE	Bender element test
CPTu	Cone penetration test with pore pressure measurement
CSR	Cyclic stress ratio ($= q_{cyc}/p'_0$)
C_v, C_h, C_{vh}, C_{hh}	Cross-anisotropy stiffness parameters, see Equations 2-5
D_R	Relative density

DSS	Direct simple shear
D_{50}	Mean particle diameter
E'_v, E'_h	Drained vertical and horizontal Young's moduli respectively
E^u_v, E^u_h	Undrained vertical and horizontal Young's moduli respectively
E^{TXC}_u	Undrained Young's modulus determined from triaxial compression tests
E^{sec}_u	Undrained secant Young's modulus
G_{hh}	Shear modulus in horizontal plane
G_{hv}, G_{vh}	Shear modulus in vertical plane
$G_{z\theta}$	($= G_{vh}$) Shear modulus in vertical plane
G_0	Elastic shear stiffness
G_{oct}	Octahedral shear modulus
G_s	Specific gravity
HCA	Hollow Cylinder Apparatus
IDD	Intact dry density
JIP	Joint industry project
KUC	K_0 -consolidated undrained compression
KUE	K_0 -consolidated undrained extension
K_0	Coefficient of earth pressure at rest
LVDT	Linear variable differential transformer
M	Critical state q/p' stress ratio
$M^{TXC}_{cs}, M^{TXE}_{cs}$	Critical state stress ratio under triaxial compression and extension
N	Number of cycles
N_f	Number of cycles to failure
N_{kt}	Cone factor defined as $N_{kt} = (q_t - \sigma_{v0})/S_u$
OCR	Over-consolidation ratio
PI	Plasticity index
PISA	Pile Soil Analysis
PT	Phase transformation
SCPT	Seismic cone penetration test
S_r	Saturation degree
S_u	Undrained shear strength
S^{TXC}_u, S^{TXE}_u	Undrained shear strength under triaxial compression and extension
TXC, TXE	Triaxial compression and extension respectively
YSR	Yield stress ratio
α	Angle between the vertical and the direction of σ_1 axis
α_{do}	Angle between the vertical and the direction of $\Delta\sigma_1$ axis
α_f	Angle between the vertical and the direction of σ_1 axis at ultimate failure
$\gamma_{z\theta}$	Torsional shear strain
Δu	Excess pore water pressure
δ'_{ult}	Ultimate soil-structure interface shear resistance angle
$\varepsilon_a, \varepsilon_v$	Axial (vertical) strain
ε_{mean}	Permanent axial strain accumulated in cyclic tests
ε_r	Radial (horizontal) strain

ε_s	Shear strain ($= \varepsilon_a$ for undrained triaxial condition)
$\varepsilon_z, \varepsilon_r, \varepsilon_\theta$	Axial, radial and circumferential strains respectively in HCA space
$\varepsilon_v, \varepsilon_h$	Vertical (axial) and horizontal (radial) strains respectively in triaxial space
$\varepsilon_1, \varepsilon_3$	Major and minor principal strain respectively
η	Mean stress ratio ($= q_{mean}/p_0'$)
ρ_{bulk}	Bulk density
$\sigma_1, \sigma_2, \sigma_3$	Major, intermediate and minor principal stresses respectively
σ_v', σ_h'	Vertical and horizontal effective stress respectively
σ_{vy}'	One-dimensional (1-D) compression yield stress
σ_{v0}'	In-situ vertical effective stress
$\tau_{z\theta}$	Torsional shear stress
ϕ'	Effective angle of shearing resistance
ϕ_{cs}'	Critical state shear resistance angle
ϕ_{peak}'	Shear resistance angle at peak

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REVIEW ARTICLES

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Measuring deformation processes in granular materials from intragranular to bulk scales

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Review Article

Keywords

Granular mechanics
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Micro-structure
Micro-mechanics
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Abstract

Granular materials (assemblies of contacting grains with inter-granular porosity) form a vast family of materials important in both natural and industrial contexts. Under mechanical loads such systems show complex behaviours relating to the inter- and intra-granular stresses, strains and force transmission as well as the individual grain kinematics and interactions leading to multi-scale, heterogeneous behaviour. To understand such systems requires detailed understanding of these different structures and processes at the pertinent scales. This paper provides a brief overview of some new opportunities for enhanced experimental description of structures and mechanisms relating to the deformation of granular materials exploiting the penetrating power of x-rays for both imaging, giving access to granular structures and their evolution, and diffraction-based measurements, giving access to intra-granular grain structure and changes as well as inter- and intra-granular force transmission.

1. Introduction

Granular materials (assemblies of contacting grains with inter-granular porosity) form a vast family of materials important in both natural and industrial contexts, e.g., slope stability, avalanches, pharmaceutical tablet compaction, building foundations, powder-based additive manufacturing and food processing, to name but a few. Such materials are highly complex and can behave as liquids, gases or load-bearing solids, depending on their confinement, such that granular media are often considered to be another state of matter (e.g., Jaeger et al., 1996). The complex behaviour of these granular systems stems from the mobility and collective interactions of relatively simple building blocks (i.e., contacting particles) that together produce the bulk material behaviours and structural evolution. The processes controlling the bulk behaviours originate in mechanisms over a range of spatial and temporal scales from intra-granular mechanisms to contacting-particle interactions, intermediate (meso-) scale correlations and structure formation (e.g., localised deformation such as shear bands) to long-range pattern formation. These kinematic effects are associated with the build-up of stress, within grains and across granular structures, and relaxation via structural reorganisation and particle damage or breakage.

Granular mechanics have been studied for centuries, starting perhaps with the work of Coulomb in the late 1700s, but there remain key pieces of information regarding the (micro)mechanics that have been challenging to characterise. In particular, the measurement and understanding of force

transfer and stress heterogeneity has, until recently, been missing in experimental studies. Furthermore, the role and evolution of heterogeneity at different scales is generally not captured in experiments and is, thus, often neglected in modelling. Therefore, more detailed experimental investigations and new techniques with a focus on capturing the necessary details for more accurate simulation models are required. Recent experimental developments are yielding new data and new understanding on heterogeneity, kinematics and force transfer from intra-granular and grain-grain contact scales to the granular-system scale that should provide improved support for advances in theoretical and numerical modelling of granular systems.

On the flip side to the experimental understanding, ever more sophisticated and detailed models and numerical simulation approaches are being proposed that aim to capture the complexities of granular micro-mechanics. These models can involve explicit representations of the granular frame, e.g., Discrete Element Method type approaches, or continuum models with constitutive laws based on phenomenological or micromechanics representations of the granular structure and evolution implemented in, for example, a finite element framework. However, in the words of Richard P. Feynman, “*It doesn’t matter how beautiful your theory is, it doesn’t matter how smart you are. If it doesn’t agree with experiment, it’s wrong*”. More precisely (for granular mechanics), if the appropriate experimental data do not exist then a theory cannot be fully developed or validated, i.e., higher-order modelling requires equivalent higher-order experimental data for parameterisation, calibration and validation, as

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well as to identify the mechanisms and phenomena that models should include (or not) for accurate representation of material responses.

This paper provides a brief overview of some new opportunities for enhanced experimental description of structures and mechanisms relating to the deformation of granular materials motivated by the need for ever more accurate modelling capabilities. The experimental approaches described exploit the penetrating power of x-rays for both imaging, giving access to granular structures, and diffraction-based measurements, giving access to the intra-granular crystalline grain structure and changes. It is noted that this is not a thorough review of the field, rather an overview of recent developments in which the author has been involved.

2. Full-field imaging of deformation, granular structure and structural evolution

It is well understood that mechanical loading of granular systems leads to localised deformation, consequently standard experimental approaches that capture just limited measurements (e.g., axial force, piston displacement, confining pressure, pore pressure) at a sample boundary can only provide a partial understanding of the processes occurring during the deformation. Localisation implies heterogeneity, which can also exist due to sample preparation or natural structure. Measurements at the sample boundary can only be interpreted with an assumption of homogeneity and be considered as some form of sample-averaged response. Such measurements can be used to calibrate/validate material models, if the full sample is modelled (e.g., with a finite or discrete element simulation), including the correct boundary conditions, but there will be ambiguity in this and little constraint on the evolving heterogeneity in structure and deformation. There can also be ambiguity as to the actual boundary conditions applied, as opposed to those that were envisaged, e.g., due to the contact of the sample with the loading pistons. Full-field measurements (e.g., Viggiani & Hall, 2012) aim to provide a field of measurements over a whole sample, which, when combined with “in-situ” imaging (i.e., imaging during the experiment with the sample set-up in-situ in the imaging system) can be used to track the evolution of the material and assess the actual boundary conditions experienced by the sample.

One of the most powerful tools for full-field imaging with in-situ experiments is x-ray (micro) tomography. Already in the 1960s x-rays were being exploited in geomechanics to follow internal deformation mechanisms in soil testing. This was mostly with radiography of 2D experiments, but some developments towards 3D were reported (e.g., Arthur & Dunstan, 1969). These works revealed, for example, dilation associated with shear localisation in granular media (e.g., Stone, 1985). In the 1980s, Desrues et al. (1996), utilised medical x-ray tomography to reveal the internal structural

evolution of shear bands in triaxial tests on sand (e.g., Desrues et al., 1996). Importantly, in this work, in addition to characterising the 3D structure of the strain localisation, quantitative information was extracted from the images to reveal details about the material behaviour, such as the local evolution of void ratio and the local applicability of critical state theory. The past 20 years have seen significant developments in the x-ray tomography technique as well as in its accessibility and utilisation. In particular, laboratory x-ray microtomographs are now present in many labs around the world enabling micron to 10s-micron resolution 3D imaging of samples, including during experiments performed inside an imaging device (called herein in-situ experiments). As such, the experiments of Desrues et al. (1996) were repeated in recent years to reveal even finer details of the strain localisation evolution, including through quantified measurements of the strain field by Digital Volume Correlation (DVC) (discussed below) (Desrues et al., 2018).

An alternative to laboratory x-ray microtomography is to use a synchrotron facility, such as the European Synchrotron X-ray Facility (ESRF) in Grenoble, France. Such facilities provide higher-speed, higher resolution imaging for more detailed studies of evolving deformation. Early work using synchrotron tomography in granular mechanics includes that of Hall et al. (2010a), in which a triaxial compression experiment was performed in-situ in the synchrotron imaging station (beamline ID15 at the ESRF). Figure 1 shows images from the experimental set-up as well as the sample stress-strain response and a 3D rendering of the x-ray tomography image in the initial state. A general limitation of synchrotron imaging is the field-of-view (FoV) that is possible, although this is becoming less of a constraint with the development of the facilities and the techniques. Furthermore, the spatial resolution is generally linked to the FoV: the higher the resolution required, the smaller the FoV. Consequently, synchrotron imaging is generally performed on reduced sized samples, compared to standard laboratory testing. In the case of Hall et al. (2010a, b), the sample size was 22 mm high and 11 mm in diameter, but this enabled a voxel size (a voxel is a 3D pixel) of $14 \times 14 \times 14 \mu\text{m}^3$ giving about 20 voxels across a grain diameter. From such images, it is possible to separate the grains from the pore space and to distinguish individual grains. Consequently, through 3D image analysis techniques, porosity fields can be derived as well as measurements on all the grains in the sample (in this case about 50 000 grains), such as their size and shape, plus the contacts between grains can be identified and quantified, e.g., in terms of orientation distribution (fabric) (see Figure 2).

In-situ experiments, such as those described above, involve loading the specimen inside the imaging set-up, which enables 3D images to be acquired through the loading. This provides 4D data sets (3D + time) enabling the evolution of the measurements, such as those highlighted in Figure 2, to be tracked through the experiment. Therefore, porosity or contact evolution can, for example, be monitored in a full-field sense.

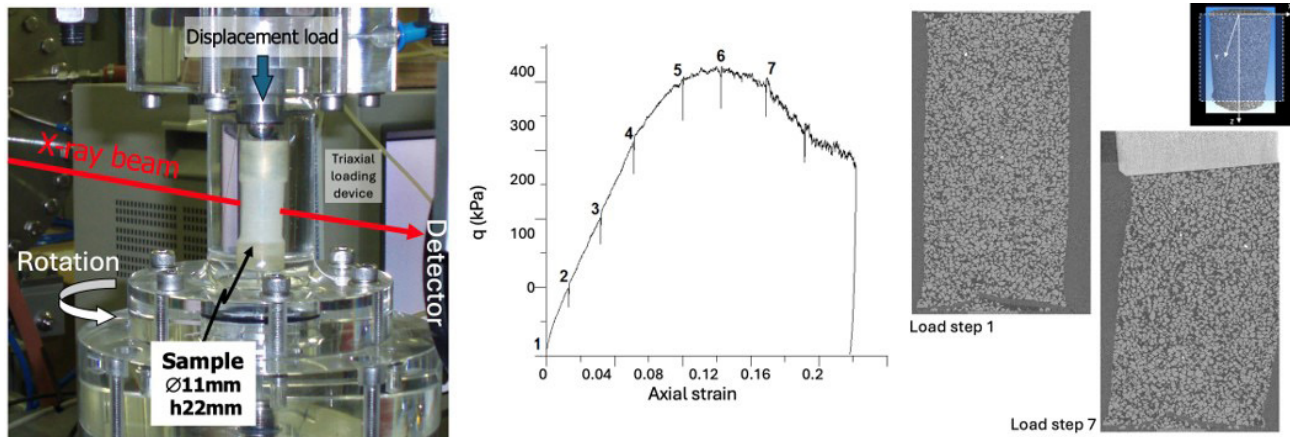


Figure 1. Left: Mini-triaxial compression experimental set-up of Hall et al. (2010a) at the ID19 beamline of ESRF. Middle: deviatoric stress (q) versus axial strain curve with imaging points indicated by numbers (the stress drops are due to stopping of the piston displacement during the imaging). Right: slices through the 3D x-ray tomography volumes acquired at the start of deviatoric loading and at load step 7 (plus inset showing the slice plane through the volume).

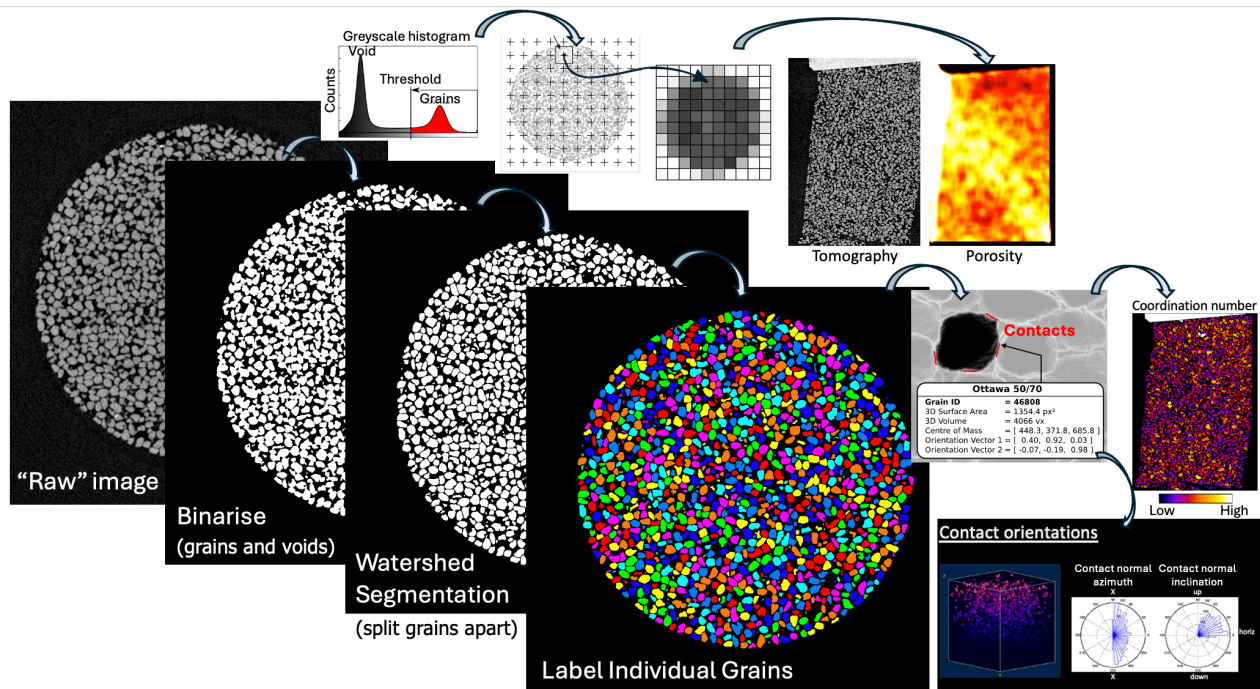


Figure 2. Schematic of the 3D image quantification process indicating how the “raw” greyscale 3D x-ray tomography images can be binarized by a simple thresholding approach to be used directly to calculate porosity fields by concatenation of the black:white voxel count over small image subsets or to separate the individual grains (e.g., by a watershed algorithm) to enable quantification of the individual grain characteristics, and contacts (images adapted from Andò, 2013; Andò et al., 2012; Hall et al., 2010a).

Furthermore, DVC can be used to follow parts of the image from one step to the next to derive 3D vector displacement fields from which, by differentiation, 3D tensor strain fields can be derived (Figure 3) (see, for example, Hall et al., 2010a; Hall, 2012). The standard DVC approach considers the image simply as a set of voxels with different intensities, ignoring any microstructure. An alternative approach to DVC is to respect

the microstructural information contained in the images and, for the case of granular materials, track the individual grains instead of just cubic subsets of the image (see Figure 4). This approach requires a segmented image volume where all the grains are individually labelled. The outputs from such “discrete DVC” (D-DVC) are the full kinematics of every grain in the sample through the loading experiment (e.g.,

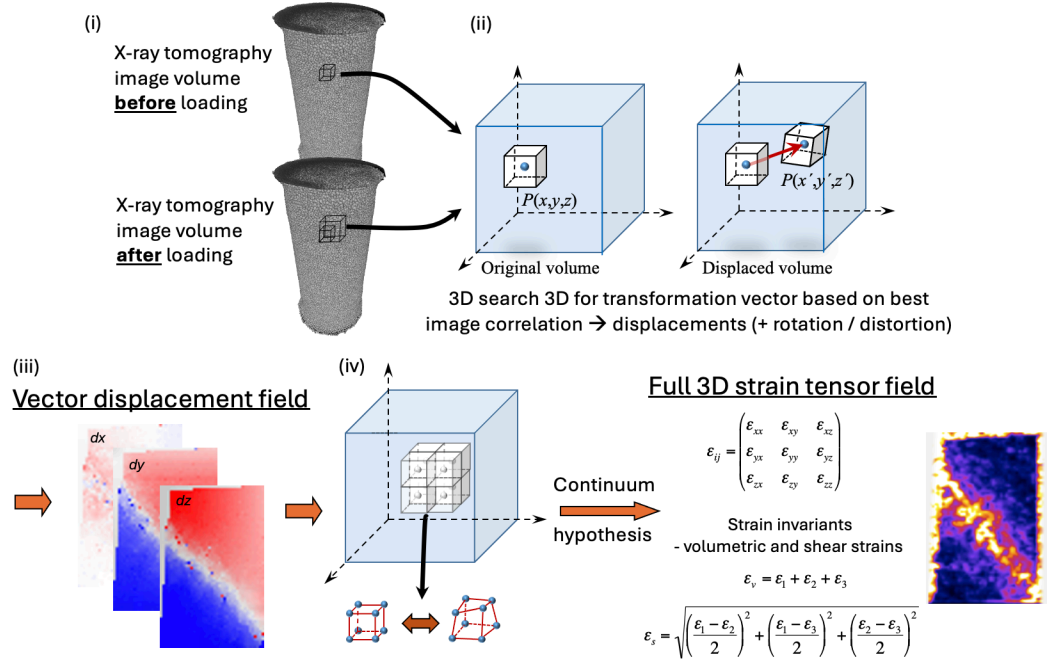


Figure 3. Schematic of a DVC involving: (i) defining a 3D grid of analysis points (nodes) in the initial 3D x-ray tomography volume; (ii) small image volumes are extracted from around each node; (iii) displacements of each node from the first image to the next is determined by a search for the translations (plus eventually rotations and deformations) that give the best correlation to equivalent volumes in the second image (usually derived with sub-pixel resolution); (iv) the resulting 3D vector field of nodal displacements (indicated by the 3 slices through the dx , dy , dz volumes) can be used to define, by differentiation, the 3D tensor strain field that is often represented in terms of strain invariant scalars, as in the slice through the shear strain field volume shown.

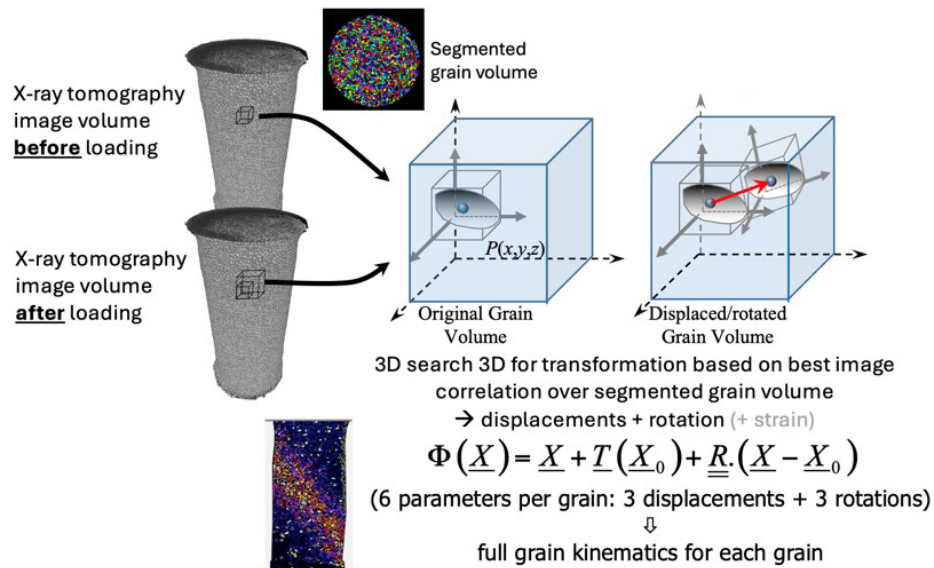


Figure 4. Schematic of a D-DVC approach, utilises the segmented grain map (see Figure 2) to define the grain centres and a mask to isolate the grain voxels in a small volume around the grain centre. A correlation search is performed, in a similar way to standard DVC (Fig. 3), refining for displacement and rotation with an assumption (usually) of incompressible grains (although grain strain can be assessed in the SPAM implementation (Stamati et al., 2020)). The output is a set of 3-component displacement vectors and 3-component rotation vectors for all the grains in the assembly. Continuum strain fields can be derived by tessellation of the grain centres and differentiation of the grain displacements.

Hall et al., 2010a). Put together, the in-situ experiments with x-ray tomography and 3D/4D image analysis can provide a quite complete picture of granular evolution during triaxial experiments, as indicated in Figure 5.

Building on the work of Hall et al. (2010a, b), Andò (2013) produced significant developments to the above-described granular experimental micromechanics approach including running similar, grain-resolution experiments with a laboratory tomograph, acquiring many more images per test (10s), comparing different grain types and advancing the image analysis algorithms. Furthermore, the SPAM software for DVC has subsequently been developed with specific adaptations for the analysis of granular materials, including D-DVC for grain tracking (Stamati et al., 2020). The experimental and image analysis advances, including the development of SPAM, have promoted a better understanding of the micromechanics of granular materials (e.g., Andò et al., 2012; Andò et al., 2013; Wiebicke et al., 2020; Zhao et al., 2021; Pinzón et al., 2023).

3. Grain scale imaging of granular deformation, granular stress/strain and force transfer

Whilst the experimental and analysis approaches with in-situ x-ray tomography described above have provided

rich, previously inaccessible details on granular deformation mechanisms, they remain limited to measurements of structure and kinematic quantities. Mechanical analysis, however, must also consider the transfer of forces and stress in the system for a complete picture. As with kinematic measurements, forces are generally only measurable at the boundary of a specimen and only some components of the stress can be evaluated, even in the advanced approaches described above. Furthermore, as with kinematic measurements, standard measurements do not give a complete picture of the actual boundary conditions and stress conditions experienced by the sample, e.g., due to limited measurement possibilities and friction effects. To address this challenge, the use of x-ray (and neutron) diffraction approaches has been proposed (e.g., Hall et al., 2011) to measure the crystallographic (elastic) deformation of the constituent grains in a granular sample, which can be interpreted in term of stress through Hooke's law.

X-ray diffraction occurs through coherent elastic scattering of x-rays from the electrons surrounding atoms. The well organised distribution of atoms in crystal structures leads to constructive interference of the scattered x-rays to give, so-called, diffraction peaks at well-defined scattering angles. This phenomenon can be described by Bragg's law where the diffraction angle, 2θ , between the incoming x-rays and the outgoing diffracted x-rays is related to the x-ray wavelength, λ , and the crystallographic spacing, d_{hkl} of each

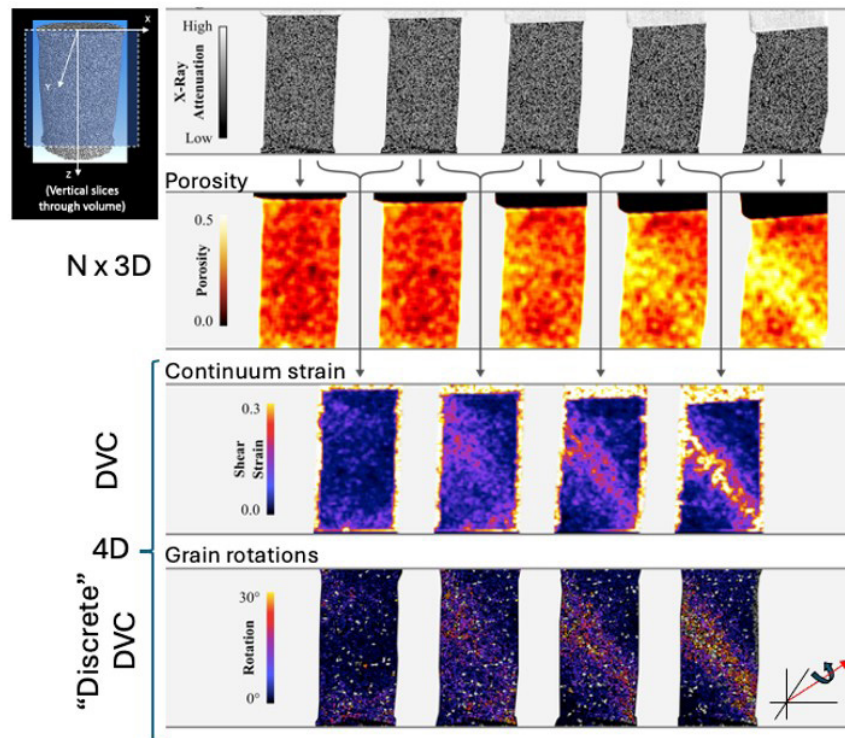


Figure 5. Example of the possible outputs from 3D and 4D image analysis of x-ray tomography data (top row) in terms of porosity field mapping, DVC-derived strains and grain kinematics from D-DVC. All analysis is in 3D but slices through the volumes are shown presenting representative components of the vector or tensor fields. (images adapted from Hall et al., 2012).

specific set of crystallographic planes in a crystal described by the Miller indices, h, k, l . For the case of single crystals, a few well-defined diffraction peaks can be measured in different directions relative to the incoming beam and sample orientation. Rotation of the sample can provide a set of peak measurements that can be interpreted to give the crystal orientation and the spacing of the crystallographic planes. In the case of a polycrystalline material consisting of many individual crystal grains, additional analysis is required to separate the signals from each grain; this is known as 3DXRD as described by, e.g., Poulsen (2008). For the case where the x-ray beam illuminates the full sample (e.g., Figure 6) this can be referred to as full-field 3DXRD (ff3DXRD). 3DXRD was first developed for metal samples but has also been applied to granular materials consisting of crystalline grains, as first demonstrated by Hall et al. (2011).

Figure 6 indicates a combined ff3DXRD and x-ray tomography workflow for granular mechanics studies. The sample is fully illuminated by a broad x-ray beam, which could involve full illumination of the sample diameter, and, with stitching, several vertically offset measurements can be used to cover the sample height. Rotation of the sample around the vertical axis provides a set of 2D diffraction patterns that can be analysed (e.g., by imageD11; Wright, 2020), to yield the individual diffraction patterns for each

grain. This allows the evaluation of the grains' centres of mass, crystallographic orientations, and unit cell parameters (which describe the crystallographic structure such as the lattice spacings). Repeating such measurements at different stages of loading can, thus, provide measurements of grain rotations (from the orientation change - usually at higher resolution than possible from tomography images) and grain-averaged strain tensors (from the changes in the unit cell parameters), as well as continuum strain fields through tessellation of the grain centre positions and their displacements. Figure 7 shows example results from Hall & Wright (2015) on the evolution of the granular strains as a function of loading for a small assembly of single crystal quartz grains showing the evolution of the strain tensors for different grains and 3D renderings of the strain tensors on the grains shapes derived from x-ray tomography images of the sample acquired concurrently with the ff3DXRD. Here it can be seen that some of the grains experience significant levels of strain and large differences in the maximum and minimum principal strain values with increasing macroscopic load, whilst other grains appear to remain largely unloaded. The loaded grains form a pathway across the sample suggesting force-chain like structures. It can also be noted that, upon unloading, some grains continue to show elevated strain levels indicating grain locking.

