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Davor Marušić and [Vedran Jagodnik](#)*

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Article

Monotonic Behaviour and Physical Characteristics of Silty Sands with Kaolinite Clay

Davor Marušić ¹  and Vedran Jagodnik ^{1,2*} 

¹ Faculty of Civil Engineering, University of Rijeka, Radmile Matejčić 3, 51000 Rijeka, Croatia

² Center for Artificial Intelligence and Cybersecurity, University of Rijeka, Radmile Matejčić 2, 51000 Rijeka, Croatia

* Correspondence: vedran.jagodnik@gradri.uniri.hr; Tel.: +385-51-265-941

Abstract

This study investigates the behaviour of dense clayey and silty sands with kaolinite clay under static drained/undrained conditions at low confining stress. Conventional laboratory tests assessed the mixtures' physical properties, but standard void ratio methods proved inadequate for silty sands with clay. Despite targeting 80% relative density, specimens exhibited loose sand behaviour in both drained and undrained tests. Analytical redefinition of minimum/maximum void ratios (based on sand-clay volumetric fractions) improved specimen reconstitution, yielding dense behaviour matching host sand performance. However, these methods are limited to non-expansive, low-plasticity clays. Void ratios in clayey silty sands depend on confining stress, drainage, saturation, clay content, and sand skeleton structure. Unlike pure sands, these mixtures experience variable void ratios due to clay phase changes under different saturation levels. A new evaluation method is needed, accounting for clay composition, saturation-dependent consistency, and initial sand skeleton configuration to accurately characterise these soils. The findings highlight the limitations of conventional approaches and stress the need for advanced frameworks to model complex soil behaviour in geotechnical applications.

Keywords: silty sand; kaolinite; void ratio redefinition; relative density; static undrained test; low confining stress; specimen reconstitution

1. Introduction

The conventional understanding of soil behaviour generally focuses on the individual behaviour of coarse-grained, granular soils (sands and gravels) and fine-grained (fines), cohesive silts and clays. However, the monotonic behaviour of transitional, silty sands is still a subject of ongoing research [1–7]. It was shown that in silty sands, below the fines threshold content where the interparticle stress transfer should still be governed by sand skeleton structure, the fines affect both drained and undrained soil behaviour [1,8–12]. The fines interact with the sand skeleton structure by sticking to the sand grains' surfaces, filling the existing pore spaces, and forming narrow “bridges” between the sand grains [13,14]. This affects silty sand behaviour differently, considering the variations in fines and sand particle sizes and shapes, mineral composition, sandy soil's relative density, and outer conditions like governing effective stresses or draining conditions. The behaviour of granular soils is mostly influenced by their texture (particle size distribution and shape) and relative density [13,14]. In contrast, the behaviour of fine-grained soil is influenced by its structure, which is conditioned by clay mineral fabric (particles' geometric arrangement) and interparticle forces. The interparticle shape and distribution of pores are mainly conditioned by clay mineralogy and the amount and angularity of the present silt fraction. Fine-grained fractions' behaviour and strength depend on the soil's consistency or firmness, conditioned with clay mineralogy, particle size distribution, and the corresponding Atterberg limits. Unlike granular soil, fine-grained soil's characteristics vary significantly with the water content. Also, the gravels, sands, and silts are represented by equidimensional particles, while the clay particles are mostly shaped like plates, elongated flakes or needles. This effect influences the stress-related orientation of particles, causing different soil behaviour in different effective stress directions [14].

1.1. Research Overview

The present research focuses on silty sand soil behaviour under static drained and undrained conditions and low confining stress, using the reconstituted specimens artificially made with uniform quartz sand and kaolinite silt as fines fraction. Silty sand mixtures are reconstituted for monotonic compression tests in the triaxial apparatus. Taking undisturbed samples for the triaxial shear tests is complicated in relatively cohesionless sandy soils, and the silty sand specimens are usually reconstituted using representative natural soil sediments or artificial mixtures. Soil reconstitution enables the specimen's relative density control, while in the artificial mixtures, it additionally enables control over particle sizes and shapes, fines content, and specimen mineral composition. Soil mixtures tested within the present research are evaluated as granular materials (sands) since they were reconstituted below the fines threshold content [15–17]. Determination of silty sand soils' physical-mechanical properties is challenging due to the differences in the sand and fines characteristics and their mixtures' characteristics deviation from the conventional soil mechanics. Existing research [18,19] used the identical soil mixtures as within the present research and implemented standard laboratory tests [20,21] to determine the mixtures' minimum and maximum void ratios, which were later used in physical modelling. However, existing standards limit the testing to silty sand mixtures with non-plastic fines and free-draining conditions. Contrary to these limitations, kaolinite clay exhibits plasticity, while its hydrophilic properties contradict free-draining properties. Therefore, standard methods should be inapplicable for determining the minimum and maximum void ratios of silty sands with clay minerals. To evaluate the validity of previously determined void ratios [18,19], the mixtures' physical characteristics were determined with laboratory tests, implementing some existing modifications and recommendations [1,22] for otherwise conventional sandy soil characteristics determination methods. The equivalent void ratio and relative density [15,16] for mixtures tested were also evaluated within this research. Alternatively, the existing suggestions for the analytical redefinition of the mixtures' minimum and maximum void ratios [23,24] were used to reconstitute another set of specimens for testing. Both sets were tested in monotonic compression, and the data obtained were correlated and evaluated. Special focus was given to the physical-mechanical properties of the reconstituted specimens and their response to drained and undrained, strain-controlled shearing.

1.2. Void Ratio and Relative Density for Sands

Phase or volumetric relations represent an idealised and simplified interaction between the solids (soil), fluids (usually water) and gas (usually air) within a given specimen. Any arbitrary volume of sandy soil will consist of both solids and pores, with the pores being filled with water, air or both. Phase relations within an arbitrary specimen are graphically presented in Figure 1, with mass and volume ratios for individual media within a soil (solid, fluid, gas).

Phase relations define the void ratio, representing the important physical-mechanical soil parameter that defines the specimen volume change and relative density of the granular soil. The void ratio dependence on water content and saturation degree is expressed with (1).

$$e \cdot S_r = w \cdot G_s \quad (1)$$

where: e stands for global void ratio, S_r is the degree of saturation, w is the water content, and G_s is the specific gravity of solids.

General control within reconstituted granular material (sand) can be achieved with a targeted relative density (D_r) as a relative position between the minimum and maximum void ratios (the densest and loosest possible soil states, respectively) (2).

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \cdot 100\% \quad (2)$$

where: D_r is the relative density, e is the specimen's void ratio (global void ratio), e_{min} is the specimen's minimum void ratio, and e_{max} is the specimen's maximum void ratio.

With the known total volume and specific gravity of the reconstituted specimen, one can easily determine the sand fraction mass for a given water content using the conventional phase relations. Sand parameters like minimum and maximum void ratios and specific gravity can be obtained by conducting simple laboratory tests [14,25]. The targeted sand specimen’s density has a firm theoretical and practical background and can also be evaluated by comparing it with the existing research on similar sandy soils.

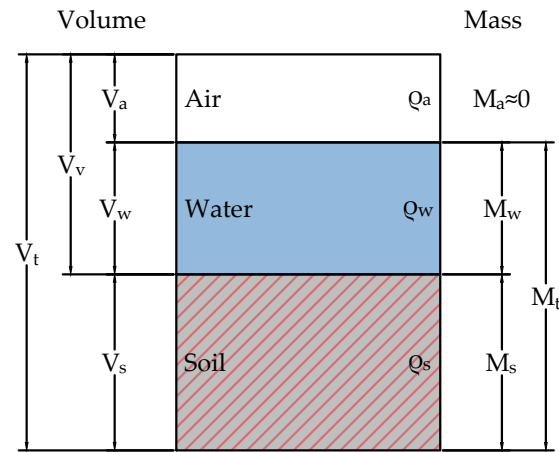


Figure 1. Phase relations schematic diagram – adopted from [14].

1.3. Equivalent Void Ratio and Relative Density for Silty Sands

Previous studies [1,2] indicate that a portion of fines in silty sands tends to fill the voids in the sand skeleton structure, consequently lowering the overall void ratio of the soil mixture. On the other hand, a portion of fines also forms “bridges” between sand grains, actively participating in soils’ structural force transfer, separating sand grains and increasing the host sand’s void ratio [15]. The effect of fines content on silty sands’ void ratio variations is schematically presented in Figure 2 and Figure 3.

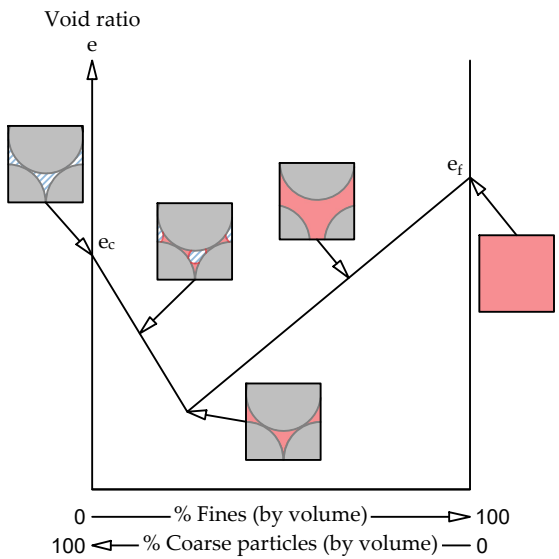


Figure 2. Schematic diagram of a theoretical variation of minimum void ratio in binary packings with % fines – adopted and modified from [1].

