

The role of shear dilation in the stability of boreholes in unconsolidated sands

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ARTICLE INFO

Editors-in-Chief:

Jean-Michel Pereira, Alessio Ferrari and Panos Papanastasiou

Keywords:

Borehole stability
Unconsolidated sands
Shear dilation
Triaxial tests
Cavity contraction

ABSTRACT

Boreholes are commonly used in the shallow and deep subsurface for the extraction or injection of fluids, e.g. for drinking water production, geothermal energy extraction or fluidic waste disposal. Many boreholes have been drilled successfully in unconsolidated sediments, however the common approach to evaluate borehole stability predicts failure when boreholes are drilled according to standard practice for groundwater wells, i.e. with little overpressure (< 40 kPa) in unconsolidated sand. To explore this mismatch, this study investigates the role of shear dilation in explaining the existence of stable boreholes in unconsolidated cohesionless sand through experimental and numerical analyses. Firstly, consolidated drained triaxial tests along different stress paths are performed on proxy and cored sand specimens. The results demonstrate the strong and immediate dilative behaviour of sand subjected to lateral unloading. Subsequently, a calibrated SANISAND material model is used in the numerical analysis of a cavity contraction problem. The results indicate that the amount of deformation and extent of a hoop stress reduction zone depend on the initial state of the sand. The early onset of irreversible deformation observed in both the experimental and numerical findings suggests that the standard practice for determining the minimum borehole overpressure is too conservative for unconsolidated sands. It is concluded that the initial state of the sand and associated shear dilation plays a prominent role in the stability problem of boreholes drilled with overpressure $p_w \sim 10$ kPa.

1. Introduction

When drilling a borehole, the surrounding formation has to support the load that was previously taken by the removed material in order to maintain a state of equilibrium, leading to stress concentrations around the borehole.¹ A drilling fluid (known as mud) is used, which is usually designed to form a filter cake on the borehole wall,² allowing mud overpressure to be maintained in the borehole without continuously adding additional mud. The mud (over)pressure exerted by the drilling fluid in the borehole (partially) compensates for the loss of support caused by the removal of material. Moreover, the drilling fluid additives which form a filter cake locally reduce permeability and increase cohesion, thereby contributing to wellbore strengthening.^{2–4}

Over the past decades, research on borehole stability has been driven by the oil and gas industry, leading to a good comprehension of the different borehole failure mechanisms in rocks under high stress regimes.^{1,5–7} To determine the corresponding minimum mud pressure, it is accepted practice to combine a linear elastic model with a rock failure criterion.^{8–13} While there is consensus on the conservative nature of this approach, it is generally deemed the most appropriate,

since a more complex but improved description of material behaviour is often considered unfeasible due to a lack of field data, and alternative definitions of borehole stability are thus far relatively arbitrary.¹² In addition, conservatism regarding borehole stability is often preferred, and thus staying well within the safe limits of the mud pressure window is desired.

Groundwater wells are often drilled in relatively shallow aquifers, which generally consists of much weaker material, such as unconsolidated sand. It is also common practice to drill these type of wells with borehole overpressure close to zero.¹⁴ In evaluating these conditions using simply an elastic solution and a shear failure criterion, e.g. the Mohr–Coulomb,¹⁰ the theory usually predicts failure with the low borehole overpressures. However, given the fact that numerous wells have been drilled successfully in sand formations, this exposes a significant discrepancy between the existing theory and practice. A detailed understanding of borehole stability becomes even more relevant when considering borehole expansion techniques, as described by Van der Schans et al.¹⁴

The elastic effective stress distribution around a vertically drilled borehole in an isotropic medium with an initially uniform horizontal

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<https://doi.org/10.1016/j.gete.2026.100890>

Received 17 March 2026; Received in revised form 20 August 2026; Accepted 20 August 2026

Available online 28 August 2026

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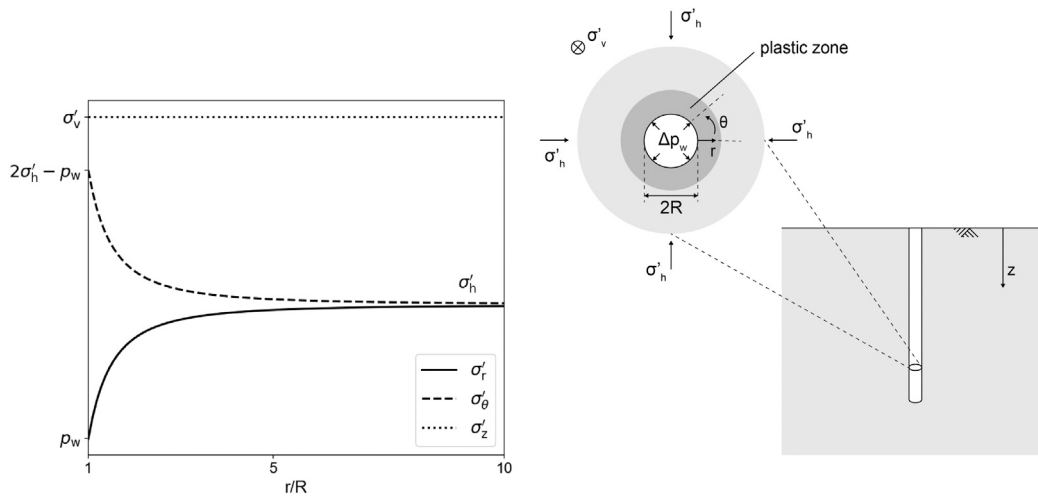


Fig. 1. Effective stress field (left) surrounding a borehole (schematically shown right) according to the elastic Kirsch solution with respect to the radial distance r .¹⁵ The effective stress field is expressed in polar coordinates, such that σ'_r is the radial stress, σ'_θ is the hoop stress and σ'_z is the stress in the z -direction. The in-situ stress state is defined by the vertical stress σ'_v and the uniform horizontal stress σ'_h , with the radius of the borehole given by R and the overpressure by p_w .

stress field according to the analytical Kirsch solution¹⁵ is shown in Fig. 1 in polar coordinates (r) normalised to the borehole radius (R). In this solution, it is assumed that there is no pressure loss over the filter cake and that a uniform horizontal stress field exists where $\sigma'_v > \sigma'_h$. In line with the geomechanical sign convention, compressive stresses are denoted as positive (+). The elastic solution shows that at the borehole wall ($r/R = 1$), the effective radial stress σ'_r drops with respect to σ'_h and equals the borehole overpressure p_w . The effective hoop stress σ'_θ , on the other hand, symmetrically increases compared to the drop in effective radial stress σ'_r , such that the mean stress remains constant. The difference between the effective hoop and radial stresses leads to shear stress and potentially (plastic) shear deformation and/or failure. In this case, a zone of plastic deformation is expected to form around the borehole,^{16,17} which can lead to borehole caving and eventually collapse.

From the elastic stress distribution shown in Fig. 1, it can be deduced that, at the borehole wall, $\sigma'_r \approx 0$ when the overpressure p_w is close to zero as commonly applied in the field. Given the fact that sand can reasonably be assumed to be cohesionless, the question arises of how a borehole can be stable when the minimum principal stress $\sigma'_3 = \sigma'_r = 0$ which would imply that the material is failing by the most common failure criteria, i.e. Mohr–Coulomb. Even if the formation may have some cohesion due to the presence of natural fines or locally to the well borehole due to filter cake additives,³ the existence of stable boreholes when drilled under the given conditions cannot be explained at any significant depth. Moreover, while sandcastles might be kept standing because of the suction forces at play,^{18,19} this explanation does not hold in a fully saturated aquifer (due to high permeabilities and the availability of water).