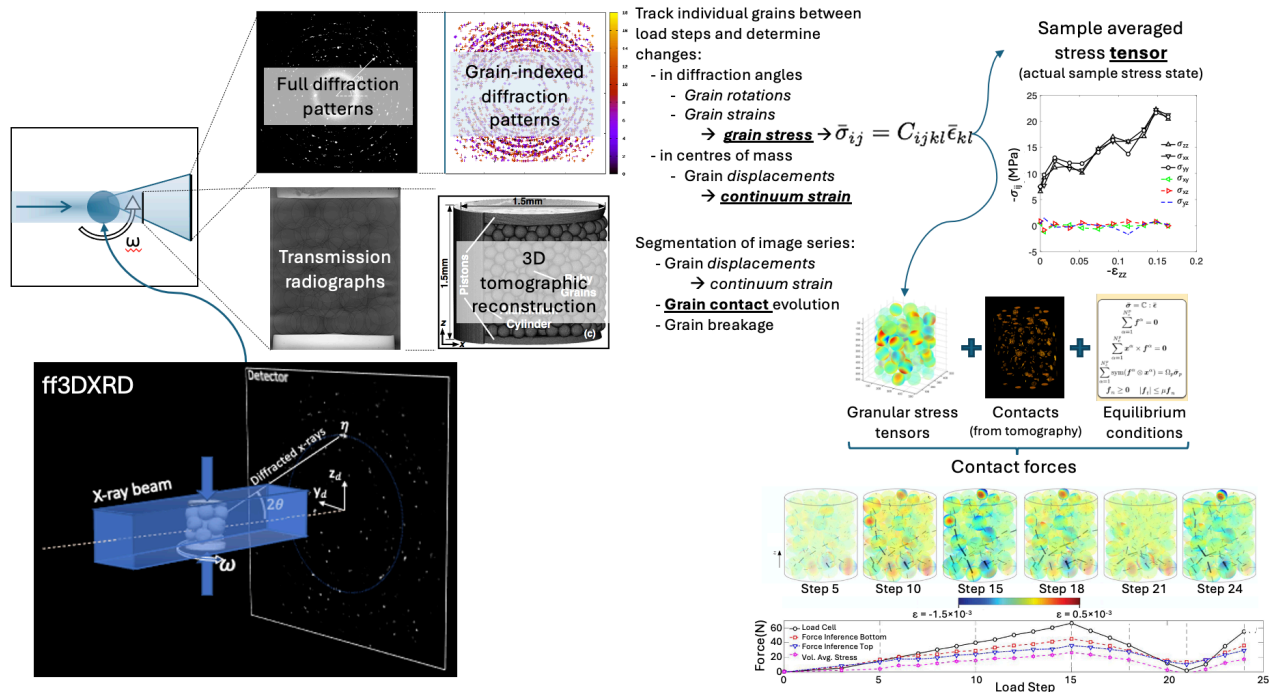


Figure 6. Schematic of the ff3DXRD process for granular mechanics in which a sample is fully illuminated across its diameter by an x-ray beam and either projection (radiography) or diffraction 2D images are acquired during rotation of the samples around the vertical axis. The radiography images provide 3D images of the sample structure. Diffraction peaks corresponding to the individual grains are separated out from the 2D diffraction images and used to determine the crystallographic unit cell parameters for each grain, which provide grain orientation and lattice spacings and their changes across load steps to yield grain rotations and strains. The grain strains, the known anisotropic stiffness tensor, and grain orientation provide the grain-averaged stress tensor, which with contact positions from the tomography images yield the contact forces. (Some images adapted from Hurley et al., 2016).

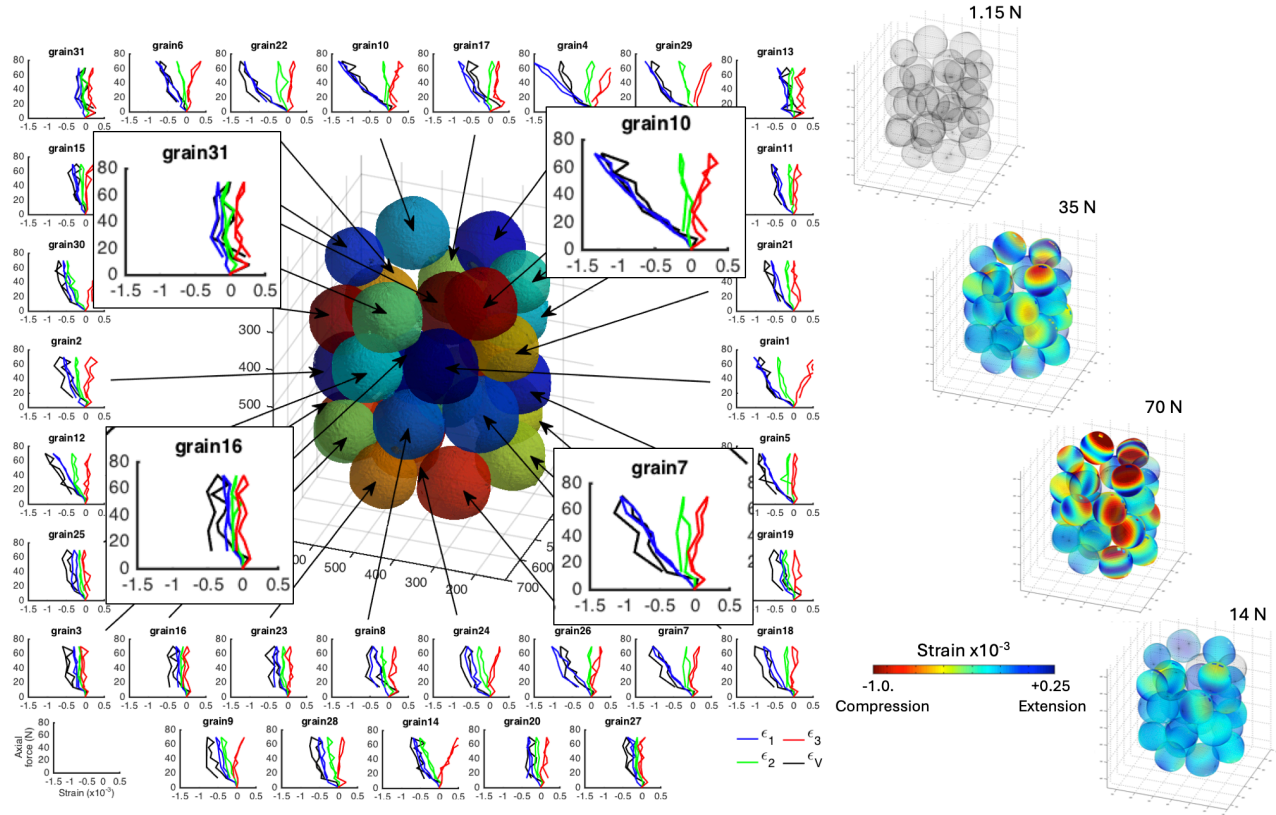


Figure 7. Summary of results from Hall & Wright (2015). Left: Rendering of the grains from the tomography data with colouring representing the individual grain label (X, Y and Z axis dimensions in voxels and the cubic voxels are $2.8 \mu\text{m}$ wide). Surrounding graphs show the strain tensor data, in terms of the principal and volumetric strains, for each of the grains, plotted against the axial force applied to the sample. The strains are relative to the respective grains' unit cell parameters at the initial load step (1.15 N axial force) and each graph has force (N) on the y-axis and grain strain ($\times 10^{-3}$) on the x-axis. Graphs for four representative grains, either low- or high-loaded, are expanded to show details. Right: 3D renderings of the grain shapes (from the tomography) with the strain ellipsoids rendered onto the surfaces with grain transparency proportional to the volumetric strain and colours indicating the strain magnitude in different directions.

The measured granular strains from 3DXRD are related to the elastic strains in the crystals. Therefore, Hooke's law can be used to determine the per-grain average stress tensors utilising the full anisotropic stiffness tensor and the grain orientations from the 3DXRD. These measurements can then be analysed on a grain-by-grain basis, in terms of the assembly behaviour, or averaged to give the sample-averaged 3D stress state. Consequently, ff3DXRD gives access to the full tensorial stress-strain response of the sample. For a large enough sample, this averaging can be applied to sub regions of the sample to investigate local variability in the stress-strain behaviour (e.g., Hurley et al., 2017). Furthermore, Hurley et al. (2016, 2017), developed a methodology based on the 3DXRD-derived granular stresses, granular contacts derived from segmented x-ray tomography images and satisfying equilibrium to infer the contact forces in the imaged granular network, as indicated in Fig. 6. This enables the preferential pathways observed in, e.g., Figure 7, to be characterised as force chains, which was only previously possible through photoelasticity-based (2D) experiments (e.g., Majmudar & Behringer, 2005; Hurley et al.,

2014) or with soft particles (e.g., Saadatfar et al., 2012; Brujic et al., 2003; Zhou et al., 2006).

Figures 8 and 9 show representative results from an in-situ triaxial compression experiment on single crystal ruby grains to demonstrate the information that can be derived from the combined ff3DXRD and tomography approach for granular mechanics studies. Figure 8 shows the experimental set-up and tomographic images of the sample evolution plus the force-displacement curve recorded at the sample boundary and the tensor stress-strain response derived from the ff3DXRD measurements. Whilst this experiment was designed to be a standard (albeit small) triaxial compression test, the confining membrane used was rather stiff leading to a more isotropic stress state in the sample, as indicated by the stresses derived from the ff3DXRD; without the 3DXRD data this would not have been detected. Furthermore, the ff3DXRD provides the sample stress tensor data plus tensor strain fields (from the grain displacements) and not just the axial values. These data also show quite a different sample response than that indicated by the boundary measurements,

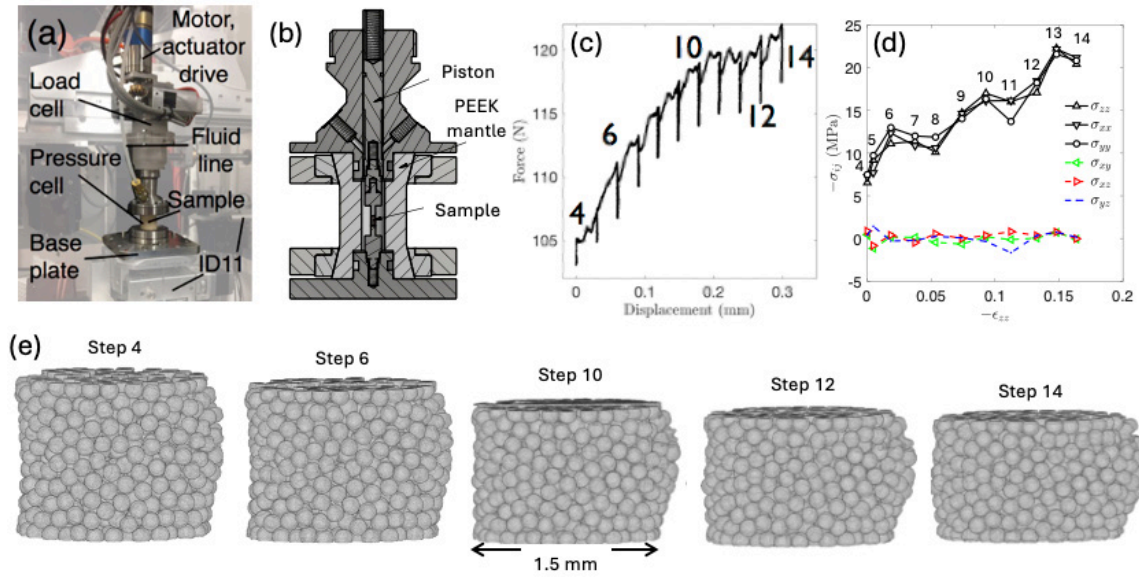


Figure 8. (a) Photograph of the mini triaxial experimental set-up at the ID11 beamline of the ESRF. (b) Cross section of the triaxial device design. The PEEK tube has an internal diameter around the sample of 2.8 mm and the sample diameter was 1.5 mm. (c) Force-displacement curve recorded by the external gauges of the device. (d) Stress-strain curve determined from the volume averaged grain stresses (tensor components) from 3DXRD and the displacements of the grains at the top of the sample versus those at the bottom, respectively. (e) Renderings of the x-ray tomography volumes at selected load steps showing the structural evolution of the sample.

with the latter showing a force plateau that is not really seen in the ff3DXRD stress-strain data.

The ff3DXRD results in Figure 9 reveal the local details of the sample response in terms of the continuum strain, grain rotations and grain stresses (with just the hydrostatic component of the stress tensor shown) plus the inferred contact force distributions. These figures indicate complex, non-homogeneous sample responses with the development of localised forces/stress in regions of the sample generally offset from regions of higher grain rotations. Further details and discussions can be found in Zhai et al (2019, 2022). The final part of Figure 9 shows 3D contact force histograms from the whole sample at different load steps. These are separated in weak and strong forces and the weaker forces are concentrated more towards the horizontal and stronger forces more towards the vertical (the macroscopic loading axis). Furthermore, evolutions of the contact force distributions can be seen such that, for example, between load step 6 and 8 plus between steps 10 and 12, the dominant orientation of the strong contact forces appears to rotate more towards the vertical and then “release”, which coincides with plateaus/slight-reduction in the stress before a stress increase. This indicates the evolution of a “contact force fabric”, which is significant in that contact fabric has been cited as being important in defining the mechanical response of granular systems (e.g., Oda, 1972), but the effective fabric will be the force carrying contact network.

Developments and applications of ff3DXRD granular mechanics studies have been presented for artificial quartz, ruby and real sand samples, which can be found in, for example,

Alshibli et al. (2013), Cil et al. (2014), Hall & Wright (2015), Hurley et al. (2016), Hurley et al. (2017), Zhai et al. (2019), Imseeh et al. (2020), Shahin et al. (2022), Zhai et al. (2022) and Hurley et al. (2023). Key hurdles to greater exploitation include sample size (which is, in part, limited by the size of the x-ray beam available at synchrotron facilities; most studies have so far been limited to about 1.5 mm diameter samples, limiting the number of grains that can fit in the FoV), as well the number of grains that can be effectively analysed without their diffraction patterns overlapping (which would prevent their separation and individual grain analysis) and the crystal quality of the grains that can be handled (which is also coupled to the number of grains). The latter refers to the fact that the greater the variation in the crystal structure within each individual grain leads to more spread-out diffraction spots, making it harder to separate the diffraction from different grains due to spot overlap as well as meaning spot positions (diffraction angles) are harder to define and track between load steps.

4. Intragranular stress/strain

The ff3DXRD approach described above provides valuable new insights into force transfer and stresses in granular materials under load, including spatial variability and localisation, e.g., force chains. Such spatial variability is not, however, limited to inter-granular variation. The stress, strain and crystal orientation fields within individual grains will also vary due to loading, especially as each grain is loaded by

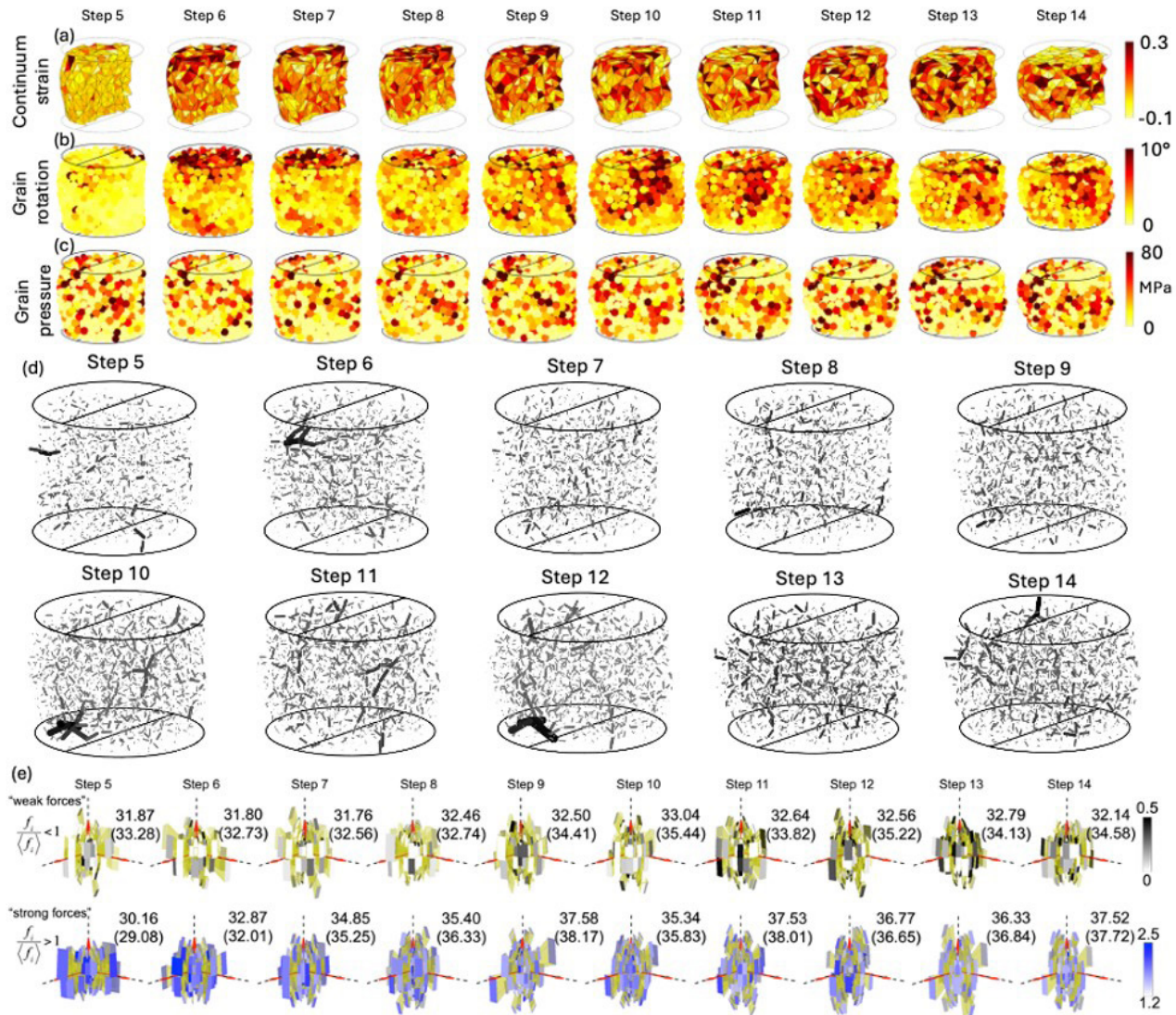


Figure 9. Results from 3DXRD analysis of a system of ruby grains under triaxial compression loading. (a) Incremental continuum strains from triangulated grain centres and their displacements (compression is positive) – the volume cut to reveal the inner structure of the field. (b) Incremental grain (crystallographic axes) rotations. (c) Grain pressures per grain, from the trace of the 3DXRD-derived stress tensor. (d) Inferred contact forces for steps 5-14 (line thickness indicates the contact force magnitude). (e) Spherical histograms of weak and strong contact forces, respectively (excluding boundaries forces) were defined using angular bins directions and the averaged force for each bin is indicated by the colour of the end of each bar. Red arrows indicate x-axis, y-axis and z-axis with lengths corresponding to force distribution probabilities of 0.02, 0.02, 0.005, respectively. The numbers indicate the average inclination of the contacts and force-weighted contact orientations (in parentheses). C. Zhai is acknowledged for producing the images.

multiple contacts. Furthermore, natural variations in the crystal parameters with or without applied loads, are likely in natural sand grains. To investigate the internal granular structure and stress/strain fields requires an alternative approach that can yield the fields of these properties within each grain. This can be achieved by scanning a sample across a small x-ray beam by vertical and horizontal translations (in Δy and Δz in Figure 10) such that each recorded 2D diffraction pattern represents the diffraction along a line integral defined by the intersection of the beam and the sample. If at each Δy position for each Δz height the sample is rotated over 180°, tomographic principles can

be used to reconstruct the unit cell parameters for each voxel within a single Δz slice; this can be referred to as scanning-3DXRD (s3DXRD) (Henningsson et al., 2020; Henningsson, 2023). Repeating this procedure over a range of Δz positions allows a full 3D volume to be defined in which each voxel contains the crystallographic parameters for that point in the grain assembly. Simplifications of the calculations allow the crystallographic strain to be more easily determined, but the full unit cell parameters can also be defined on a per-voxel basis (Henningsson et al., 2020; Henningsson, 2023). As with the strains from ff3DXRD, the voxel strains from s3DXRD can be

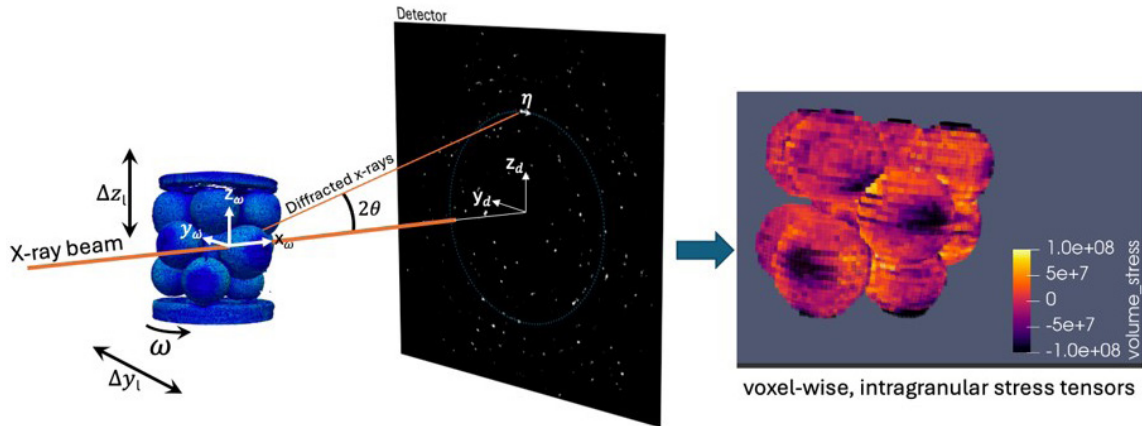


Figure 10. Left: schematic of the s3DXRD experimental approach where the sample is scanned through a small x-ray beam over a range of Δy_l positions with 180° sample rotation at each position to record a series of 2D x-ray diffraction images that can be treated to yield voxel-wise tensorial measures of the crystallographic strain/stress. Right: example of a s3DXRD-derived 3D stress field (volumetric component of the stress tensors shown) for an assembly of 12 single crystal quartz-grains loaded under confined 1D compression (upper and lower pistons and the side walls; not shown in rendering) (Vestin, 2022). The sample width is about 1.4 mm and the grains are about 300 μm in diameter.

interpreted in terms of the local stress tensor through Hooke's law and the 3D stiffness tensor of the grain material rotated to the local crystallographic orientation. Figure 10 outlines the experimental measurement set-up for an experiment at ID11 at the ESRF and an example of the output from the s3DXRD in the form of the volumetric components of the voxel-wise defined stress tensors in an assembly of single-crystal quartz grains loaded in confined uniaxial compression. The rendering indicates regions of compression in the grains where they contact with the boundaries and at the granular contacts, as well as dilating regions between the contacts.

The potential for the s3DXRD approach for improved understanding of granular systems is significant in terms of gaining access to the intra-granular stress fields and how, for example, these evolve towards eventual grain failure and for a more detailed understanding of contact mechanisms (particularly if combined with high-resolution tomography imaging), all in the context of a multi-grain 3D system. Further results will be presented in upcoming papers.

5. Imaging of granular dynamics

The preceding examples have demonstrated the increasing detail that it is possible to achieve through advanced 3D x-ray measurement techniques, but are all limited to quasi-static experiments. X-ray tomography can now be run for small samples at synchrotron facilities with time resolutions in the order of a second (the resolution is defined by how long it takes to acquire the data to generate the 3D image). This means that continuous loading experiments can be run if the loading rate is slow enough to provide less than about half a voxel width displacement during a scan. However, many processes in granular mechanics are more dynamic and 1 tomography/s is too slow to capture such phenomena

and faster tomographic imaging requires fast rotations (e.g., García-Moreno et al., 2019), which can influence the sample response. An alternative is to limit the imaging to 2D, which has previously been done with high-speed photography of a sample surface (e.g., Borg et al., 2013), but viewing the sample surface leaves uncertainties about what is happening in the bulk of the sample versus surface interaction effects. High-speed x-ray radiography provides the possibility to follow sample evolution also in the bulk of a sample, albeit in 2D and with averaging through the sample thickness (e.g., for granular materials, Royer et al., 2005, 2007; Gupta et al., 2021; Méjean et al., 2022; Thakur et al., 2024).

Figure 11 shows an example of high-speed x-ray radiography (100 000 frames/s) of a gravity-driven impact of a steel ball into a dry granular bed (glass beads) acquired at the ID19 beamline of the ESRF. The top row of images shows a series of radiography images of the impact 10 frames (0.1 ms) apart in time. From these images, the impact of the ball and the resulting cratering and ejector can be observed. The full radiographic sequence provides a 3D volume defined by the horizontal (x) and vertical (y) axes plus the time (t) axis. This volume can be sliced along the time axis to give a (t, y) image in which gradients indicate the vertical velocity of the different components of the system (Figure 11, middle row). Consequently, the trajectory and, thus, the velocity of the impacting ball and its deceleration can be determined from the (t, y) gradient (as indicated in the lower image). This (t, y) image can be enhanced by, for the whole time-sequence, subtracting the preceding image to provide an incremental difference image (i.e., $I_{diff}(t) = I(t) - I_{(t-1)}$), which highlights changes in the image between time frames. The bottom row of Figure 11 shows such an I_{diff} image and the propagation of a shockwave through the granular bed from the impact point can be clearly seen and its velocity measured from the

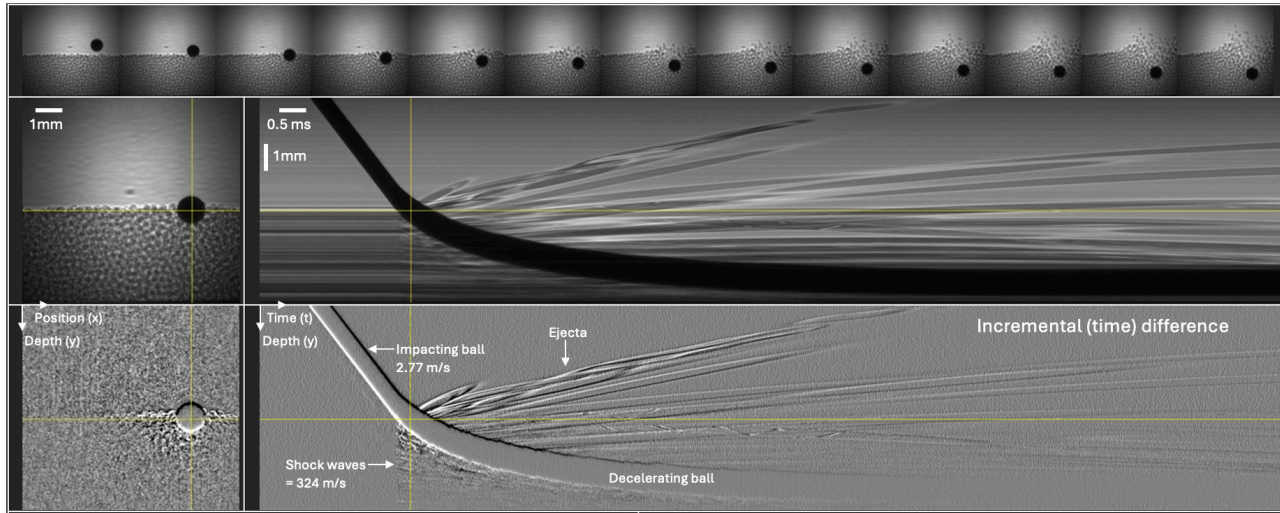


Figure 11. Fast x-ray radiography of the impact of a 3 mm diameter iron ball traveling at about 2.77 m/s into a bed of 0.3 mm diameter glass beads (100 kHz image acquisition, 8.39 μ s shutter speed and 20 μ m pixel size). Top: snapshots from image sequence over 1.2 ms (every 10th image shown – interval 0.1ms). Middle: x-y (radiography) image for a single time frame and y-t slice from the 3D (x,y,t) volume. Bottom: the same slices extracted from the incremental difference $I_{diff}(x,y,t)$ volume. The initial propagating wave (about 324 m/s) can be seen below the first point of impact. Images acquired at ID19 at the ESRF.

gradient. The shockwave velocity is estimated to be about 324 m/s, although it is noted that the time resolution possible with the 100 kHz imaging plus the given pixel size and FoV, limit this quantification.

The described high-speed radiography measurements have the potential to reveal new details towards understanding dynamic processes such as impacts into granular beds, but the limitation to 2D will always require adapted experiments and appropriate interpretation considering the 2D boundary conditions, even if some 3D information can be derived from dynamic 2D imaging and associated static 3D images (e.g., Gupta et al., 2021; Kuwik et al., 2024). The possibility to perform high-speed 3D x-ray imaging of dynamic processes without sample rotation is currently a key topic of research, but requires further development of both acquisition methodologies and data analysis approaches (e.g., Henningsson & Hall, 2021; Villanueva-Perez, et al., 2023, Asimakopoulou et al., 2024).

6. Summary and conclusion

This paper has provided an overview of some experimental developments and results from the last 15-20 years that have provided new opportunities for enhanced experimental description of deformation processes in granular materials. This overview has described, primarily, work of the author with different collaborators and is not a comprehensive view of the field but aims to give a snapshot of some of the new possibilities in experimental granular mechanics. The described experimental approaches exploit x-ray imaging and 3D x-ray diffraction to describe granular structures and their evolution under different mechanical loads at the granular assembly,

inter-granular and intra-granular scales, including continuum strains and granular strains/stresses. The different methods provide access to different structural scales and their use in combination can provide important multi-scale insight. Continuing developments are pushing the limits in terms of temporal and spatial resolutions, as well as possible FoVs, which are all essential to enable experiments of increasing relevance in terms of granular systems with reduced boundary influences. The consequence of the increased resolution and FoV size is the increase in data volumes, which will require even more efficient data analysis algorithms, but will also offer great possibilities for our understanding of evolving granular systems.

Acknowledgements

The work described in this paper is a summary of a number of projects over the past 20 years involving many collaborators. In particular, E. Andò, G. Viggiani, N. Lenoir and J. Desrues (Laboratoire 3SR, Grenoble) contributed greatly to the developments described concerning the 4D x-ray microtomography imaging of in-situ triaxial compression tests on sand (and continue to make important advances in this area). Parts of Figure 2 were originally created by E. Andò. The 3DXRD developments would not have been possible without the support and expertise of J. Wright (ID11, ESRF). The described ESRF beamtimes for 3DXRD at ID11 were also supported by M. Majkut and W. Ludwig (ESRF). R. Hurley (John Hopkins University) was key to the developments in force transfer analysis from the 3DXRD experiments and has since driven forward the use

of 3DXRD in geomechanics, as described, for example, in the cited references. C. Zhai (John Hopkins University) is acknowledged for providing the images in Figure 9. The dynamic x-ray imaging experiment at ID19 was run by J. Åhlund (Scandiflash) with E. Larsson (Lund University) and supported by B. Lukic and A. Rack (ID19, ESRF). A. Henningsson (Lund University) has driven the development of s3DXRD and the results shown were produced by P. Vestin (Lund University), both under supervision of the author. J. Engqvist (Lund University) was key to the development of many of the described experimental set-ups. The described ESRF experiments were performed under granted beamtimes, ma280, ma828, ma1216, ma1913, ma2665, ma3373, ma4200, ma4493 and in1126. Some of the work was performed within the framework of funding from Vetenskapsrådet projects (#2012-04234 and #2015-04398).

Declaration of interest

The author has no conflicts of interest to declare.