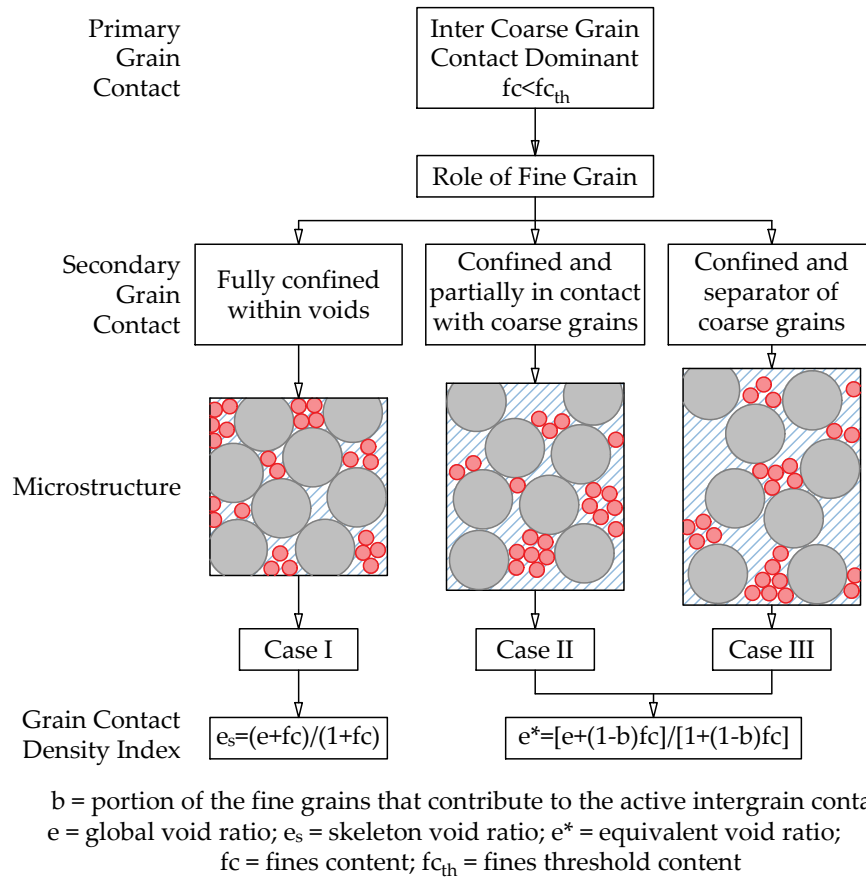


Figure 3. Intergranular soil mix classification for coarse-grain soil mix – adopted and modified from [15].

According to [1], within fine sand and silt regions with non-spherical particles, friction, adhesion, and bridging between the fine particles form structures controlled by interparticle forces. Increasing angularity enhances the particle bridging effects and produces higher void ratios than equivalent spheres. Van der Waals attraction and double-layer repulsion in clay minerals and aggregates compete with gravitational forces to control the packing mechanisms and arrangements. These forces are dependent on the distance between the particles and are further enhanced in water, where the buoyed particles may pack more loosely than under dry conditions. Previous research [16,26] suggested analytical approaches to solve these issues, which continue to be successfully implemented for the silty sand relative density approximation [7,17,27]. The skeleton void ratio given by Equation (3) [2] and the equivalent void ratio by Equation (4) [15,16,26] were introduced as an approximation of the void ratio of silty sands at low fines content.

$$e_s = \frac{e + f_c}{1 + f_c} \quad (3)$$

where e_s is the skeleton void ratio, f_c is the fines content, and e is the global void ratio.

$$e^* = \frac{e + (1 - b) \cdot f_c}{1 + (1 - b) \cdot f_c} \quad (4)$$

where: e^* is the equivalent void ratio, b is the embedding coefficient.

The embedding coefficient (b) represents the fines fraction participating in the granular force structure. Theoretically, if ($b=0$), all the fines are inactive as they fill the host sand pore spaces. Alternatively, ($b=1$) indicates that all the fines participate in the granular force structure. It should be noted that additional research [3,28] pointed out the dependency of the (b) coefficient value on the fine content and state of the mixture. This also indicates the possible dynamic changes in the b

coefficient value with changes in the silty sands' relative density and governing confining stresses. The determination of the (b) coefficient and fines threshold was given by Equations (5) and Equation (9), respectively.

$$b = \left[1 - \exp\left(-0.3 \cdot \frac{f_c / f_{th}}{k}\right) \right] \cdot \left(r \cdot \frac{f_c}{f_{th}} \right)^r \quad (5)$$

$$\gamma = \frac{D_{10}}{d_{50}} \quad (6)$$

$$r = \frac{1}{\gamma} \quad (7)$$

$$k = 1 - r^{0.25} \quad (8)$$

where γ is the particle size ratio, r is the ..., k is the ..., D_{10} is the size of the particle at 10% passing from sand distribution and d_{50} is the size of the particle at 50% passing from silt distribution.

$$f_{th} = 0.4 \cdot \left(\frac{1}{1 + \exp(\alpha_r - \beta_r \cdot \gamma)} + \frac{1}{\gamma} \right) \quad (9)$$

where $\alpha_r = 0.5$ and $\beta_r = 0.13$ for $2 \leq \gamma \leq 42$.

However, the equivalent void ratio determination method with the embedding coefficient given by [16] presented in Equations (5) and (9) was calibrated on the non-plastic, semi-spherical silty sand specimens. The specimens tested within the present research consisted of subangular uniform sand and platy kaolinite clay particles. Additionally, the value of the particle size ratio (γ) was 46, which falls beyond the applicable range for the α_r and β_r values given in Equation (9). For these reasons, the existing equivalent void ratio method proved inapplicable for mixtures tested within the present research.

1.4. Alternative Void Ratios

As mentioned, it was shown that the fines influence the sands' void ratios, conventionally determined with standardised testing procedures and phase relations [2,29,30]. An analytical approach to redefine the silty sands' minimum and maximum void ratios has been introduced [23,24] and evaluated by researchers [24,31]. To determine the minimum void ratio for various silty sand mixtures in the sand grain packing structure domain, the analytical approach uses the mixtures' volume content of sand and fines individually, with their corresponding minimum void ratios in a simple relation by Equation (10).

$$e_m^{min} = e_s^{min} \cdot y_s + e_f^{min} \cdot y_f - a \cdot \left(1 + e_f^{min} \right) \cdot y_f \quad (10)$$

where: e_m^{min} is the redefined minimum void ratio of the mixture; e_s^{min} is the minimum void ratio of the sand fraction; e_f^{min} is the minimum void ratio of the fines fraction; y_s is the volume content of the sand fraction; y_f is the volume content of the fines fraction; a is the filling coefficient.

In contrast to the embedding coefficient (b), the filling coefficient (a) represents the fines fraction filling the sand skeleton pore spaces. If ($a = 1$) all the fines are inactive as they fill the host sand pore spaces, while ($a = 0$) indicates that all the fines participate in the granular force structure. The predictions for a coefficient values were calculated and experimentally evaluated for various sand-silt mixtures [23]. The condition $a \approx 1$ corresponds to the limiting case where the size of small particles is much smaller than the size of large particles. This relation is schematically shown and described in the microstructure and soil behaviour framework [15] as case I, Figure 3, with the given condition of mean particle sizes of sand and fines $D_{50}/d_{50} > 6.5$ for spherical particles, and $f_c < f_{th}$. For comparison, the mean particle size ratio for the mixtures tested within the present research was $D_{50}/d_{50} = 71.5$. Regarding the particle sphericity or angularity, it was shown that angular particles enhance the particle bridging on the void ratios of clean sand with non-spherical particles and consequently tend to produce

higher void ratios than equivalent spheres [29], which can schematically be represented as cases II and III, Figure 3.

Considering the linear relationship between the e_{min} and e_{max} , an Equation (11) for the redefined maximum void ratio e_m^{max} was also given by [24].

$$e_m^{max} = \alpha \cdot e_m^{min} + \beta \tag{11}$$

The coefficients α and β are determined by the Equations (12) and(13).

$$\alpha = \frac{e_s^{max} - e_{fmax} + a \cdot \left(1 + e_f^{max}\right)}{e_s^{min} - e_{fmin} + a \cdot \left(1 + e_f^{min}\right)} \tag{12}$$

$$\beta = e_s^{max} - \alpha \cdot e_s^{min} \tag{13}$$

This analytical method was used within the present research to determine an alternative minimum and maximum void ratios for the silty sand mixtures. Using the alternative void ratios, the mixtures were further reconstituted and tested.

2. Methodology

The soil mixtures’ characteristics and parameters are essential to establish solid control over the tests. These can be determined by simple laboratory tests combined with existing theoretical relations. To reevaluate the mixtures’ previously determined void ratios [18,19], the conventional laboratory tests [20,21] were performed, incorporating some existing modifications and recommendations [22,29]. Additional mixtures were reconstituted using the analytically determined alternative void ratios (Chang et al., 2015, 2016). Reconstituted specimens were tested in monotonic compression, and the data obtained were correlated with existing research and evaluated.

2.1. Silty Sand Mixtures

Silty sand mixtures were prepared by mixing kaolinite silt and the "Modland" uniform quartz sand specimens with a predetermined mass ratio (Table 1). The basic physical characterisation of the used material is documented in the existing literature [32–34].

Table 1. Tested specimens’ kaolinite (K) and sand (S) content.

Soil type	Sand (S) [%]	Kaolinite (K) [%]
Modland (SK0)	100	0
SK10	90	10
SK15	85	15
Kaolinite (SK100)	0	100

2.2. Physical Properties Determination

2.2.1. Specific Gravity, Granulometric Analysis

The soil mixtures’ specific gravity and granulometric analysis were adopted from the existing research data [32,33]. The previous research data are briefly described below. The conventional pycnometer technique [35] and the combined sieve and hydrometer analysis [36] were used to determine the silty sands’ specific gravity and particle size distribution, respectively. The particle size distribution results obtained for the SK10 and SK15 mixtures with host sand and kaolinite are presented in Figure 4 and the existing results from previous research are presented in Table 2.