In this study, it is hypothesised that the dilative properties of sand in unloading conditions contribute to the stability of boreholes. Dilatancy, a phenomenon first observed by Rowe,²⁰ describes the tendency of a dense matrix of particles to expand (and conversely, a loose packing to contract) upon shearing, which subsequently, contributes to increasing the mobilised shear strength of the matrix.²⁰ Nowadays, this characteristic is indispensable in the description of the hydro-mechanical response of sand.^{21–25} The main aim of this research is to investigate what the role of shear dilation is in the stability problem of boreholes drilled in saturated sand with low overpressure. To address this, the dilative behaviour of sand is evaluated in a series of consolidated drained triaxial tests. The experimental methodology and results are presented first. Subsequently, an appropriate constitutive description

of the sand is determined on the basis of the triaxial test results. A Finite Element Model (FEM) is then used to analyse a cavity contraction model to evaluate the stress and strain field surrounding a borehole when it is drilled with little to no overpressure. Without proposing a new failure criterion, we explore if the initial state of the sand and associated shear dilation can (partially) explain the mismatch between theory and practise regarding the stability of boreholes.

2. Consolidated drained triaxial tests and constitutive model

A series of consolidated drained triaxial tests have been carried out. The series comprises of conventional compression (C) and the less common, lateral extension (LE) tests.^{26,27} For the lateral extension tests, the axial stress σ_a is maintained at a constant level during shearing while the radial stress σ_r is gradually reduced. Somewhat contradictory to what the name of the test suggests, the resulting loading path is still under compression throughout but has a reducing confining stress. The choice for this less well-known test is that these loading conditions replicate better the lateral unloading of the formation when drilling a borehole.

2.1. Experimental method

2.1.1. Material

Two sands are tested: (i) a proxy sand – a clean fine quartz sand obtained from a mine, and (ii) a cored sand – acquired from a test drilling for an ATEs project. The core was obtained from the Maass-luis formation²⁸ at 128–132 m depth in Delft, the Netherlands. Fig. 2 displays the Particle Size Distribution (PSD) of both sands. Both materials with 99% of the PSD < 400 μm are classified as fine sands. The tail ends of the two PSD differ slightly, with the cored sand (B) showing a wider particle distribution. For the proxy sand (A), no fines (< 63 μm) are detected, whereas the cored sand (B), has 3% fines. The index properties of the sand are reported in Table 1. The roundness R_p and sphericity by area S_A are determined according to the method described by Zheng and Hryciw.²⁹

2.1.2. Sample preparation

The sample preparation of the two materials is almost identical. The proxy sand (A), which is stored dry, is mixed with demineralised water (10% water content) directly prior to the set up of the test. The cored sand (B), which is stored in a semi-saturated state at around 5 $^\circ\text{C}$, has

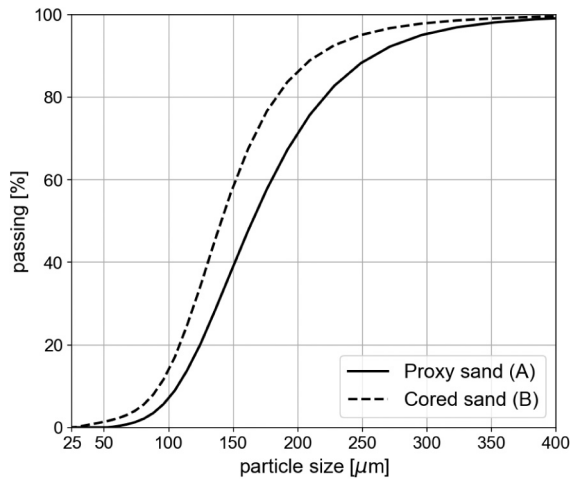


Fig. 2. Particle size distribution (PSD) of the proxy sand (A) and the cored sand (B).

Table 1

Index properties of the proxy sand (A) and cored sand (B).

Property	A	B
d_{50} [μm]	161	136
ρ_p [g/cm ³]	2.65	2.71
e_{min}/e_{max} [-]	0.59/0.99	0.62/1.07
R_p [-]	0.46	0.47
S_A [-]	0.69	0.71

an initial water content of approximately 21%. The cored sand (B) is therefore dried in the oven at 65 °C for a minimum of 72 h, then it is mixed with demineralised water to match the 10% water content of the proxy sand (A), and left in a sealed container for a minimum of 48 h, to ensure uniform moisture distribution in the sample including the fine pores from the limited fines content.

The samples are prepared in place using a three piece mould (height = 100 mm and diameter = 50 mm) using the moist tamping method,³⁰ which ensures a relatively consistent initial void ratio across the samples. Nonetheless, it should be noted that the type of preparation method can affect the soil response. The procedure is as follows: the sample is constructed in five layers of equal height. For each layer, 1/5 of the total mass is weighed and tamped until it reaches the desired height (20 mm). Before continuing with the next layer, the surface of the sample is lightly scored to allow for better continuity between the layers. The initial height h_0 and diameter d_0 of the specimens are measured using a calliper at three different locations. The resulting mean values are used to calculate the sample volume. The initial void ratio prior to testing is targeted at 0.80. This gives a relative density D_r of 47.5% and 60% for the proxy sand (A) and cored sand (B) samples, respectively.

2.1.3. Triaxial testing procedure

A total of eight consolidated drained triaxial tests have been performed: five on the proxy sand (A) and three on the cored sand (B), see Table 2. To speed up the saturation process, samples of the proxy sand (A) are initially flushed with demineralised water equal to at least one pore volume. In order to flush the sample, a pressure gradient is applied over the sample of 15–20 kPa ($p_{bottom} > p_{top}$), while maintaining a relatively high isotropic effective stress, i.e. 300 kPa by imposing a back and cell pressure of 10 kPa and 310 kPa, respectively, to ensure sample stability. For the cored sand (B), with a higher fine content, no flushing during saturation is applied, due to possible removal of the fines.

For all samples, the saturation is completed by slowly increasing the back pressure to 395 kPa and the cell pressure to 400 kPa, preserving an effective stress of 5 kPa throughout this stage. After allowing time for any remaining air bubbles to fully dissolve in the water, the saturation stage is concluded when the B-value is measured to be ≥ 0.95 . For the flushed samples, this takes around 3–4 days, whereas for the remaining cored sand (B) samples, this requires 9–15 days.

After the saturation stage, all samples are isotropically consolidated up to either 400 kPa or 1200 kPa effective consolidation stress p'_c . In some cases, the samples are partially unloaded again to achieve overconsolidated conditions (OCR = 3). While the high consolidation stress $p'_c = 1200$ kPa is more representative of the field conditions, the lower stress allows for a wider variety of loading paths well within the allowable ranges of the pressure transducers and load cell. During the shearing stage, the samples are subjected to two different drained triaxial loading paths: compression and lateral extension loading conditions. For the conventional compression tests, the radial stress is kept constant while the axial deformation rate is approximately 5%/hour. In case of the lateral extension tests, which are carried out in a stress-controlled manner, the axial stress remains constant, while the radial stress is gradually decreased at 1 kPa/min.

The triaxial tests are performed using a load frame type GDS triaxial setup.³¹ The axial deformation is measured using an external Linear Variable Differential Transformer (LVDT) sensor. Cross section area correction, assuming an equivalent cylinder, is used in the axial stress calculation.³⁰ All results have been corrected for the initial seating effect as described in Perbawa et al.³² Furthermore, it is assumed that the volume correction as a result of membrane penetration effects is negligible, due to the fineness of both sands in combination with the relatively low lateral stress.³³ Additionally, the error in the applied deviator stress is < 5%.³⁴

2.2. Experimental results

An overview of the test results is given in Table 2. The initial void ratio e_0 in Table 2 refers to the void ratio at the start of the shearing stage. Table 2 also reports the corresponding relative density D_r and state parameter Ψ at the onset of shearing, with the critical state line used to determine the initial state parameter provided in the Supplementary Material. The shear modulus G is inferred from the

Table 2

Overview of the triaxial tests carried out and corresponding results of the proxy (A) and cored (B) sand. The test name indicates the stress path: lateral extension (LE) or conventional compression (C).