Data availability

The raw data from the ESRF experiments in this paper are available through the ESRF data portal.

List of symbols and abbreviations

2D	two dimensional
2θ	diffraction angle in Bragg's law multiplied by 2,
3D	three dimensional
3DXRD	three-dimensional X-ray Diffraction
4D	four dimensional
d_{hkl}	crystallographic plane spacing for planes defined by the Miller indices h, k, l
dx, dy, dz	displacements in x,y and z directions from DVC
h, k, l	Miller indices describing different planes in a crystal lattice
q	deviatoric stress in triaxial compression testing
s3DXRD	scanning Three-dimensional X-ray Diffraction
x, y, t	2D spatial axes (x,y) plus time axis (t)
DVC	digital Volume Correlation
D-DVC	discrete Digital Volume Correlation
ESRF	European Synchrotron Radiation Facility
FoV	Field of View
Ff3DXRD	Full-field Three-dimensional X-ray Diffraction
I_{diff}	difference image derived from $(I_t - I_{t-l})$
I_t	image at time t
I_{t-l}	image at time $t-l$
SPAM	Software for Practical Analysis of Materials
$\Delta y, \Delta z$	motor translations used in scanning 3DXRD (y – horizontal, z – vertical)
λ	x-ray wavelength,

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Emerging technologies and advanced analyses for non-invasive near-surface site characterization

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Keywords

Shear wave velocity
Rayleigh waves
DAS
Damping
FWI
Machine learning

Abstract

The in-situ small-strain shear modulus of soil and rock materials is a parameter of paramount importance in geotechnical modeling. It can be derived from non-invasive geophysical surveys, which provide the possibility of testing the subsurface in its natural and undisturbed condition by inferring the velocity of propagation of shear waves. In addition, for soil dynamics and earthquake engineering applications, the small-strain damping ratio plays a relevant role, yet its estimation is still challenging, lacking consolidated approaches for its in-situ evaluation. Recent advancements in instrumentation, such as distributed acoustic sensing (DAS), combined with advanced analysis methodologies for the interpretation of seismic wave propagation (e.g., machine learning and full waveform inversion), open new frontiers in site characterization. This paper presents and compares some advanced applications of measuring 1D and 2D variations in shear wave velocity and attenuation in-situ with reference to a specific case history.

1. Introduction

Non-invasive subsurface imaging techniques utilizing seismic wave propagation have garnered escalating attention in recent decades owing to their remarkable cost-effectiveness compared to conventional invasive site characterization methods and their potential to cover large areas. These imaging techniques primarily focus on capturing two crucial soil parameters of particular interest in geotechnical engineering: the small-strain shear wave velocity (V_s) and the small-strain damping ratio in shear (D_s). V_s is directly related to the small-strain shear modulus (G_0 or G_{max}), representing the stiffness of the soil, while D_s quantifies the soil internal energy dissipation at low strains. This paper provides a review of some of the latest advancements in noninvasive subsurface imaging techniques for the estimation of V_s and D_s and their practical application at a well-characterized case history site called the Hornsby Bend site in Austin, Texas, USA. The subsequent paragraphs highlight the importance of V_s and D_s in geotechnical engineering applications and discuss the challenges and advancements in the noninvasive techniques developed for their estimation.

V_s and D_s play a key role in evaluating the response of soil deposits to both general dynamic loading and ground

motion amplification caused by earthquakes. In regard to seismic loads, V_s and D_s are especially important parameters to quantify when the soil is subjected to low-intensity shaking (e.g., Tao & Rathje, 2019; Rodriguez-Marek et al., 2021; Fernandes et al., 2023). For instance, Rodriguez-Marek et al. (2021) observed that D_s is the most influential parameter at high frequencies, with an impact even more relevant than V_s , whereas the low-frequency soil response is mainly affected by V_s of shallow layers. According to Foti et al. (2021), D_s has a substantial influence on the seismic amplification in deformable soil deposits. However, this influence is less pronounced under conditions of strong shaking that strain the soil sufficiently to induce nonlinear soil behavior. Additionally, in a site-specific study, Foti et al. (2021) compared the amplification resulting from the epistemic uncertainty in D_s with that caused by V_s and the nonlinear soil behavior modeled using modulus reduction and damping (MRD) curves. They found that a change in D_s leads to a significant variation in amplification compared to the overall variability of the results. This effect is particularly relevant at high frequencies and near the resonance peak, even under higher seismicity conditions.

Furthermore, studies such as Kouroussis et al. (2011), Papadopoulos et al. (2019), and Santos et al. (2016) highlight the importance of V_s and D_s in assessing vibrational impact.

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The small-strain and anelastic properties of the soil significantly influence the energy transmission from the source, its propagation, and the resulting motion at the receiver. These properties directly impact the amplitude and frequency content of the vibrations. For instance, Lombaert & Degrande (2003) and Lombaert et al. (2006) observed that when dealing with rail traffic as the noise source, uncertainties in defining the spatial variation of dynamic soil characteristics lead to poor agreement between simulated and experimental data. Rail vehicles mainly generate high-frequency signals, reaching up to 200 Hz (Pyl, 2004). Due to their short wavelengths, these signals are highly sensitive to local heterogeneities in the soil deposit. Moreover, Schevenels (2007) demonstrated the impact of uncertainties in D_s and V_s on free-field wave propagation, whose variability exponentially increases with the frequency, especially at large distances from the source.

The small-strain dynamic soil properties also play an important role in soil-structure interaction problems, where the deformability of the supporting soil impacts both the fundamental period and the energy dissipation of the system (Veletsos & Meek, 1974). Among these properties, V_s holds particular significance as it directly governs soil deformability, which is a key factor in this phenomenon (Veletsos & Meek, 1974). Energy dissipation, on the other hand, arises from various sources, including inelastic phenomena within the structure, hysteretic dissipation in the soil deposit, and radiation damping. Radiation damping refers to the geometric effect of waves radiating from the foundation-soil interface, carrying energy away from the foundation system as they propagate outward. It is commonly assumed that radiation damping represents the primary mechanism for energy dissipation, especially at small strains. However, Martakis et al. (2017) observed through centrifuge tests that significant dissipation also occurs due to hysteretic effects linked to the intrinsic dissipation of the soil, even at small strains.

The preceding paragraphs underscore the importance of accurately estimating V_s and D_s for dynamic modeling of seismic and general vibration problems. Traditionally, these small-strain soil properties have been estimated through laboratory testing or empirical relationships (e.g., Darendeli, 2001; Menq, 2003; Ciancimino et al., 2020). However, their in-situ estimated values often deviate from those obtained in the laboratory. This deviation can be attributed to the disturbances that inevitably occur during the acquisition of soil samples for laboratory testing, leading mostly to a reduction in soil stiffness (e.g., Stokoe & Santamarina, 2000). Additionally, at the site scale, complex wave propagation phenomena (e.g., wave scattering) result in additional energy dissipation beyond material dissipation, which cannot be captured accurately through laboratory tests (e.g., Stewart et al., 2014; Tao & Rathje, 2019). Geophysical field measurements offer the advantage of estimating the ground response in its natural state thus mitigating the uncertainties associated with sample disturbance and scale effects often encountered when working with rock-like materials. Overall, non-invasive

techniques investigate a large volume of the medium, whose size depends on the array geometry (Comina et al., 2011; Passeri, 2019), providing parameter estimates at a scale compatible with those of geotechnical systems. Furthermore, some design criteria in geoen지니어ing directly rely on these testing procedures. For instance, rail infrastructure design requires the train speed to be smaller than a “critical” speed, corresponding to the Rayleigh phase velocity, V_R , of the underlying medium (e.g., Connolly et al., 2015). Indeed, at higher speeds, the amplitude of track vertical deflection dramatically increases (Timoshenko, 1927; Krylov, 1995; Madshus & Kaynia, 2000; Madshus et al., 2004). For this reason, the rail operational speed is often determined through a dispersion diagram, which involves the experimental V_R at various frequencies as an input parameter, determined through surface wave-based geophysical techniques (Thompson, 2009). Finally, the field-based small-strain estimates of V_s and D_s can be used in conjunction with laboratory tests to map and un-normalize the nonlinear mechanical response of soil, which is most easily characterized in the lab at strains ranging from moderate to large. This approach enables the development of advanced numerical models or simplified procedures for evaluating the behavior of geotechnical systems subjected to either static or dynamic loading (e.g., settlement of shallow foundations and seismic site response) across a broad range of induced strains.

Given the importance of accurately estimating V_s and D_s in situ, the field of geophysical imaging based on seismic wave propagation is continuously advancing, introducing new innovations aimed at increasing imaging resolution and reducing uncertainty. These innovations encompass improvements in both data acquisition systems (DAQ) and imaging methodologies. A notable recent development in data acquisition is the utilization of distributed acoustic sensing (DAS) for seismic wave measurements. DAS offers unprecedented spatial resolutions (on the order of meters) and length scales (on the order of tens of kilometers), surpassing conventional sensing technologies (Soga & Luo, 2018). Further details on DAS technology, which is employed as the DAQ for most of the 1D and 2D imaging techniques discussed in this paper, are provided in a separate, dedicated section later in this paper. This paper also highlights some of the significant advancements in imaging techniques, encompassing both 1D and 2D approaches. In the field of 1D imaging, two notable developments have emerged. Firstly, the utilization of DAS as the DAQ for 1D multichannel analysis of surface waves (MASW). Secondly, the joint estimation of phase velocity and phase attenuation data within a 1D MASW test setup, achievable using either geophones or DAS as the DAQ. This paper demonstrates the pioneering use of DAS for jointly characterizing the stiffness and dissipative parameters of a soil deposit. In the realm of 2D imaging, the presented techniques comprise the application of 2D MASW using DAS data, the utilization of machine learning for 2D imaging, and the use of full waveform inversion (FWI) with DAS data.

All of these 1D and 2D imaging techniques were successfully applied at a well-characterized case history site called the Hornsby Bend test site, providing a valuable opportunity to compare and discuss their results.

The subsequent sections of the paper are organized as follows. First, a brief overview of DAS technology is presented, highlighting its key features and capabilities. Following that, the testing conducted at the Hornsby Bend site is discussed. The paper then delves into the advancements in 1D and 2D imaging techniques in regard to measuring V_s and D_s at the Hornsby Bend site. Then, a discussion about the advantages and disadvantages of each technique is presented. Lastly, a comprehensive subsurface imaging experiment conducted at the Newberry site in Florida, USA utilizing some of the latest sensing technologies is showcased.

2. The DAS technology

DAS is an innovative technique that transforms fiber-optic cables into a distributed array of ground motion sensors (Cox et al., 2012; Yu et al., 2019). This rapidly evolving technology allows for the simultaneous collection of high-resolution data with small channel separations (e.g., 1-m) over long arrays spanning kilometers, making it highly suitable for near-surface imaging applications. Unlike traditional geophones that measure particle velocity ($\partial u/\partial t$) at discrete points along the acquisition array (Figure 1a), DAS records the spatially-averaged axial strain $e(r,t)$ induced on the fiber-optic cable by the passing wavefield. Here, “ r ” represents the location coordinate and “ t ” represents the time instant. When properly coupled with the ground, the passage of mechanical waves generates an axial strain in the fiber-optic cable that coincides with the horizontal, in-line strain $\epsilon(r,t)$ in the ground. An interrogator unit (IU) reads the consequent shift in phase lag of a laser pulse traveling in the cable, induced by the variation in the length of the cable.

Thus DAS measurements represent the variation in phase difference over a reference length $2g$, called gauge length, around the investigated location, from which the average strain $e(r,t)$ is derived (Figure 1b; Grattan & Sun, 2000). The resulting average strain at each measurement point can be linked with the displacement $u(r,t)$, as it equals the difference of the radial displacement at two points separated by a distance equal to the gauge length (Mateeva et al., 2014; Bakku, 2015; Jousset et al., 2018; Vantassel et al., 2022b):

$$e(r,t) = \frac{1}{2g} [u(r+g,t) - u(r-g,t)] \quad (1)$$

It is noteworthy that the gauge length is not necessarily linked with the channel separation (i.e., the distance between two subsequent measurement points). The gauge length plays a crucial role in the spatial sampling quality, as it limits the range of investigable wavelengths. In the simple scenario of a spatially harmonic radial displacement field, it is demonstrated that the averaging procedure (i.e., the mapping from $\epsilon(r,t)$ to $e(r,t)$) is equivalent to applying a lowpass filter (such as a sinc filter) in the wavenumber domain (e.g., Bakku, 2015). This filtering tends to attenuate wave components with shorter wavelengths λ , particularly affecting the high-frequency components of the Rayleigh wavefield (Figure 1c). Increasing the gauge length results in a greater loss of information for short-wavelength data, although it improves the overall signal quality and signal-to-noise ratio (e.g., Bakulin et al., 2020). Therefore, selecting an optimal gauge length is a critical task that should consider various factors such as the acquisition setup, source quality, magnitude of incoherent noise, and the desired range of investigated wavelengths. A possible strategy to overcome the resolution issues induced by spatial averaging is to conduct multiple measurements where the gauge length is modified at each

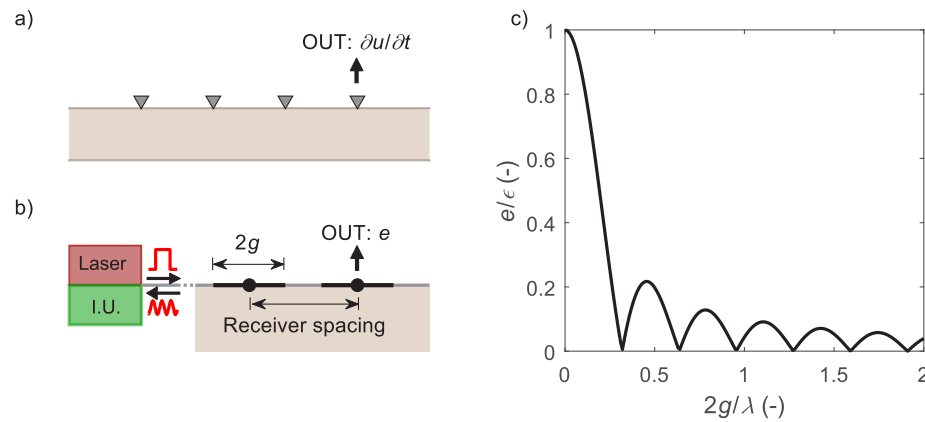


Figure 1. a) Schematic model of an acquisition system based on geophones, wherein the output (labeled as “OUT”) is the particle velocity $\partial u/\partial t$; b) Schematic model of the DAS system, where a source generates a laser pulse which is then interpreted by an interrogator unit (labeled as I.U.) and the output (labeled as “OUT”) is the average strain e [modified from Bakku (2015)]; c) Amplitude response in terms of e/ϵ ratio, as a function of the wavelength-normalized gauge length $2g/\lambda$.

step (Bakku, 2015). It is worth noting that interpreting DAS-recorded data, which provides a spatially averaged measure of the strain field, is slightly more complex compared to conventional acquisition devices. Nonetheless, DAS is increasingly being used for invasive geophysical tests (e.g., Mateeva et al., 2014; Kuvshinov, 2016), ambient noise vibrations (e.g., Hornman et al., 2013; Freifeld et al., 2016; Yavuz et al., 2016; Ajo-Franklin et al., 2017) and MASW testing (Galan-Comas, 2015; Lancelle, 2016; Costley et al., 2018; Song et al., 2018; Vantassel et al., 2022b).

3. Innovative processing techniques and DAS

3.1 Reference dataset: Hornsby Bend

The Hornsby Bend site (HB, 30°13.918'N, 97°38.631'W – in the WGS84 Datum) is located on the outskirts of Austin, Texas, USA. This site has been the subject of extensive invasive and noninvasive site characterization studies in recent years. The noninvasive testing conducted at the site and utilized in this paper involved the deployment of two parallel 200 meter-long fiber-optic cables (refer to Figure 2a), one manufactured by NanZee and the other by AFL. These cables were interconnected at the far end of the array by splicing the NanZee and AFL cables together, enabling simultaneous recording on both cables. On the near-side of the array, the NanZee cable was connected to an OptaSense ODH4 IU, while the AFL fiber was properly terminated to minimize end-reflections. The ODH4 IU was configured with the gauge length

and channel separation set to 2.04 m and 1.02 m, respectively. These values represent the shortest gauge length and channel separation allowed by the ODH4. Consequently, the DAS recorded waveforms represent an average response over the 2.04 m gauge length surrounding each channel location (i.e., every 1.02 m). The IU sampling frequency, or ping rate, was set at 100 kHz. After acquiring the data, the raw measurements underwent down sampling to 1 kHz and high-pass filtering above 3 Hz to remove low-frequency artifacts linked with laser drift and static strains.

Two geophone arrays were also deployed in conjunction with the fiber-optic cables: a vertical geophone array and a horizontal geophone array oriented in line with the DAS fiber optic cables. Each array comprised 48 geophones, uniformly spaced at 2 m intervals, resulting in a total array length of 94 m as shown in Figure 2a. To capture the geophone signals, four interconnected 24-channel Geometrics Geode seismographs were utilized, enabling simultaneous recording from both the vertical and horizontal geophone arrays. All signals were acquired using a sampling rate of 1 kHz. The geophone array and DAS fiber-optic cables were employed to simultaneously record actively-generated surface waves from various sources. These sources encompassed highly-controlled vibroseis shaker trucks and more variable impact sources. The vibroseis sources comprised the three-dimensional shaker, T-Rex, and the highly-mobile one-dimensional shaker, Thumper, both from the Natural Hazards Engineering Research Infrastructure at the University of Texas at Austin (NHERI@UTexas) experimental facility (Stokoe II et al., 2020). Additionally, an

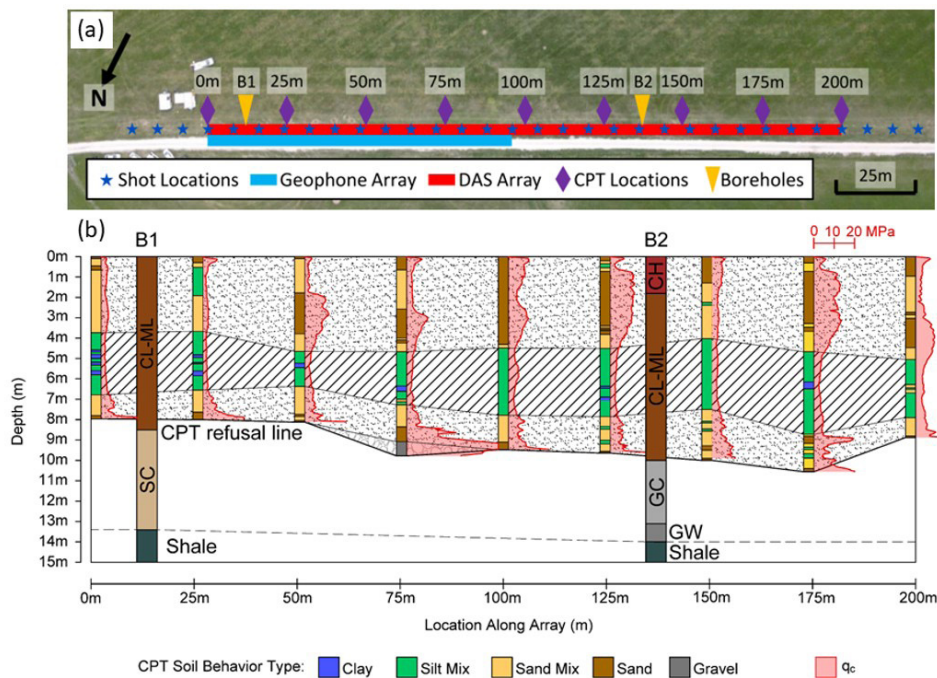


Figure 2. a) Aerial view of the Hornsby Bend test site showing the locations of CPT tests and boreholes as well as the DAS fiber optic cable, the geophone array, and the vibroseis shot locations; b) Geological cross section.

instrumented 5.4 kg sledgehammer from PCB Piezotronics was used as an impact source. T-Rex was utilized for shaking in all three directions: vertically, horizontally in-line, and horizontally cross-line. It generated a 12-second chirp signal with frequencies linearly swept from 3 to 80 Hz, providing a maximum force output of approximately 270 kN in the vertical direction and 130 kN in the horizontal directions. Thumper was used to produce a 12-second chirp signal in the vertical direction with frequencies linearly swept from 5 to 200 Hz, offering a maximum force output of approximately 27 kN in the vertical direction. For the vibroseis sources, three sweeps were conducted at each source location, whereas five impacts were performed using the sledgehammer. These sources were used at various locations around the site, however, for the purposes of this paper, only the source locations along the linear array alignment will be discussed. A significant portion of the noninvasive dataset from testing conducted at the Hornsby Bend site is accessible to the public through DesignSafe-CI (Vantassel et al., 2022a).

In addition to the noninvasive testing, nine cone penetration tests (CPT) soundings were carried out at 25 m intervals along the fiber-optic cable, covering the range from 0 to 200 m, as indicated in Figure 2a. These CPT soundings were performed on three different dates, spaced out over a span of approximately eight months. Yust et al. (2022) utilized the soil behavior type index value (Ic) method developed by Robertson (2009) to analyze the collected CPT data. Based on their analysis, they developed the subsurface cross section shown in Figure 2b down to the depths of CPT refusal. According to Yust et al. (2022), the site comprises three distinct layers above the depth of CPT refusal. These layers include a shallow granular layer (between depths of approximately 0 to 4 m) consisting of sand and sand mix, an intermediate cohesive layer (between depths of approximately 4 to 7 m) comprising clay and silt, and a deeper granular layer (between depths of approximately 7 to 10 m) composed of sand and sand mix. The depth of CPT refusal along the cable varied between 7.96 and 10.56 m, with an average depth of 9.15 m. In this study, the original cross-section developed by Yust et al. (2022) has been

further extended from its original depth of approximately ten meters down to 15 m, as shown in Figure 3b. This extension enables the depiction of the shale layer depth, which was determined using data acquired from two recently drilled boreholes at the Hornsby Bend site. Both the first borehole (B1) positioned 12.5 m from the starting point of the geophone array, and the second borehole (B2), located 137.5 m away (refer to Figure 2b), confirmed the existence of a shale layer at an approximate depth of 13.5 m beneath the ground surface. Furthermore, seismic downhole (DH) testing was conducted in borehole B1, reaching a depth of 24 m with a receiver interval of 1 m, which led to the identification of four distinct velocity layers in the subsurface (discussed later in the paper).

3.1.1 1D MASW processing for the joint estimation of S-wave velocity and damping ratio

A promising technique for obtaining in-situ estimates of V_s and D_s relies on MASW (Nolet & Panza, 1976; McMechan & Yedlin, 1981; Gabriels et al., 1987; Park et al., 1999; Foti, 2000). This technique relies on the measurement of propagation characteristics of surface waves (typically, Rayleigh waves) and the testing procedure can be divided into three main steps:

- Data acquisition: a waveform generated by an artificial source is recorded along a linear array of sensors (typically, geophones) on the ground surface.
- Data processing: based on variations of phase lag and amplitude of surface waves along the array, the corresponding propagation speed (i.e., the phase velocity V_R) and spatial attenuation of the amplitude (i.e., the phase attenuation α_R) are derived, as a function of the frequency. The frequency-dependence of V_R and α_R is a combined effect of geometric dispersion, which results from the variation of mechanical properties with depth, and intrinsic dispersion, due to the constitutive behavior of linear viscoelastic media.
- Inversion: the V_s and the D_s profile with depth are obtained through an inversion scheme, where a

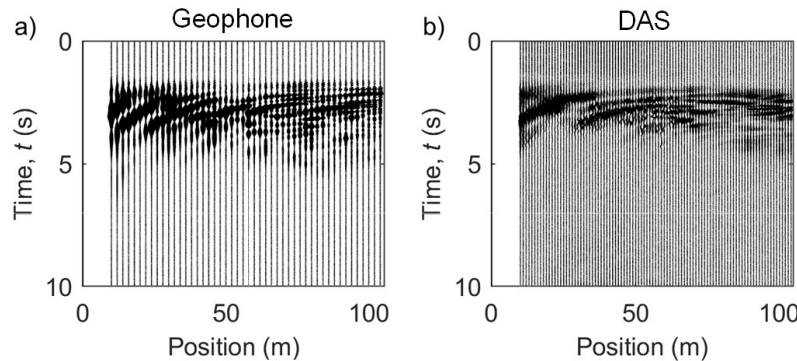


Figure 3. Recorded data at the Hornsby Bend site: a) Time histories of particle velocity recorded by the geophone array; b) Time histories of average radial strain recorded by DAS. Data refer to the wavefield generated from the active source located at an offset equal to 10 m.

theoretical soil model is calibrated to match the experimental V_R and α_R .

In MASW testing, the main advantage of the DAS technology with respect to conventional acquisition devices is the enhanced spatial resolution using low-cost instrumentation. Indeed, DAS allows for dense spatial sampling of the wavefield, potentially along a broad array extent. Conversely, achieving the same spatial resolution with ordinary receiver arrays would require a large number of sensors, entailing severe economic and logistic issues. Furthermore, high quality measurements can be even obtained from conventional fiber-optic cables, that are not specifically designed for seismic investigation and already deployed in the ground (e.g., the telecommunication infrastructure; Jousset et al., 2018). Therefore, the per-channel cost is moderately low. Applications of this technology to MASW surveys demonstrated that the dispersion estimates well match those obtained from geophone measurements (Galan-Comas, 2015; Vantassel et al., 2022b). Furthermore, the DAS acquisition tends to better identify higher propagation modes (Galan-Comas, 2015). However, fiber-optic systems are uniaxial devices, recording only perturbations acting in the longitudinal direction, and the correct location of measurement points may be uncertain in some cases (e.g., in the case of fiber overstuffing; Bakku, 2015). Also, the signal-to-noise ratio of measured data is lower compared to geophones. The lower quality in recorded traces limits the repeatability of the survey (Costley et al., 2018) and the reliability of the estimated wave parameters in the presence of weak signals (Mestayer et al., 2012). Finally, as explained above, the measurement technique involved in this technology partially limits the minimum investigable wavelengths at greater values than the one defined by the Nyquist-Shannon theorem (Lancelle, 2016; Bakulin et al., 2020). Therefore, the characterization of high-frequency R-wave data might be challenging.

Different acquisition layouts were investigated at the Hornsby Bend site to assess the influence of the DAQ type on the estimated dispersion and attenuation data, through the canonical 1D processing procedure. The first 94 m section of the fiber-optic cable, which is adjacent to the geophone array, is selected in this study to ensure a consistent comparison between results, as they sample a comparable volume of the soil deposit. This study refers to waveforms generated at shot points located at 5 m, 10 m, 20 m, and 40 m offset from the closest measurement point (i.e., 0 m; refer to Figure 3a). Additionally, it only utilizes waveforms created by the Thumper truck, which generated a 12-s long chirp signal, with frequency shifting from 5 Hz to 200 Hz. As an example, Figure 3a-3b reports the time histories of recorded data in the Hornsby Bend site using the geophone and the DAS arrays, with the active source located at an offset equal to 10 m.

As noted above, Vantassel et al. (2022b) showed that it is possible to extract equivalent surface wave dispersion data from seismic measurements made using a traditional geophone array and DAS. Moreover, they demonstrated that frequency-dependent normalization of the dispersion image

removes the effect of scaling, integration, and differentiation on the acquired waveforms, thereby mitigating the need to convert the measurements into consistent engineering units prior to comparing dispersion data. Thus, Vantassel et al. (2022b) rigorously demonstrated the potential for extracting high-resolution, multi-mode surface wave dispersion data using DAS measurements and MASW-type processing.

Aimar et al. (2023) showcased the potential of exploiting DAS data for the joint estimation of the phase velocity and phase attenuation data. These quantities were obtained from the vertical geophone waveforms through application of the recently developed Cylindrical Frequency-Domain BeamForming – Attenuation algorithm with Modal Filtering (CFDBFaMF; Aimar et al. (2024a)). The extraction of the R-wave parameters from DAS data adopts a modified version of the CFDBFaMF, which implements an average strain-based beamforming [further details are available in Aimar et al. (2024a)]. In this way, the procedure properly models the spatial variation of the amplitude and phase of the average radial strain. Figure 4 compares the estimated modal dispersion and attenuation data for the first two propagation modes (labeled as R0 and R1, respectively), obtained from the interpretation of the DAS and the geophone array data. In this case, the data distribution is represented by the interval around the median value, the width of which equals one logarithmic standard deviation. Data statistics are obtained by combining results from different source offsets, in consistency with the multi-offset approach (Cox & Wood, 2011).

In general, dispersion and attenuation data well match with each other, particularly for the R0 mode. For the R1 mode, the DAS data does not allow phase velocity and attenuation estimates to be made over as broad a frequency range, being more limited at lower frequencies. This partially limits the capability of the DAS system in characterizing deeper layers. However, the corresponding degree of variability in the phase velocity and attenuation data derived from DAS is generally less than or equal to the variability affecting the geophone-based parameters. This result is quite surprising, as the signal-to-noise ratio of DAS records has been reported in other studies to be slightly lower than geophone records, hence, higher variability in the DAS-derived attenuation was expected. A possible reason behind the low data scatter can be the remarkably larger number of measurement points that the DAS system includes, that provides a more exhaustive dataset of wavefield values to better constrain the velocity and the attenuation estimates.