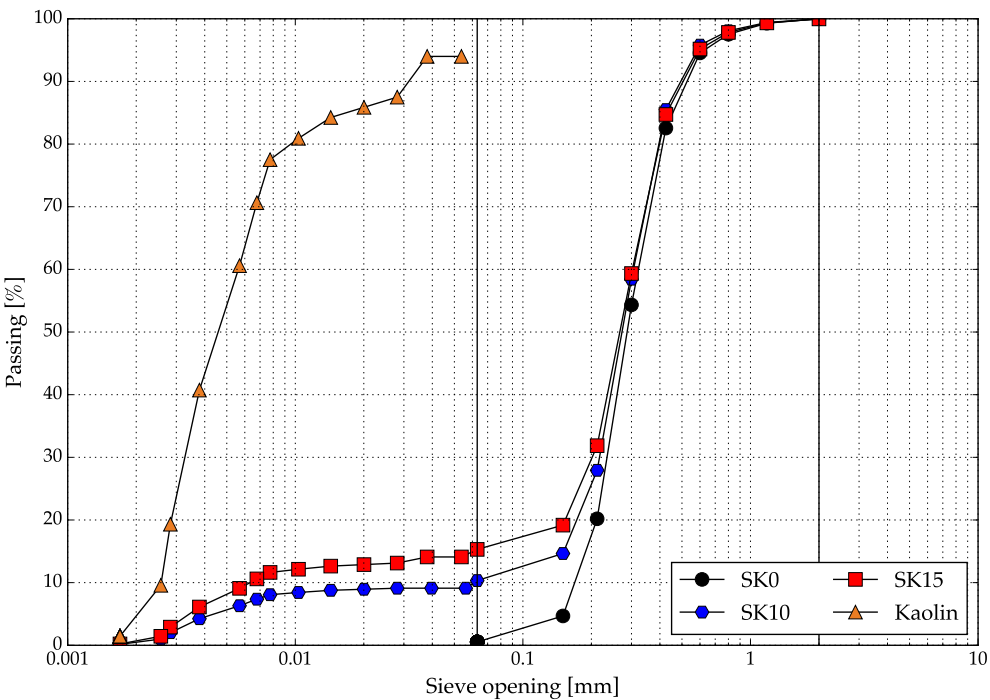


Figure 4. Particle size distribution of tested soil specimens.

Table 2. Tested soil specimens’ physical properties.

Soil Type	Specific Gravity (g_s)	D_{10} (mm)	D_{50} (mm)	d_{50} (mm)	Liquid Limit (%)	Plastic Limit (%)
SK0	2.70	0.1713	0.2888	N/A	N/A	N/A
SK10	2.69	0.0535	0.2757	N/A	N/A	N/A
SK15	2.67	0.0063	0.2701	N/A	N/A	N/A
SK100	2.60	0.0026	0.0047	0.0047	53	30

2.2.2. Conventional Minimum and Maximum Void Ratio Determination

Both minimum and maximum void ratios of sands are used with the soil’s phase relations to define the soil mass amount needed to achieve the targeted density within a given volume. By implementing some existing modifications and suggestions to conventional laboratory tests [22,29], an attempt was made to determine the mixtures’ minimum and maximum void ratios. The host sand void ratios’ values were adopted from previous research [37]. According to the existing research, using cylindrical molds of different volumes that ranged from 31 cm^3 to 2000 cm^3 for minimum and maximum void ratios determination produced consistent void ratio values. The alternative cylindrical mold with a volume of 196.35 cm^3 , a height of 10 cm, and 50 mm in diameter was used to determine the minimum and maximum void ratio. The rest of the determination process was conducted according to [22,29]. For the maximum void ratio determination, the dry soil of constant mass was poured from zero height through a funnel into a cylindrical mold. After careful soil surface levelling, the height difference between the soil surface and the top of the cylindrical mold was measured with the caliper. Similarly, for the minimum void ratio determination, the dry soil of constant mass was poured into a cylindrical mold and densified. The densification process included 1000 strikes with an 80 g wooden handle, 2 cm in diameter. Strikes were done perpendicularly at the four sides of the cylindrical mold at its base. In other words, the soil was densified with 4 sets of strikes, with 250 strikes per set, turning the cylindrical mold by 90° after each set. Again, the height difference between the soil surface and the top of the cylindrical mold was measured with the caliper. The height difference enabled the calculation of the minimum and maximum void ratio using “phase relations”. As mentioned, the existing standards limit the testing to silty sand mixtures with non-plastic fines and free-draining

conditions, and should be inapplicable for silty sands with clay minerals. These tests were performed within the present research to evaluate the validity of void ratios used in previous research [18,19]. The plastic and liquid limits were used to determine the minimum and maximum void ratio of the kaolinite (SK100) specimen. The liquid limit represents the highest void ratio and water content in the clay fraction at which the soil behaves as a solid. The shrinkage limit theoretically represents the lowest clay volume in fully saturated conditions. Existing researchers [38,39] have determined that the shrinkage limit approximates the plastic limit for kaolinite samples. Additionally, according to [40], the plastic limit is the result of either cavitation or air entry, resulting in the water phase ceasing to act as a continuum within the soil thread. For these reasons, the plastic limit value was used to define the minimum void ratio of kaolinite used in mixtures. The kaolinite plastic and liquid limits were adopted from the existing research [32,33] and presented in Table 2).

2.2.3. Alternative Minimum and Maximum Void Ratio Determination

The alternative minimum void ratios for the SK10 and SK15 mixtures were analytically determined using Equation (10), [23]. The determination procedure involved the calculation of the individual sand and fines fractions' volume contents and their corresponding minimum and maximum void ratios. Since the mixtures' void ratios could not be validated experimentally, a different approach was used to determine the representative filling coefficient (a). Based on the existing test data [23], the filling coefficients (a) are fitted by a power function of particle size ratio d_{50}/D_{50} , Equation (14). The limiting cases are given with $a = 1$ at $d_{[50]}/D_{[50]} = 0$, and $a = 0$ at $d_{50}/D_{50} = 1$.

$$a = \left(1 - \frac{d_{50}}{D_{50}}\right)^p \quad (14)$$

where p is the fit parameter.

The best-fit curve with the corresponding p-value was obtained using the complete available data set. With the obtained a coefficient and corresponding minimum void ratio, the maximum void ratio could be obtained using the obtained parameters α and β with Equations (11), (12), and (13) [24]. Specimens reconstituted this way were further evaluated with the triaxial test results and correlated with the conventionally reconstituted specimens' test results.

2.3. Monotonic Compression

2.3.1. Triaxial Testing System

The silty sand mixtures drained and undrained static behaviour was tested with an automated triaxial system. Triaxial tests were performed at the University of Rijeka, Faculty of Civil Engineering's Geotechnical Laboratory. The triaxial system, shown in Figure 5, is manufactured by Wyckeham Farrance, Controls Group, and equipped with a computer-controlled system for easy test performance [41]. The triaxial testing system used in this study included a 50 kN (Figure 5(a)) loading frame, a triaxial cell that can withstand pressures up to 2500 kPa (Figure 5(b)), and a 25 kN submerged-type load cell (Figure 5(c)) for precise force measurement under saturated conditions. The triaxial system uses transducers to measure cell (Figure 5(d)), pore (Figure 5(e)), and back pressures (Figure 5(f)) during the saturation, consolidation, and shear phases. The vertical linear variable displacement transducer (LVDT) (Figure 5(g)) measured the vertical displacement during the test. Volumetric change was monitored with the volume change device (Figure 5(h)). Membrane corrections were incorporated into the analysis [42,43].

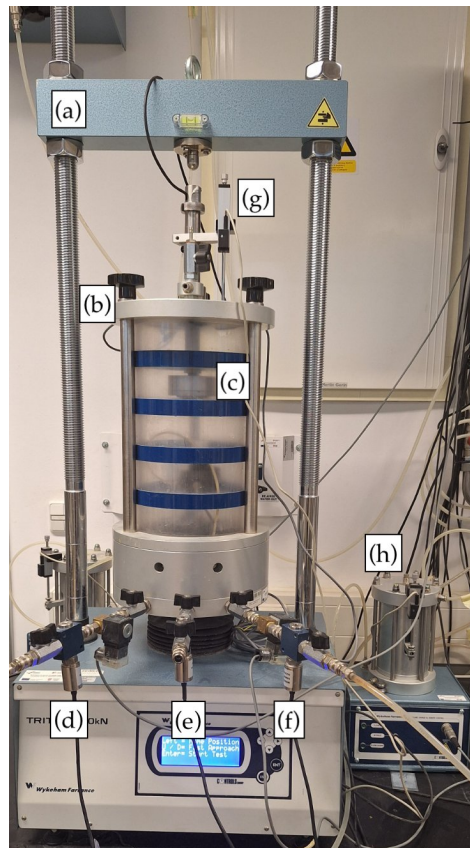


Figure 5. Triaxial system used: (a) Triaxial frame, (b) Triaxial cell, (c) Submerged load cell, (d) Cell pressure transducer (e) Pore water pressure transducer, (f) Back pressure transducer, (g) LVDT and (h) Volume change device.

All tests were conducted according to the ASTM standards for consolidated drained [43] and undrained [44] triaxial compression tests.

2.3.2. Sample Preparation Method

Two sets of silty sand mixtures with changing fines content were reconstituted at 80% relative density for triaxial testing. The first set, including the host sand, was reconstituted using void ratios determined by conventional methods and recommendations used for sands and silty sands [22,29]. Another set of SK10 and SK15 mixtures was reconstituted using analytically determined alternative void ratios [23,24]. Test results were correlated and evaluated. For easier evaluation, the mixtures reconstituted using conventional determination methods will be referred to as "A-mixtures", and the mixtures reconstituted using alternative void ratios will be referred to as "B-mixtures". The specimens were reconstituted within a latex membrane in a cylindrical mold of 50 mm in diameter and 100 mm in height Figure 5. The undercompaction method [45] was used. This moist tamping method ensures continuous density throughout the sample size and reduces particle segregation, thus providing repeatable and comparable test results. Undercompaction degree of 5% was implemented in the specimen preparation. The soil mixtures were reconstituted at targeted water content and compacted in 10 layers of the same mass. The height of each layer was controlled with a standard laboratory caliper.