Test	OCR	p'_c [kPa]	p'_0 [kPa]	e_0 [-]	D_r [%]	Ψ [-]	G [MPa]	ϕ_p [°]	ψ_{max} [°]
A-LE1	1	400	400	0.78	53	-0.012	43	37.1	11.0
A-LE3	3	400	133	0.78	53	-0.012	26	37.0	10.6
A-C3	3	400	133	0.78	53	-0.012	33	36.1	8.9
A-C1	1	400	400	0.77	56	-0.022	44	35.8	8.5
A-LE1 ¹²⁰⁰	1	1200	1200	0.77	56	0.012	71	34.9	8.2
B-LE1	1	400	400	0.79	62	-0.002	68	38.9	12.0
B-C1	1	400	400	0.77	67	-0.022	60	36.0	5.9
B-LE1 ¹²⁰⁰	1	1200	400	0.74	73	-0.018	122	36.8	8.5

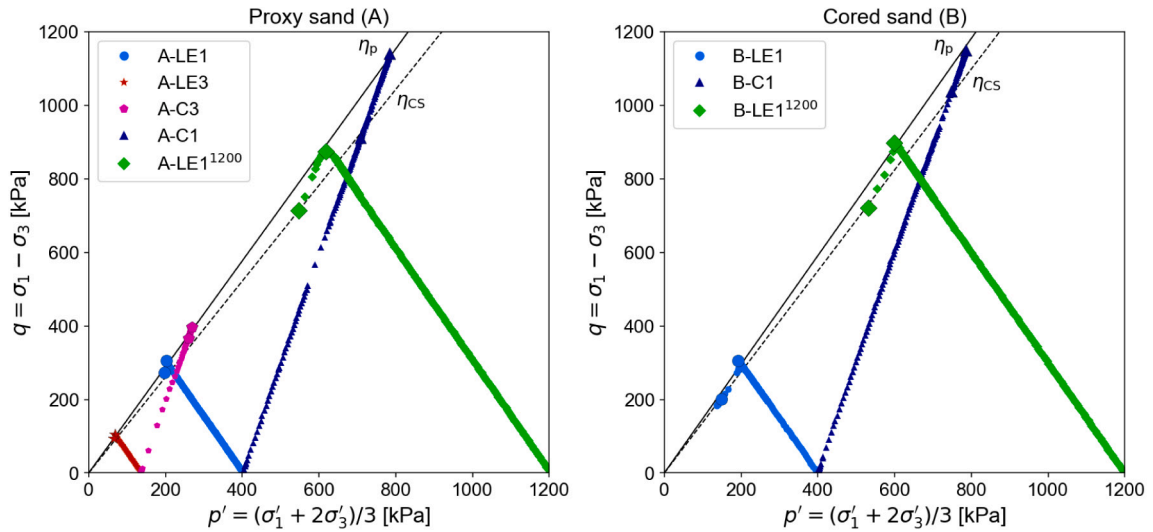


Fig. 3. Stress paths plotted in the mean p' and deviatoric q stress space for (a) the proxy sand (A); and (b) the cored sand (B). The solid and dashed lines represent the trends through the peak η_p and critical stress ratio η_{cs} , respectively.

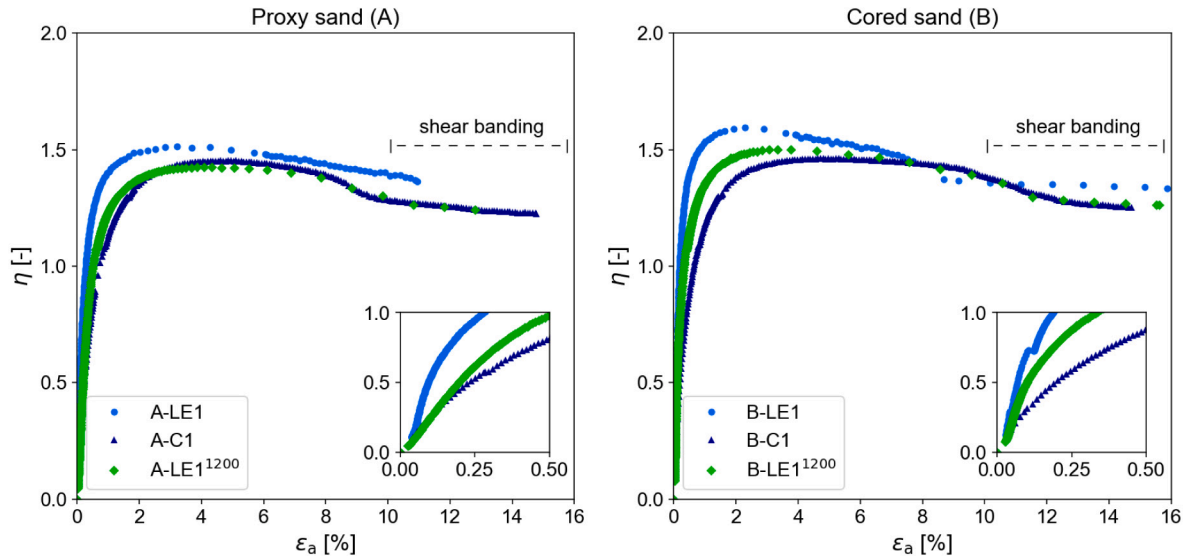


Fig. 4. The development of the stress ratio η with the axial strain ϵ_a for (a) the proxy sand (A); and (b) the cored sand (B). The small inset axes shows the evolution at small strains $\epsilon_a < 0.5\%$. Also indicated in the figure is the occurrence of shear band formation.

response at small strains. More details on the elastic parameters can also be found in the Supplementary Material.

Fig. 3 displays the stress paths in the mean effective stress p' and deviatoric stress q space from the start of shearing. The peak friction angle ϕ_p corresponding to the maximum stress ratio η_p for each test is given in Table 2. These values fall within a similar range, both across tests, as well as, between the two sands. As a result, the peak stress ratio is well described by a linear fit, shown in Fig. 3.

The development of the stress ratio η and the volumetric strain ϵ_{vol} are plotted versus the axial strain ϵ_a in Figs. 4 and 5, respectively, for the LE1, C1 and LE1¹²⁰⁰ tests, which have been performed on both the proxy sand (A) as well as the cored material (B). Based on both figures, it can be concluded that the proxy sand (A) and the cored sand (B) behave very similarly. Fig. 4 shows that the samples reach a peak stress ratio η_p of around 1.5. After reaching the peak shear stress, softening behaviour is observed in some of the tests, while others seem

to continue shearing until approximately 10% ϵ_a . This 9%–10% ϵ_a also coincides with the initiation of local deformation in the compression tests, *i.e.* shear banding. For tests A-LE1 and B-LE1¹²⁰⁰, the specimen showed bulging at 10% ϵ_a followed by the development of a shear band at higher axial strain. The measurement of volumetric deformation at large axial strain ($> 10\%$) may therefore no longer be representative of the element volume.