Finally, experimental Rayleigh-wave data were mapped into profiles of V_s and D_s versus depth, by means of the inversion procedure developed by Aimar et al. (2024b). This operation was carried out through an improved Monte Carlo scheme, which implements a smart sampling technique of the model parameter space, by exploiting the scaling properties of the Rayleigh wave parameters in linear viscoelastic media. These properties allow a significant saving in computation time, preserving the quality of the resulting ground models at the same time. For simplicity,

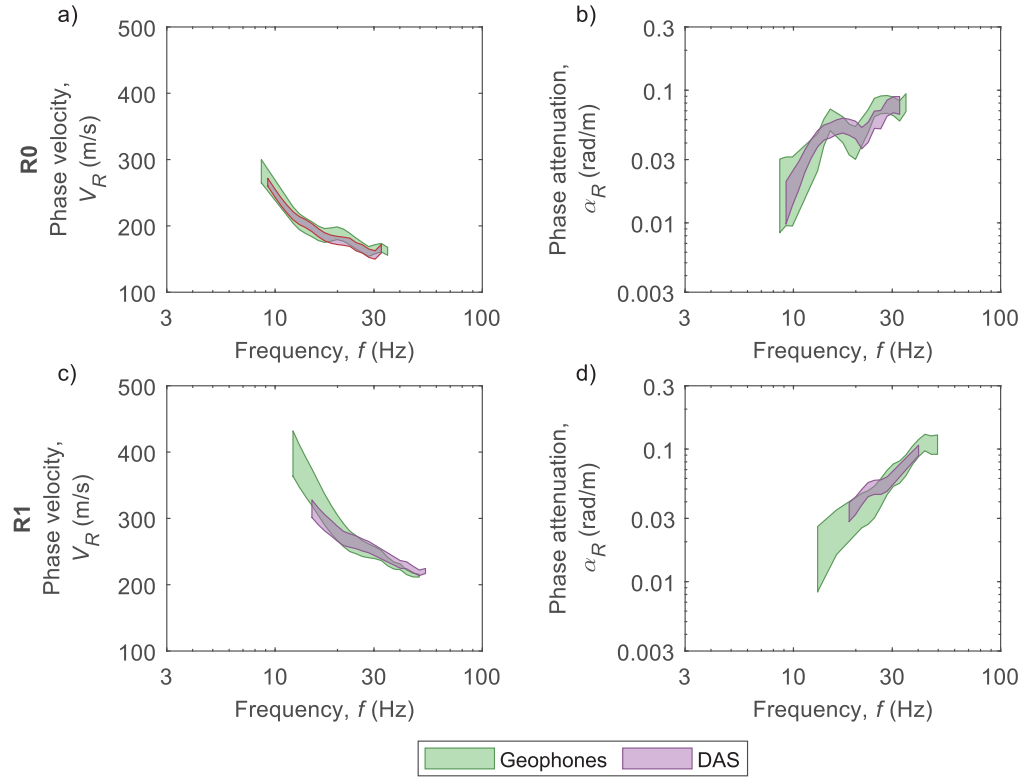


Figure 4. Comparison between the estimated dispersion and attenuation curves from the DAS and the geophone data at the Hornsby Bend site: a-b) Resulting dispersion (a) and attenuation (b) curves for the fundamental mode, R0; c-d) Resulting dispersion (c) and attenuation (d) curves for the first higher mode, R1. Estimated data are represented in terms of intervals given by one logarithmic standard deviation around the median value; after Aimar et al. (2023).

the model identification adopts a three-layer ground model, consistently with the stratigraphy inferred by cone penetration soundings carried out close to the DAS array (refer to Figure 2b). For each layer, an adequate range of layer thicknesses, S-wave velocities and damping ratios were investigated, whereas the mass density and Poisson's ratios were fixed at realistic values. The inversion was run using 10,000 trial earth models. Forward dispersion and attenuation modeling was carried out through the EDT toolbox (Schevenels et al., 2009). Model selection was based on a proper misfit function, wherein fitting errors between theoretical curves and experimental data were weighted as a function of the uncertainties affecting $V_R(\omega)$ and $\alpha_R(\omega)$.

Figure 5 shows results for the best fitting 30 models. Inverted S-wave velocity and damping ratio profiles are relatively well constrained, and the velocity and dissipation structures are clearly identifiable in the near-surface layers. Specifically, the resulting V_s model exhibits a gradual increase in stiffness with depth. The depths of the identified layers interfaces are about 4 m and 12-13 m. This result is consistent with the main geological interfaces inferred at the site and with information from past geophysical surveys. The estimated D_s profiles are affected by greater variability, which increases with depth. Specifically, D_s is about 5% in the near-surface layer, and increases to around 8% in the layer

below. As for the half-space, the variability in both V_s and D_s dramatically increases with respect to shallow layers. Indeed, the few amounts of experimental data at long wavelengths does not allow an effective constraints of estimated profiles at greater depths. Thus, the V_s oscillates between 330 m/s and 400 m/s, whereas D_s spans over a much broader range, mostly between 0.5% and 5% (that is, the variation is about one order of magnitude). This is the combined effect of the large variability in low-frequency attenuation data, the relevant influence of V_s on phase velocity and attenuation data, and the moderately low sensitivity of theoretical attenuation curves to D_s at great depths (e.g., Verachtert, 2018), that does not allow a constraint on D_s as effective as in the stiffness modeling. On the other hand, it should be noted that, particularly in the near-surface layers, both the velocity and dissipation structures show well defined trends.

In summary, the DAS technology can be successfully used to jointly estimate the phase dispersion and attenuation data, obtaining the same level of reliability of the canonical geophone array. Furthermore, the potentially stronger influence of incoherent noise on DAS data is balanced by the significant increase in the number of measurement points,

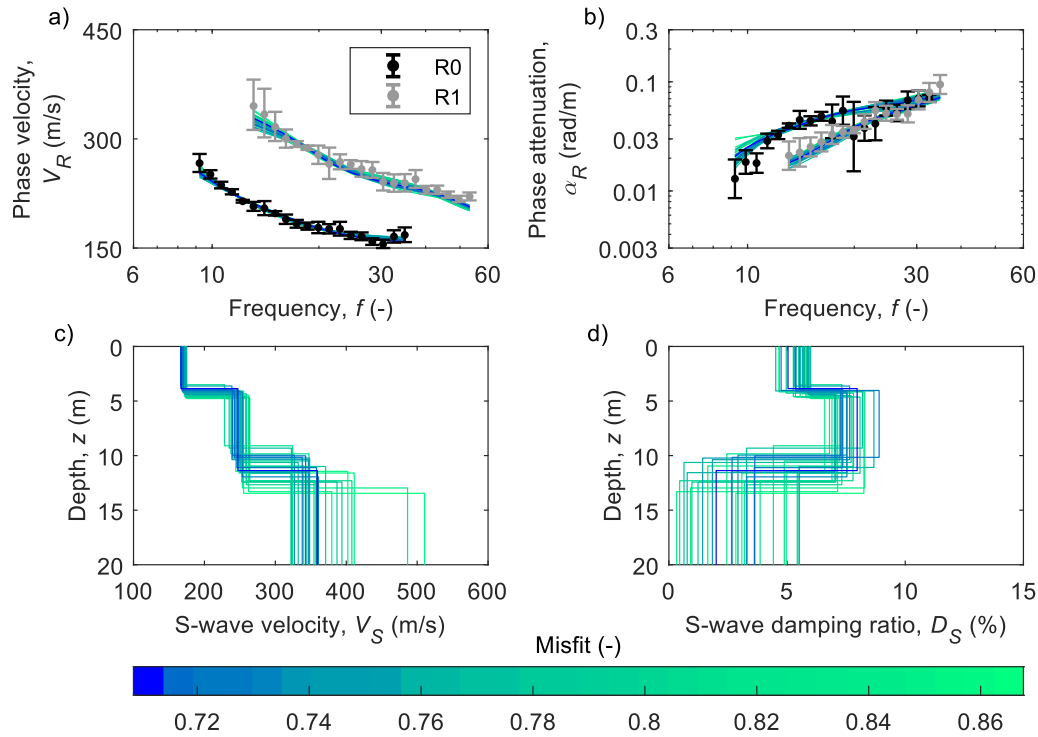


Figure 5. Best fitting inverted ground models to DAS experimental data from the Hornsby Bend site: a-b) Theoretical and experimental data for the phase velocity (a) and phase attenuation (b); c-d) Resulting S-wave velocity (c) and damping ratio (d) profiles; after Aimar et al. (2023).

thus resulting in a reduction in data variability, entailing an improvement in the accuracy of this system.

3.1.2 2D MASW using DAS

Two-dimensional (2D) MASW (e.g., Park, 2005) is a technique used to produce a pseudo-2D V_S cross-section of the subsurface by expanding upon the 1D MASW approach. This technique relies on spatially interpolating numerous 1D MASW V_S profiles obtained from overlapping sub-arrays along a linear testing alignment. One of the main challenges when performing 2D MASW using conventional equipment, such as geophones and 24-channel seismographs, is that the geophone spacing, the length of the sub-arrays, the spatial interval between sub-arrays, and the positions of shots relative to the sub-arrays must be determined prior to/during data acquisition, making it difficult to adjust them during data processing (i.e., after data acquisition). This poses a challenge because the geophone spacing and sub-array length can have a significant impact on the lateral resolution, maximum characterization depth, and anomaly detection capabilities (Yust et al., 2022). Therefore, the ability to modify parameters such as the sub-array length after initial processing results have been investigated can be highly advantageous. According to a study by Yust et al. (2022), the use of DAS rather than traditional 2D MASW

equipment can be highly advantageous in addressing these challenges.

In their recent study, Yust et al. (2022) investigated the effects of 2D MASW sub-array length using the DAS data collected at the Hornsby-Bend site. Three sets of sub-arrays with varying lengths were used to develop pseudo-2D V_S cross-sections along a 200 m long DAS line (refer to Figure 2a). The sub-arrays investigated by Yust et al. (2022) consisted of: (a) 12-channel sub-arrays approximately 11 m long, (b) 24-channel sub-arrays approximately 23 m long, and (c) 48-channel sub-arrays approximately 47 m long. They used an equivalent sub-array spatial interval of four channels (approximately 4 m) for all sub-arrays and performed 129 individual MASW analyses in total. The pseudo-2D V_S cross-sections obtained using the 12-, 24-, and 48-channel sub-arrays had lateral extents of 187.68 m, 175.44 m, and 150.96 m, respectively, while maintaining a consistent depth of 15 m, as shown in Figure 6a, 6b, and 6c, respectively. These cross-sections are quite similar over the top 7-8 m, but show noticeable differences at greater depths, highlighting the sensitivity of 2D MASW results to the choice of sub-array length. Nonetheless, each cross-section obtained from the different sub-array lengths was found to correlate better with a different key feature of the subsurface, as verified by comparisons with invasive data collected along the array alignment. For instance, Yust et al. (2022) found that, for the

Hornsby-Bend site, the most prominent impedance contrast in the V_s cross-section obtained using 12-channel sub-arrays corresponded to the depth of CPT refusal (see Figure 6a), while the V_s cross-section obtained using longer, 48-channel sub-arrays corresponded to the deeper shale impedance contrast, as indicated by the boring lithology log superimposed on the V_s cross-section (see Figure 6c). It is worth noting that Yust et al. (2022) were able to investigate these different sub-array lengths post data acquisition, owing to the flexibility allowed by DAS technology. Unlike traditional seismic equipment, DAS records the wavefield generated at each of the considered shot locations simultaneously along the entire length of the array at a constant channel separation, eliminating the need to pre-determine sub-array length and spatial sampling interval during data acquisition. This feature enables the investigation of multiple sub-array geometries during the processing stage of the analysis, thereby providing greater flexibility and control over acquisition parameters.

The study by Yust et al. (2022) shed light on the advantages of utilizing DAS in 2D MASW, and emphasized the significance of incorporating a priori information, such as invasive testing data, to fine-tune the 2D MASW analysis and achieve project-specific objectives, whenever feasible. In cases where there is insufficient conclusive information to constrain subsurface layering, Yust et al.

(2022) recommended examining multiple 2D MASW sub-array configurations to gain a comprehensive understanding of the subsurface conditions and accurately assess the uncertainty of the results.

3.1.3 Machine learning

In recent years, there has been a growing interest in utilizing deep learning/machine learning/artificial intelligence (DL/ML/AI) techniques for non-invasive subsurface imaging (Adler et al., 2021). For example, with a fully trained and adaptable neural network it would theoretically be possible to swiftly generate subsurface images directly from wavefield measurements without the need to perform costly and complicated inversions. Furthermore, this type of imaging could be performed by an analyst without any significant understanding of machine learning. This would enable the reuse of fully trained neural networks to rapidly produce site-specific results, eliminating the need for specialized expertise, which is often required by conventional inversion methods. To train a neural network for subsurface imaging, a large dataset consisting of numerous input-output image pairs is required. The input can be a representation of the raw data acquired from the field, or some post-processed version

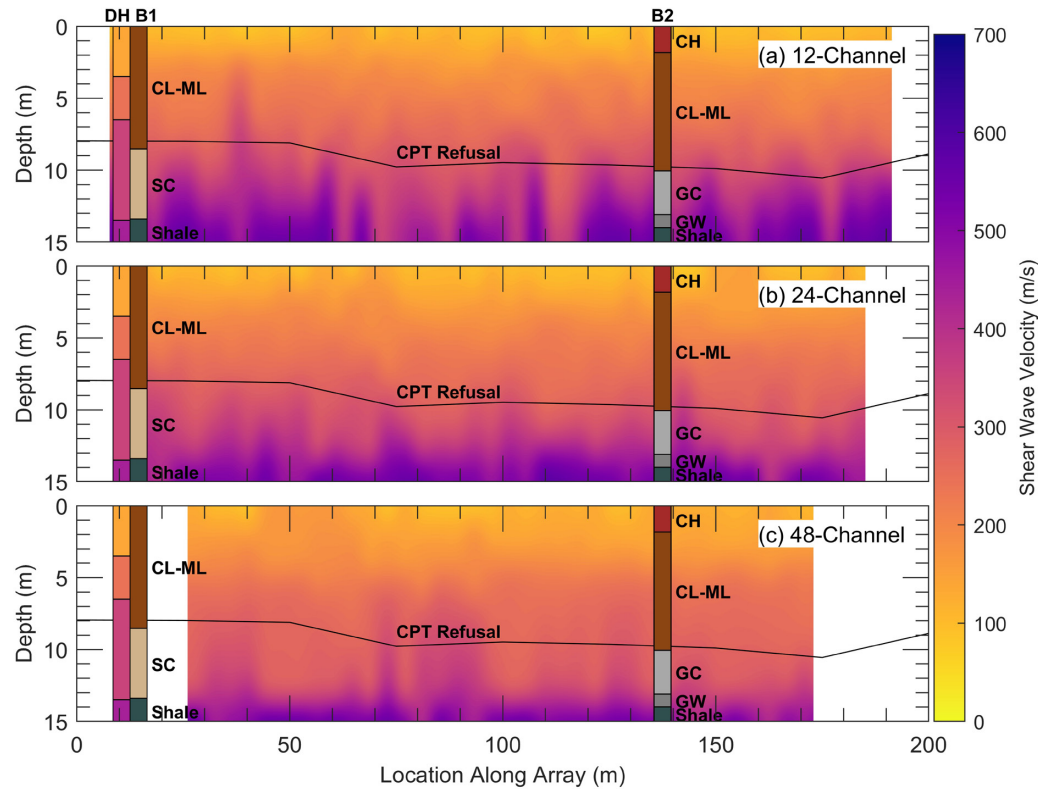


Figure 6. Pseudo-2D V_s cross-sections after Yust et al. (2022) from the: (a) 47, 12-channel MASW sub-arrays, (b) 44, 24-channel MASW sub-arrays, and (c) 38, 48-channel MASW sub-arrays inverted using a 15-layer inversion parameterization. The depths of refusal for 9 CPT soundings along the array are shown on all plots with a solid black line.

of it, while the output can be a subsurface image of the parameter of interest, such as a V_s cross-section. Due to the considerable number of image pairs needed to train a neural network, all research studies thus far have relied on numerically-developed image pairs, as in the case of the convolutional neural network (CNN) developed by Vantassel et al. (2022c), which takes a seismic wavefield input image and outputs a 2D V_s image. A significant challenge that has impeded the use of machine learning as an end-to-end imaging technique for real-field applications is the lack of generalizability (Li et al., 2020; Feng et al., 2022). In other words, these neural networks often struggle when presented with real-field data, particularly if the data was acquired using a different acquisition configuration than the one used during network training (e.g., Vantassel et al., 2022c).

Abbas et al. (2023b) have recently introduced a CNN that shows promise for rapidly generating 2D V_s images of near-surface soil-over-bedrock geology using real-field data. To train and test their CNN, they utilized 100,000 synthetic near-surface models with varying soil-over-bedrock conditions. Their CNN takes a frequency-dependent normalized dispersion image as input (rather than a seismic wavefield image) and produces a 2D V_s image as output, as illustrated schematically in Figure 7. Abbas et al. (2023b) demonstrated that while using different testing configurations in terms of source type, source offset, number of receivers, and receiver spacings leads to significantly different measured wavefields for the same subsurface structure, the normalized dispersion images processed from these different wavefields are quite similar, although not identical. Abbas et al. (2023b) leveraged this feature and demonstrated that a CNN trained on normalized dispersion images processed from wavefields acquired using a specific testing configuration can still perform well when presented with dispersion images processed from wavefields acquired using different testing configurations. This acquisition flexibility significantly improves the CNN's generalization capability, enabling it to be utilized as an end-to-end imaging method or as a tool to create rapid starting models for full-waveform inversion (FWI).

Abbas et al. (2023b) demonstrated the practical capability of their CNN by applying it to experimental field data collected at the Hornsby Bend site. Their CNN generated a high-resolution 48 m wide by 24 m deep V_s subsurface image, which agrees well with the actual subsurface structure determined through invasive tests conducted at the site, as shown in Figure 8, thereby establishing the CNN's promise in handling real-field data. While Abbas et al. (2023b) used geophone-derived dispersion images in their study, their approach is equally applicable to DAS-derived dispersion images, as geophone and DAS-derived dispersion images are shown to be equivalent by Vantassel et al. (2022b).

3.1.4 Full wave form inversion using DAS

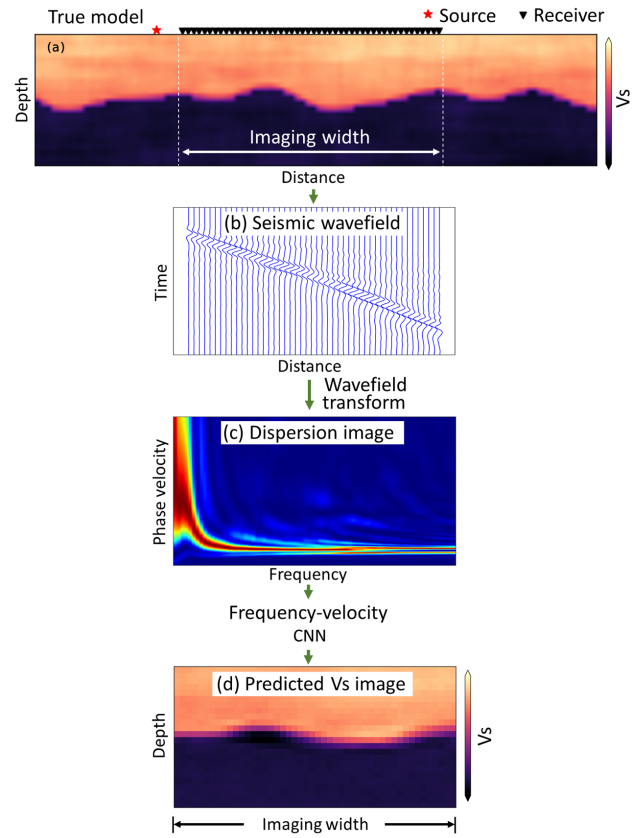


Figure 7. Illustrates the Frequency-velocity CNN framework introduced by Abbas et al. (2023b) for 2D V_s imaging of near-surface soil-over-bedrock geology. Panel (a) showcases a soil-over-bedrock 2D model featuring a 47-meter array of receivers and a single source located off the array's end. In panel (b), an example seismic wavefield recorded by the 48 receivers shown in panel (a) from a Ricker source is depicted. Panel (c) displays the associated dispersion image, serving as the input to the Frequency-velocity CNN. Finally, in panel (d), the Frequency-velocity CNN's predictions of the true synthetic 2D V_s images presented in panel (a) are showcased.

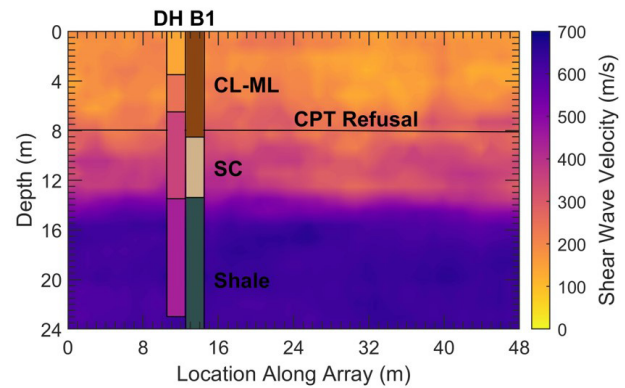


Figure 8. The frequency-velocity CNN output 2D V_s image for the Hornsby Bend site after Abbas et al. (2023b). For comparison with actual field conditions, a borehole log (i.e., B1) is superimposed on the predicted V_s image at 12.5 m, which is the location where the boring was conducted.

Full-waveform inversion (FWI) is a robust imaging technique that produces 2D/3D images of the subsurface by matching a synthetic seismic wavefield, generated by numerically solving the wave equation, to an experimental seismic wavefield acquired in the field. The process involves an iterative data-fitting procedure that requires modifying an initial model through which the synthetic waveforms propagate until the synthetic and experimental wavefields match, based on a selected inversion objective function. This is achieved by adjusting the material properties of the synthetic subsurface model, thereby reducing the misfit between the synthetic and recorded data. Unlike 2D MASW, which can only produce pseudo-2D images, FWI generates true 2D and 3D images by leveraging all available information in the seismic wavefield, including phase and amplitude. This sets FWI apart from other approaches, such as seismic refraction that relies only on wavefield first arrivals or surface wave testing that uses only Rayleigh dispersion. In addition to providing insights into material parameters like V_s and compression wave velocity (V_p), which are of high interest to engineers, FWI can also be used to assess any other material properties that impact seismic wave propagation, such as density and damping ratio.

Despite its numerous advantages, FWI also encounters significant challenges. The computational costs associated with FWI are considerable, and the accuracy of its results can be heavily influenced by the initial model (Vantassel et al., 2022c; Yust et al., 2023), particularly in near-surface applications such as imaging the top 30 m. In the near surface, material properties tend to exhibit rapid variations over short distances, leading to a complex mixture of different wavefield components. This amalgamation includes compression, shear, and surface waves, which have not yet propagated far enough to separate from each other.

Traditionally, FWI analysis has utilized data acquired through geophones, which capture particle velocity wavefields. However, the high spatial resolution provided by DAS has garnered interest for use in FWI studies. Nonetheless, there is a dearth of literature on the use of FWI with DAS data for near-surface characterization, particularly in regards to directly inverting DAS strain data without first converting it to particle displacement or velocity.

In a recent study conducted by Yust et al. (2023), a direct inversion of strain measurements obtained from DAS was performed at the Hornsby Bend site. The study aimed to image a cross-section measuring 200 m wide and 30 m deep. Four distinct 1D and 2D starting models specifically tailored to the site were utilized. The first model was based on a 1D V_s profile obtained through traditional MASW testing. The second starting model utilized a 1D V_s profile derived from seismic down-hole invasive testing. The third starting model relied on the CNN 2D V_s subsurface model developed by Abbas et al. (2023b) and depicted in Figure 8. Lastly, the fourth model was constructed using the 2D MASW analysis with the 48-channel subarrays developed by Yust et al. (2022) and discussed in the 2D MASW section (refer to Figure 6). The four FWI starting models used by Yust et al. (2023) are visually depicted in Figure 9. Yust et al. (2023) inverted for V_s , V_p , and density, ρ , while assuming constant quality factors to characterize the attenuation of compression and shear waves in the model. Nonetheless, they noted that the attenuation parameters significantly impact the FWI results and noted that additional studies are required to help constrain these values. Yust et al. (2023) observed enhancements in each of the initial models through the implementation of FWI. This improvement was evident when comparing the misfit between synthetic and recorded waveforms for both the initial and updated models. For example, Figure 10 illustrates the misfit between the synthetic and experimental waveforms of

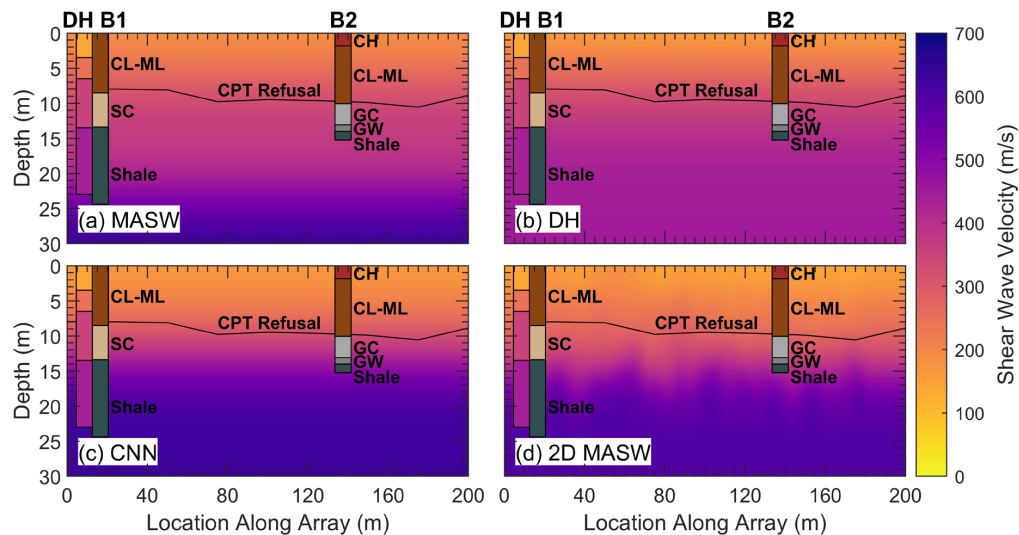


Figure 9. The four smoothed 2D V_s starting models used by Yust et al. (2023) for FWI based on: (a) 1D MASW, (b) downhole testing (DH), (c) CNN machine learning, and (d) pseudo-2D MASW.

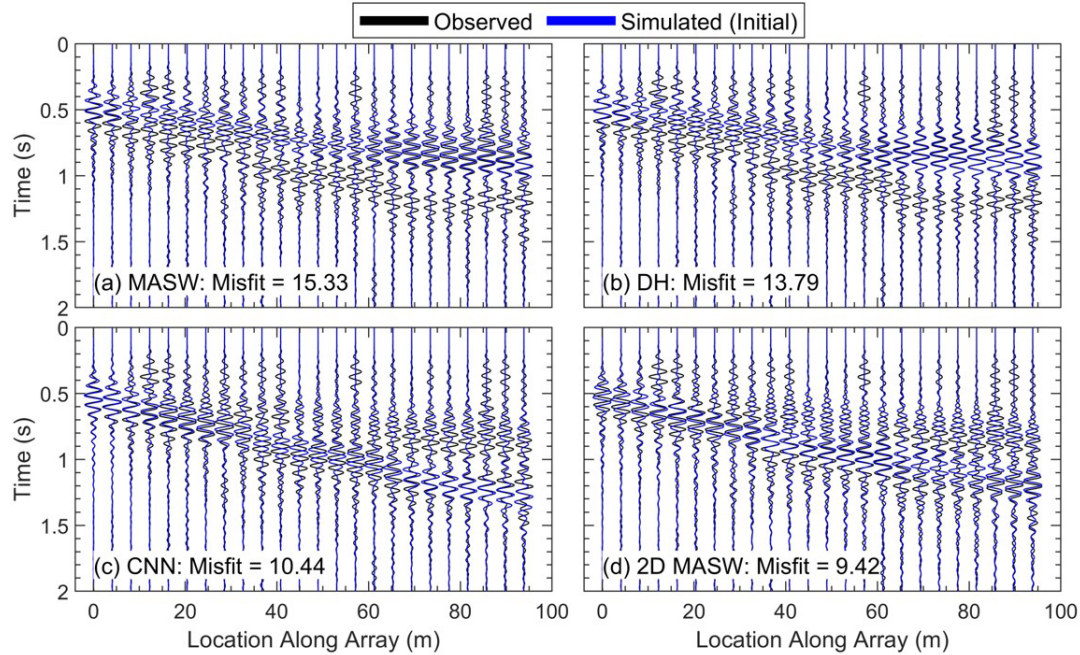


Figure 10. Normalized observed and simulated waveforms by Yust et al. (2023) from Shot 1 (−24 m) of Stage 1 (10 to 15 Hz) for the: (a) MASW, (b) downhole testing (DH), (c) CNN, and (d) 2D MASW starting models. GSOTD misfit values for each set of simulated waveforms are shown in the bottom left of each plot. Note that for clarity purposes, the waveforms are only shown for every fourth channel used for FWI.

the initial models, while Figure 11 displays the misfit for the FWI updated models following the first stage of a four-stage FWI. By comparing the waveform misfit values in Figure 10 and Figure 11, which are based on a graph space optimal transport distance (GSOTD) algorithm, one can clearly see that the FWI iterations in the first stage resulted in better waveform matches (i.e., lower misfit values) for all four starting models. This reduction in misfit values continued through each stage of the FWI process, ultimately resulting GSOTD misfit values for the four final models that were very similar to one another and only varied between 1.91 to 1.46. Despite the similarities in the final waveform misfit values, the final subsurface models did not vary significantly from their respective starting models and exhibited noticeable visual differences from one another, as depicted in Figure 12. Within the upper 7 m, the final V_s images are quite similar. Yet, below this depth there are evident disparities in the magnitudes and rates at which V_s increase. This finding highlights the intrinsic non-uniqueness associated with the FWI process, underscoring the importance of incorporating multiple starting models. Assessing the sensitivity of results to the choice of starting model and attaining consistent outcomes across different models engenders confidence in the subsurface regions where the models yield congruent results.

3.2 Discussion

In the preceding sections, various subsurface imaging techniques have been reviewed. This section aims to

provide an analysis of the advantages and disadvantages associated with each technique. The first method reviewed was 1D MASW, which is a well-established technique for V_s imaging using geophones. Extensive research supports this method, offering best practices and recommended workflows (Foti et al., 2018; Vantassel & Cox, 2022). However, this study presents two advancements to the conventional 1D MASW workflow. The first involves using DAS instead of geophones for data acquisition, while the second is the joint inversion for damping and V_s . The utilization of DAS significantly enhances measurement scales and spatial resolution beyond the capabilities of traditional measurement technologies. Moreover, when appropriate precautions are taken, the measurements obtained through DAS exhibit good agreement with those acquired using traditional methods like geophones (Daley et al., 2016; Hubbard et al., 2022; Vantassel et al., 2022b). It should be noted, though, that DAS provides deformation measurements only along the fiber optic cable direction, while geophones can capture particle motion point measurements in all three directions individually or simultaneously.