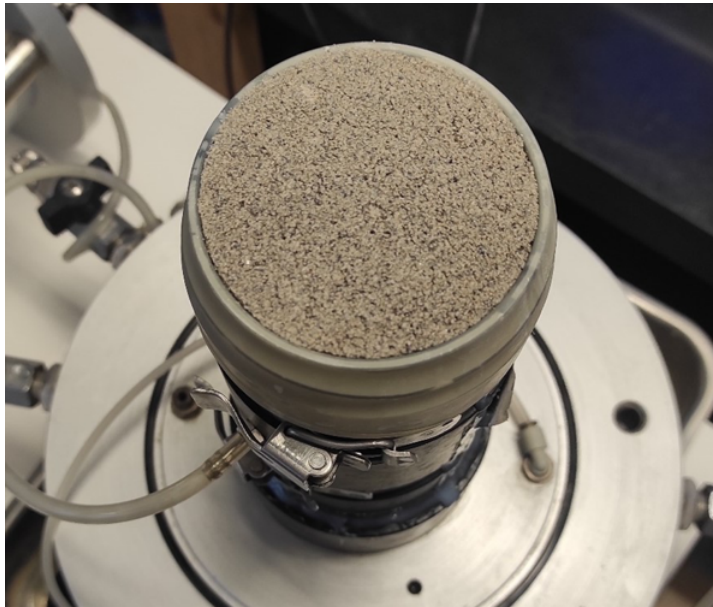


Figure 6. Prepared sample for monotonic triaxial test

2.3.3. Triaxial Testing Program

The reconstituted specimens were tested with the triaxial compression test in drained and undrained conditions, under two different initial confining stresses, 25 and 50 kPa. All tests were performed as strain-controll tests. The triaxial testing program is summarised in Table 3.

Table 3. Tested soil specimens’ physical properties

Test ID	Soil Type	Mixture Type	Relative Density D_r [%]	Effective Mean Confining Stress p'_0 [kPa]	Test Type
050_SK0_DR_EFF25	SK0	N/A	80	25	CID
041_SK0_DR_EFF50	SK0	N/A	80	50	CID
015_SK10_A_DR_EFF25	SK10	A	80	25	CID
021_SK10_A_DR_EFF50	SK10	A	80	50	CID
018_SK15_A_DR_EFF25	SK15	A	80	25	CID
012_SK15_A_DR_EFF50	SK15	A	80	50	CID
049_SK10_B_DR_EFF25	SK10	B	80	25	CID
046_SK10_B_DR_EFF50	SK10	B	80	50	CID
048_SK15_B_DR_EFF25	SK15	B	80	25	CID
047_SK15_B_DR_EFF50	SK15	B	80	50	CID
042_SK0_UND_EFF25	SK0	N/A	80	25	CIU
015_SK0_UND_EFF50	SK0	N/A	80	50	CIU
016_SK10_A_UND_EFF25	SK10	A	80	25	CIU
020_SK10_A_UND_EFF50	SK10	A	80	50	CIU
007_SK15_A_UND_EFF25	SK15	A	80	25	CIU
054_SK15_A_UND_EFF50	SK15	A	80	50	CIU
043_SK10_B_UND_EFF25	SK10	B	80	25	CIU
035_SK10_B_UND_EFF50	SK10	B	80	50	CIU
044_SK15_B_UND_EFF25	SK15	B	80	25	CIU
045_SK15_B_UND_EFF50	SK15	B	80	50	CIU

A – reconstituted using void ratios determined with conventional methods for sands (A-mixtures). B – reconstituted using analytically determined alternative void ratios (B-mixtures). CID - Consolidated Isotropically Drained. CIU - Consolidated Isotropically Undrained.

Before the start of the tests, the reconstituted specimens were saturated in three steps: percolation with CO₂, percolation with de-aired water. Saturation was performed in the back pressure increments. By utilising lower values of back pressure, specimens were subjected to the CO₂ flow for 15-30 min according to the existing suggestions [41]. After the percolation with CO₂, specimens were percolated with de-aired water until the water visibly started to be pushed out of the specimens. The saturation was performed in several stages by successive increases of cell and back pressure increments. The degree of saturation was monitored with Skempton's B coefficient [42,46]. After reaching Skempton's B coefficient value B=0.99 the sample is considered fully saturated and can be consolidated. For all tests performed, the isotropic consolidation was performed in a single stage since the targeted effective stresses of 25 and 50 kPa represent low confining stresses, with low resulting hydraulic gradients during consolidation. The technique of slow drainage valve opening was used here as in other research by authors [41,47,48] to reduce the effect of the unfavourable increase in hydraulic gradient during saturation and consolidation. As mentioned, the reconstituted specimens were tested with the triaxial compression test in drained and undrained conditions, under two different initial confining stresses: 25 and 50 kPa. Strain-controlled tests were performed to evaluate the tested silty sands' drained and undrained behaviour at constant relative density but different initial confining stresses. The monotonic shearing (compression) stage was performed at a 4% per hour axial strain rate.

3. Results and Discussion

3.1. Conventional Minimum and Maximum Void Ratio

The cylindrical mold with a volume of 196.35cm³, a height of 10cm, and 50mm in diameter, was used to determine the minimum and maximum void ratio of the host sand and "A-mixtures" tested. The determination procedure was conducted according to [22,29]. The plastic and liquid limits were used with existing relations Equation 1 to determine the minimum and maximum void ratio of the kaolinite (SK100) specimen. The host sand (SK0) void ratios' values were adopted from previous research [37]. Conventionally determined minimum (e_{min}) and maximum (e_{max}) void ratios for tested soils are presented in Table 4.

Table 4. Conventionally determined minimum (e_{min}) and maximum (e_{max}) void ratios.

Specimen Type	Minimum Void Ratio e_{min}	Maximum Void Ratio e_{max}
SK0	0.641	0.911
SK10	0.596	1.022
SK15	0.640	1.128
SK100	0.780	1.378

A visible segregation of kaolinite particles was present while performing the minimum void ratio determination tests on the A-mixtures. During the densification process, kaolinite particles began to surface through the soil mixture and cover the top layer in the testing mold. This is caused by electrostatically charged large area surfaces of platy kaolinite particles, which overcome the gravitational forces in the process. The dry testing condition enables free movement of the microscopically small kaolinite particles, and vibration causes the kaolinite particles' large and negatively charged surfaces to come in contact and repel each other. Within the laterally confined mold and continuously densified sand fraction, the kaolinite particles that overcome gravitational forces gradually segregate upwards to the specimen's surface. Observations on using the conventional tests [20,21] for silty sands with clay minerals relate well to the inapplicability of these testing procedures for such mixtures, even within the sand-dominated soil structure. This also implies the mis-calculated targeted relative density within the present and the existing research [18,19].

3.2. Alternative Minimum and Maximum Void Ratios

The existing analytical approach [23,24] was used to redefine the mixtures’ minimum and maximum void ratios. The minimum void ratio is obtained by determining the mixture’s specific parameters a and p using the Equation 14. Since the mixtures’ void ratio values could not be physically determined (as previously discussed with the conventional determination method for minimum and maximum void ratio), the complete available data set [23] was used to obtain the best-fit curve with the corresponding p parameter and a coefficient Figure 7.

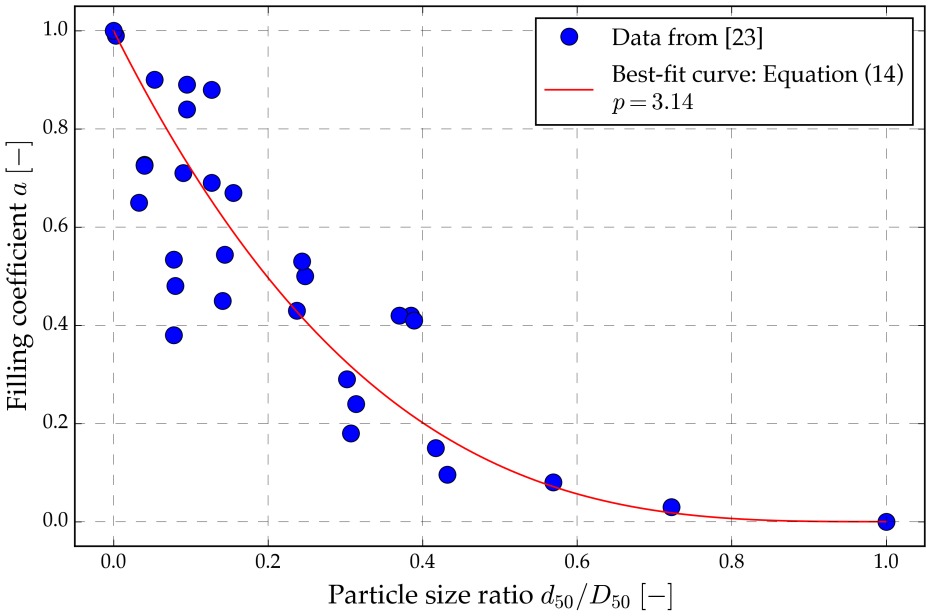


Figure 7. The filling coefficient a relation to a particle size ratio d_{50}/D_{50} , and computed best-fit power trend, [23].

With the obtained p parameter, the a coefficient, and the corresponding minimum void ratio, the maximum void ratio Equation (11) was obtained using the parameters α Equation (12) and β Equation (13), [24]. The values of determined B-mixtures parameters used for alternative void ratios determination are presented with Table 5.

Table 5. Determined B-mixtures parameters used for alternative void ratios determination.

Filling Coefficient α	Mean Grain Size Ratio d_{50}/D_{50}	Power Fit Parameter p	Filling Constant α	Filling Constant β
0.957	0.014	3.14	1.151	0.173

The value of the filling coefficient $a = 0.957 \approx 1$, approximates the limiting case condition with all the fines filling the host sand pore spaces, and none participating in the mixture’s structure. This condition can also be identified as the intergranular [2] or the skeleton [3] void ratio. It relates well to case I condition Figure 3, with the mean particle sizes of sand and fines $D_{50}/d_{50} > 6.5$ for spherical particles, and $f_c < f_{thr}$, [15], and to the filling coefficient’s power trend relation to a particle size ratio Figure 7. The obtained alternative (B-mixtures) void ratios are presented and compared to the conventionally determined (A-mixtures) void ratios are presented in Table 6.

Table 6. SK10 and SK15 alternative void ratios (B-mixtures) comparison with the ones obtained with conventional methods (A-mixtures).

Soil Type	Void Ratio	B - Mixtures	A - Mixture
SK10	e_{min}	0.479	0.596
	e_{max}	0.725	1.022
SK15	e_{min}	0.399	0.64
	e_{max}	0.632	1.128

It can be observed that the alternative void ratio values are significantly lower than the conventionally determined ones. Moreover, the SK15_B specimen’s maximum void ratio was higher in value than the conventionally determined minimum void ratio. Regardless, both sets of mixtures were successfully reconstituted and further tested.