In Fig. 5, it is apparent that all loading paths eventually lead to dilation. The conventional compression tests show initial contraction until $\epsilon_a \approx 4\%$ followed by dilation. On the contrary, the samples subjected to the lateral extension (LE) tests display immediate dilative behaviour with an almost linear relationship to the axial strain, resulting initially in a constant dilation angle until seemingly reaching a critical state plateau. The maximum values of the dilation angle are given in Table 2. For the conventional compression (C) tests, the dilation angle ψ evolves as the axial loading progresses. Here, an asymptotic critical

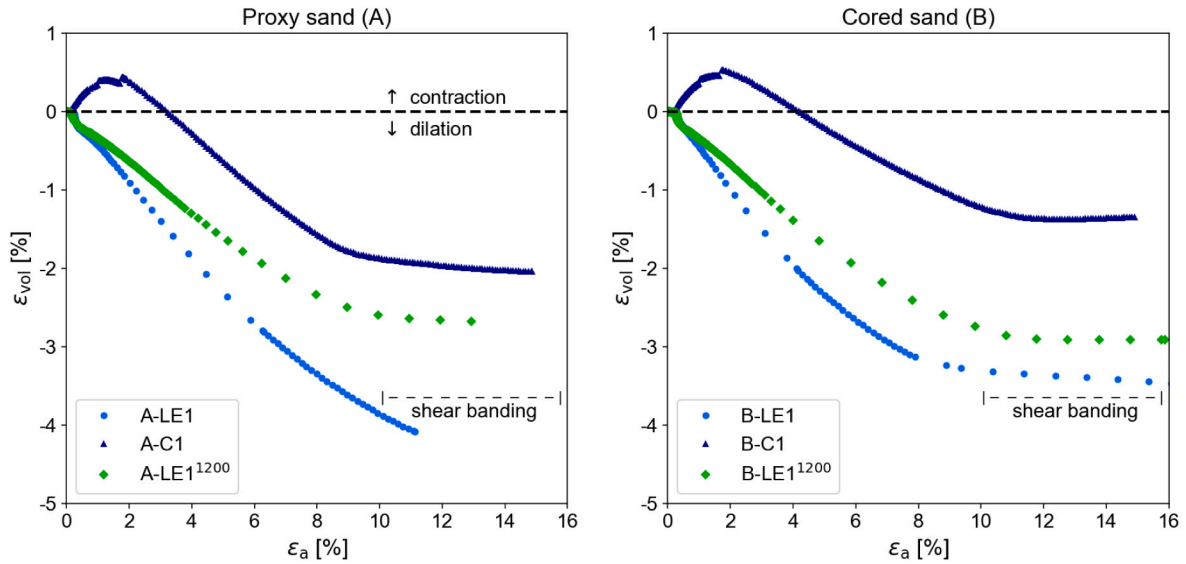


Fig. 5. The change in volumetric strain ϵ_{vol} with increasing axial strain ϵ_a for (a) the proxy sand (A); and (b) the cored sand (B). Also indicated in the figure is the occurrence of shear band formation.

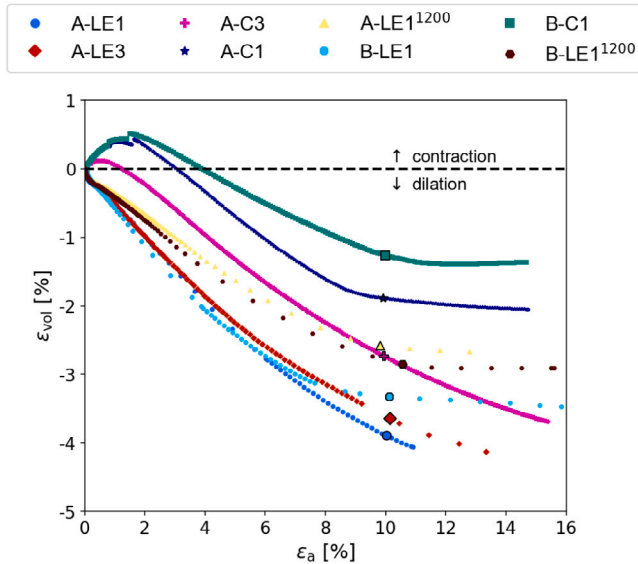


Fig. 6. The development of the volumetric strain ϵ_{vol} versus the axial strain ϵ_a for all eight triaxial tests. The assumed critical state point is indicated by the large marker.

state plateau can again be observed in some of the tests, which is roughly reached at $\epsilon_a = 10\%$.

To determine the critical state, the evolution of volumetric strain ϵ_{vol} with axial strain ϵ_a has been evaluated for all eight tests and is shown in Fig. 6. For most tests, ϵ_{vol} tends to stabilise beyond approximately $10\% \epsilon_a$, indicative of critical state conditions. For tests A-LE1, A-LE3 and A-C3, which do not exhibit a clear plateau, the critical state is taken at the same $10\% \epsilon_a$ threshold, consistent with the onset of localised deformations observed from that point onwards. The associated critical stress ratio η_{cs} is reported for both materials in Fig. 3, yielding a critical state friction angle $\phi_{cs}^A = 32.4^\circ$ for the proxy sand (A) and $\phi_{cs}^B = 34.0^\circ$ for the cored sand (B).

Loading and unloading cycles were imposed in the initial phases of tests A-C1, B-C1 and B-LE1 to investigate whether the samples exhibited elastic or non-elastic behaviour (see Fig. 7). All three tests display non-recoverable axial deformation from the start of deviatoric stresses,

i.e. the samples display immediate plasticity, which for test B-LE1 is accompanied by instantaneous dilation, see Fig. 7.

2.3. Constitutive modelling calibration

The experimental results show that the volumetric behaviour of the sand evolves under continuous shearing until most specimen reach a constant volume at large axial strain. This type of behaviour is typically well captured by the critical state modelling framework. Different critical state constitutive models exist that are capable of describing the response of sand, such as NorSand,³⁵ CASM³⁶ and SANISAND.³⁷ However, another key feature emerging in the experimental observations is the early onset of irreversible deformation and absence of a purely elastic region, which the SANISAND model is best suited to capture. This feature seems to be relevant especially in the lateral extension (LE) tests. The advanced material model SANISAND is therefore calibrated for the tests presented, since it also allows for contractive and dilative behaviour dependent on the evolving sample state, and post-peak softening behaviour as observed in the triaxial tests.

SANISAND is formulated as a bounding surface plasticity model³⁸ defined in terms of a state parameter, which combines the effect of the stress level and void ratio in characterising the state of the sand.³⁹ SANISAND has been developed over the years to its current advanced state,^{37,40} which is particularly useful for, yet not limited to, capturing the cyclic behaviour of sand.⁴¹

The model has been implemented as a user-defined material model in the 3D version of the Finite Element Software (FEM) PLAXIS.⁴² This version of the model (the SANISAND-MS variant⁴³) requires the definition of 16 dimensionless parameters. The first 13 parameters are determined in a relatively straightforward manner based on monotonic triaxial tests, with this calibration procedure described in the Supplementary Material and the values presented in Table 3. The final three parameters relate to the cyclic behaviour of the material, the calibration of which requires the results of cyclic triaxial tests. The borehole stability problem does not involve any cyclic loading. However, the Plaxis implementation requires the definition of the three parameters even for a monotonic problem, and therefore the same values found for Karlsruhe sand⁴¹ have been adopted. It is noted that due to the similarity of the triaxial results of the two sands tested here, the calibration has been undertaken with both sets of results.

Fig. 8 shows the comparison between the experimental results (markers) and the numerical simulations (dashed line) performed in

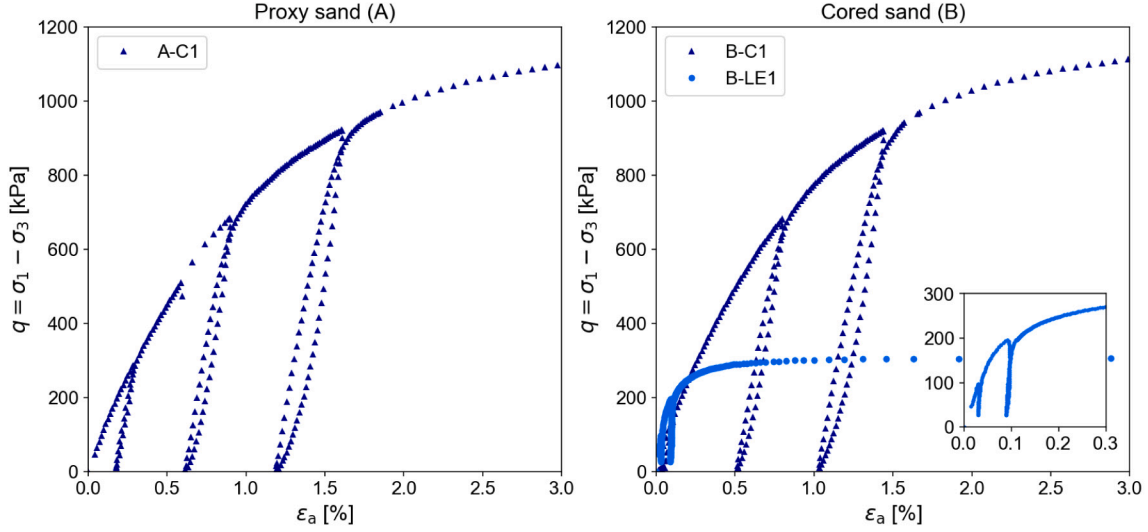


Fig. 7. The loading and unloading regimes at relatively small axial strains imposed on (a) the A-C1 sample of the proxy sand (A); and (b) the B-C1 and B-LE1 samples of the cored sand (B), where the inset axis shows the B-LE1 results at an even greater zoom.