The second advancement focuses on the joint inversion for damping and V_s using data acquired through a 1D MASW setup, which represents a significant breakthrough, particularly due to the crucial role of damping in various domains, including site response. Aimar et al. (2023) showed that their approach is applicable on both geophone and DAS acquired data. Furthermore, this paper illustrates the innovative application

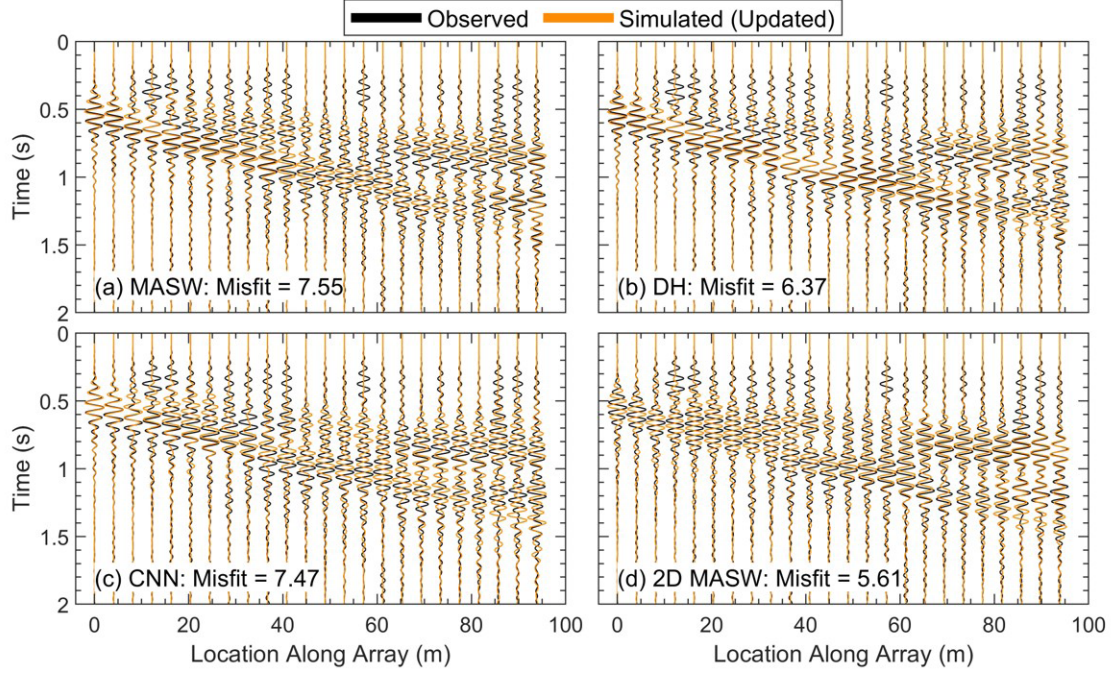


Figure 11. Normalized observed and simulated waveforms by Yust et al. (2023) from Shot 1 (–24 m) for the updated models at the end of FWI Stage 1 (10 to 15 Hz) based on the: (a) MASW, (b) downhole testing (DH), (c) CNN, and (d) 2D MASW starting models. GSOTD misfit values for each set of simulated waveforms are shown in the bottom left of each plot. Note that for clarity purposes, the waveforms are only shown for every fourth channel used for FWI.

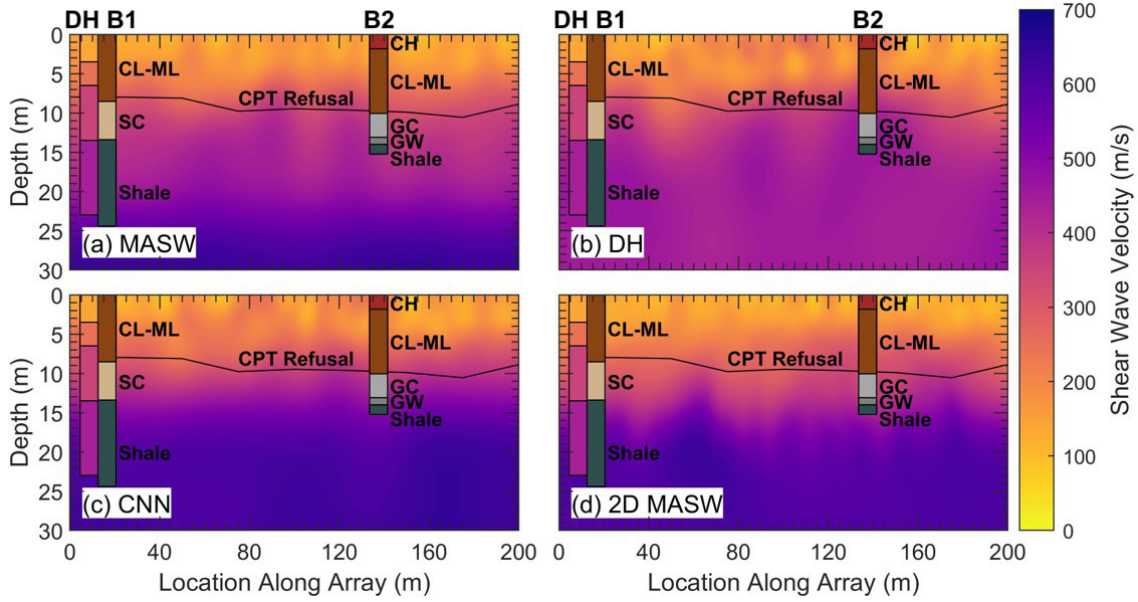


Figure 12. Borehole logs, downhole V_s results, and the depth to CPT refusal overlaid on the final, updated 2D V_s images at the end of FWI Stage 4 (10 to 30 Hz) for the: (a) MASW, (b) downhole testing (DH), (c) CNN, and (d) 2D MASW starting models after Yust et al. (2023).

of DAS in jointly characterizing the stiffness and dissipative parameters of a soil deposit based on a fiber-optic array. Figure 4 illustrates the favorable agreement between attenuation estimates obtained using both DAS and geophones for the Hornsby Bend site, with DAS-based measurements

exhibiting lower uncertainty compared to geophone-based measurements. However, the uncertainty in attenuation estimates generally tends to be higher than that observed for dispersion estimates, as detailed by Aimar (2022). In addition, the results presented were obtained from a portion

of a large experimental dataset. Future studies will thoroughly investigate these data to obtain a more comprehensive view of DAS performance compared to geophones and to seek for improved estimates of the phase attenuation, especially at low frequencies. It is worth noting that the 1D MASW technique generates a 1D subsurface profile by averaging the soil properties beneath the instrumentation array. To obtain a 2D representation of the subsurface, the 2D MASW technique was utilized. However, 2D MASW generates a pseudo 2D image rather than a true one, as it involves interpolating between multiple 1D MASW-derived soil profiles. Yust et al. (2022) demonstrated that this method produced V_s images that aligned with significant features at the Hornsby-bend site. Nevertheless, it was also observed that the resulting V_s image is reliant on the specific parameters chosen for data acquisition.

Another approach presented in this paper for 2D V_s imaging is the machine learning CNN proposed by Abbas et al. (2023b). This approach holds great promise as it can generate true 2D images of the subsurface, in contrast to the pseudo 2D images obtained through 2D MASW. Additionally, the machine learning approach is notable for its remarkable speed, as a trained neural network can deliver imaging results within seconds. However, this technique is still in its early stages of development and requires extensive research and testing before it can be reliably employed for subsurface imaging purposes.

The final imaging approach explored in this paper is 2D FWI. What sets FWI apart from the previously discussed methods is its utilization of the entire measured wavefield to generate true 2D and 3D subsurface images. Moreover, FWI differs from the machine learning approach by not operating as a black box method. As FWI continues to undergo further development, it is anticipated to become the preferred imaging technique. However, FWI currently faces several challenges, including notable computational costs and time-consuming complexity of the analysis process. Furthermore, studies by Yust et al. (2023) and Vantassel et al. (2022c) have demonstrated that the imaging results obtained through FWI are significantly influenced by the starting model used.

4. Reference dataset: Newberry site, Florida

The preceding paragraphs have showcased the latest developments in seismic wave-based noninvasive subsurface imaging, with a practical demonstration of their potential at the Hornsby Bend site. However, it should be noted that the Hornsby Bend site is a relatively simple site with no documented underground anomalies. Furthermore, the seismic data from the Hornsby Bend site was collected using relatively short and linear DAS and geophone arrays. To provide researchers with a more comprehensive dataset at a more challenging site, Abbas et al. (2024) conducted a field test in Newberry, Florida, at a site known for its spatial variability, karstic voids, and underground

anomalies. The experiment utilized cutting-edge sensing technologies, including a two-kilometer DAS fiber optic cable, forming a dense 2D array of 1920 channels, and a 2D array of 144 SmartSolo three-component nodal stations to sense active-source and passive-wavefields, as illustrated schematically in Figure 13. The active-source data was generated using a powerful three-dimensional vibroseis shaker truck and impact sources, and it was simultaneously sensed by both the DAS and nodal stations. The vibroseis truck was used to vibrate the ground in the three directions (two horizontal and one vertical) at 260 locations inside and outside the instrumented array, while the impact sources were used at 268 locations within the array (refer to Figure 13). The passive wavefield data, recorded using the nodal stations, consisted of 48 hours of ambient noise collected over four days in four twelve-hour time blocks. The active-source and passive-wavefield DAS and nodal station data have been preprocessed and organized in an easy-to-navigate folder structure. The raw and processed data, along with detailed

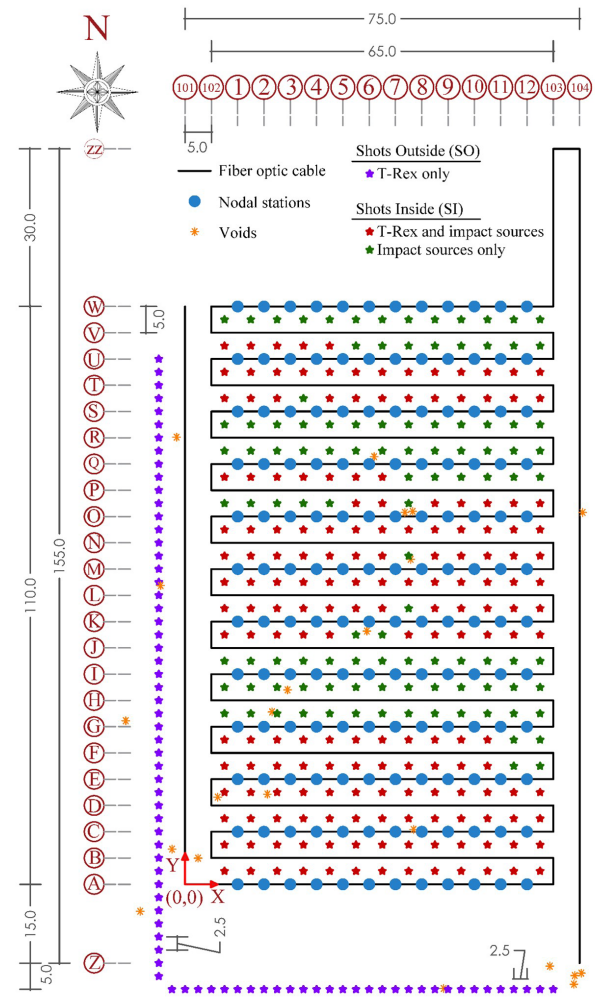


Figure 13. Newberry site testing configuration after Abbas et al. (2024).

documentation of the experiment and Python tools to aid in visualizing the DAS dataset have been archived and made publicly available on DesignSafe (Abbas et al., 2023a). The Newberry dataset (Abbas et al., 2024; Abbas et al., 2023a), featuring a powerful, triaxial vibroseis shaker and 3C sensors, in conjunction with the dense DAS array, offers a valuable resource for researchers exploring novel noninvasive subsurface imaging approaches that utilize seismic waves.

5. Conclusion

The paper covers advancements in noninvasive subsurface imaging technologies that utilize seismic waves for site characterization. These advancements span both innovative sensing technologies and advanced 1D and 2D imaging techniques for retrieving the small-strain shear modulus and damping ratio. Regarding sensing innovations, a particular focus is placed on the use of DAS as the data acquisition system for seismic wave sensing in near-surface imaging applications. In terms of imaging techniques, the joint inversion of attenuation and dispersion data is presented, utilizing 1D MASW in conjunction with either DAS or traditional sensing techniques such as geophones. With this approach, 1D shear wave velocity and damping profiles can be obtained. The presented 2D imaging techniques comprise 2D MASW using DAS, machine learning for 2D subsurface imaging, and 2D FWI using DAS data. All of these advancements, whether in sensing technologies or analysis methods, were implemented at the well-characterized Hornsby Bend site in Austin, Texas, enabling a discussion of the advantages and disadvantages associated with each method. Finally, a comprehensive and open-access subsurface imaging experiment conducted in Newberry, Florida, has been presented, where state-of-the-art technologies for sensing seismic waves and generating wavefields have been implemented.

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

Aser Abbas: conceptualization, writing – original draft, formal analysis, methodology, software. Mauro Aimar: conceptualization, writing – original draft, formal analysis, methodology, software. Michael Yust: visualization, formal analysis, software. Brady R. Cox: conceptualization, funding acquisition, project administration, writing – review & editing, supervision. Sebastiano Foti: conceptualization, funding acquisition, project administration, writing – review & editing, supervision.

Data availability

Experimental datasets are available from the authors upon request.

List of symbols and abbreviations

$2g$	gauge length
e	average strain, computed over the gauge length along the fiber-optic cable direction
r	spatial coordinate
t	time
u	ground displacement
D_s	shear wave damping ratio
V_p	compressional wave velocity
V_R	Rayleigh wave phase velocity
V_s	shear wave velocity
α_R	Rayleigh wave phase attenuation
λ	wavelength
ρ	density (mass per unit volume)

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Investigating the Martian soil at the InSight landing site

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Review Article

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Abstract

The InSight mission is a geophysical mission aimed at better understanding the structure of Mars and of the other rocky planets of the solar system. To do so, a lander accommodating two cameras, a very sensitive seismometer, and a dynamic self-penetrating heat probe nicknamed the mole were placed on the Mars surface by the Instrument Deployment Arm (IDA). Besides geophysical data (which definitely enriched the existing knowledge on the structure of Mars), the InSight instruments significantly increased the knowledge of the geological and geotechnical characteristics of the surface material at the InSight site. Small strain (elastic) parameters were derived from wave velocity measurements during the hammering sessions between the self-penetrating probe and the seismometer. A detailed observation of the soil profile along a depth of 37 cm was made possible thanks to the photos taken by the cameras, and to a detailed analysis of the mole penetration process. Further information was provided by an intense campaign of scraping and piling conducted by the IDA on the surface sand/dust layer. It was shown that the soil profile was composed of a surface 1 cm thick sand/dust layer, overlaying an around 20 cm thick loose duricrust made up of a cohesive matrix containing some pebbles, located above a 12 cm layer of sand overlaying a gravel/sand deposit. It is believed that the geology and soil mechanics data provided by the InSight mission will help for further robotic exploration of Mars.

1. Introduction

Mars is one of the four rocky planets of the Solar system, with a Martian year as long as around two times the Earth year, an average diameter of 6779 km (0.553 that of the Earth) and,

as a result, a smaller gravity of 3.721 m/s². The atmosphere on Mars is mainly composed of CO₂ with an average atmospheric pressure of 610 Pa. As a consequence, the surface on Mars is submitted to wind, with significant consequences on the surface deposits, both in terms of particle shape and surface morphology.

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The InSight mission on Mars (Interior Exploration using Seismic Investigations, Geodesy and Heat Transport) is a geophysical mission aimed at better understanding the structure of Mars and of the other rocky planets of the solar system (Banerdt et al., 2020). The mission, managed by NASA with European Space Agencies (e.g. CNES, France and DLR, Germany) is based on using a lander accommodating various instruments.

Besides geophysical data, that definitely enriched existing knowledge on the structure of Mars, some of the InSight instruments that interacted with the soil significantly enlarged the knowledge of the geological and geotechnical characteristics of the surface soil at the InSight site, that may also be useful for future robotic exploration of Mars. The data on the soil behaviour gained during the InSight mission are presented in this paper.

2. The InSight mission

The InSight lander, that landed at the surface of Mars in western Elysium Planitia on 26 November 2018, is represented in Figure 1 (this date corresponds to sol 1, sol is the name of a Martian day). The artist view accommodates various instruments, including a very sensitive seismometer called SEIS (Seismic Experiment for Interior Structure), comprising a very-broad-band seismometer (VBB) funded by CNES, and a short-period (SP) one developed at Imperial College (Lognonné et al., 2019). In the Figure, SEIS is covered by a white hemispherical wind and thermal shield (WTS).

The lander also included a low-velocity penetrator – nicknamed the Mole – as part of the Heat flow and Physical Properties Package HP³ (Spohn et al., 2018, 2022a, b). The package was provided by the German Aerospace Center (DLR) through its Institute of Planetary Research. The probe is 40 cm long and 2.7 cm in diameter. The HP³ was designed to measure the thermal conductivity and the thermal gradient down to 3-5 metres below the surface. The heat flow from the interior of the planet can be calculated as the product of the thermal conductivity and the temperature gradient and provides important constraints on the composition and the evolution of the planet. To do so, the mole was supposed to drag a Kapton® tether to depth equipped with temperature sensors positioned to optimize the gradient measurement (Spohn et al., 2018). Further details on the penetration process of the mole are given in Section 5.2.

Figure 2a (Golombek et al., 2020a) shows an image of the InSight lander, heat shield and parachute taken on 6 December 2018 from the HiRISE (High Resolution Imaging Science Experiment) camera on board the Mars Reconnaissance Orbiter (MRO). More details are also given for the heat shield (b), lander (c) and parachute (d). The 20 m radius dark spot around the lander corresponds to the surficial dust layer that has been expelled around by retrorockets during landing. Its extension to the southeast is due to prevailing wind direction from the northwest in this area, confirmed by the InSight instruments later on. The photo also provides more information about the landing site, with circular impact craters at various degradational states, due to wind and aeolian

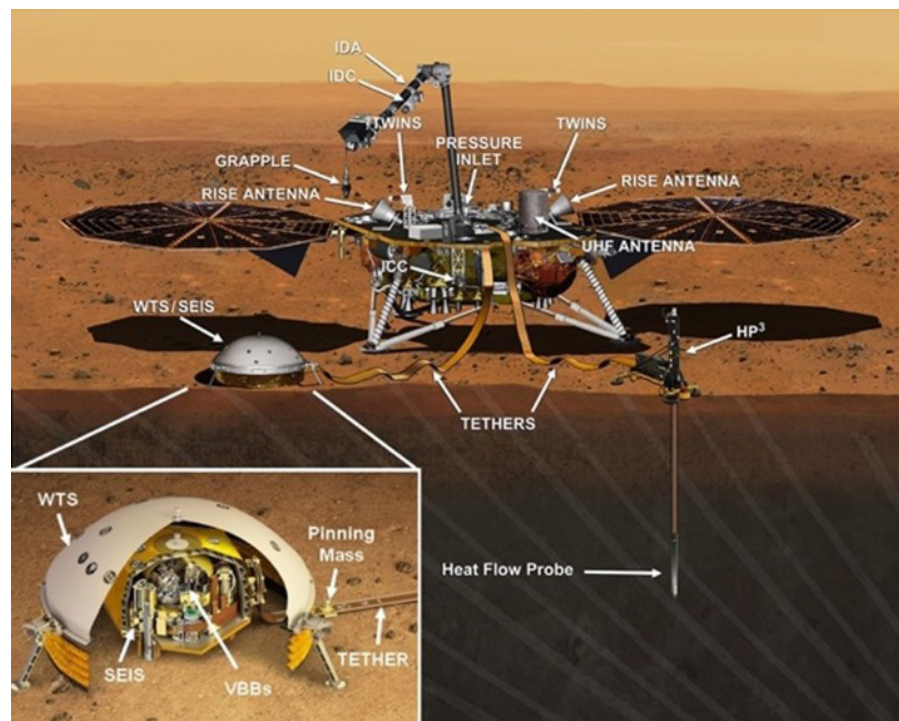


Figure 1. Artist view of the InSight lander and of the SEIS seismometer (image credit: JPL-NASA and IPGP D.Ducros).

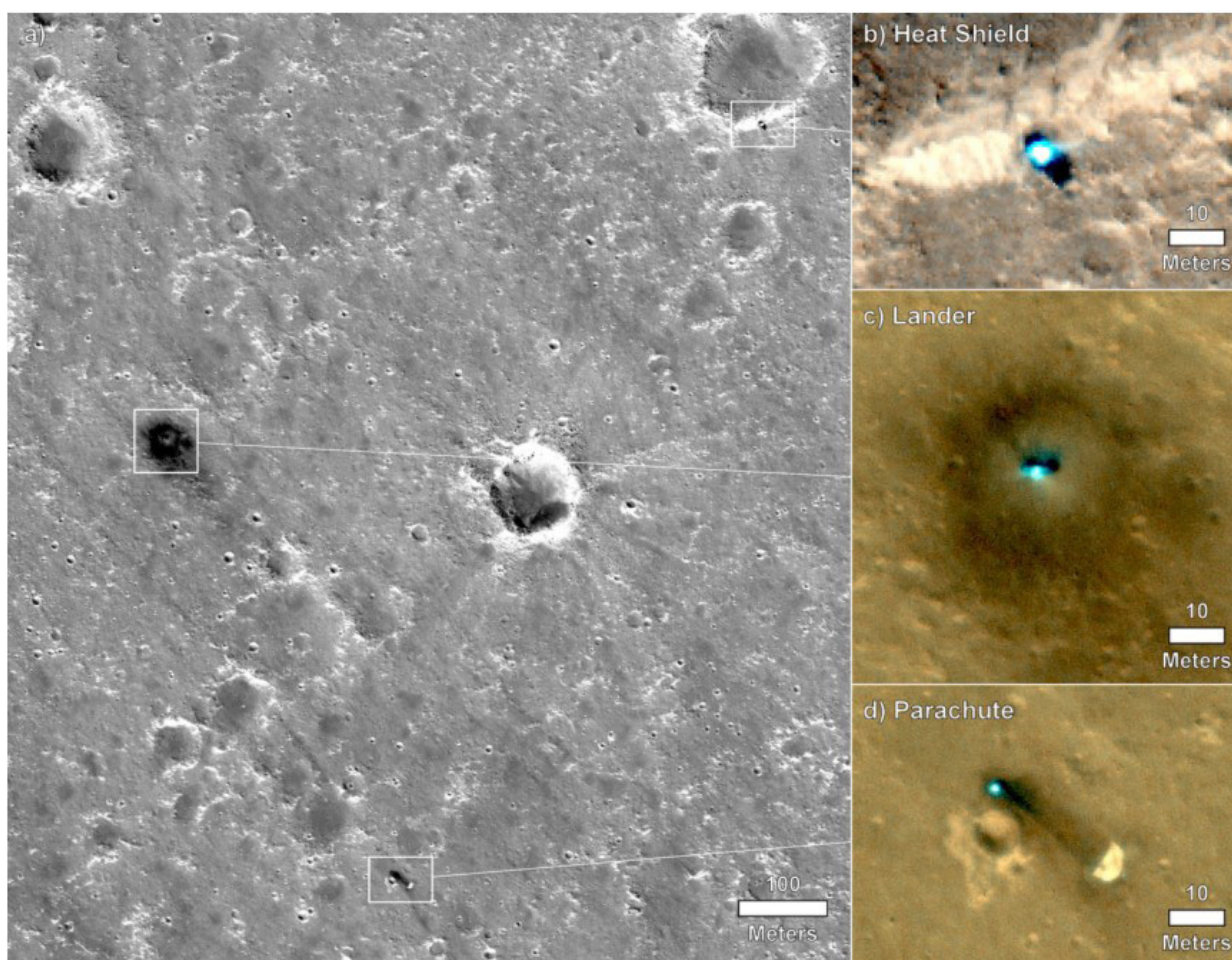


Figure 2. HiRISE image from the InSight landing site taken from the Mars Reconnaissance Orbiter: (a) General view; (b) Heat shield; (c) Lander; (d) Parachute (Golombek et al., 2020a).

transportation of surface particles. A relatively fresh crater is observed at 400 m to the east of the lander.

The SEIS and HP³ instruments have been placed on the surface of Mars thanks to the grapple located at the end of the 2 m long Instrument Deployment Arm (IDA) of the lander (Trebi-Ollennu et al., 2018). The IDA (see Figure 1) also accommodates a scoop that can be used for pushing on the surface and scraping and pouring regolith (Golombek et al., 2023). Direct visual observation of the soil around the lander is made possible by two cameras, the wide-angle Instrument Context Camera (ICC), placed on the lander and the Instrument Deployment Camera (IDC), with a better resolution, fixed on the IDA (Figure 1). The lander includes two solar arrays (2.2 m diameter, 700 W each on clear days when clean) that provide energy to the lander and instruments. Energy is transmitted to the SEIS and HP³ by tethers that also transmit data. Among other instruments, the lander accommodates a meteorological station continuously monitoring wind and temperature (Banfield et al., 2018), a magnetometer and a precision tracking system (Folkner et al., 2018).

The ICC photo of Figure 3, taken at the beginning of the mission (early 2019), shows on the left-hand side the HP³ support structure (dark in shadow) containing the mole prior to penetration in the soil. One also sees the SEIS covered by the white hemispherical (clean) WTS. The distance between them is close to 1.18 m. The Figure also provides a view of the landing area, that was selected to be flat, with low rock abundance and mainly composed of sandy regolith (Golombek et al., 2017).

Figure 4, extracted from the Mars Orbiter Laser Altimeter (MOLA) shaded topographic map of Mars (Golombek et al., 2018), shows the landing sites of InSight (NSY) in Elysium Planitia, together with those of i) the Viking Lander 2 (VL2, 1976) in Utopia Planitia, ii) of the Mars Science Laboratory (MSL) Curiosity rover in Gale crater (2012) and iii) of the Mars Exploration Rover (MER) Spirit in Gusev crater (2004). InSight landed on ~200 m thick lava flows from Early Amazonian-Hesperian period (1.8 - 3.6 billion years) (Golombek et al., 2018, 2020a), underlain by sedimentary rocks of likely Noachian age (around >3.6 billion years, Pan et al., 2020) possibly containing phyllosilicates. The

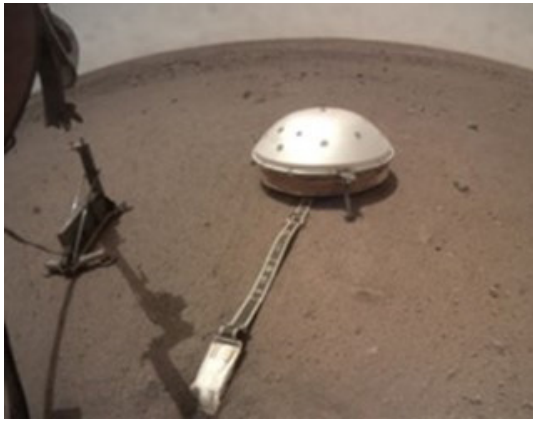


Figure 3. Image taken from the Instrument Context Camera (ICC) on sol 87 (local time 11:54:07 LMS) showing the location of SEIS and of the HP³ support structure, 1.18 metre apart from each other (image credit: JPL/NASA).

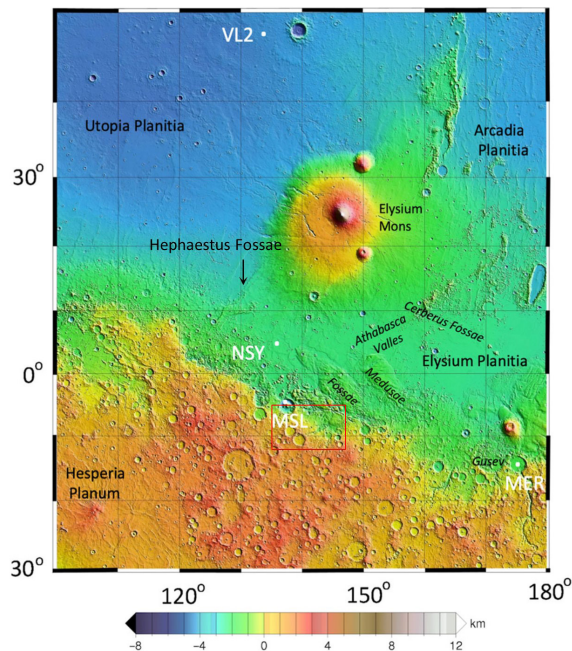


Figure 4. Locations of the InSight lander (NSY) close to the equator (4.502°N/135.623°E), not far from the dichotomy boundary that separates, on Mars, the southern heavily cratered older highlands from the northern lowlands (see colour scale). The map is a portion of the Mars Orbiter Laser Altimeter (MOLA) shaded topographic map of Mars (after Golombek et al., 2020a).

Figure also shows Mars' dichotomy, with the northern hemisphere (mainly in blue, corresponding to a -4 km altitude) at lower elevation than the southern hemisphere (red colour is $+3.5$ km). The southern hemisphere is much more heavily cratered (older) than the northern plains.

Figure 5 shows an ICC photo taken in Autumn 2022 after 4 years of mission. The photo shows that all devices are covered by a dust layer that also covered the solar arrays, finally reducing the energy production. As a consequence, the mission ended on 15 December 2023 for lack of energy.

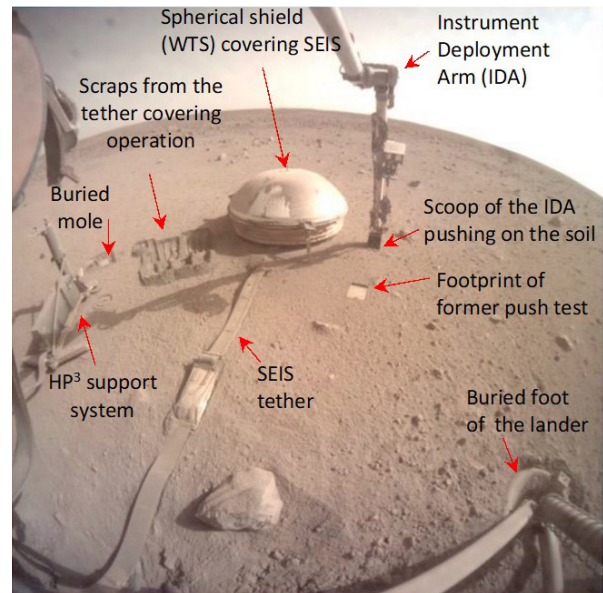


Figure 5. Image taken by the Instrument Context Camera on sol 1170 showing the SEIS WTS (diameter 720 mm) with the Instrument Deployment Arm (IDA) vertically pushing on the soil at the right of the SEIS. As described in more details in Section 5.3.2, the IDA is performing the second elastic pushing experiment, resulting in a tilt monitored by SEIS. The rectangular imprint of the back of the scoop is from the indentation and first elastic experiment. All instruments are covered by a dust layer progressively brought by the Martian winds (image credit: NASA-JPL).

In the Figure, the IDA (Golombek et al., 2023) is pushing on the regolith surface in an attempt to further identify the elastic parameters of the surface thanks to the tilt detected by SEIS, as will be commented in Section 5.3.2. A flat footprint made by a former push test by the IDA can also be observed. The HP³ support system is to the left of the SEIS and the location where the mole is buried (from which the support system has been removed, see below) is indicated. Some scraps made by the IDA scoop to cover the SEIS tether by pouring regolith from the scoop for a better thermal insulation and for cleaning the solar arrays can also be seen at the left of the WTS (Golombek et al., 2023). The WTS is also covered by dust, except in the zone where the regolith has been poured on it, resulting in a cleaner white section.