3.3. Sample Preparation

Since this research evaluates existing recommendations on silty sands’ physical characteristics determination, additional observations on the sample reconstitution process and visible characteristics are given. Compared to the host sand specimen, significantly less effort was made to densify the A-mixtures reconstituted at 80% relative density. Minimum to no compaction effort was applied to the lowest undercompacted layers. The reconstituted specimens physically appeared loose and were susceptible to deformation during the preparation for the triaxial test. A-mixtures also exhibited numerous macropores during the saturation stage, which is characteristic of loose specimens. In contrast, slightly more effort was made to densify the B-mixtures reconstituted at 80% relative density than the host sand specimen. The reconstituted B-mixtures physically appeared dense, exhibiting no macropores, and were also stable during the preparation for the triaxial test. An increase in the fines content causes an increase in the densification effort, indicating a possible increase in the relative density of reconstituted specimens with increasing fines content. Observations on reconstituted dense silty sands’ appearance and preparation process relate better to the physical characteristics defined with the alternative minimum and maximum void ratios than those determined by conventional methods for sands.

3.4. Physical Characteristics of Clayey and Silty Sands with Clay Minerals

New findings regarding the clay minerals’ influence on silty and clayey sands below the fines threshold are based on observational and experimental data from this research. Clay particles in dense, saturated, and isotropically confined silty sands will likely get crushed or slip between the sand grains in the sand skeleton pore spaces. The mechanism for such particle migration below the fines threshold is highly dependent on the saturation level and relative density, with clay minerals absorbing various amounts of water that infiltrate the available pore spaces. The clay mineral’s hydrophilic property will ensure the complete absorption of the free pore water within the sand skeleton. Additionally, the volume change with increasing saturation should be considered with expansive clays. Due to the differences in sand to clay particle shapes, sizes and material hardness, the crushing and rearranging of clay particles can occur in relatively dry specimens. In partly or fully saturated specimens, the water content will govern the behaviour of the clay fraction. With increasing water content, the clay will go from plastic and paste-like behaviour up to the liquid limit water content, to slip or flow beyond the liquid limit water content [49]. The slip and flow mechanisms are especially unfavourable in drained conditions, since they cause selective particle migration (internal erosion or suffusion) and loss of specimen homogeneity. In contrast, within unsaturated mixtures (below the clay fractions’ liquid limit water content), the angular clay particles can enhance the particle bridging and consequently tend to produce higher void ratios than equivalent spheres [29], schematically presented as case III Figure 3, [15]. The void ratios of silty sand mixtures with clay minerals appear to be highly dependent on the governing confining stresses, draining conditions, saturation degree, clay mineral composition and

volume fraction, and the configuration of the sand skeleton. Unlike pure sands with well-defined and stable structure, silty sands with clay minerals will be subjected to varying void ratios caused by the clay’s fraction phase transformation at different saturation degrees. Due to the clay fractions’ slip or flow-like behaviour, the particle bridging is improbable above the clay fractions’ liquid limit water content. This also implies that below the fines threshold, soil structure presented with cases II and III Figure 3 will gradually collapse to case I with increasing saturation degree, due to the loss of clay cohesive forces and its inability to form stable bridges between the sand grains. Considering the present research with triaxial tests on fully saturated soil specimens, the analytically determined alternative void ratios with the skeleton void ratio approximation relate well to the case I condition, and the corresponding observations given. The observations also indicate that the conventional and analytical determination methods for silty sands with clay minerals do not provide exact or credible minimum and maximum void ratio values, as is obtainable for clean sands.

3.5. Drained Compression Tests

A total of 10 drained monotonic compression tests were performed on two types of SK10 and SK15 mixtures and the host sand (SK0) specimens. Each specimen was reconstituted at 80% relative density and isotropically consolidated to two varying initial confining stresses (25 and 50 kPa). After the saturation and consolidation stages, specimens were subjected to drained compression at a 4% axial strain rate per hour. The quantitative results of the drained compression tests are summarised in Table 7.

Table 7. Summary of specimens with relative density of 80%, initial conditions and critical state values.

Test ID	Initial Conditions		Peak State Values	Critical State Values		
	Initial void ratio e_0 [-]	Confining mean effective stress p'_0 [kPa]	Stress ratio at peak η_{peak} [-]	Void ratio at critical state e_c [-]	Stress ratio at critical state η_{cs} [-]	Mean effective stress at critical state p'_{cs} [kPa]
050_SK0_DR_EFF25	0.695	25	1.454	0.693	1.096	42.55
041_SK0_DR_EFF50	0.695	50	1.545	0.690	1.265	88.17
015_SK10_A_DR_EFF25	0.681	25	1.274	0.652	1.274	45.19
021_SK10_A_DR_EFF50	0.681	50	1.272	0.652	1.272	83.78
018_SK15_A_DR_EFF25	0.738	25	1.318	0.705	1.318	48.00
012_SK15_A_DR_EFF50	0.738	50	1.275	0.705	1.275	90.00
049_SK10_B_DR_EFF25	0.522	25	1.663	0.519	1.313	44.59
046_SK10_B_DR_EFF50	0.522	50	1.536	0.519	1.270	80.89
048_SK15_B_DR_EFF25	0.436	25	1.677	0.435	1.250	56.00
047_SK15_B_DR_EFF50	0.436	50	1.683	0.434	1.333	100.00

For A-mixtures reconstituted using conventional methods for sands, the deviatoric stress to axial strain relation diagram Figure 8(a) indicates a constant increase in deviatoric stress with increasing axial strain. The critical state was reached at approximately 15% axial strain for the SK10_A specimen and 20% axial strain for the SK15_A specimen. At 50 kPa initial confining stress, the SK15_A responded with a lower peak deviatoric stress than the SK10_A. The volumetric response to axial strain Figure 8(b) was relatively consistent for all initial confining stresses. At approximately 15% axial strain, a constant volumetric strain of 2.3% for the SK10_A and 3.0% for the SK15_A, was reached.

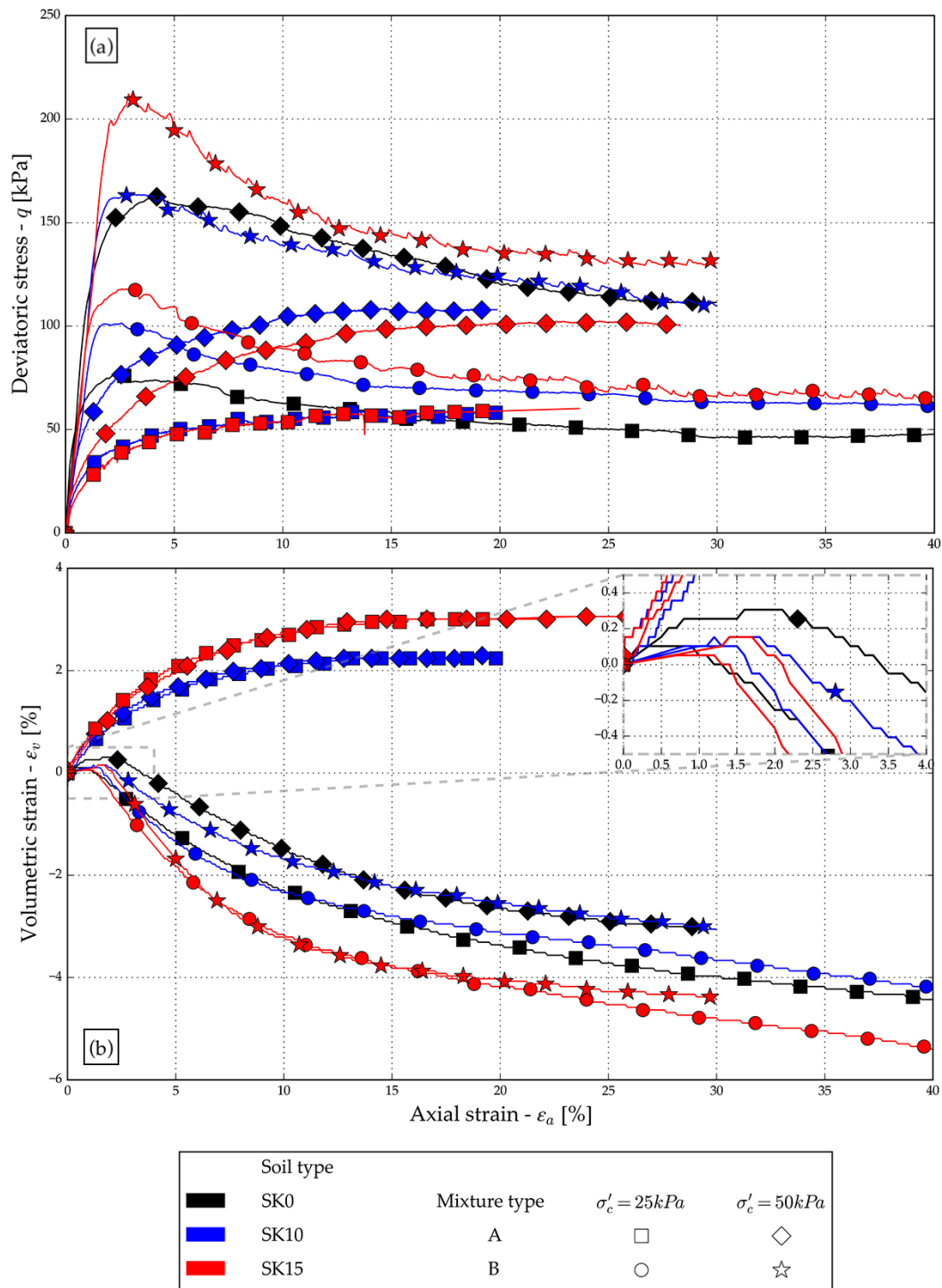


Figure 8. Results of a drained tests: (a) stress-strain curves and (b) volumetric strain vs axial strain.