Table 3

Calibrated (dimensionless) SANISAND parameters based on the combined experimental data of the proxy (A) and cored (B) sand.

G_0	ν	M_c	c	m	$e_{p=0}$	λ_c	ξ	n^b	n^d	A_0	c_h	h_0
105	0.27	1.34	0.7	0.01	0.82	0.01	0.73	4.3	8	1.1	1.1	15

the PLAXIS 3D Soiltest environment using the calibrated parameters for SANISAND. Here, it should be noted that the initial void ratio e_0 has been treated as an additional calibration parameter per test, given both the uncertainty in the measurement of this state variable during the experiments (± 0.01), as well as the sensitivity of the results to the parameter. The variations of e_0 used are within reasonable limits, i.e., $0.74 \leq e_0 \leq 0.77$. The overall agreement between the experiments and simulation is good for all tests. The adopted constitutive model replicates several important features observed in the experiments, such as the contractive to dilative behaviour seen in some of the tests, as well as hardening and softening behaviour. Considerable deviations occur in the prediction of the volumetric deformation at large axial strain. This is attributed to the local deformation mechanisms observed in the triaxial tests, the development of which was observed to initiate at an axial strain of about 9%–10%.

3. Cavity contraction model

A cavity contraction model is adopted here to replicate a cylindrical borehole supported by a gradually reducing overpressure p_w applied to the borehole wall, see Fig. 9. Essentially when a borehole is drilled, a soil volume is removed, causing the formation to unload. The resulting deformation and stress development is well captured by the cavity contraction model. The cavity contraction problem is the less well-known counterpart of the cavity expansion problem, which finds value in many different geotechnical applications, such as in-situ testing, pile foundations and underground excavations.⁴⁴ This simplified representation of a borehole configuration allows for the evaluation of the stress and strain field as the borehole overpressure is reduced. The aim is to investigate the role of shear dilation in the borehole stability problem.

3.1. Numerical domain and boundary conditions

The cavity contraction problem is translated into a 3D configuration using PLAXIS 3D finite element analysis software.⁴² It is assumed that

the in-situ stresses are such that $\sigma'_v > \sigma'_h$ with $K_0 = 0.5$. The user-defined SANISAND material model is only implemented in the 3D version of PLAXIS,⁴³ and thus imposes the full 3D evaluation of stresses and strains. To limit computational costs, only a quarter of the domain is simulated. We assume the formation to be fully drained, thereby evaluating the purely mechanical problem in terms of effective stresses only. Furthermore, under the assumption that there is no pressure loss over the filter cake, the effective radial stress σ'_r is equal to the borehole over pressure p_w at the borehole wall. To represent the in-situ stresses efficiently in a 3D model, a 1 m thick virtual layer with unit weight equivalent to the omitted overburden is simulated in combination with the PLAXIS 'K0-procedure'.⁴⁵ The virtual layer is described by simple linear elastic material model with elastic properties similar to the formation material. All far field boundaries are supported in the normal direction. To replicate the unloading of the borehole, the borehole overpressure on the borehole wall is gradually reduced with each load step, starting from an initial value equal to the depth-dependent in-situ horizontal stress.

The simulated borehole has a radius R of 0.3 m, which is in line with the typical diameter of a groundwater well.¹⁴ The numerical domain is $15 \times 15 \times 10$ m, where the centre corresponds to a depth of $z = 150$ m. A sensitivity analysis has shown the results to be insensitive to increasing the domain size. Assuming an overburden with an average effective unit weight of 8 kN/m^3 , the equivalent effective unit weight of the virtual layer therefore is 1160 kN/m^3 . A schematic of the numerical model is given in Fig. 9. Mesh refinement is applied with respect to the radial distance from the borehole wall by defining three zones with increasing coarseness: 1–2R m, 2R–5R m, 5R–10R m and 10R–15 m. The resulting finite element mesh consists of approximately 1.8×10^5 10-node tetrahedral elements.

3.2. Numerical results

Fig. 10 shows the results of the numerical cavity contraction simulations, with the formation described by the calibrated SANISAND material model. As the calibration procedure indicated a sensitivity to the initial state of the material, the simulations have been repeated for three different initial void ratios $e_0 = 0.6, 0.7$ and 0.8 (porosities of $n_0 = 0.38, 0.41$ and 0.44), while keeping all other parameters and boundary conditions constant. At an in-situ mean stress $p' = 800 \text{ kPa}$, the critical state void ratio for the calibrated material $e_{CSL} = 0.77$. Therefore, the initial void ratios e_0 of 0.6, 0.7, and 0.8 correspond to