3. The geology of the InSight landing site

The investigation of the surface deposits on Mars has been considered in detail since the landing of Vikings 1 and 2 in 1976 (e.g. Moore & Jakosky, 1989). It has afterwards been completed by data from landers and rovers of other missions. Global thermal inertia (see Section 4.1) and albedo measurements of Mars along with surface investigations by nine landers and rovers show that Mars is covered by a surface layer of generally fine-grained materials that is metres thick (Christensen & Moore, 2008; Golombek et al., 2008). These materials have been mostly modified by mass

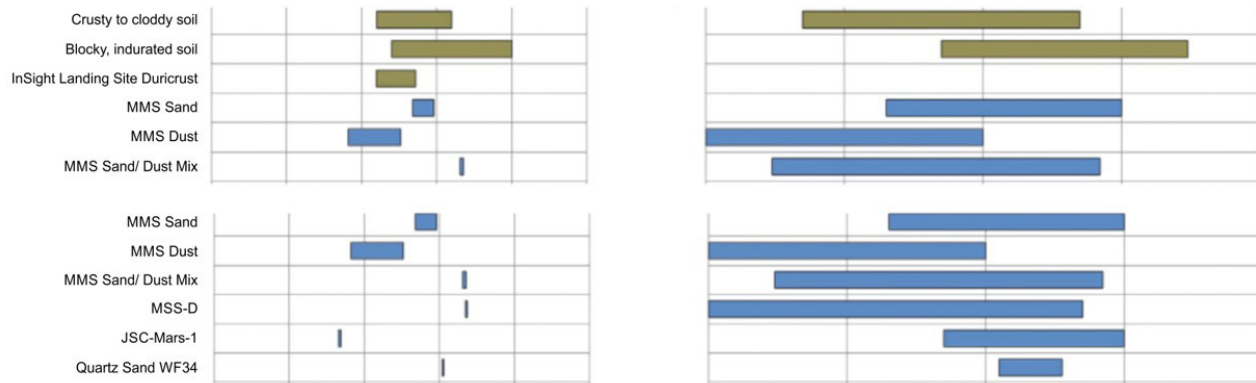


Figure 6. Various soils observed at the surface of Mars, after Spohn et al. (2022b), based on compilations from Golombek et al. (2008) and Herkenhoff et al. (2008).

wasting, aeolian and impact processes. All of the landers and rovers obtained information about the surface materials at their landing sites and along rover traverses using cameras (some high-resolution), arms and scoops, drills and rock abraders, and rover wheels. Most of the soils encountered are composed of dominantly sand size materials with some cohesion (Christensen & Moore, 2008; Herkenhoff et al., 2008). The sand is basaltic in composition, mostly 0.05 to 0.3 mm diameter (very fine to fine sand) and individual grains are generally equant to very equant and subrounded to rounded (Yen et al., 2005; Goetz et al., 2010; McGlynn et al., 2011, 2012; Minitti et al., 2013; Ehlmann et al., 2017) suggesting an origin via impact and aeolian activity (McGlynn et al., 2012; Golombek et al., 2020d).

A synthesis of the soils observed on Mars is presented in Figure 6, in which a distinction is made between i) drift materials, observed at surface and made up of very small particles between 1 and 10 μm with densities between 1000 kg/m^3 and 1300 kg/m^3 , sand (between 1100 kg/m^3 and 1300 kg/m^3), crusty to cloddy soil and blocky, indurated soils.

At the InSight landing site, the granular regolith results from the long-term and successive impacts of meteors into the lava flows of Elysium Planitia, that progressively broke the basalt parent rock into smaller and smaller rocks and particles, that are afterwards submitted to erosion and sorting by gravity and winds, to end up with a sand-like surface regolith (Golombek et al., 2017). Estimates of the regolith thickness is schematically represented in Figure 7, where two impacts are represented (Warner et al., 2017; Golombek et al., 2020d). When the impact takes place only in the regolith (left hand side of the Figure, smaller impactors), the ejecta around the crater is only composed of regolith. When the impactor comes in contact with the bedrock (right hand side, bigger impactors), rocks are present in the ejecta around the crater.

This can be seen in the photo of Figure 8, taken from the HiRISE (High Resolution Imaging Science Experiment) camera on board the Mars Reconnaissance Orbiter (MRO). The fresh crater A is 112 m diameter with rocky ejecta around

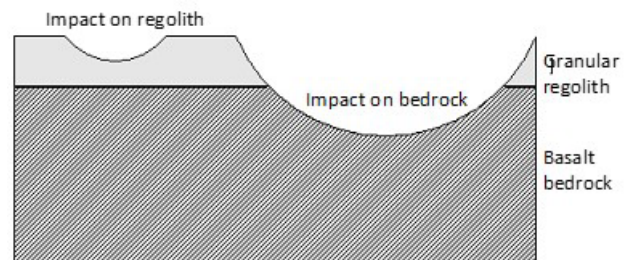


Figure 7. Schematic representation of the regolith formation from impacts (with impactor coming at 5 km/s) either touching the bedrock (right hand side, larger impactors), with blocky ejecta around the crater, or only touching the regolith (left hand side), with no blocky ejecta around the crater.

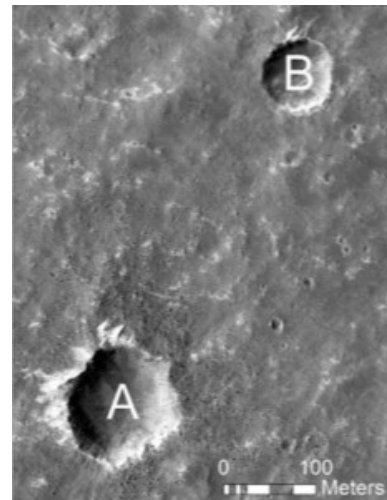


Figure 8. HiRISE image showing fresh crater A (112 m diameter), with rocky ejecta around and B (75 m) with no rocky ejecta around. The depth at which ejecta is sourced in fresh craters is 0.084 times the diameter (Golombek et al., 2017). Note that Warner et al. (2017) and Golombek et al. (2020c) have a more specific treatment of the regolith depth.

(little black dots), whereas fresh crater B (75 m diameter) has no rocky ejecta. Given that the depth at which ejecta is sourced in fresh craters is 0.084 times the diameter,

the image indicates that the bedrock depth is larger than $0.084 \times 75 = 6.3$ m and smaller than 9.4 m. This was an important criterion for site selection (Golombek et al., 2017), since it was planned during the mission to penetrate the HP³ thermal probe down to 5 m.

Aeolian activity (motion of small particles by the wind) is common on Mars, with some active dunes and ripples. At InSight, aeolian bedforms are rare, with some ripples near relatively fresh impact craters (e.g. Figure 8). The ripples are dusty, suggesting relative inactivity (Golombek et al., 2020a) and a surface with little aeolian activity except for wind vortices that produce little net motion of grains in the modern era (Charalambous et al., 2021; Baker et al., 2021).

The stratigraphy is illustrated by the HiRISE photo of Figure 9, showing a steep scarp of Hephaestus Fossae exposure (see location in Figure 4), a fracture ~900 km northwest of the InSight landing site that cuts through a potentially analogous terrain (Warner et al., 2017, 2022; Golombek et al., 2018). The photo shows ~4-5 m thick, relatively fine-grained regolith overlying blocky ejecta that grades into strong, jointed bedrock. Given that the bedrock is composed of basalt lava, the regolith is mainly basaltic as well.

The image analysis carried out on Figure 10 (Warner et al., 2022) shows the profile of the steep scarp with, on top, a 4-5 m thick granular layer (with a maximum slope of 20°, smaller than the angle of repose estimated at 30°), underlain by a 13-29 m thick boulder-dominated cliff-forming unit, underlain by another cliff-forming unit (112-133 m thick) that may be the primary bedrock, with a talus at its base inclined at 32° (close to the estimated angle of repose of loose granular materials, see Delage et al., 2017). The estimation of the grain distribution of the talus material is not straightforward, but it is probable that it is composed from part of fine regolith that flowed down from the top, together with other rocky debris with size comprised between 1 m and 10 m coming from the upper layer of blocky ejecta. The entire stratigraphic sequence is consistent with an impact-comminuted lava

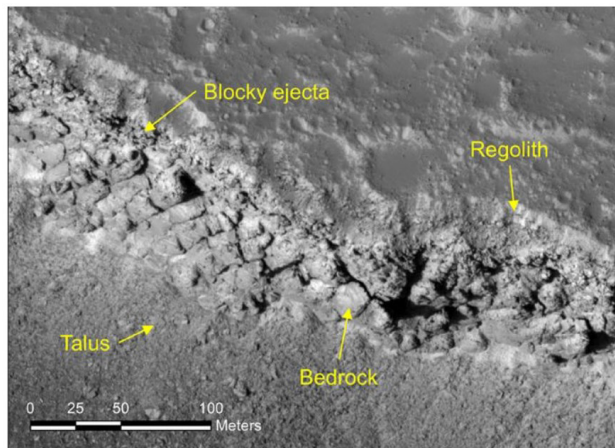


Figure 9. A portion of the exposed steep scarp of Hephaestus Fossae in southern Utopia Planitia (Golombek et al., 2018).

plain with a gardened regolith that is up to 10 metres thick. The regolith fines upwards is a result of the higher frequency of small (order of 1 to 10 m) impacts.

4. Physical and mechanical data from previous investigations

Some physical properties of the regolith at the landing site, estimated prior to landing, either from orbiter measurements or from lab testing on regolith simulants, are now presented.

4.1 Thermal inertia

The main information about the nature of the surface soil gained from orbiters is the thermal inertia I , that is used to estimate the average grain size of surface sandy deposits. Thermal inertia (unit $\text{Jm}^{-2}\text{K}^{-1}\text{s}^{-1/2}$) is given by:

$$I = \sqrt{k\rho c_p} \quad (1)$$

where k is the thermal conductivity, ρ the density, and c_p the specific heat capacity of the material. The principle that allows derivation of average particle size from thermal inertia is simple: for the same mass, the changes in temperature of a particle

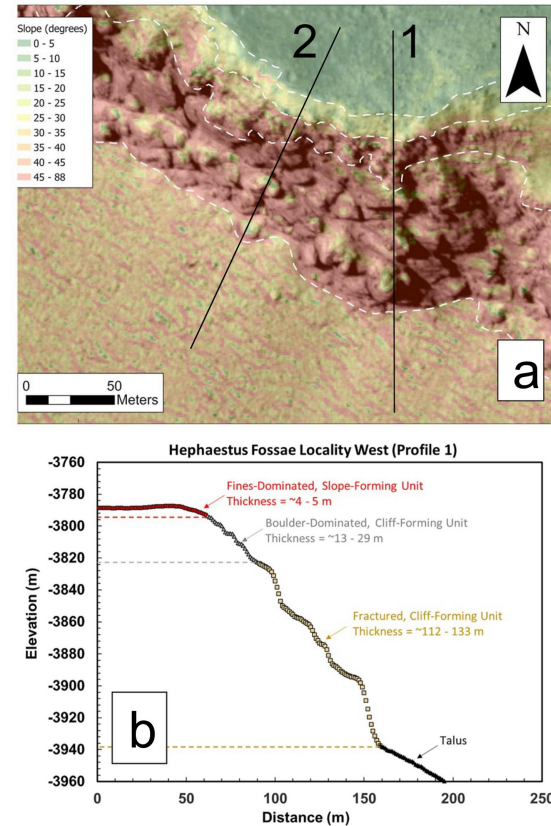


Figure 10. (a) HiRISE image of the Hephaestus Fossae exposure, a fracture ~900 km northwest of the InSight landing site (see Figure 4); (b) Slopes determined along profile 1 (Warner et al., 2022).

(the average temperature of the atmosphere on Mars changes as much as 100 °C between days and nights) are slower for larger particles (small specific surface) and faster for small particles (high specific surface). Note however that inter-grains cementation in slightly cohesive granular materials may increase the thermal inertia. While density and specific heat capacity vary little for different soils, thermal conductivity can vary by orders of magnitude, depending on bulk porosity, composition, grain size and the state of cementation or induration.

Figure 11 shows a colour map of thermal inertia values characterised around the InSight landing site by the THEMIS (Thermal Emission Imaging System) instrument on board of the Mars Odyssey spacecraft launched in 2001 (e.g., Palluconi & Kieffer, 1981; Mellon et al., 2000; Putzig & Mellon, 2007; Golombek et al., 2008).

The image in Figure 11 shows an area of relatively constant thermal inertia, in accordance with what is observed around the lander (see Figure 3). Orbital thermal inertia measurements indicated values between $160 \text{ J m}^{-2} \text{ K}^{-1} \text{ s}^{-1/2}$ and $230 \text{ J m}^{-2} \text{ K}^{-1} \text{ s}^{-1/2}$ (Golombek et al., 2017), which were later confirmed by local measurements using the InSight HP³ Radiometer (Spohn et al., 2018; Golombek et al., 2020a; Mueller et al., 2021; Piqueux et al., 2021). The average grain size derived from this value is around 170 μm . In Figure 11, high thermal inertia data are in red. Some red dots can be observed in rocky ejecta craters with I around $300 \text{ J m}^{-2} \text{ K}^{-1} \text{ s}^{-1/2}$ (craters A, B), comparable with what can be directly observed in Figure 8.

4.2 Wave velocities and elastic properties of regolith simulants

A previous estimation of wave velocities at surface was helpful to interpret the surface wave data recorded by SEIS. To do so, a laboratory investigation of wave velocities was carried out in the laboratory, prior to the mission, on various regolith simulants. The selection of a relevant Martian regolith simulant (Seiferlin et al., 2008) is not an easy task. In the case of InSight, four simulants have been studied to match the range of parameters expected on Mars (Golombek et al., 2008, 2017; Herkenhoff et al., 2008, see Figure 6). These simulants, often considered for missions on Mars, are the Mojave Mars Simulant (MMS, Peters et al., 2008), the Eifelsand and the MSS-D (Mars Soil Simulant-D) simulants

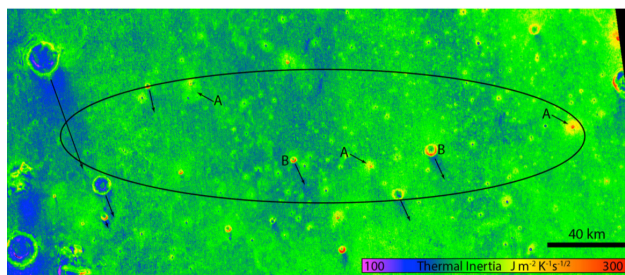


Figure 11. Thermal inertia values at the InSight landing site (Golombek et al., 2017).

(see Delage et al., 2017), together with a fourth simulant, the Fontainebleau sand (NE34). Compared to the first three that were too angular, the Fontainebleau sand is a well sorted sub-rounded to rounded sand, preferable to reflect the mechanical effects of long-term saltation on the grains of Martian surface regoliths. A detailed description of the MMS simulant, of the Eifelsand and MSS-D simulants are given in Delage et al. (2017). The Fontainebleau sand, located in the Fontainebleau Forest near Paris, France, is almost exclusively composed of hyaline quartz (Andria-Ntoanina, 2011).

The grain size distribution (GSD) curves of the four simulants, presented in Figure 12, indicate that the GSD of MSS-D is clearly bi-modal (with both a 50% sand fraction - larger than 100 μm - and a 50% fraction of fine-grained crushed olivine, smaller than 100 μm in the silt size range). The GSDs of MMS and Eifelsand simulants are somewhat comparable, with however a greater percentage of smaller particles in the Mojave simulant. The Fontainebleau sand has a smaller average grain diameter $D_{50} = 220 \mu\text{m}$, a little bit larger than the 170 μm estimated value at the InSight site. The sand is quite well sorted around its mean diameter, with a uniformity coefficient $C_u = 1.57$.

The seismic velocities were measured by using bender elements inserted in top and bottom of triaxial specimens and able to generate and receive both shear and compression waves. The size of the specimen for the MMS, Eifel and MSS-D simulants was 50 mm in diameter and 100 mm in height. These simulants were tested from low to high confining stresses (i.e. 20 to 500 kPa), with densities of 1670 kg/m^3 for MSS-D, 1550 kg/m^3 for MMS, and 1350 kg/m^3 for Eifelsand (Delage et al., 2017). When testing the Fontainebleau sand (Castillo-Betancourt et al., 2023), one tried to better replicate the Mars conditions of low density and low stresses. A larger cylindrical specimen (100 mm in diameter and 170 mm in height) was used to have a longer wave travel path. To achieve a low density, the pluviation technique (Kolbuszewski, 1984) was used with very small fall height, allowing a low value of dry density of about 1400 kg/m^3 , corresponding to a density index I_d of 6%. To minimize stress and have a homogeneous stress field, bender elements

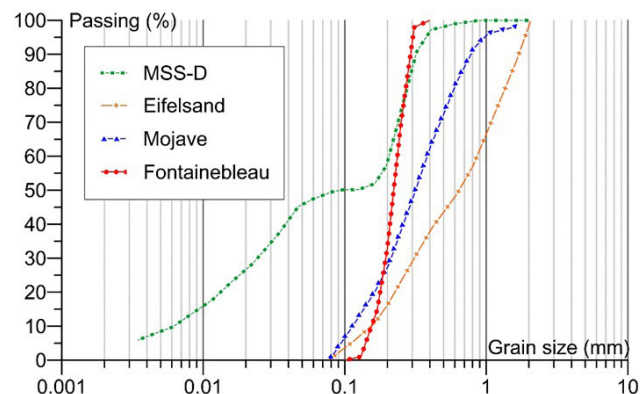


Figure 12. Grain size distribution of the Martian simulants.

measurements were carried out on horizontal samples, with careful calibration of stresses due to the membrane and to self-weight. Stress was changed by applying vacuum to the sample, up to 80 kPa. Bender measurements could be carried out at stress as low as 1 kPa (with however more dispersion in this stress range), enlarging at low values the range of stress generally considered in geotechnical engineering. The travel time was determined by using the common peak-to-peak approach, as shown in Figure 13. The Figure also shows the interaction between both P and S waves: some slight perturbation is observed in the S wave when the P wave first arrives and the same holds for the P wave at arrival of the S wave.

Figure 14 shows the evolution of the P and S wave velocities with respect to the effective isotropic confining stress p . In a standard fashion, the increase in velocity is non linearly related to the increase in intergranular forces resulting from the increase of confining stress. For the three simulants tested at higher stresses (i.e. Mojave, Eifel and MSS-D, Delage et al., 2017), the compression wave velocity increased from approximately 250 m/s at 25 kPa to around 600 m/s under 500 kPa. The measurements were carried out at lower stress in the Fontainebleau sand, down to 2.6 kPa (Castillo-Betancourt et al., 2023) with a value of 110 m/s, increasing

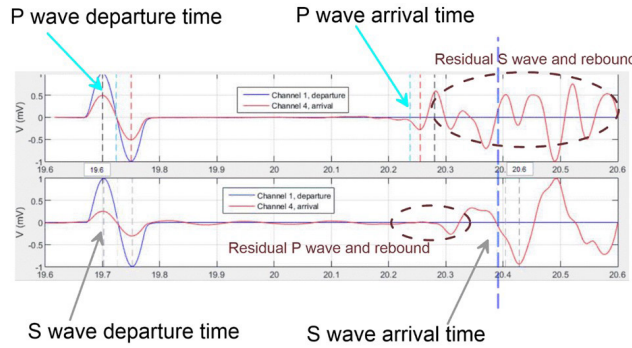


Figure 13. Peak-to-peak approach and S-wave arrival (Castillo-Betancourt et al., 2023).

to 360 m/s at 80 kPa, close to the values of the Mojave and Eifelsand simulants. A good comparability between those three simulants is observed, whereas the data of the finer MSS-D simulant are distinct, with larger velocities. The data also provide an estimation of the velocities at surface, close to 110 m/s for V_p and 70 m/s for V_s .

The changes in seismic velocities with respect to the confining stress p are defined by an empirical power law (e.g. Santamarina et al., 2001), as follows:

$$V_p = \alpha_p \left(\frac{p}{1 \text{ kPa}} \right)^{\beta_p}, \quad V_s = \alpha_s \left(\frac{p}{1 \text{ kPa}} \right)^{\beta_s} \quad (2)$$

in which parameters α_i and β_i are experimentally determined; α_i is the velocity under 1 kPa confinement.

As recalled by Santamarina et al. (2001), the stiffer the particles and the denser the packing, the higher the value of α and the lower the β exponent. Note that theoretical values of β are 1/6 for Hertzian contacts between elastic spherical spheres, 1/4 for cone-to-plane contacts (typical of rough or angular particles) and 1/4 for spherical particles with yield. As shown in Figure 6, the values reported for sands by Santamarina et al. (2001) are between 0.12 and 0.28, with a linear relationship between β_s and α_s . Interestingly, our values are in line with Santamarina's et al. (2001) data (Figure 15).

The measurements of V_p and V_s make it possible to calculate the Poisson's ratio (ν) over the studied range of stresses using the following equation:

$$V_p / V_s = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}} \quad (3)$$

giving:

$$\nu = 0.5 \frac{(V_p / V_s)^2 - 2}{(V_p / V_s)^2 - 1} \quad (4)$$

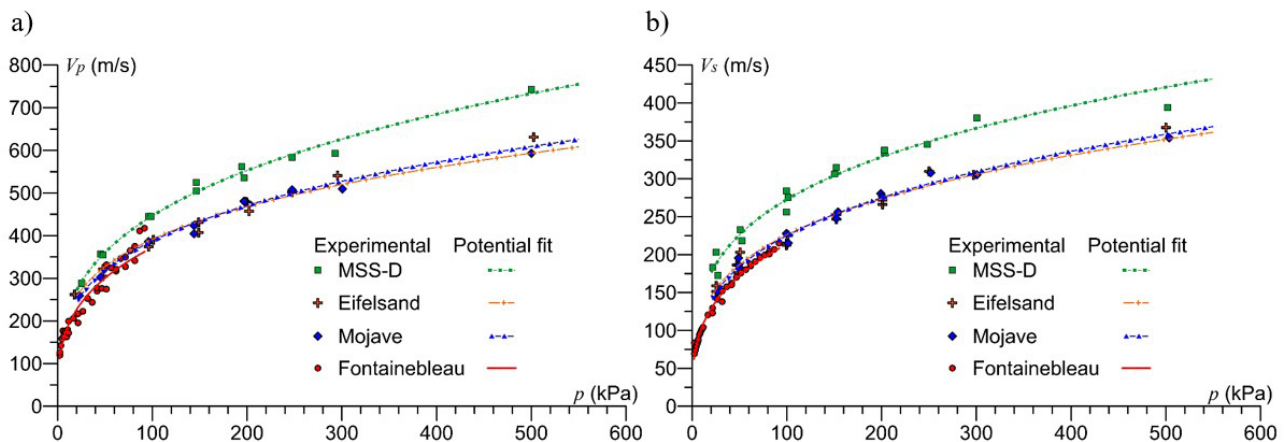


Figure 14. Compressive wave velocity of the four simulants: (a) Compression waves V_p ; (b) Shear waves V_s .

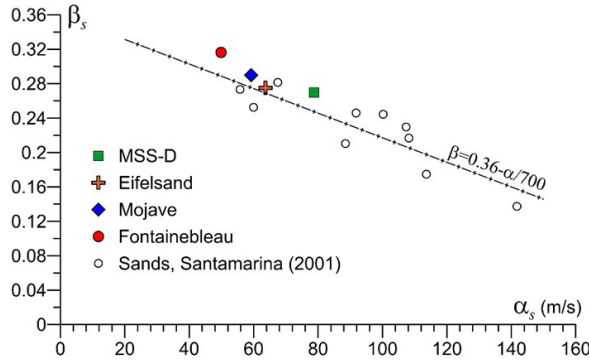


Figure 15. Values of α_s and β_s of sands and the tested regolith simulants (see Equation 2).

As shown in Delage et al. (2017), comparable values of v around 0.22 were obtained for the three simulants investigated (MSS-D, Eiffelsand and Mojave). Conversely, a smaller value around 0.14, with no significant change with respect to stress above 10 kPa, was determined for the Fontainebleau sand, as seen in Figure 16. This is in agreement with the data obtained in-situ by Bachrach et al. (2000) on a beach sand. The change of v at low stresses (< 5 kPa) is more difficult to characterise, due to significant dispersion in a zone where the bender elements become less operational. Further investigation is needed in this zone of low stresses.

Knowing V_p , V_s and the density also allows to determine the Young's modulus E , according to the following expression:

$$E = \rho V_p^2 \left(\frac{3 - 4 / (V_p / V_s)^2}{(V_p / V_s)^2 - 1} \right) \quad (5)$$

The changes in Young's modulus for the four simulants investigated are presented in Figure 17.

In a standard fashion, the modulus increases with the confining stress (Jamiolkowski, 2012). The curves also indicate an order of magnitude of the Young's modulus at low stress at surface, with values around 20 MPa for the Fontainebleau sand (for which smaller stresses have been considered in the experimental investigation), a little bit larger for the other simulants. Note that this value is in good agreement with those derived from a small strain investigation of the interaction between the foot of the SEIS seismometer and loose samples ($\rho = 1400 \text{ kg/m}^3$) of Fontainebleau sand (Delage et al., 2022).

Since roughness effects at the inter-grains contact are suspected to be more significant at low stresses like in the case of Mars, a detailed analysis based on the mechanics of contacts has been carried out by Caicedo et al. (2023) based on the works of Bachrach et al. (2000), Bahrami et al. (2005) and Butt et al. (2015). Figure 18 shows the changes in wave velocity with respect to stress for the Fontainebleau sand. They are no longer based on a semi-empirical power law like in Figure 14, but result from the contact theory with an

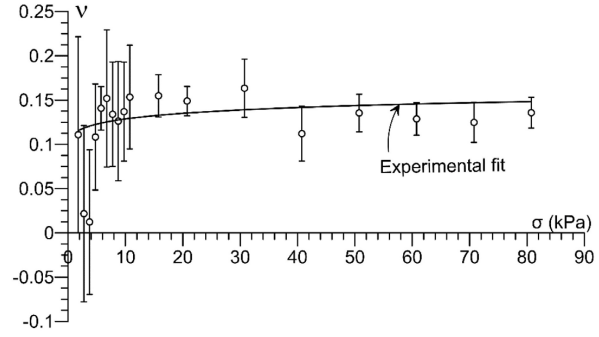


Figure 16. Poisson's ratio calculated for the wave velocities measured on Fontainebleau sand (Castillo-Betancourt et al., 2023).

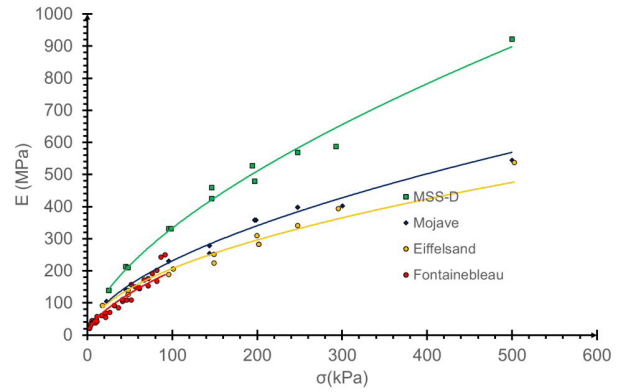


Figure 17. Changes in Young's modulus with respect to the confining stress.

average roughness parameter σ_{rms} between $0.6 \mu\text{m}$ and $0.8 \mu\text{m}$ (a little bit larger than the $0.2 \mu\text{m}$ identified by Atomic Force Microscopy on a Fontainebleau sand grain, see Caicedo et al. 2023). Based on Bachrach et al. (2000), Figure 18b presents the changes in Poisson's ratio with stress obtained with a fraction ξ of non-slipping grains between 0.55 and 0.65, with best fit for $\xi = 0.6$. The Figure also shows that the Poisson's ratio is constant down to 5 kPa, whereas the dispersion at low stress does not allow to draw a conclusion on a possible decrease below.

5. Direct determination of the regolith properties

5.1 Direct observations at surface

Direct observation of the regolith particles on Mars has been made by the microscopic imagers of the Spirit, Opportunity (McGlynn et al., 2011) and Curiosity rovers, the Phoenix lander and by the atomic force microscope of the Phoenix mission (Pike et al., 2011). They revealed sorted dark grey basaltic sub-rounded to rounded fine sand particles due to wind saltation, with diameter between 80 and 200 μm , as seen in Figure 19 from the Phoenix mission (Goetz et al., 2010). This range is compatible with the average diameter (170 μm) derived from thermal

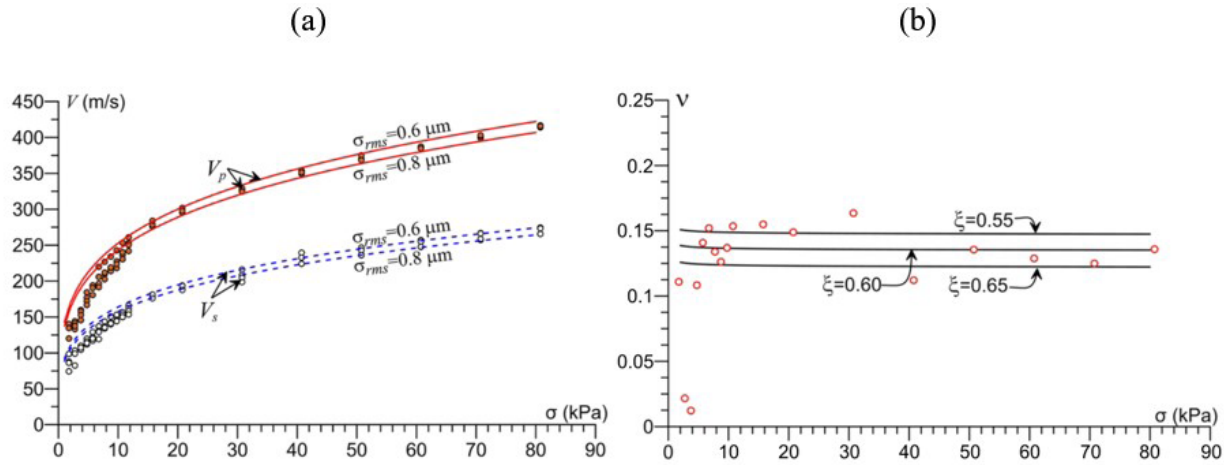


Figure 18. (a) Comparison between the theoretical contact model and test data on Fontainebleau sand (σ_{rms} is the asperity parameter); (b) Changes in Poisson's ratio with stress (ξ is the fraction of non-slipping grains, Caicedo et al., 2023).