The peak deviatoric stress was reached within 2-4% of axial strain for the host sand and B-mixtures reconstituted using alternative void ratios Figure 8(a). At higher axial strains, the deviatoric stress begins to drop to a critical state value at approximately 30% axial strain. The SK15_B specimens exhibited a slightly higher deviatoric stress increase than the host sand and SK10_B specimens at 25 and 50 kPa initial confining stresses. The SK10_B also exhibited a slightly higher deviatoric stress increase than the host sand at 25 kPa initial confining stress, while at the 50 kPa initial confining stress, the SK10_B and host sand showed similar deviatoric stress to axial strain relation. The host sand and B-mixtures initially undergo compression in the early stages of axial loading, enlarged part of Figure 8(b). The highest positive volumetric strain of 0.3% at 2% axial strain is recorded for the host

sand tested at 50 kPa confining stress. The B-mixtures initially exhibited smaller volumetric strain than the host sand at 50 kPa confining stress. After initial compression, the volumetric strain begins to drop with an increased negative value at higher axial strains as the specimens undergo constant dilation Figure 8(b). The SK15_B specimens at 25 and 50 kPa initial confining stresses resulted in the highest negative volumetric strain. The host sand and SK10_B specimens show similar dilative behavior, which increases at lower confining stress. The deviatoric stress and effective stress ratio response to the axial strain graphically represents the loose behavior for A-mixtures and dense behavior for the host sand and B-mixtures. A constant positive volumetric strain for the A-mixtures indicates compression, a characteristic of loose sand specimens. The host sand and B-mixtures exhibited initial compression at very low axial strains and dilation with decreasing negative volumetric strain at higher axial strains, which is characteristic of medium-dense and dense sand specimens.

Figure 9 shows the complete stress paths from performed drained tests, while peak and critical state values are presented in Figure 10 The A-mixtures’ peak deviatoric stress values are positioned lower in the $p' - q$ plane than those of the host sand and B-mixtures Figure 9 and 10, resulting in a lower inclination of the projected critical state lines (CSL) (Figure 10 and Table 7). An increase in the CSL angle with an increase in fines content can be observed with B-mixtures.

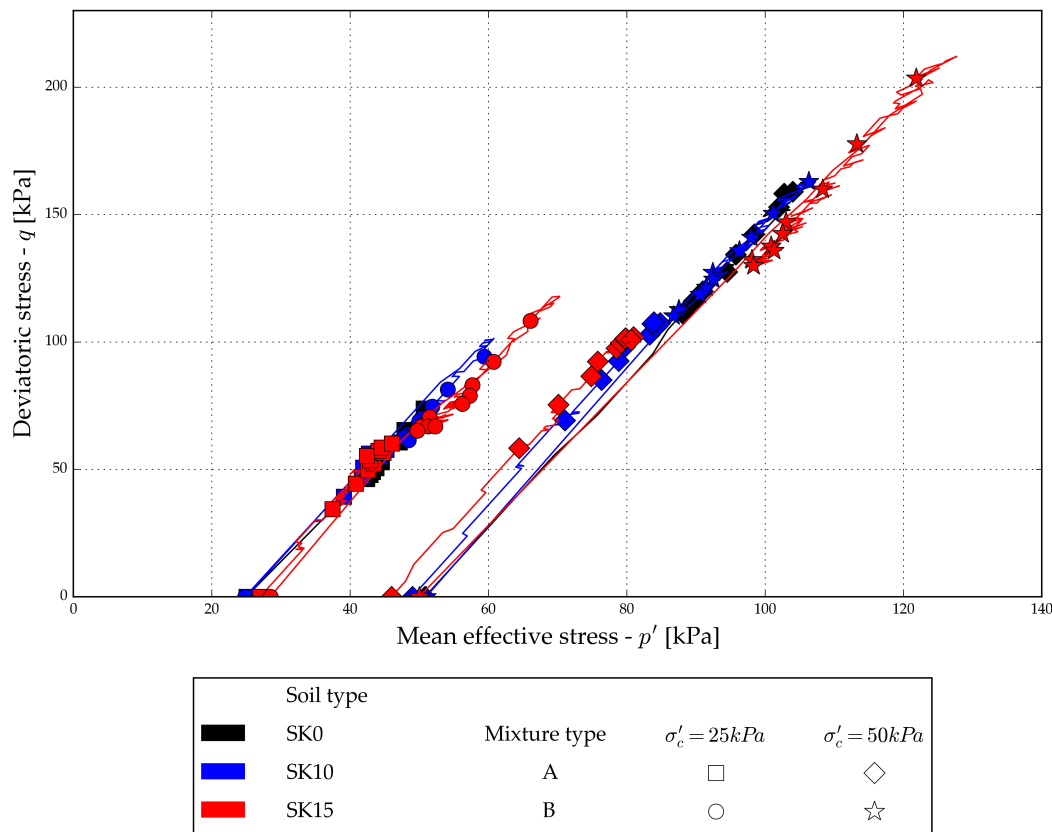


Figure 9. Drained stress-path presentation of the tested mixtures compared with the determined critical state line for each specimen type.

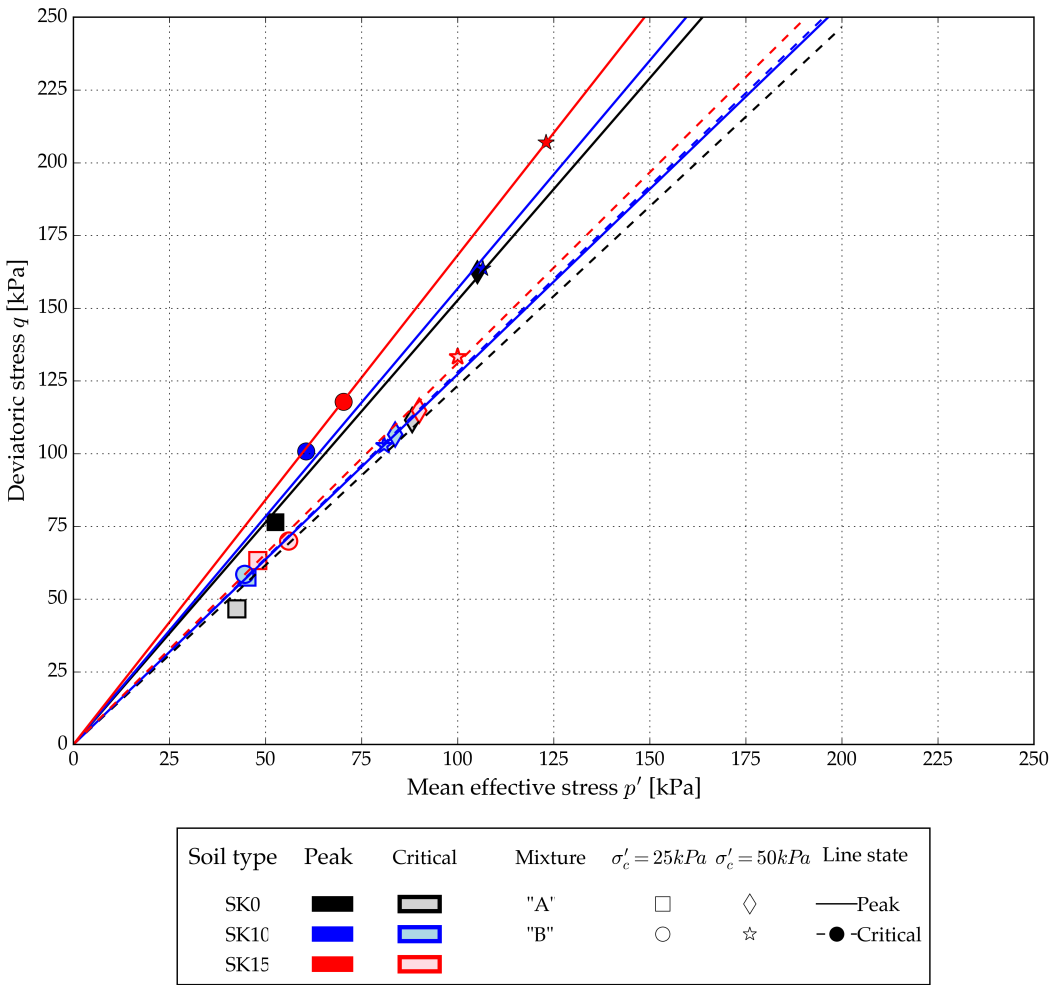


Figure 10. Peak and critical state values with assumed peak and critical lines.

For A-mixtures, deviatoric stress and volumetric strain response to axial strain in drained conditions correlate well with the lower CSL position than the host sand’s CSL, defining samples reconstituted at a looser state than the host sand. Since the specimens tested were fully saturated, the water content surpassed the clay fraction’s liquid limit. With consequential loss of cohesive forces, the unfavourable slip and flow mechanisms will govern the specimen’s clay fraction in drained conditions, with the potential of selective particle migration (internal erosion or suffusion) and loss of specimen homogeneity. Regarding the B-mixtures, a small change in soil behaviour with a change in fines content could be attributed to the specimen’s varying relative density due to the inaccurately analytically determined void ratios, to the changing clay consistency at different saturation degrees, or the potential structural rearrangement of the clay fraction during the test (loss of specimen homogeneity).

3.6. Undrained Compression Tests

A total of 10 undrained monotonic compression tests were performed on two types of SK10 and SK15 mixtures and the host sand (SK0) specimens. Each specimen was reconstituted at 80% relative density and isotropically consolidated to two varying initial confining stresses (25 and 50 kPa). After the saturation and consolidation stages, specimens were subjected to undrained compression at a 4% axial strain rate per hour. The quantitative values of the undrained compression tests are summarized in Table 8.

Table 8. Summary of undrained tests with initial conditions, undrained instability state and steady state values.