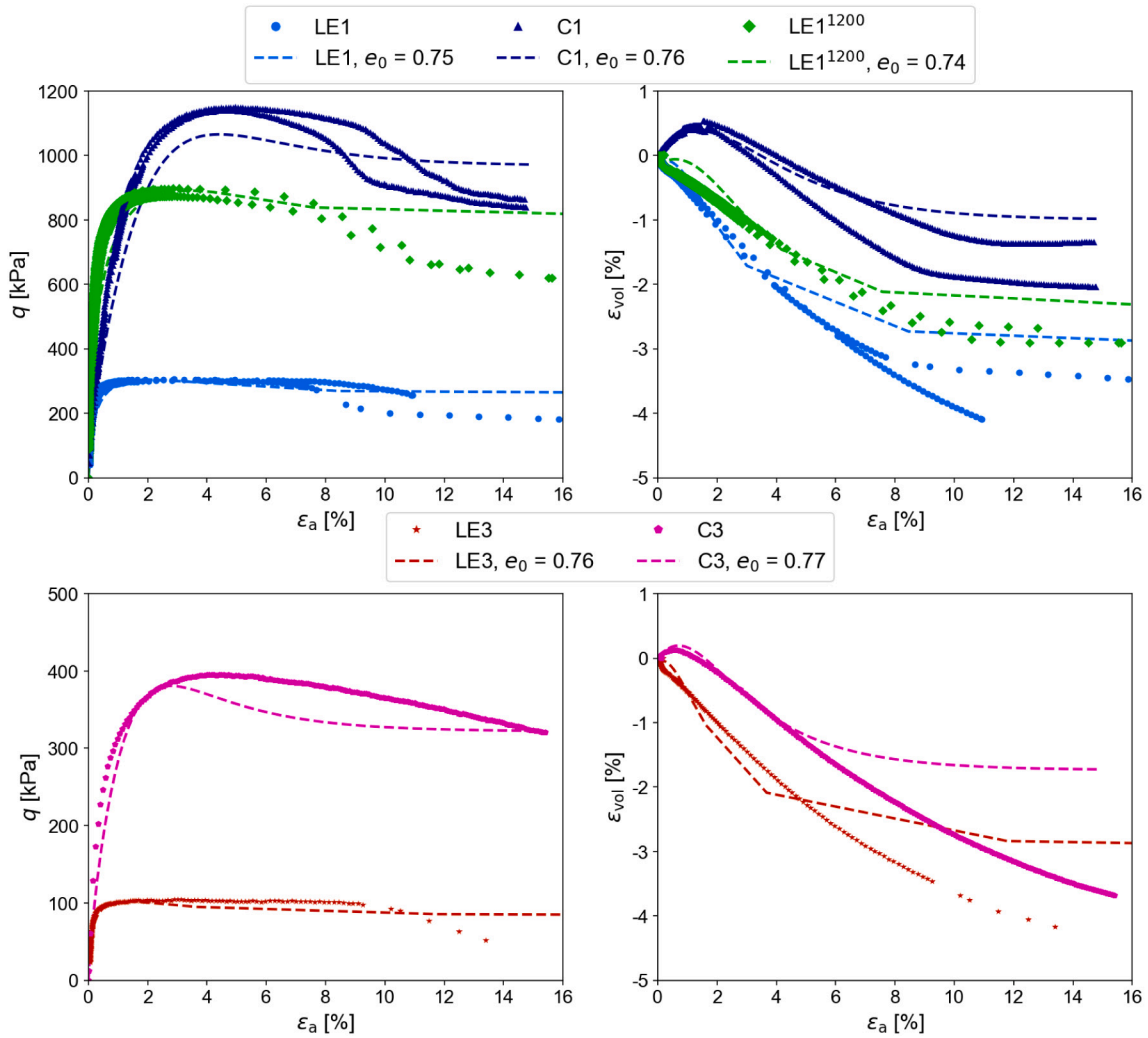


Fig. 8. The comparison between the combined experimental results (markers) and calibrated numerical simulations (dashed lines).

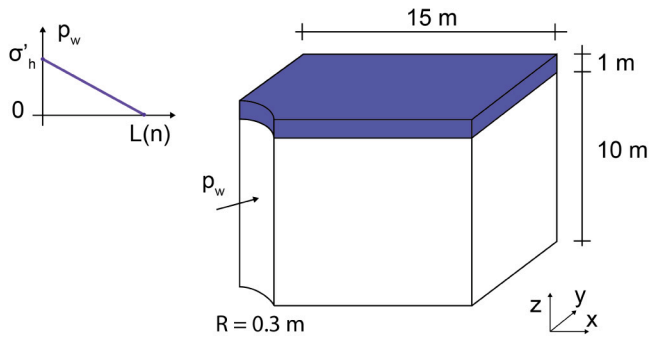


Fig. 9. A schematic of the numerical domain. The virtual layer representing the effective overburden is indicated in blue. The subfigure on the left hand side shows the applied borehole overpressure with each loading step $L(n)$.

a very dense, medium dense and loose sand, respectively. Assuming averaged values of $e_{\min} = 0.6$ and $e_{\max} = 1$ for the cored and proxy sand, the initial void ratios under consideration have approximate relative densities D_r of 100%, 75% and 50%. Relative densities ranging between 50%–75% are close to what has been tested in the triaxial tests and in line with expected field conditions. The void ratio $e_0 = 0.6$ and $D_r = 100\%$ represents an extreme condition, outside of the range of the calibrated range of initial void ratios. However, the purpose of this case

is to explore the density state limit and the associated effect of strongly enhanced dilative behaviour on the borehole stability problem.

The results are presented at the normalised radial distance r/R from the borehole wall. As mentioned previously, these simulations have been performed assuming a borehole radius $R = 0.3$ m. While the spatial evolution of the stresses and strains scales with the borehole radius, the magnitude is independent of the radius.

Fig. 10a–c show the results of these three simulations at the borehole wall, i.e. $r/R = 1$, in terms of the deviatoric stress $q = \sqrt{1/2(\sigma_r - \sigma_\theta)^2 + 1/2(\sigma_\theta - \sigma_z)^2 + 1/2(\sigma_z - \sigma_r)^2}$ against the mean effective stress $p' = (\sigma'_r + \sigma'_\theta + \sigma'_z)/3$ (Fig. 10a), the radial strain against the borehole overpressure (Fig. 10b) and the volumetric strain against the borehole overpressure (Fig. 10c). In Fig. 10d–f, the polar stresses are presented against the normalised radius r/R for $e_0 = 0.6$, with the polar stresses presented in Fig. 10g–i and 10j–l for $e_0 = 0.7$ and $e_0 = 0.8$, respectively. The vertical dashed lines in Fig. 10b and c refer to the specific loading steps which are presented in Fig. 10d–l.

In Fig. 10a, all the stress paths start from the same initial stress state ($p' = 800$ kPa, $q = 600$ kPa) and progress differently as the borehole overpressure p_w and therefore σ'_r is subsequently lowered. Due to the reduction of radial support, the surrounding formation deforms towards the centre of the borehole, causing the borehole to contract radially. This is denoted by the negative radial strain of the sand shown in Fig. 10b. The very dense sand ($e_0 = 0.6$) develops the strongest shear resistance as it dilates under the loading, resulting in an increase in effective hoop stress σ'_θ (Fig. 10e) that almost exactly counterbalances