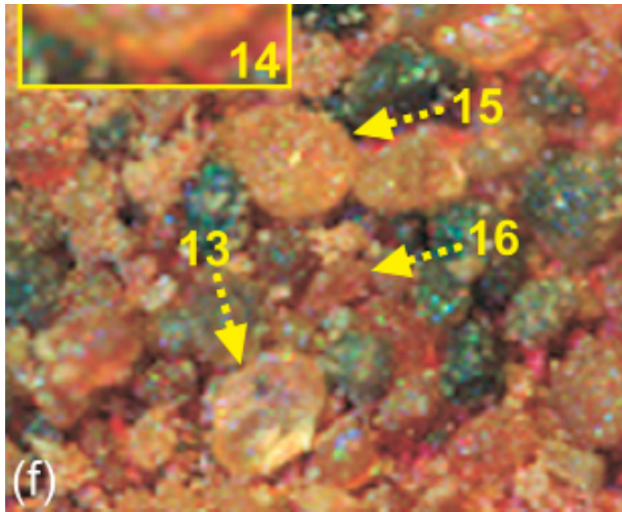


Figure 19. Microscopic observation of sub-rounded to rounded particles of Mars regolith at the Phoenix site. Picture width is 500 μm (Goetz et al., 2010).

inertia measured at the InSight landing site. Bright red dust particles (2-5 μm) of slightly chemically altered basalt are also observed (Arvidson et al., 2008, 2009, Goetz et al., 2010).

The colour of the particles in Figure 19 varies from dark grey (black) to slightly reddish. This probably depends on the degree of contamination of their surface by red fines, or on surface stains (Goetz et al., 2010).

Figure 20 shows the walls of the various pits (P1, P2 and P3) formed during the excavation of the surface by the rocket exhaust beneath the lander during landing. It provides some information on the true nature of the regolith at the InSight landing site. The Figure shows that, in fact, the soil is not a well sorted subrounded to rounded (cohesionless) sand, as suggested from thermal inertia measurements (further observations - see more details later

on - indicated that this is however true for the 1 cm surface layer). One observes in the Figure, along the 10 - 12 cm height of the steep pit walls, a more resistant layer having enough cohesion to allow the static stability of the walls (and the stability with respect to the rocket exhaust, which would probably require a much larger cohesion). Note that the layers of crust and duricrust could be cemented by salts deposited by thin films of water via interactions of atmospheric water vapour and soils, as suggested by chemical measurements by Viking and Mars Exploration Rover spacecraft (Banin et al., 1992, Haskin et al., 2005, Hurowitz et al., 2006). The effects of rocket exhaust are quite localized, with an average pit diameter of 50 cm. One observes, on the eroded walls ~ centimetre-size pebbles embedded in a finer cohesive grain matrix. Observation of pits (*Pi*) and of other excavations (*di*) also show some larger black angular likely basaltic pebbles.

5.2 Mechanical and thermal data from the self-penetrating HP³ device

The Heat Flow and Physical Properties Package HP³ was planned to measure the temperature profile and a thermal conductivity profile down to a depth of 3-5 m, together with related physical properties. The temperatures would have been measured using sensors on a Kapton tether which a small self-penetrating device (the mole) would have dragged down to the targeted depth (Spohn et al., 2018). Pausing penetration at depth intervals of 50 cm, a thermal conductivity profile would have been measured using sensors embedded in the mole (e.g., Grott et al., 2019; Spohn et al., 2018) and thermal diffusivity would have been determined using the temperature gradient and the thermal conductivity, the heat flow from the interior of Mars would have been calculated (provided that a minimum depth be reached to avoid perturbation caused by seasonal variations of the surface

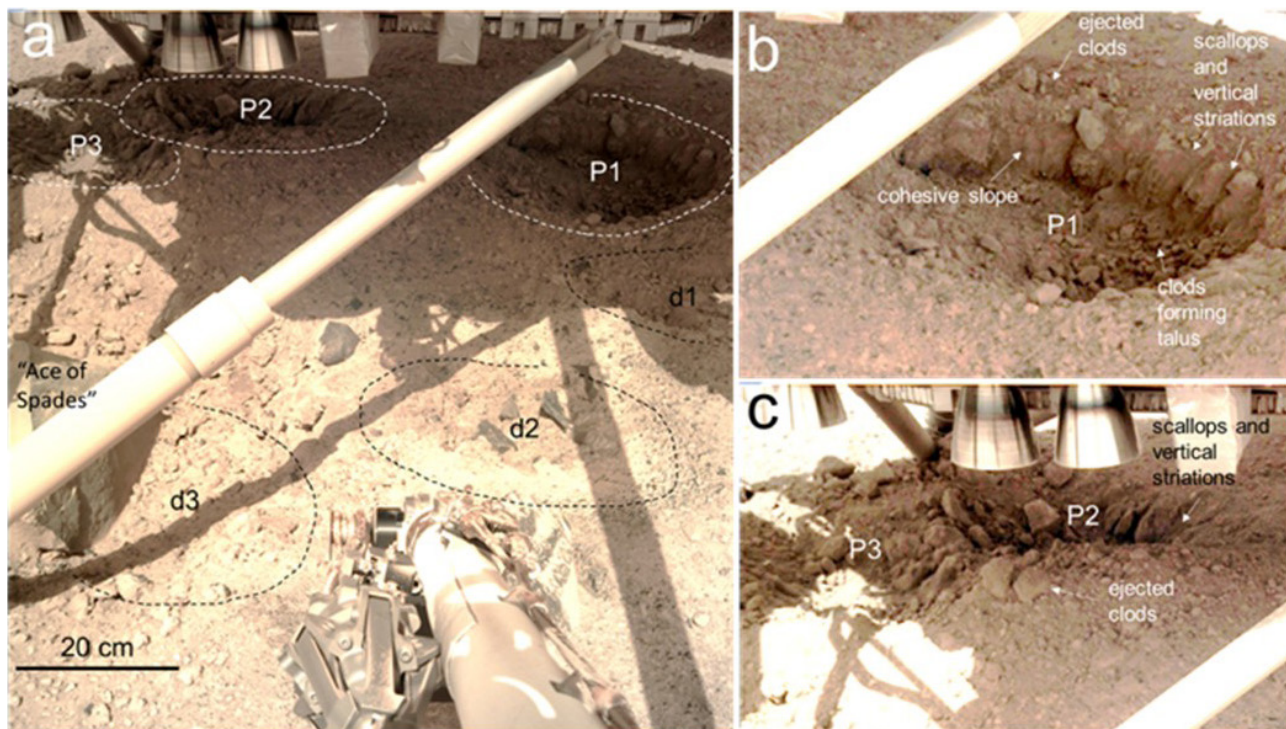


Figure 20. IDC images taken beneath the lander on sol 18. (a) The pits and depressions were excavated by the lander's retrorockets. P1 and P2 are the largest and deepest pits; (b) P1 is ~50 cm in diameter and reveals ~12 cm of the shallow stratigraphy; (c) View in more details of pits P2 and P3. The pit walls are vertically striated and steep (up to 65°) and expose an up to ~10 cm-thick cohesive duricrust comprised of cohesive fine sand. The floor of each pit contains abundant reddish, pebble-size clasts or clods of material that broke off the steep pit walls. Dark-grey pebbles of likely basaltic composition are visible within the matrix of fine sand (Warner et al., 2022).

temperature). Inversion of the thermal conductivity and diffusivity measurements would have allowed an estimate of the density as a function of depth, assuming the soil's heat capacity to be known. A mechanical strength profile for the soil would have been derived from the penetration rate that would have been recorded.

5.2.1 Mole penetration

Unfortunately, the mole did not penetrate deeper than ~37 cm at a final tilt of 30°, bringing the mole back-end 1-3cm below the surface. The record of the mole penetration attempts and related activities and inferences on the properties of the soil have been described in Spohn et al. (2022a). Lessons learned from the mole about penetrators have been discussed in Spohn et al. (2022b). Penetration attempts had started on sol 92 with 4000 hammer strokes. Instead of reaching the targeted mole tip depth of 70 cm, the tip was stuck at a then unknown depth (it was later determined that it had reached 32 cm depth). After a second attempt on sol 94 that brought no measurable progress and an intense campaign of diagnostics an almost two-year campaign of mole recovery was started on sol 308. Figure 21 shows a record of that period until the final penetration test on sol 754. Among the initial hypotheses for the mole's failure to penetrate were a) a stone blocking penetration and b) a lack

of friction on the mole hull from the cohesive duricrust to balance recoil. The duricrust had already been observed in the pits excavated by the rocket exhaust below the lander (Figure 20). To provide additional friction, the mole was first pinned (pinning 1) using the scoop at the end of the Instrument Deployment Arm (Golombek et al., 2023). The techniques to help penetration are schematically shown below the graph. The schemes also indicate the 7 cm deep and 5-6 cm wide pit that had formed during the sol 92 and 94 hammering sessions by the mole precessing in the duricrust about a point midway along its hull.

The penetration attempts with pinning between sols 308 and 318 were successful bringing the mole 5 cm deeper in. This essentially disproved the conjecture of a stone blocking the way forward and supported the conjecture of a lack of friction from the duricrust. Because the mole was then too deep in for pinning to continue, it was tried to provide friction by pressing the scoop onto the surface next to the mole. The vertical stress would increase the overall stress around the mole and provide friction. This attempt failed and resulted in a reversal upward movement of around 28 cm (reversal 1) with an increase in tilt from 10° to 17°. The reversal was explained by sand filling in underneath the mole when it moved upwards during bouncing motions (the bouncing motions being the result of insufficient friction). The observation that the reversal motion ended when the mole tip was at a

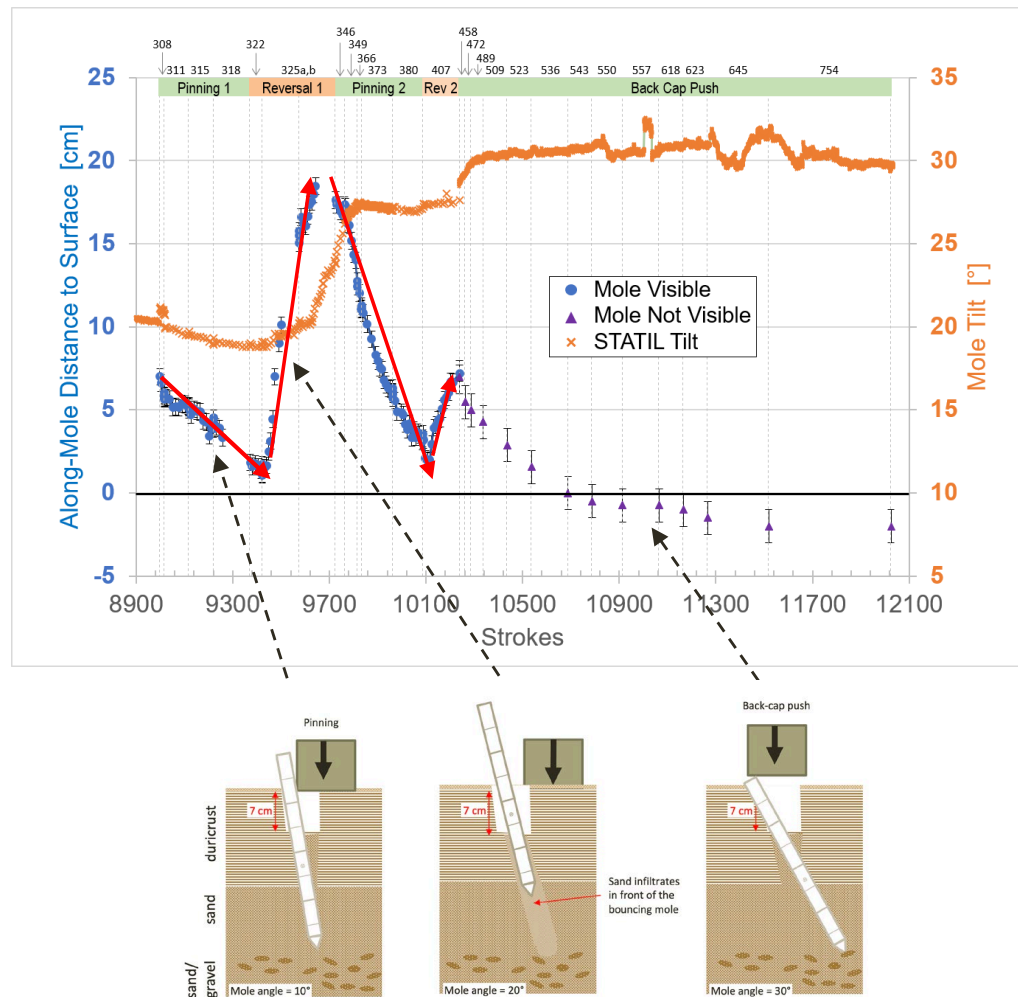


Figure 21. Mole penetration, strokes 8900 to 12100, described by the blue points in terms of distance between the back-end of the mole and the surface (given in cm in the left-hand vertical axis, - 0 means that the mole is fully below the surface). The orange points give the mole inclination to vertical (right-hand vertical axis, in degrees). Individual sols when hammering occurred are indicated along the top border; vertical dashed lines show the boundaries of each sol's planned hammer strokes. The major periods of successful mole penetration (Pinning 1, Pinning 2, and Back Cap Push) are indicated by green horizontal bars along the top, while major periods of mole reversal (Reversal 1 and Reversal 2) are in orange (after Spohn et al., 2022a).

depth of about 20 cm was taken as indication that the mole had reached the bottom of the duricrust where sand was no longer available to freely fall in underneath the mole tip.

A second penetration (pinning 2), comparable to the first one was achieved at almost the same penetration degree, followed by a second reversal (rev. 2). Finally, it was decided to press the scoop directly on the top of the mole, achieving full penetration at sol 540 (with a final tilt of 30°).

Figure 22 presents an IDC photo of the “back cap push” session, in which the scoop of the IDA is compressing the regolith above the hole that developed during hammering, with the tether appearing on the left side of the scoop. The photo confirms the sand-like aspect of the surface regolith, with no rock in the area and some centimetre-sized pebbles around. Note also the perfectly smooth imprint of the scoop observed between the right side of the scoop and the elongated pile on the right. Such smooth surfaces have also been observed



Figure 22. IDC photo showing the scoop of the Instrument Deployment Arm pressing on the mole during the Back Cap Push session, with the tether on the left side. The scoop is 7.6 cm in width and 10 cm in length (image credit JPL/NASA).

on rover tracks in other missions and indicate that the fine particles contained in the sandy layer provide some degree of cohesion (Golombek et al., 2020a). A smooth surface was also observed during the indentation and pressing experiments carried out with the scoop, as shown in Figure 5 (see Section 5.3.2).

The mole was designed to penetrate cohesionless soil like quartz sand, which was initially expected to provide a good mechanical simulant material for Martian sand (see Section 4.2). The sand would provide friction to the buried mole hull to balance the remaining recoil of the mole hammer mechanism that drives the mole forward. Although a suppressor mass and spring in the hammer mechanism absorbed much of the recoil, the available mass did not allow a system that would have eliminated the recoil altogether (Spohn et al., 2022b). The root cause of the failure - as was determined through an extensive, almost two years long campaign (Spohn et al., 2022a, b) - was thus a lack of friction in an unexpectedly thick cohesive and brittle duricrust. In addition, it was found that the Martian soil provided unexpected levels of penetration resistance. A final “Free Mole Penetration” test in January 2021 to see whether the mole could continue penetrating without arm support failed, unfortunately, and the campaign was ended, also because of dwindling energy resources on the lander. The mole body remained 2-3 cm below the surface at which depth thermal conductivity measurements could be performed (Grott et al., 2021, 2023) for almost a Martian year.

5.2.2 Mechanical and thermal data derived from HP³ measurements

The penetration record of the mole, the thermal conductivity and density measurements, the radiometer data and the seismic data recorded during the hammerings were used to derive a model of the properties of the first ~40 cm of the Martian soil at the landing site (Spohn et al., 2022a). Accordingly, as shown in the schemes of Figure 21, a duricrust of about 20 cm thickness is found underneath an about one-centimetre-thick unconsolidated sand and dust layer. Beneath the duricrust, a sand layer of about 10 cm thickness is found followed by a layer of a sand mixed with gravel, possibly consisting of debris from a small impact crater.

The penetration resistance of the sand/gravel layer is best constrained by the penetration data reported in Spohn et al. (2022a) and was found to be 5.3 MPa while the duricrust has a 5 to 10 times smaller penetration resistance. Compaction during the initial about 9000 strokes hammerings may have caused some densification of the lower layer. Applying cone penetration theory (e.g. Terzaghi & Peck, 1947; Poganski et al., 2017) the resistance of the duricrust was used to estimate a cohesion of the latter of 4-25 kPa, depending on the assumed internal friction angle of the duricrust. Pushing the scoop with its blade into the surface and chopping off a piece of duricrust provided another estimate of the cohesion of about 6 kPa.

There are no direct measurements of soil density at the InSight landing site, but inversion of the thermal conductivity measurements performed by the HP³ TEM-A (Thermal Excitation and Measurement-Active mode, Grott et al., 2021) after the mole was fully buried allows an estimate of the average density of the top 40 cm of the soil of 1211^{+149}_{-113} kg/m³.

The average thermal conductivity was found to be 0.039 ± 2 W/m K (Grott et al., 2021). Based on the scheme of Figure 21, Spohn et al. (2022a) suggest that the thermal conductivity increases from 0.014 W/m K to 0.034 W/m K through the one-centimetre thick surface sand/dust layer, keeps the latter value in the duricrust and the sand layer underneath and then increases to 0.064 W/m K in the sand/gravel layer below. The density is 1300 kg/m³ in the sand/dust layer, 950-1100 kg/m³ in the duricrust, 1300-1500 kg/m³ in the sand layer and 1600 kg/m³ in the sand/gravel layer beyond 31 cm depth. These values are in good agreement with the thermal inertias at the InSight landing site, that correspond to thermal conductivities $k = 0.041 \pm 0.013$ mW m⁻¹ K⁻¹. Due to low atmospheric pressure and gravity typical of Mars, these are very low values compared to those of current terrestrial soils, where k is close to 1 W m⁻¹ K⁻¹.

Grott et al. (2023) have studied the variation of the thermal conductivity through the Martian seasons (solar longitude between 8° and 210°) and with the atmosphere pressure. They found that the thermal conductivity varied by $\pm 5\%$; the error of the measurements being $\pm 2.6\%$. Their results confirm that atmosphere gas contributes in a major way to the heat transfer in the regolith, as experimentally demonstrated by Nagihara et al. (2022). For cohesionless and non-cemented soils, the measured thermal conductivity values indicate that 85-95% of all particles are smaller than 104-173 μ m (Presley & Christensen, 1997; Grott et al., 2021; Piqueux et al., 2021).

Further observation of the near-surface has been made possible by the pit that developed around the mole during hammering, observable once the support structure was removed by the IDA.

The photo of Figure 23 shows the pit created around the inclined mole (inclined 15°), with an almost vertical wall made up of resistant layers containing some pebbles. The steep edge and some overhangs indicate the existence of cohesion. Pebbles are embedded in a fine-grained matrix, indicating cementation. The resulting cohesion is at the origin of the lack of friction that jeopardized the mole penetration.

5.2.3 Pressing experiments on the mole pit

The shear strength parameters of the regolith were determined by analysing experiments conducted with the robotic arm between sols 240 and 250 (Spohn et al., 2022a; Golombek et al., 2023; Marteau et al., 2021, 2022, 2023). In these experiments, pressure was applied on the ground using the robotic arm's scoop near the open pit that formed



Figure 23. Image of the pit (7 cm deep, 5-6 cm wide) created by the HP3 mole during hammering (inclined $\sim 15^\circ$), with the scoop appearing on top right (image credit JPL/NASA).



Figure 24. IDC photo showing the scoop of the IDA compressing the regolith with the mole (not in contact with the scoop, with the tether clearly visible) moved up during a reversal period (see Figure 20). The mole pit can be observed at the left of the scoop (image credit JPL/NASA).

around the HP³ mole during initial hammering, as showcased in Figure 24.

The scoop can be positioned to exert pressure on the ground either with its flat surface (Figure 24) or with its tip. A three-dimensional slope stability analysis was conducted, considering an elasto-plastic Mohr-Coulomb failure criterion as the soil's material model. The soil unit weight in the Martian environment was set at 4.52 kN/m³, which corresponds to a unit mass of 1220 kg/m³ under the gravity of Mars (3.721 m/s²). A friction angle $\phi' = 30^\circ$ was fixed for all calculations, and the cohesion required to fulfil the stability criteria with a safety factor of 1 was determined iteratively.

The dimensions of the HP³ mole pit, as determined by the images acquired by the arm-mounted camera, are 0.045 m in width and 0.07 m in height. The slope inclination of the pit was measured at 85° , indicating a steep slope. The force exerted by the scoop on the ground was estimated based on an algorithm, that takes into account the joint position of the arm as well as the current and torque measurements from the actuators on Mars. However, it should be noted that the acquisition rate of these measurements is low and that they may be subjected to noise, potentially leading to inaccuracies in the force measurements. The force exerted during the flat push was measured to be 29 N, resulting in a pressure of 7.9 kPa, whereas the force measured during the tip push was 45 N, corresponding to a distributed load of 91.9 kPa. The inferred minimum cohesion values for the flat push and tip push were found to be $c' = 0.4$ kPa and 5.8 kPa, respectively (Marteau et al., 2021, 2022, 2023).

5.3 In-situ determination of elastic properties

The elastic properties of the regolith have an important influence on the seismic wavefield and travel times as recorded by the SEIS instrument. A specific example is site effects, i.e., the influence of shallow near-receiver structure on seismic recordings, which are well known in terrestrial seismology. Knowledge of the elastic properties of the regolith will help to better understand these effects which will need to be considered when analysing signals in the affected frequency range.

5.3.1 Local wave velocity measurements

The seismometer SEIS was placed at a distance of around 1.227 m from the HP³ self-hammering mole (Figure 25, see also Figure 3 and Figure 5, Golombek et al., 2020b). At some point during the mission preparation, it was realised that recording the mole hammering signals offered a unique opportunity to study the elastic properties at the landing site, comparable, in terrestrial geotechnics, to some kind of cross-hole test. However, because SEIS was designed to record mars quakes and a direct link between the HP³, SEIS and lander clocks was not established, a series of adaptations had to be implemented. During hammering of the HP³ mole, SEIS was set to a specific mode to enable recording the seismic hammering signals in high temporal resolution beyond the nominal sampling frequency of 100 Hz. Furthermore, an adapted time keeping a clock correlation scheme was implemented to estimate the travel times and, hence, the seismic velocity of the regolith volume between the HP³ mole and SEIS (Kedar et al., 2017; Sollberger et al., 2021; Brinkman et al., 2022).

Brinkman et al. (2022) processed the SEIS waveform recordings of around 2000 hammer strokes (Figure 26). The three-component seismic data were rotated to separate the *P*- and *S*-wave components of the wavefield such that

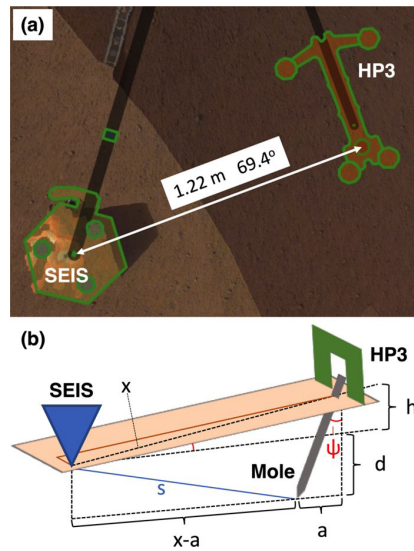


Figure 25. (a) Graphical representation of SEIS and HP³; (b) Schematic illustration of the SEIS, HP³ and mole geometry. SEIS and HP³ are separated by a distance $x = 1.227$ m. The 40 cm-long mole is inclined by an angle $\psi \approx 30^\circ$ (see Figure 20). Figure modified from Brinkman et al. (2022).

travel times for both the P - and S -wave first arrivals could be picked manually for 2271 hammer stroke recordings. The location of the mole tip, which was assumed to have acted as seismic source when hammering, was reconstructed from images taken by the two cameras (Figure 25). The distance s between the tip of the 40 cm-long mole and the centre of SEIS could be determined for 1518 hammer strokes, and varied between 1.08 m and 1.17 m depending on the (varying) inclination ψ of the mole (Figure 26b).

Given the manually picked travel times and the distance s between the mole tip and SEIS, apparent seismic velocities were computed obtaining velocity values $V_p = 119^{+45}_{-21}$ m/s and $V_s = 63^{+11}_{-7}$ m/s, respectively (Figure 27). The estimates follow log-normal distributions with given mode and 68.3% confidence interval values. Because the P - and S -wave travel times show a similar scatter in terms of magnitude, the relative error of V_p is larger than the relative error of V_s . A complementary study of the apparent incidence angle of 2'461 P -wave first arrivals revealed an independent V_p/V_s ratio estimate of $1.84^{+0.89}_{-0.35}$, which is in good agreement with the travel time-derived value of $1.86^{+0.42}_{-0.25}$. These velocity estimates likely reflect properties of the bulk volume of the uppermost 10s of cm of the regolith layer.

Because SEIS was located only around one seismic wavelength away from the source, Brinkman et al. (2022) addressed concerns that near-field effects could affect their analyses and confirmed their results with an extensive numerical full-wavefield simulation study.

Assuming a density of 1200 kg/m^3 (see above), we can convert the V_p and V_s estimates by Brinkman et al. (2022) into a bulk, shear, and Young's modulus of $9.8 \pm 6.8 \text{ MPa}$, $4.3 \pm 1.0 \text{ MPa}$, $11.3 \pm 2.9 \text{ MPa}$, respectively, and a Poisson's

ratio ν of 0.31 ± 0.15 . When interpreting these numbers, one has to keep in mind that they are effective values of a homogeneous half-space and resulted from seismic observations with a dominant frequency of around 60 Hz.

Comparisons of the prelanding predicted low seismic regolith velocities on Mars with terrestrial soil and planetary regolith studies have extensively been discussed in Morgan et al. (2018). Similar low P -wave velocities of 100-120 m/s have been observed during laboratory tests with different Martian regolith simulants and low overburden pressure (Delage et al., 2017, see also Figure 14). For the Moon, active source (e.g. Cooper et al., 1974) and passive (e.g. Sens-Schönfelder & Larose, 2010) seismic experiments from Apollo 14, 16, and 17 as well as laboratory studies on lunar regolith samples (Johnson et al., 1982) found P -wave velocities in the range of around 100-125 m/s at, or close to, the surface. Published lunar S -wave velocities at the surface range between around 30 and 60 m/s (e.g. Dal Moro, 2015; Larose et al., 2005; Tanimoto et al., 2008), and reported Poisson's ratios range between 0.23 and 0.43 (e.g. Larose et al., 2005). Interestingly, these Poisson's ratios are generally higher than the predicted value for the InSight landing site that was estimated prelanding (i.e. 0.22 by Delage et al., 2017) but agree reasonably well with the Poisson's ratio of 0.28 found in this study. The elastic parameter estimates are also in general agreement with values derived with other approaches, e.g., SEIS/soil interaction on sandy simulants (Fayon et al., 2018; Delage et al., 2022), analysis of SEIS and lander resonances (e.g. Lognonné et al., 2020; Stott et al., 2021) and HP³ mole recovery activities (Spohn et al., 2022a).

5.3.2 Elastic properties experiments

Some experiments to measure the elastic properties of the Martian regolith by pressing on the Martian ground using the scoop on the robotic arm were carried out (Golombek et al., 2023). This can be observed in the photo of Figure 5, where the scoop is pressing the soil on the right-hand side of SEIS. One can also observe in the photo an imprint of a former press tests.

Pressing the soil with the scoop causes the ground to incline slightly towards the scoop, generating a tilt of SEIS that is observable on the horizontal component of the seismometer. Assuming that the inclination angle (θ) is very small, the acceleration due to the local ground tilt can be approximated as $g\theta$, where g is the local gravitational acceleration ($g = 3.721 \text{ m/s}^2$). A quasi-static elastic surface deformation approach (as described in Murdoch et al., 2017a, b) can then be used to invert θ in order to retrieve the mean elastic parameters of the ground between the scoop and SEIS.

These experiments require some measurements of the vertical (F_v) and radial (F_r) force components, that are derived from the electric intensity in the IDA motors during the test. This was made through calibration tests carried out in the JPL testbed.