Test ID	Initial Condition		Undrained Instability State		Steady State	
	Void ratio e_0 [-]	Confining mean effective stress p'_0 [kPa]	Mean effective stress p'_{uis} [kPa]	Deviatoric stress q_{uis} [kPa]	Mean effective stress p_{ss} [kPa]	Deviatoric stress q_{ss} [kPa]
042_SK0_UND_EFF25	0.695	25	425.68	605.05	425.68	605.05
015_SK0_UND_EFF50	0.695	50	435.49	630.03	435.49	630.03
016_SK10_A_UND_EFF25	0.681	25	19.52	10.56	0.05	0.16
020_SK10_A_UND_EFF50	0.681	50	36.11	24.23	4.01	3.03
007_SK15_A_UND_EFF25	0.738	25	21.17	12.52	0.22	0.65
054_SK15_A_UND_EFF50	0.738	50	35.93	20.79	2.63	1.9
043_SK10_B_UND_EFF25	0.522	25	383.14	486.42	383.14	486.42
035_SK10_B_UND_EFF50	0.522	50	574.56	769.69	579.28	720.83
044_SK15_B_UND_EFF25	0.436	25	647.11	891.32	647.11	891.32
045_SK15_B_UND_EFF50	0.436	50	729.41	1033.24	729.41	1033.24

For A-mixtures, the peak deviatoric stress, ranging between 10 and 25 kPa, was reached at 0.3% axial strain Figure 11. After reaching the peak value, the deviatoric stress decreases to 0 with a further increase in axial strain at approximately 3% axial strain for A-mixtures tested at 10 and 25 kPa and at approximately 5% axial strain at 50 kPa. The pore water pressure changes to axial strain relation diagram Figure 11 shows an increase in the positive pore pressure with increasing axial strain to an approximately constant value at 2-3% axial strain. The pore water pressure increase appears unaffected by the fines content.

The peak deviatoric stress was reached within 7-15% of axial strain for the host sand and B-mixtures Figure 11(a). With increasing axial strains, the deviatoric stress gradually declines. The host sand and B-mixtures initially show a positive pore water pressure change (Fig.9d) in the early stages of axial loading (up to 1% axial strain). The initial pore water pressure increase is unaffected by the fines content. With increasing axial strain, the pore water pressure changes drop to a constant negative value at approximately 7-25% axial strain Figure 11(b). The decrease in the rate and value of pore water pressure is affected by the fines content but is unaffected by changing confining stresses. The higher the fines content, the higher the increase in negative pore pressure. The peak deviatoric stress achieved with A-mixtures was significantly smaller than that achieved with host sand and B-mixtures. This relates well to the deviatoric stress to axial strain response, which graphically represents the loose behaviour for A-mixtures and dense behaviour for the host sand and B-mixtures. Deviatoric stress rapidly drops to zero value with increasing axial strain for A-mixtures, indicating a complete liquefaction of the specimens. The change in pore water pressure with increasing axial strain indicates constant compression for A-mixtures, a characteristic of loose sand specimens. The initial pore water pressure increase, followed by a decrease to a constant negative value with increasing axial strain for the host sand and B-mixtures, indicates an initial compression followed by dilation, a characteristic of dense and mid-dense sands. Again, a good relation with the deviatoric stress to axial strain response, graphically representing the loose behaviour for A-mixtures and dense behaviour for the host sand and B-mixtures. Both A-mixtures develop similar stress-paths during undrained shearing at low initial confining stresses Figure 12, and a full liquefaction effect can be observed. The A-mixtures' undrained steady state lines (SSL) exhibit a smaller inclination in the p' - q plane than those of the host sand and B-mixtures. For the A-mixtures, the undrained instability state lines (ISL) are projected through the measured peak deviatoric stress values according to [50], (Figure 13).

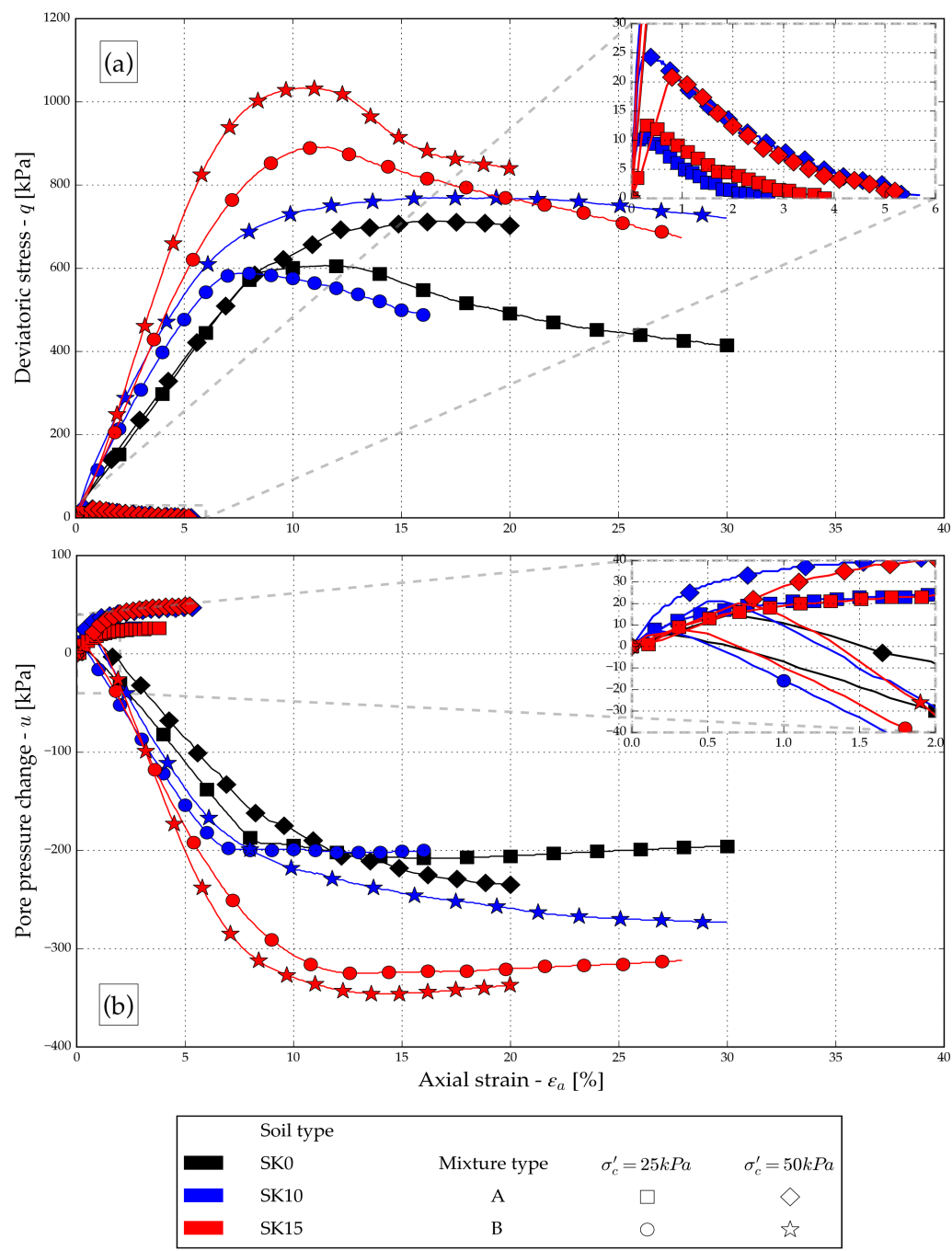


Figure 11. Results of a drained tests: (a) stress-strain curves and (b) volumetric strain vs axial strain.

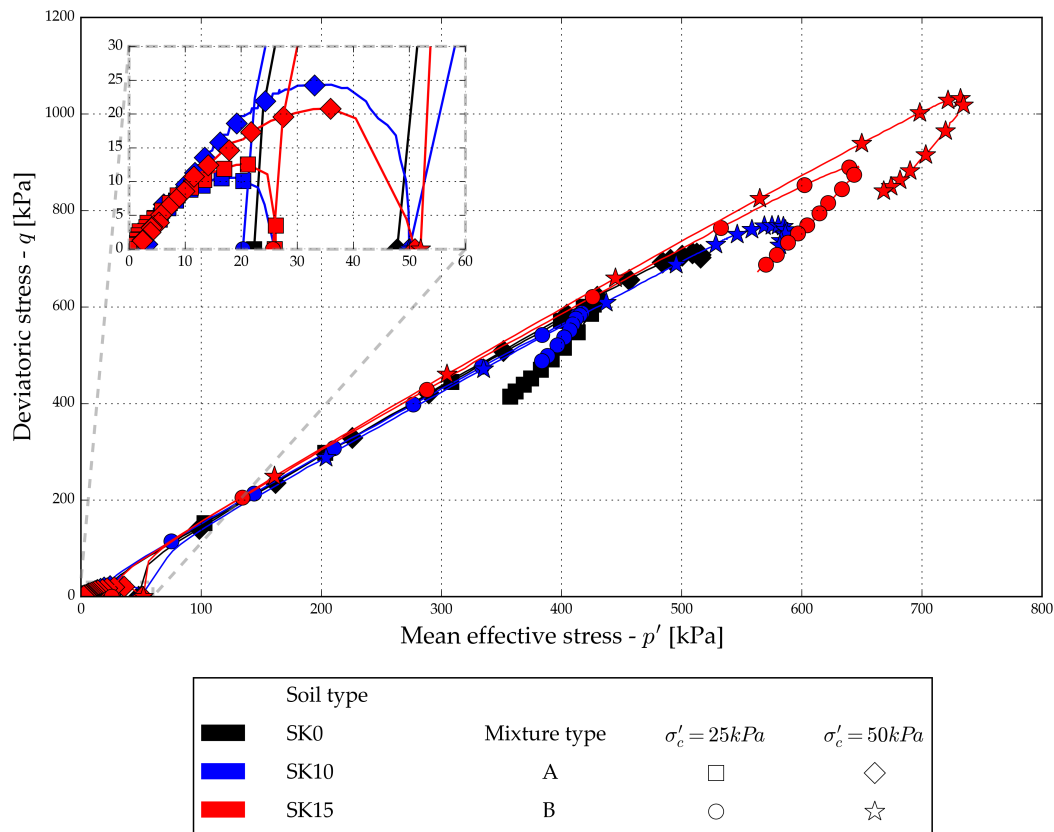


Figure 12. Undrained stress-path presentation of the tested mixtures compared with the determined steady state line for each specimen type.