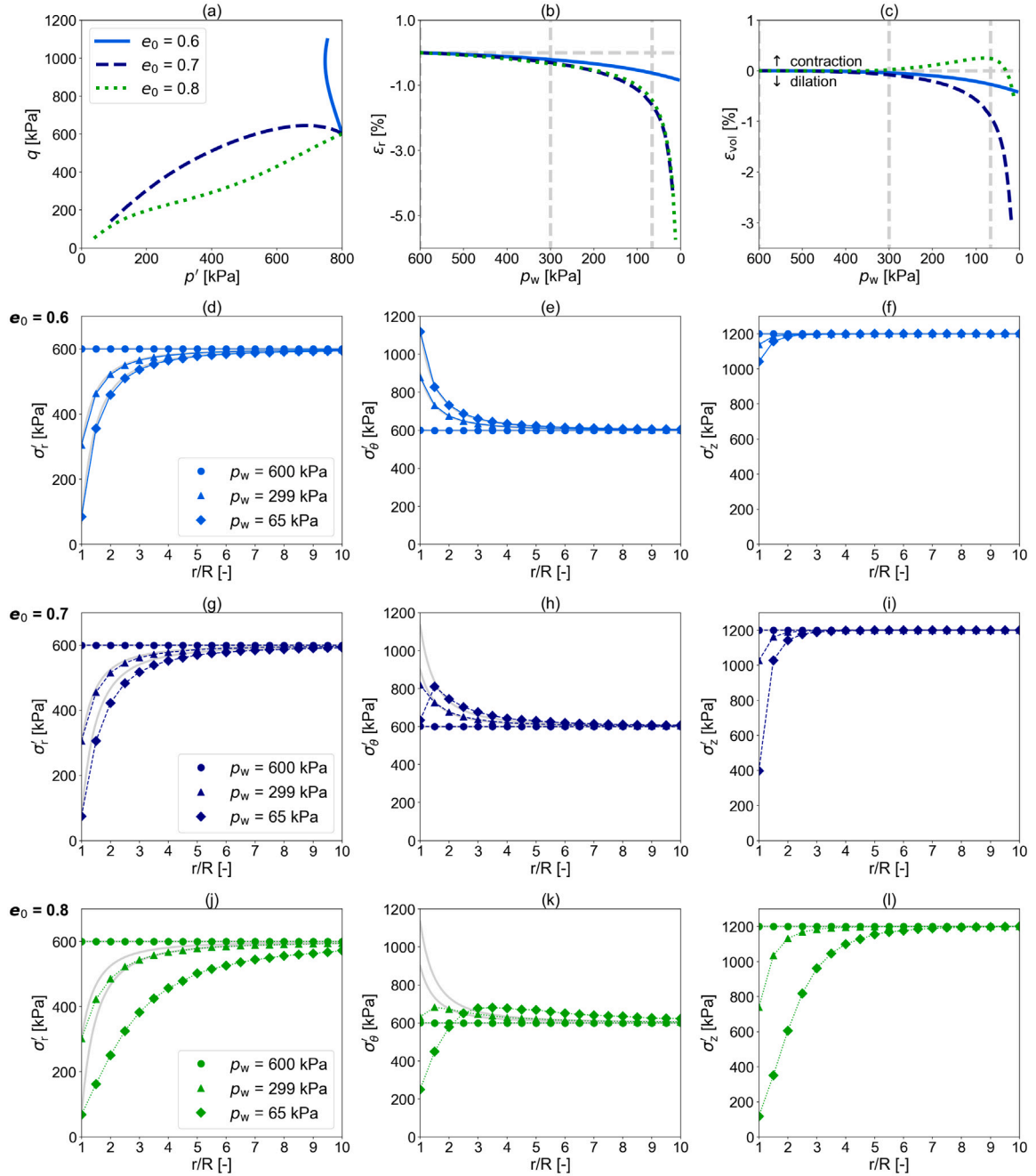


Fig. 10. The numerical results at the borehole wall $r/R = 1$ with initial void ratios $e_0 = 0.6, 0.7$ and 0.8 showing (a) the stress paths in the q - p' space and the development of (b) the total radial strain ε_r and (c) volumetric strain ε_{vol} with borehole overpressure p_w . The vertical dashed lines correspond to the loading steps at which the results for subfigures (d)–(l) are presented. The effective radial stress σ'_r versus the normalised radial distance r/R is plotted for various e_0 in subfigures (d), (g) and (j). Similarly, the effective hoop stress σ'_θ and effective vertical stress σ'_z are displayed in subfigures (e), (h) and (k), and (f), (i) and (l), respectively. In these subfigures, the grey solid lines denote the elastic solution at the corresponding levels of borehole overpressure.

the enforced decrease in σ'_r . As a result, the shear stress which the dense sand is able to sustain increases and the radial deformation is limited (Fig. 10b). The stress field very closely resembles the elastic solution, indicated by the light grey lines. Only the effective vertical stress σ'_z in Fig. 10f shows a small decrease close to the borehole wall with respect to the elastic response, which predicts a constant σ'_z equal to the in-situ vertical stress given that the horizontal stresses are equal. This indicates that some yielding has occurred causing the stresses to redistribute. The initial response of the medium dense sand ($e_0 = 0.7$) is similar to that of the dense sand, but to a more limited degree. This primary response is illustrated by an increase in shear strength with decreasing mean stress p' (Fig. 10a) and moderate deformation. As the borehole overpressure

gets close to zero, however, the medium dense sand is no longer able to sustain the increasing shear stress, causing σ'_θ and σ'_z to drop (Fig. 10h and i) and the deformation to increase.

The loose sand ($e_0 = 0.8$), on the other hand, displays a significantly different behaviour. In line with expectations, the initial response of the loose sand is to contract upon shearing, accompanied by a loss of shear strength. Due to the reduction in σ'_r not being counterbalanced by an increase in σ'_θ , the radial deformation for the loose sand is the largest. Similarly, the redistribution of the stresses causes σ'_z to reduce significantly, nearly equalling zero, see Fig. 10l. As the sand continues to contract due to the decreasing mean effective stress p' , the relative state of the sand evolves. When p_w is reduced to around 70 kPa, the

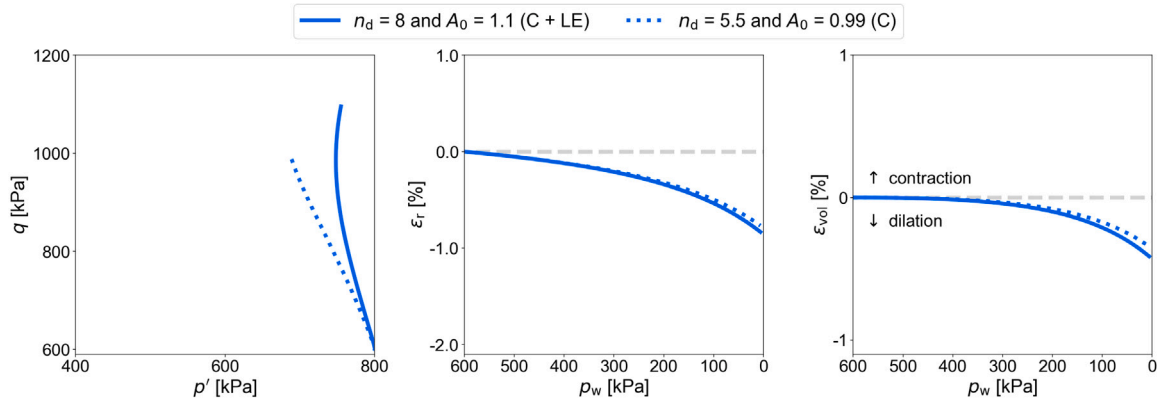


Fig. 11. The results of the numerical simulations taken at the borehole wall $r/R = 1$ with initial void ratio $e_0 = 0.6$ for two different sets of parameters determined based on the calibration of both compression and lateral extension tests (C + LE) or compression tests (C) only.

material changes from contractive to dilative behaviour, see Fig. 10c. This change in behaviour occurs because the response to the imposed decreasing mean stress p' becomes dominant over the shear response.

In Fig. 10h and k, which show the development of the hoop stress σ'_θ with respect to the radial coordinate for $e_0 = 0.7$ and $e_0 = 0.8$, it can be observed that at a certain load step the imposed reduction of σ'_r is no longer counterbalanced by an equivalent increase in σ'_θ . When $p_w = 35$ kPa, the inflection points roughly occur at $1.5 r/R$ and $5 r/R$ for the medium dense and loose sand, respectively. The inflection point marks a zone surrounding the borehole characterised by a hoop stress reduction. The extent of this zone depends on the borehole overpressure and the initial state of the sand. For the dense sand, no such zone can be identified.

In the calibration of the SANISAND material model, the parameters n_d and A_0 have been optimised such that the numerical simulations of the triaxial tests represent better the instantaneous dilation observed in the lateral extension (LE) loading paths. This calibration procedure is described in the Supplementary Material. Without the optimisation, the values of the parameters determined based on only the conventional compression tests (C) are: $n_d = 5.5$ and $A_0 = 0.99$, which are both lower than the values derived on the combination of the compression and lateral extension tests (C + LE) $n_d = 8$ and $A_0 = 1.1$. The need to include the unconventional lateral extension test to describe the sand behaviour is evaluated by comparing the simulation results using the two sets of values for n_d and A_0 , while keeping all other parameters the same. Fig. 11 shows this comparison for the very dense sand where $e_0 = 0.6$. Here, it is observed that the difference is negligible for the strains, and has limited differences in q - p' space. For the medium dense ($e_0 = 0.7$) and loose sand ($e_0 = 0.8$), which results are not shown, the differences are even smaller.

4. Discussion

In this work, we hypothesise that as the formation is laterally unloaded during drilling, the resulting dilation of the sand contributes to the stability of the borehole. If this behaviour is accounted for in the description of the material, it represents a step towards explaining the existence of stable boreholes in sand, even when they are drilled with borehole overpressures close to zero (< 40 kPa). The findings in this study demonstrate the prominent role of dilation and the state of the sand in the stability problem. This is consistent with Le et al.⁴⁶ who showed that a higher dilation angle promotes the formation of stable arches in rock caving processes.

4.1. Dilation in unloading conditions

To relate the volumetric response of the sand to the loading conditions exhibited in a borehole configuration, the lesser known lateral

extension tests have been performed. As the volumetric response of the soil matrix upon shearing depends on the material state, *i.e.* the mean stress and void ratio, care is needed in the comparison of the different loading paths. By comparing triaxial tests performed at the same initial mean stress p'_0 (see Table 2) and assuming that the differences in initial void ratio are negligible, it is seen that the lateral extension tests (A-LE1 and B-LE1) result in a dilation angle that is a few degrees higher than their conventional compression counterparts (A-C1 and B-C1). The trajectories from the initial state to the peak deviatoric stress are plotted in the q - p' and the e - p' spaces with respect to the Critical State Line (CSL) in Fig. 12a and b, respectively. Moreover, Fig. 12c shows the corresponding development of the state parameter Ψ for the different loading conditions. These figures show that the reduction in mean effective stress p' imposed by the compression in combination with the reduction of effective radial stress σ'_r in the lateral extension tests results in potentially more negative values of state parameter. This leads to earlier onset of dilation and larger peak angles of shearing resistance.