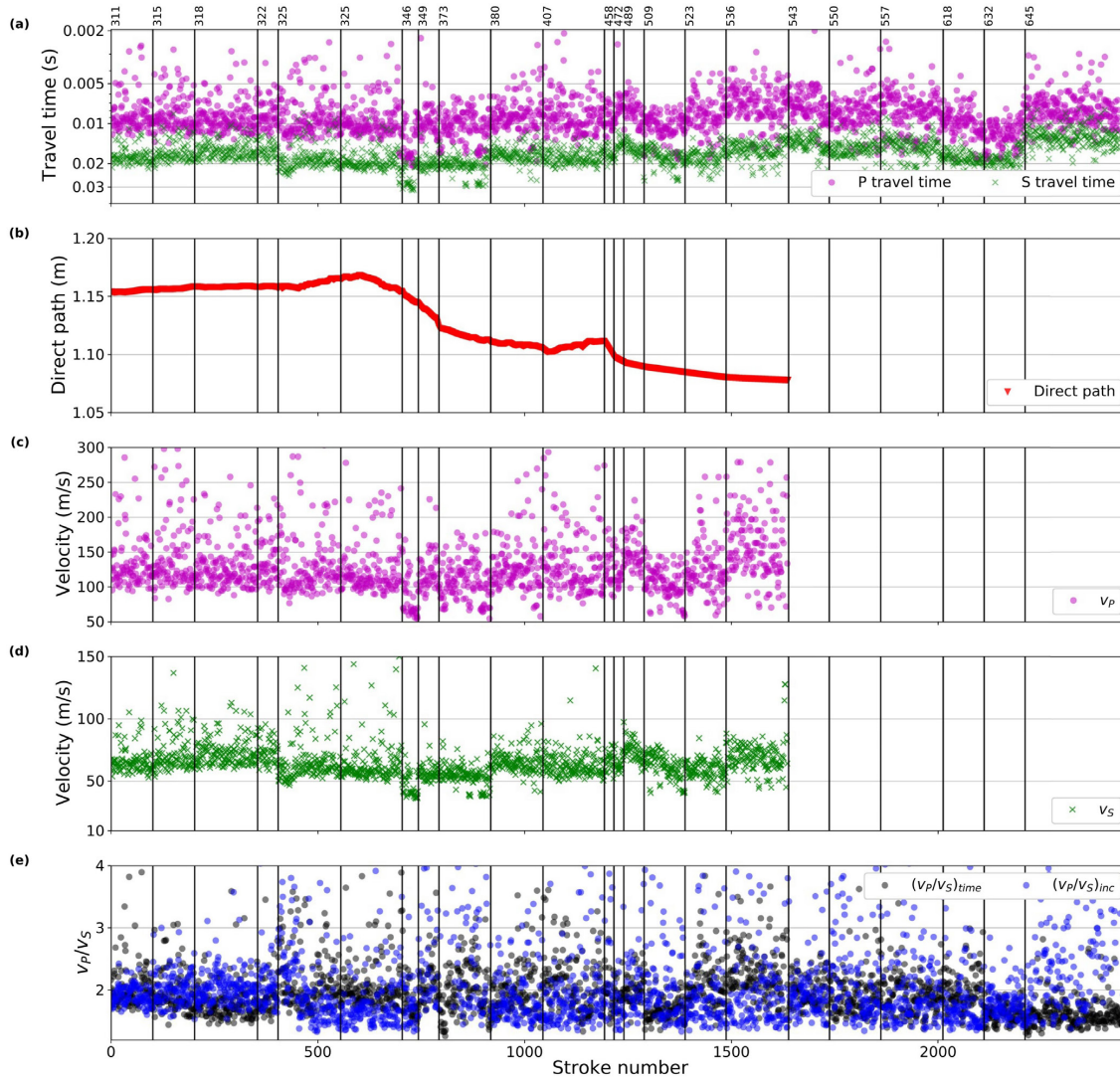


Figure 26. (a) First-arrival P- (t_p) and S-wave (t_s) travel time picks for the hammer sessions conducted between sols 311 and 645; (b) Distance between the HP³ mole tip and the SEIS seismometer; (c) Effective P- (V_p) and (d) S-wave velocity (V_s) estimates based on the travel times and travel path distances shown in (a) and (b), respectively; (e) $(V_p/V_s)_{\text{time}}$ ratio estimates derived from t_s/t_p using the travel time data displayed in panels (a) and (b) plotted together with the incidence angle-derived $(V_p/V_s)_{\text{inc}}$ (Brinkman et al., 2022).

The first scoop pressing experiment, performed on sol 1156, involved pressing on the ground with a (near) vertical force. This was to ensure that the tilt signal on SEIS was detectable, to reduce the error in the knowledge of the force applied by the scoop, and because this allowed the elastic properties experiment to be combined with an indentation experiment that required vertical forcing. The location selected to implement this experiment (see Figure 5) was chosen to be suitable for both the indentation and elastic properties experiments, and feasible for the IDA team. A second scoop pressing experiment was performed on sol 1170, with a larger radial force component (F_r), in order to further constrain ground properties. With only vertical forcing (F_v) it is possible, in principle, to constrain the relationship between elastic

parameters, but adding radial forcing allows to simultaneously resolve all the elastic ground parameters. The location of the second elastic properties experiment, about 48 cm outboard from the first experiment (see Figure 5), was selected such that F_r/F_v was about 0.93, based on calibrations done in the InSight JPL testbed. These data are still being analysed, but initial results suggest Poisson's ratio is around 0.25 and the shear modulus is around 2 MPa, which are comparable to those derived from SEIS recordings of hammering experiments (Brinkman et al., 2022). Note however that further investigations have to be conducted to account in more details, in the comparison of the elastic parameters obtained, for the morphology of the surface layers given in Spohn et al. (2022a) and further commented in Section 6.

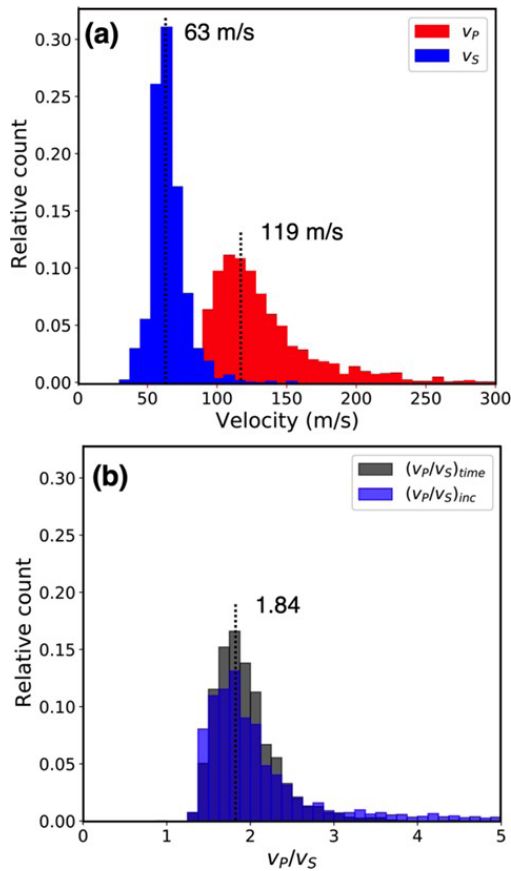


Figure 27. (a) Histogram of 1 518 P- (V_p) and S-wave (V_s) velocity estimates; (b) Histogram of 2 271 (V_p/V_s)_{time} values determined from the picked travel times and 2 461 (V_p/V_s)_{inc} values determined from the P -wave incidence angles. The dashed lines mark the mode of each distribution. Figure modified from Brinkman et al. (2022).

5.4 Scraping and dumping activities

The robotic arm and scoop were also utilised to bury the SEIS tether so as to reduce the perturbations due to the change in temperature of the tether and to its interactions with the wind. This was made by creating numerous scrapes, scooping regolith, and dumping it. Several scraped piles were created from which regolith was collected using the scoop, (Golombek et al., 2023) as shown in Figure 5 and in more details in Figure 28.

The regolith extracted in the scoop was then dumped in piles above the tether, close to the SEIS instrument. However, some of the regolith was observed to be blown away by the wind, as reported by Golombek et al. (2023) and Verdier et al. (2023). This constrained the grain size distribution curve of the poured regolith by integrating the combined effects of wind and gravity on the poured grains.

To assess the topography of the scraped and dumped piles, digital elevation models and elevation profiles were generated from the images acquired between Sols 803 and 822. Figure 29 provides an example of the digital elevation model and elevation profiles obtained on Sol 822. The average

slope of the scraped mounds was found to be approximately 42° (with a standard deviation of 2.7°) (Marteau et al., 2022, 2023), while the slopes of the side walls created by the vertical sides of the scoop were steeper, with a slope value of 54.7° (with a standard deviation of 6.6°). These values are significantly larger than what could be derived from the friction angle of the sandy regolith, estimated to be around 30° in the loose state met at the surface (Delage et al., 2017; Morgan et al., 2018). Note however that the slopes along P1 and P2 in Figure 29 are not those of a standard pile, put of a regolith mass either pushed and remoulded by the scoop (P1), or slopes of the walls on both sides of a trench. This certainly deserves further investigation, but it seems that the particular conditions prevailing on Mars (low gravity, low atmospheric pressure) together with the presence of dust within the sandy regolith result in having some cohesion that allow steeper slopes than that corresponding to the angle of repose. It is worth noting that the cohesion is further enhanced by the low gravity conditions on Mars, as observed on lunar regolith by Walton et al. (2007).

Subsequently, the IDA scooped up the scraped material and dumped it from a height of 40 cm on top of the SEIS tether. The highest point of the dumped pile reaches a height of approximately 3 cm, with a slope value of 24.1° (with a standard deviation of 6.1°). Note that, as calculated by Verdier et al. (2023), the grain size distribution of the poured regolith is different from that of the scraped one, since the smaller particles ($200\text{ }\mu\text{m}$ and below) have been blown by the wind and deposited in plumes at distances up to 3-5 m from the scoop, whereas the particles smaller than $100\text{ }\mu\text{m}$ and below (including dust) are blown away. This can be observed in the photo of Figure 30 (Verdier et al., 2023) that shows the situation before (a) and after (b) a dump. The plume of deposited finer particles observed as a dark elongated spot in (c) has been obtained by differencing the images (a) and (b).

The smaller slope of 24.1° of the dumped pile (Marteau et al., 2023) is satisfactorily in agreement with the estimated 30° angle of friction of the loose regolith. This smaller friction value for the regolith without its finest and dust fraction is in agreement with the cohesion that is suspected to be provided by the dust fraction ($< 10\text{ }\mu\text{m}$) in the intact regolith, that no longer exist in the poured regolith due to the wind dispersion of smallest particles.

6. A model of the soil profile at the InSight landing site

The diagram of Figure 31, based on visual observations of the photos (Figures 3, 5, 20, 22, and 23) and on data from both the mole penetration process (Spohn et al., 2022a) and local wave velocity measurements (Brinkman et al., 2022) provide a synthesis of the estimated physical and mechanical properties of the near surface. Below a 1 cm thick surface dust/sand deposit, an around 19 cm thick duricrust layer rests

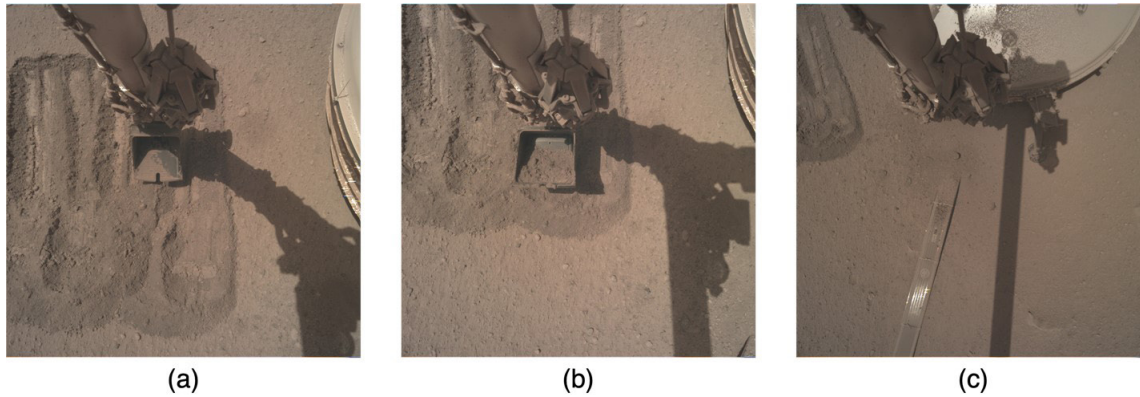


Figure 28. Photos acquired on sol 870: (a) Scraped piles; (b) Scooped regolith; (c) Regolith dumped on the SEIS tether close to the WTS (Image credit NASA/JPL).

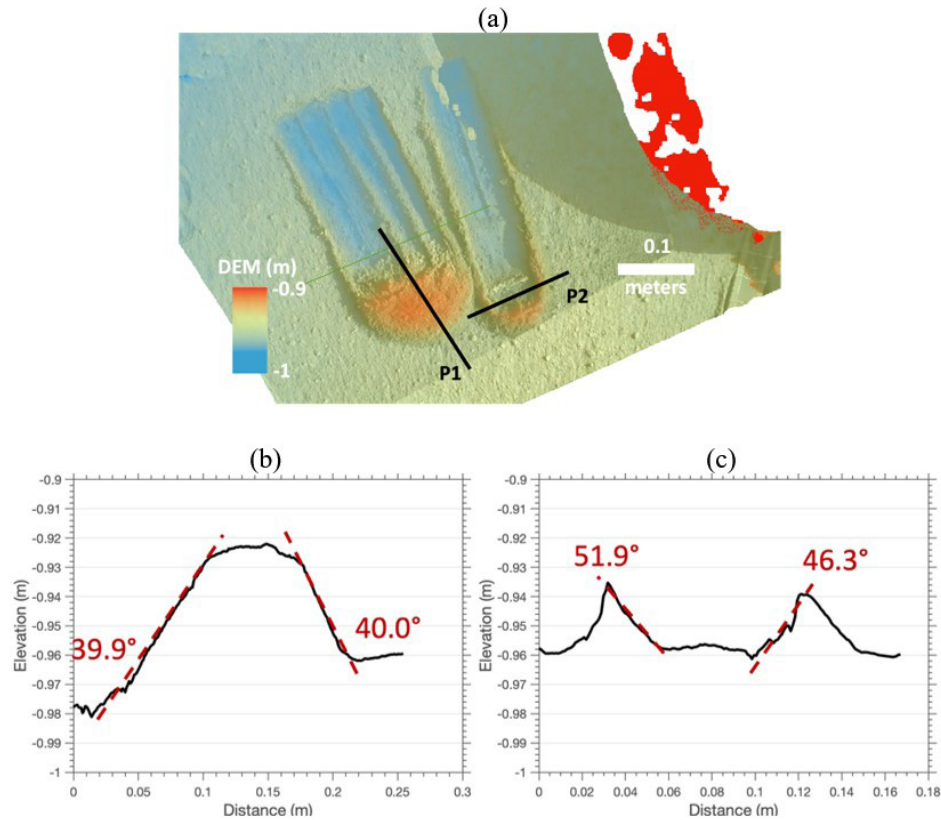


Figure 29. (a) Sol 822 Digital Elevation Model; (b) Elevation profile of scraped pile P1; (c) Elevation profile of scraped walls P2.

on an around 12 cm thick sand layer overlaying a sand/gravel layer of unknown thickness beginning at a depth of 31 cm. The Figure also shows that the elastic properties derived from the HP³/SEIS measurements are representative of the averaged elastic properties of the sand and duricrust layer, with densities of 1300-1500 kg/m³ and 950-1100 kg/m³, respectively. It is important to also remind that waves travel in surface under quite low stresses, given the smaller gravity on Mars. With $g = 3.721 \text{ m/s}^2$, for the profile of the Figure, the vertical stress σ_v is equal to around 1.35 Pa at 31 cm depth

where the waves are emitted by hammering and reduce to 0.045 Pa at 1 cm depth. Present knowledge on wave transfer in granular (and/or slightly cohesive) materials under such very low stresses is presently not well known and certainly deserves further investigation, with particular attention to the effect of particle shape and rugosity at inter-grains contacts.

Consistent with the formation process of the regolith (see Figure 7) and from observing the photos taken around the InSight landing site by the two cameras, Golombek et al. (2020a) proposed the sketch of Figure 32 to describe the

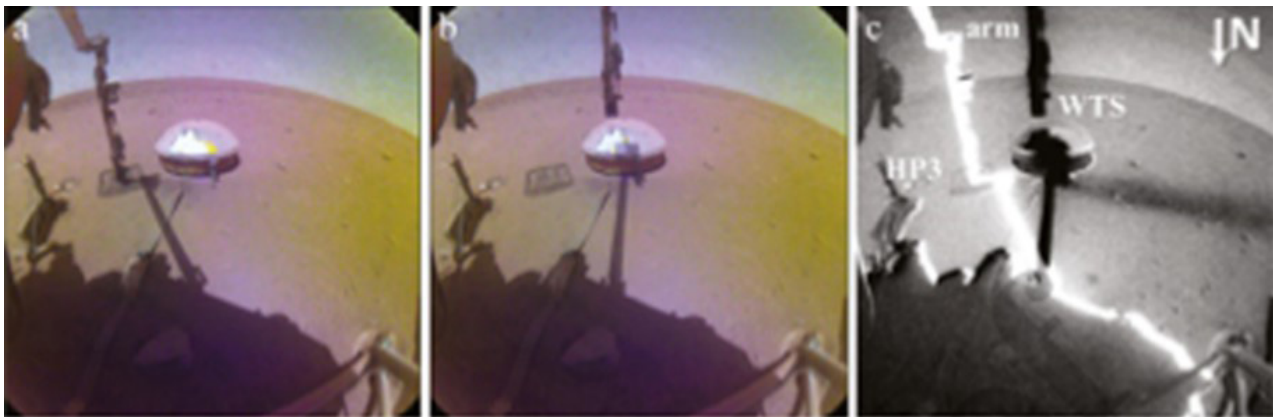


Figure 30. (a) Instrument Context Camera (ICC) image taken on sol 877 at 11:22 a.m. before dump 6 on the SEIS tether; (b) ICC image acquired on sol 877 at 11:33 a.m., after dump 6; (c) ICC image difference showing the dark plume made up of wind dispersed and transported particles. The plume extends to the right from the WTS, and its orientation corresponds to the wind direction from N14° (ESE) to N320° (WNW) (Verdier et al., 2023).

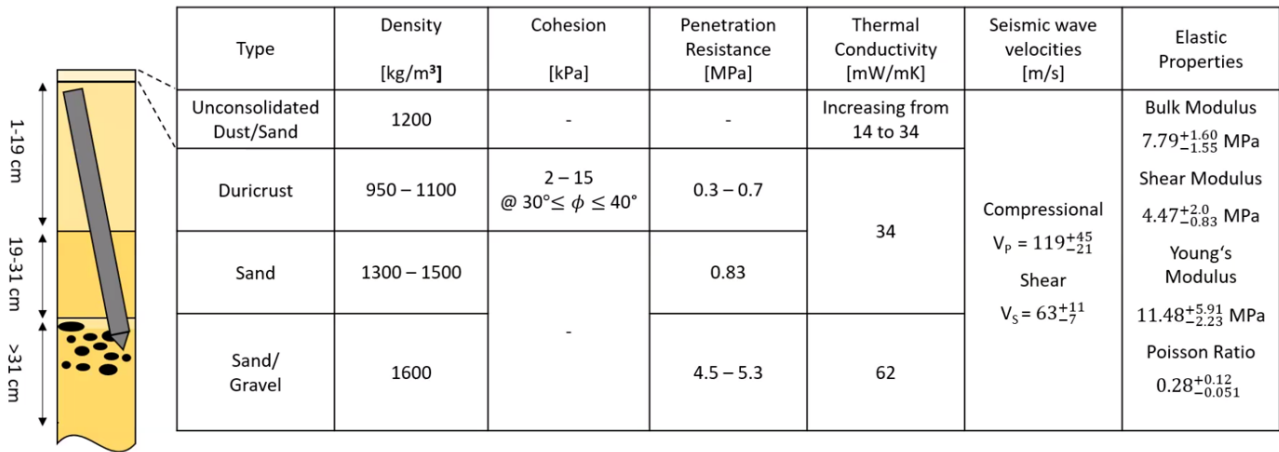


Figure 31. Estimated properties of the near surface (Spohn et al., 2022a). Thermal properties are from Grott et al. (2021); Seismic wave velocities and elastic properties are from Brinkman et al. (2022), see Section 5.3.1.

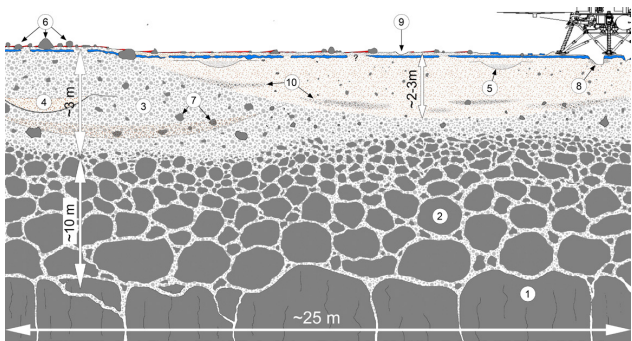


Figure 32. Interpretative cross-section of the shallow surface beneath the InSight lander. (1) Fractured basalt flow; (2) Blocky ejecta; (3) Fine-grained impact generated regolith; (4, 5) Overlapping craters; (6) Rockier area; (7) Rocks embedded in regolith; (8) Pits opened by retro rockets during landing; (9) Surface divots; (10) Lens of ejecta from other craters. (Golombek et al., 2020a).

structure of the regolith along a deeper profile in the landing area. A depression of ~27 m in diameter (in which the lander sits) interpreted as an ancient degraded and infilled impact crater in the regolith is underlain by blocky ejecta and fractured basaltic bedrock. The crater has been filled dominantly by sand moved by the wind, producing an upper layer of fine material three metres thick, consisting mostly of sand.

7. Conclusion

The InSight mission was quite successful on a geophysical point of view, but the various instruments accommodated by the InSight lander also provided significant input on the geological and geotechnical properties of the near surface. This concerns the high-quality photos provided by the

cameras, the data from the self-penetrating mole, the wave recordings by the SEIS seismometer and all the activities carried out with the Instrument Deployment Arm and its scoop. In spite of the significant difference in nature between the true soil profile and the sandy regolith simulants considered in the pre-mission investigations, a rather good agreement was found for the elastic properties at surface, derived from wave velocity measurements. These were well characterised by conducting some kind of Martian cross-hole test, by analysing with SEIS the waves resulting from the mole hammering sessions. The penetration phases of the self-penetrating mole, in spite of being problematic, provided a significant input on the soil profile along the first 30 cm penetrated, completed by the photos taken from the IDC camera located on the IDA. The profile initially observed under the lander in the area affected by the impingement of the rocket exhaust was confirmed by observing the pit around the mole and analysing the penetration data. Other significant information was given by the scraping, piling and pushing activities carried out by the Instrument Deployment Arm to help the mole penetration and to cover the SEIS tether to improve its thermal and mechanical insulations. The surface regolith, suspected to be sandy with an average grain diameter of 170 μm from orbiter thermal inertia measurements and surface radiometer measurements, appeared to have a profile in which a 1 cm sand/dust layer is overlaying a layer made up of a loose cohesive matrix with embedded pebbles, called duricrust. Below, as derived from penetration data, the profile includes a sandy layer covering a gravel/sand layer in which the mole penetration was stopped.

The scraping operations only concerned the 1 cm thick surface sand/dust layer, given that the scoop was not able to excavate deeper into the underlain cohesive duricrust. The Digital Elevation Models provided more details on the excavated trenches and poured piles, with angles of slopes that were sometimes significantly larger than the angle of repose and friction angle (estimated around 30° for the loose surface sandy deposit). The steep slopes observed in trenches are suspected to be due to some cohesion resulting from the presence of dust and some bonding agents in the surface layer, and from the effects of low gravity. Interestingly, the slope of the piles of poured regolith, in which no dust and fine particles remained due to the action of the wind, were close to the estimated angle of friction.

The observations and measurements made with the various InSight instruments definitely enriched the knowledge of the geological and mechanical properties of the surface regolith and will certainly help for further robotic exploration of the surface of Mars.

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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

Pierre Delage: supervision, conceptualization, writing – original draft, review and editing. Bernardo Caicedo: conceptualization, writing – original draft. Matt Golombek: formal analysis, data analysis, conceptualization, writing – original draft. Tilman Spohn: funding acquisition, project administration, data analysis, writing – original draft. Cédric Schmelzbach: investigation, data curation, writing – original draft. Nienke Brinkman: investigation, data curation, formal analysis. Eloise Marteau: investigation, data curation, formal analysis, writing – original draft. Naomi Murdoch: investigation, data curation, formal analysis, writing – original draft. Nicholas Warner: investigation, data curation, formal analysis, conceptualisation. Véronique Ansan: investigation, data curation, formal analysis, conceptualisation, visualisation. Bruce Banerdt: funding acquisition, project administration, conceptualisation. Juan-Pablo Castillo-Betancourt: data curation, formal analysis, visualisation. Pascal Edme: data curation, formal analysis. Annabel Gomez: investigation, data curation, formal analysis. Matthias Grott: investigation, data curation, formal analysis, writing – original draft. Ken Hurst: investigation, data curation, formal analysis, methodology. Mark Lemmon: data curation, formal analysis. Philippe Lognonné: funding acquisition, project administration, conceptualisation. Sylvain Piqueux: investigation, data curation, formal analysis, methodology. Johann Robertsson: data curation, formal analysis, conceptualisation, methodology. David Sollberger: data curation, formal analysis, conceptualisation, visualisation, methodology. Simon Stähler: data curation, formal analysis, conceptualisation. Nicolas Verdier: data curation, formal analysis, conceptualisation. Christos Vrettos: data curation, formal analysis, conceptualisation. Nathan Williams: data curation, formal analysis, conceptualisation.

Data availability

No dataset was generated or evaluated in the course of the current study; therefore, data sharing is not applicable.

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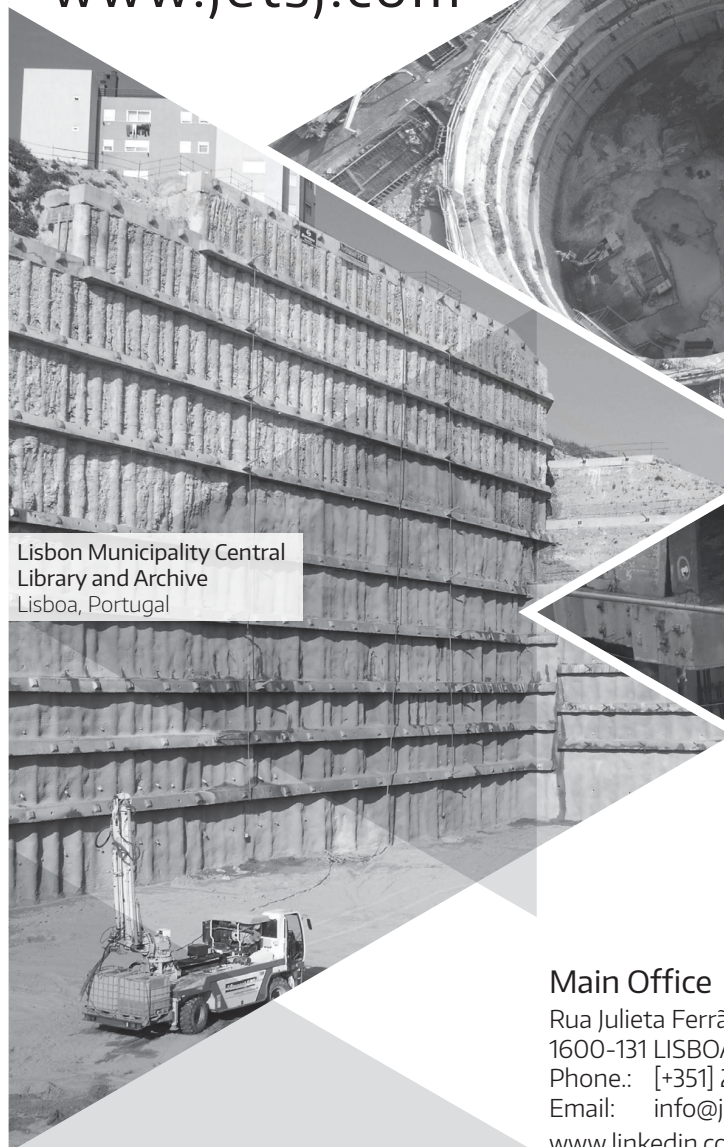
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Soils and Rocks provides a manuscript template available at:

<https://drive.google.com/file/d/17BGwQaD79WSPBVm6BAYATv1rAqNodCZW/view?usp=sharing>

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Submissions are classified into one of the following categories:

Article – an extensive and conclusive dissertation about a geotechnical topic, presenting original findings.

Technical Note – presents a study of smaller scope or results of ongoing studies, comprising partial results and/or particular aspects of the investigation.

Case Study – report innovative ways to solve problems associated with design and construction of geotechnical projects. It also presents studies of the performance of existing structures.

Review Article – a summary of the State-of-the-Art or State-of-the-Practice on a particular subject or issue and represents an overview of recent developments.

Discussion – specific discussions about published papers.

Authors are responsible for selecting the correct category when submitting their manuscript. However, the manuscript category may be altered based on the recommendation of the Editorial Board. Authors are also requested to state the category of paper in their cover letter.

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- b) assure that the paper has not been previously published, and is not being submitted to any other journal for publication.

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Full-length manuscripts (Article, Case Study) should be between 4,000 and 8,000 words. Review articles should have up to 10,000 words. Technical Notes have a word count limit of 3,500 words. Discussions have a word count limit of 1,000 words. These word count limits exclude the title page, notation list (e.g., symbols, abbreviations), acknowledgments and references, but include any appendices. Each single column and double column figure or table is considered as equivalent to 150 and 300 words, respectively.

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The manuscripts should be written in UK or US English, in the third person and all spelling should be checked in accordance with a major English Dictionary. The manuscript should be able to be readily understood by a Civil Engineer and avoid colloquialisms. Unless essential to the comprehension of the manuscript, direct reference to the names of persons, organizations, products or services is not allowed. Flattery or derogatory remarks about any person or organization should not be included.

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4.6 Data availability

A data availability statement must be included in any original research article. Authors should include information on where

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4.7 Use of generative artificial intelligence for textual, visual, and data enhancement

The use of generative artificial intelligence (GenAI) has proven to be a valuable tool for translation and for improving the quality of texts, images, graphic elements, as well as for the treatment and compilation of data and results in scientific publications. However, Soils and Rocks considers that at no time can GenAI tools be considered as the author or co-author of the work since authorship and co-authorship are exclusively attributed to human action. Such tool should be used as aid and should be always supervised by humans.

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Soils and Rocks follows the ethics of the Committee on Publication Ethics (COPE), of the Code of Ethics American Educational Research Association, the Code of Good Scientific Practice - FAPESP and the Council of Science Editors (CSE) concerning publication. Any material published in is in agreement with the SciELO Guidelines on Best Practices for Strengthening Ethics in Scientific Publication.

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IMPORTANT OBSERVATION: Mendeley software plug-in (suggested in this guide) for MS-Word can be used to include the references in the manuscript. This plug-in uses a field code that sometimes includes automatically both title and abstract of the reference. Unfortunately, the similarity software adopted by the Journal (Plagius) recognizes the title and abstract as an actual written text by the field code of the reference and consequently increases considerably the percentage of similarity. Please do make sure to remove the abstract (if existing) inside Mendeley section where the adopted reference is included. This issue has mistakenly caused biased results in the past. The Editorial Board of the journal is now aware of this tendentious feature.

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The text must be presented in a single column, using ISO A4 page size, left, right, top, and bottom margins of 25 mm, Times New Roman 12 font, and line spacing of 1.5. All lines and pages should be numbered.

The text should avoid unnecessary italic and bold words and letters, as well as too many acronyms. Authors should avoid capitalizing words and whenever possible to use tables with distinct font size and style of the regular text.

Figures, tables and equations should be numbered in the sequence that they are mentioned in the text.

Abstract

Please provide an abstract between 150 and 250 words in length. Abbreviations or acronyms should be avoided. The abstract should state briefly the purpose of the work, the main results and major conclusions or key findings.

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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:

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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
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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.”

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Standards must be cited in the text by the initials of the entity and the year of publication, e.g.: ABNT (1996), ASTM (2003).

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A customized style for the Mendeley software is available and may be downloaded from:

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Journal Article

Bishop, A.W., & Blight, G.E. (1963). Some aspects of effective stress in saturated and partly saturated soils. *Geotechnique*, 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., Konig, 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.
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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.