For host sand and B-mixtures tested at 50 kPa initial confining stress, the peak q and p' values increase with an increase in fines content. At the 25 kPa initial confining stress, the SK15_B specimen reaches higher values of q and p' than the host sand, and the SK10_B reaches similar values to those of the host sand. The A-mixtures' deviatoric stress and pore water pressure response to small axial strains in undrained conditions indicate complete liquefaction and loose samples' behaviour. This correlates well with the determined SSL and ISL linear trend, in contrast to the B-mixtures' CSL. As mentioned, with consequential loss of cohesive forces due to full saturation of the specimens, the unfavourable slip and flow mechanisms will govern the specimen's clay fraction in undrained conditions. Regarding the B-mixtures, a small change in soil behaviour with a change in fines content could be attributed to the specimen's varying relative density due to the inaccurately determined void ratios, or the changing clay consistency at different saturation degrees.

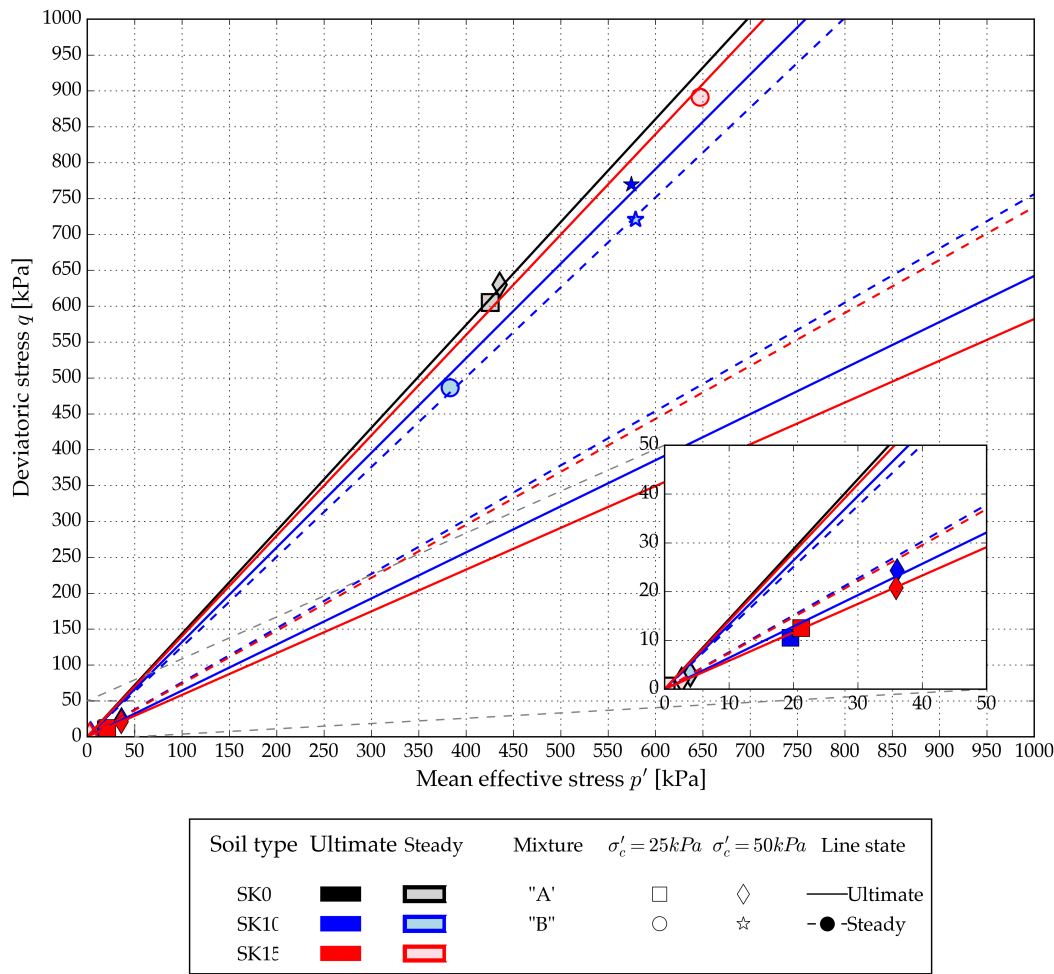


Figure 13. Ultimate state and steady state values with assumed ultimate and steady state lines.

4. Conclusions

The present research summarizes the experimental and observational findings of the clay minerals' influence on the reconstitution process and drained and undrained static behaviour of silty and clayey sands. Several concluding remarks can be drawn concerning the void ratio determination suggestions and the effect of clay mineral fines on the silty sands' void ratios. Additionally, the clay mineral influence on the silty sand's drained and undrained static behavior is validated with experimental data and further elaborated.

4.1. Physical Characteristics of Clayey and Silty Sands with Clay Minerals

The clay particles will likely be subjected to the crushing or slip/flow mechanism in dense, isotropically confined, and saturated silty sand conditions. With increasing water content, the clay will go from plastic and paste-like behaviour up to the liquid limit water content, to slip or flow beyond the liquid limit water content [49]. The slip and flow mechanisms are especially unfavourable in drained conditions, since they cause selective particle migration (internal erosion or suffusion) and loss of specimen homogeneity. The potential loss of specimen homogeneity can negatively affect the experimental results, which should be considered when choosing the test type. Within unsaturated mixtures (below the clay fractions' liquid limit water content), the angular clay particles can enhance the particle bridging and consequently tend to produce higher void ratios than equivalent spheres [29], schematically presented as case III Figure 3, [15]. Due to the clay fractions' slip or flow-like behaviour, the particle bridging is improbable above the clay fractions' liquid limit water content. This also implies that below the fines threshold, soil structure presented with cases II and III Figure 3 will gradually collapse to case I with increasing saturation degree, due to the loss of clay cohesive forces and its

inability to form stable bridges between the sand grains. The void ratios of silty sand mixtures with clay minerals appear to be highly dependent on the governing confining stresses, draining conditions, saturation degree, clay mineral composition and volume fraction, and the configuration of the sand skeleton. Unlike pure sands with well-defined and stable structure, silty sands with clay minerals will be subjected to varying void ratios caused by the clay's fraction phase transformation at different saturation degrees. Additionally, the volume change with increasing saturation should be considered with expansive clays.

4.2. Void Ratios Determination Methods

The observed clay particle segregation when using the conventional determination methods [20,21] with existing recommendations [22,29] for A-mixtures relate well to the inapplicability of these testing procedures for silty sands with clay minerals, even within the sand-dominated soil structure. This also implies the miscalculated targeted relative density within the present and the existing research [18,19]. The reconstituted specimens physically appeared loose and were susceptible to deformation during the preparation for the triaxial test. A-mixtures also exhibited numerous macropores during the saturation stage, a characteristic of loose specimens. The analytically determined void ratios for B-mixtures approximate the limiting case condition with all the fines filling the host sand pore spaces, and none participating in the mixture's structure. This condition can also be identified as the intergranular [2] or the skeleton [3] void ratio. It relates well to case I condition Figure 3, with the mean particle sizes of sand and fines $D_{50}/d_{50} > 6.5$ for spherical particles, and $f_c < f_{th}$ [15], and to the filling coefficient's power trend relation to a particle size ratio Figure 8. The reconstituted B-mixtures physically appeared dense, exhibiting no macro-pores, and were also stable during the preparation for the triaxial test. An increase in the fines content causes an increase in the densification effort, indicating a possible increase in the relative density of reconstituted specimens with increasing fines content. The conventional void ratio determination methods appear inapplicable to properly define the physical characteristics of silty sands with clay minerals. Compared to the host sand, the analytical method relates well to the appearance and preparation process, and the triaxial experimental data, indicating the dense sand behavior for silty sands with clay minerals. However, due to the varying characteristics of clay minerals, the existing determination methods for silty sands with clay minerals do not provide exact minimum and maximum void ratio values, as is obtainable for clean sands with conventional methods. The analytical methods [2,23,24] can be applied to redefine the void ratios of silty sands with clay minerals for the specimen reconstitution process and further approximation of the soil behavior in various experiments. The potential deviation from the "real" relative density should be considered when comparing the soil behavior with less significant testing condition differences. The analytical methods are based on the volumetric fractions of sand grains and clay particles, and should only be applied to non-expansive and low plasticity clay. To properly define the varying void ratio of the clayey and the silty sands with clay minerals, a new method should be developed that considers the clay mineral composition and fraction, saturation degree and changing clay consistency, and the primary configuration of the sand skeleton.

4.3. Drained Compression Tests

For A-mixtures, deviatoric stress and volumetric strain response to axial strain in drained conditions correlate well with the lower CSL position than the B-mixtures and the host sand's CSL, defining samples reconstituted at a loose state. The host sand and B-mixtures deviatoric stress and volumetric strain response to axial strain in drained conditions indicate a dense behavior. Since the specimens tested were fully saturated, the water content surpassed the clay fraction's liquid limit. With consequential loss of cohesive forces, the unfavorable slip and flow mechanisms will govern the specimen's clay fraction in drained conditions, with the potential of selective particle migration (internal erosion or suffusion) and loss of specimen homogeneity. Regarding the B-mixtures, a small change in soil behavior with a change in fines content could be attributed to the specimen's varying relative density due to the inaccurately analytically determined void ratios, to the changing clay consistency at different

saturation degrees, or the potential structural rearrangement of the clay fraction during the test (loss of specimen homogeneity).

4.4. Undrained Compression Tests

The deviatoric stress to axial strain response represents the loose behavior for A-mixtures and dense behavior for the host sand and B-mixtures. The A-mixtures' deviatoric stress and pore water pressure response to small axial strains in undrained conditions indicate complete liquefaction and loose samples' behavior. This correlates well with the determined SSL and ISL linear trend, in contrast to the B-mixtures' CSL. With consequential loss of cohesive forces due to full saturation of the specimens, the unfavorable slip and flow mechanisms will govern the specimen's clay fraction in undrained conditions. Regarding the B-mixtures, a small change in soil behavior with a change in fines content could be attributed to the specimen's varying relative density due to the inaccurately determined void ratios, or the changing clay consistency at different saturation degrees.

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Informed Consent Statement: "Not applicable".

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Conflicts of Interest: "The authors declare no conflicts of interest."

Abbreviations

The following abbreviations are used in this manuscript:

CSL	Critical State Line
SSL	Steady State Line
ISL	Instability State Line
LVDT	Linear variable differential transducers

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