A consequence of this imposed unloading in terms of volumetric behaviour also unfolds in the numerical simulations, where the loose sand, initially displaying contraction, eventually dilates as the borehole overpressure approaches zero. Huang et al.⁴⁷ made an equivalent observation for a dense sand subjected to a cavity expansion analysis, which logically showed the opposite response: initial dilation, followed by contraction. They concluded that this nuanced response is only captured by a state-dependent material model, *e.g.* SANISAND.

While the lateral extension (LE) tests replicate better the lateral unloading of the formation when drilling a borehole, a comparison between the numerical results shows that SANISAND calibrated only using the conventional compression tests sufficiently captures the sand behaviour in unloading. For the extremely dense sand, only a small difference in behaviour can be observed when comparing the results of the two sets of parameters. This minimal difference disappears when the density decreases. In practical applications, where $D_r < 100\%$, this means that an advanced constitutive model should be used, but only conventional testing is necessary.

4.2. Application to the field

The experimental and numerical findings suggest that in order to determine the minimum required borehole overpressure p_w , a material model should incorporate a description of the sand's state, preferably one that does not assume this state to be constant. Only then is it possible to overcome the observed discrepancy between the drilling practice and common theory regarding borehole stability in unconsolidated sand. The identification of the minimum required borehole pressure taking into account the state of the sand can reduce the occurrence of borehole collapse as well as decrease the use of heavy drilling fluids

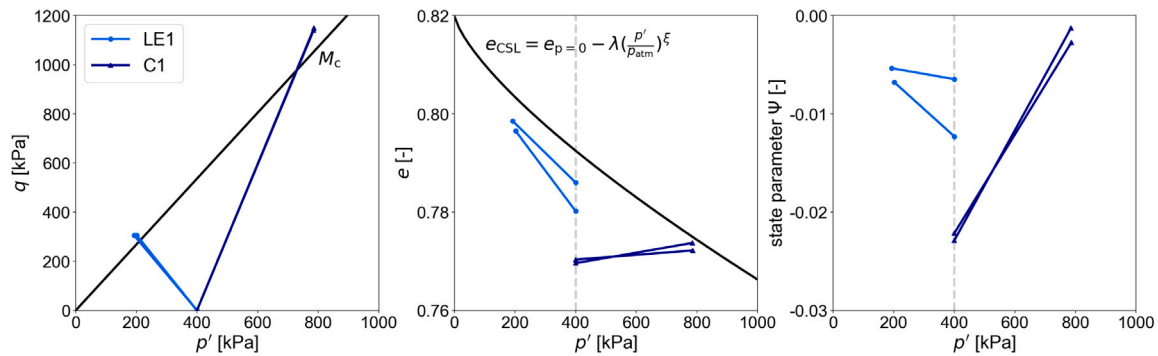


Fig. 12. The stress paths of the lateral extension (L-E1) and conventional compression (C-1) (both sheared at $p'_0 = 400$ kPa) are plotted with respect to the critical state line (CSL) in (a) the q - p' - stress space; (b) e - p' - space; and (c) in terms of the development of the state parameter Ψ .

of which the formed skin can be difficult to remove. Nonetheless, the tendency of the sand to dilate in unloading is not the only factor of relevance in a borehole stability problem. Other important factors are, for example, the borehole size,^{48,49} corresponding scale-effects and shear band formation resulting from formation heterogeneity,⁵⁰ and a potentially anisotropic horizontal stress state.⁵¹ Moreover, a certain level of cohesion provided by the presence of clay particles or layers,⁵² as well as, locally, at the borehole wall, by the filter cake^{2,3} positively contributes to the stability. Additionally, in practice, where overpressure is applied there will most likely be a pressure loss over the filter cake, which negatively affects the borehole stability problem.^{53–57}

4.3. Borehole failure

The lateral extension tests indicate plastic deformation from the start of shearing, i.e. as soon as the borehole is drilled. This is in contrast to elastic-plastic models, such as the Mohr-Coulomb model, which are often used to evaluate borehole stability in consolidated formations, with the onset of plasticity used to indicate borehole failure. The plastic deformation drives the observed dilation, which is seen to redistribute the stresses and can lead to stable conditions. The results in this paper therefore imply that the assumption of plastic deformation equalling borehole failure is too conservative for boreholes in unconsolidated formations.

The numerical results demonstrate that a zone characterised by a loss in hoop stress can develop around the borehole when the overpressure is reduced. It is seen that the occurrence, the extent of the zone and when this happens depend on the initial state of the sand. If the conservative assumption is made that no loss of hoop stress can occur, it enables the estimation of a minimum borehole overpressure as a function of the initial state of the sand. Or similarly, it can theoretically explain the existence of stable boreholes in sand drilled at $p_w \approx 0$ (< 40 kPa) provided that the sand is extremely dense. Yet, the details of how this loss in hoop stress translates to a specific type of failure mode, potentially resulting in partial or complete failure, is not yet investigated. In practice, a certain amount of caving or materials from the borehole wall falling into the borehole do not result in borehole collapse. While experimental and numerical analyses investigating the failure mechanisms in rock exist,^{49,58–60} unconsolidated sand has received little attention. An exception is the work by Hashemi et al.^{48,52} who studied borehole failure in poorly cemented granular formations. However, their analysis does not seem to account for the dilative properties and evolving state of the material. Further research is thus needed to be able to define a new failure criterion.

5. Conclusion

The common ability to create stable boreholes in unconsolidated sand drilled with very low overpressure is investigated. When a borehole is drilled, a soil volume is removed, subsequently causing the

formation to unload. These loading conditions combine an increase in shear stress while simultaneously reducing mean stress, giving rise to enhanced dilative behaviour of the sand. The magnitude of the dilative behaviour is highly dependent on the initial state of the material. The behaviour of sand is well captured by lateral extension triaxial tests, which enable an accurate calibration of the advanced material model SANISAND. While this is true, when the material model is calibrated based only on the conventional compression tests, the modelling results are very similar. Therefore, it is concluded that the unconventional lateral extension tests are not strictly necessary in describing the stability problem. Modelling using the material model reveals a zone around the borehole characterised by a loss in hoop stress, the extent of which depends on the initial state of the sand. When exploring the limits of borehole stability by evaluating extremely dense sands, this hoop stress reduction zone even disappears completely. While a new failure criterion remains yet to be defined, these findings highlight the important role of the initial state and associated shear dilation of the sand in the borehole stability problem.

CRediT authorship contribution statement

T.M. Grubben: Writing – original draft, Visualization, Validation, Software, Methodology, Investigation, Formal analysis, Conceptualization. **A. Koulidis:** Writing – review & editing, Investigation, Conceptualization. **M. Bloemendal:** Writing – review & editing, Supervision, Funding acquisition, Conceptualization. **N. Hartog:** Writing – review & editing, Supervision, Conceptualization. **S. Muraro:** Writing – review & editing, Validation, Methodology, Investigation. **P.J. Vardon:** Writing – review & editing, Validation, Supervision, Methodology, Conceptualization.

Funding

This research is funded by the European Union under grant agreement 101096566. Views and opinions expressed are however those of the author(s) only and do not necessarily reflect those of the European Union or Horizon Europe – the Framework Programme for Research and Innovation (2021–2027). Neither the European Union nor the granting authority can be held responsible for them.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgements

The authors gratefully acknowledge Cristina Caverio Panes and Luis Parra Gómez for their assistance with the triaxial tests. We also thank Mourad Chalkhane, Marien Harkes and Mandy Korff for enabling the minimum and maximum density tests at the Deltares Geotechnical Laboratory and David Taborda for initial discussions on the material model.

Appendix A. Supplementary data

Supplementary material related to this article can be found online at <https://doi.org/10.1016/j.gete.2026.100890>.

Data availability

Data will be made available on request.

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