Drilling and maintaining stable unsupported boreholes in poorly cemented sandy formations

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Thesis is submitted for the degree of Doctor of Philosophy

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-June 2015-
ABSTRACT

This thesis presents a series of journal and conference articles in which the failure behaviour of an unsupported vertical borehole drilled through poorly cemented sands is studied by analytical, numerical and experimental methods. Also, drilling field investigations were carried out to collect real samples. Three different cement contents and two borehole sizes were considered to study the effects of the bonding strength and scale-size on the particle dislocation. This study resulted in a more realistic prediction of the actual behaviour of this formation in the vicinity of a drilled borehole. Having in-depth understanding on the parameters influencing the borehole status is of significant importance in identifying borehole instability problems, designing adequate borehole supports and choosing an efficient drilling method. Due to poor cementation and therefore granular behaviour of this material, the Discrete Element Method (DEM) was identified as a well-suited tool for developing realistic numerical models. To conduct the numerical simulation, a cube of 8 m$^3$ made up of spherical particles with diameters ranging from 5 mm to 70 mm was modelled and analysed in a three-dimensional Particle Flow Code (PFC 3D). The effects of in-situ stresses around the borehole, strength of particle bonding and fluid flow pressure on the stability of the formation around the borehole have been investigated. The studies showed that when there is lack of sufficient bonding between the sand grains, the interaction between them results in their movement towards the borehole opening and thus eventuates the collapse of the borehole wall. Furthermore, the presence of high pressure water flow expedites the process of the borehole collapse.

To study the behaviour of poorly cemented sands thick-walled hollow cylinder (TWHC) and solid cylindrical synthetic specimens were designed and prepared in the laboratory. The effects of different parameters such as stress path, water and cement content, grain size distribution and mixture curing time on the characteristics of the samples were studied to identify the mixture closely resembling the formation at the drilling site. The Hoek cell was modified to allow the real-time visual monitoring of the grain debonding and borehole breakout processes during the laboratory tests. The results showed the significance of real-time visual monitoring in determining and better understanding the onset of the borehole breakout. The study on the size-scale effect revealed that with the increase in the borehole size the ductility of the specimen decreased, however the axial and lateral stiffness of the TWHC specimen remained unchanged. Under different confining pressures the lateral strain at the borehole breakout initiation point was considerably lower in a larger size borehole (20 mm) versus a smaller size one (10 mm).

Three well-known failure criterion domains; the Coulomb, Drucker-Prager and Mogi, were considered versus the laboratory test data from the TWHC tests to evaluate their ability to predict the shear failure of a borehole. The obtained results showed the significance of selecting an appropriate failure domain for predicting the shear failure behaviour of poorly cemented sands near the borehole wall. The results also showed that the Coulomb criterion is not well suited for predicting the borehole failure when there is no pressure acting inside the borehole. A failure envelope based on the Mogi domain was developed which can be used for the case of the far-field stress states. The introduced failure envelope allows predicting the stability of a borehole drilled in poorly cemented sands. The results from the video recording of the tests showed that a narrow localized zone
develops in the direction of the horizontal stress, where the stress concentration causes a full breakout in the specimens. In the TWHC specimens the dilation occurred at lower confining pressures and contracting behaviour was observed during the onset of shear bands at higher pressures. Scanning electron microscopy (SEM) studies showed that sand particles stayed intact under the applied stresses and micro- and macrocracks developed along their boundaries. The SEM imaging was used to investigate and characterize pre-existing microcracks on the borehole wall developed due to the specimen preparation. It showed that boring the solid specimen in order to produce a TWHC specimen can generate microcracks on the borehole wall prior to testing which affects the process of borehole failure development during the test. Detecting the bonding breakage point and introducing an appropriate failure criterion plays a key role in the borehole stability analysis. The total potential and dissipative absorbed strain energy per volume of material up to the point of the observed particle debonding was calculated. The results showed that the particle bonding breakage point at the borehole wall was reached both before and after the peak strength of the TWHC specimens depending on the stress path and cement content. Test results showed that the stress path has a significant effect on the onset of the particle bonding breakage. Also, it was shown that for different stress paths there is a near linear relationship between the absorbed energy and the normal effective mean stress.
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STATEMENT OF ORIGINALITY

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Conference Paper


ACKNOWLEDGEMENTS

I would like to gratefully acknowledge various individuals who accompanied me in my journey in recent years as I was working on my PhD research. Their endless support and encouragement have been invaluable in many ways.

I owe an enormous debt of gratitude to my wife, Bita. Through the struggles and trials of this thesis she has been a constant source of kindness.

I would like to thank my principal supervisor and lovely friend, Dr. Noune Melkoumian, who has kindly encouraged and supported me through the ups and downs of the PhD study. Her enthusiasm, encouragement and faith in me throughout this journey have been extremely helpful.

I would like to express my many thanks to Dr Abbas Taheri for his ideas and guidance whenever there was a problem in the research and Prof. Mark Jaksa for his support and precious suggestions.

I am very thankful to laboratory staff, Mr. Simon Golding, Mr. Adam Ryntjes and Mr. Ian Cates, for their assistance with the experimental work. Special thanks are given to Mr. Simon Golding for his support and cooperation, and experiment configurations.

The financial supports from the Deep Exploration Technologies CRC and the University of Adelaide are greatly appreciated.

Finally, to my parents, I am eternally grateful for their love, support, motivation and encouragement.
INTRODUCTION

Borehole exploration is one of the most frequently used geotechnical data collection methods in mining, petroleum and civil engineering, and the greatest yield of geotechnical data is obtained when both the borehole walls and the cores are investigated. Drilling a borehole is an excellent method not only for monitoring and petrographic assessment of borehole walls but also for measuring the spatial position of formations and for obtaining fissure characteristics. A common feature of all openings in the ground, ranging from small diameter boreholes to large excavations, is that the release of the pre-existing stresses upon creation of the opening causes increase in the tangential stress and decrease in the radial stress near the opening wall thus resulting in the elastic deformation of the formation in the vicinity of the opening at the very least. Several borehole instability problems during and after the completion of drilling a borehole have been reported by drilling companies in Australia. If the stresses around the opening are high enough they can cause inelastic deformation of the rock, which usually involves fracturing of the rock and an associated reduction in its integrity and load-bearing capability. The key to maximizing the economic value of these boreholes is in the ability to predict if, when, and how the borehole will fail and to determine and select the optimal method for controlling it. A valid prediction of the borehole closure can help in predicting many other problems, such as issues associated with pipes getting stuck during the drilling operations. Borehole stability problems can be considered from two main aspects; borehole stability must be guaranteed during the drilling operations, and a stabilized borehole is necessary if the borehole is to be used for further field investigations. To address these aspects of borehole stability a better understanding on the behaviour of poorly cemented sandy formation is required.

This thesis focuses on the borehole stability analysis in unconsolidated formations which mainly comprise of poorly cemented sands at up to 200 m underground. Borehole stability analysis is one of the main challenges in producing sustainable energies and in drilling boreholes for exploration purposes. These formations have a high failure potential and may collapse due to stress concentration or increasing tangential stress around the excavated borehole.
The first chapter of the thesis presents the fundamentals of the discrete element method (DEM) which has been developed for borehole instability investigation in high-porosity granular rocks under confining hydrogeological condition. For this purpose a series of numerical models have been simulated in the DEM using the three-dimensional Particle Flow Code (PFC 3D). Initially, the process of the sand bonding breakage was simulated by the DEM to investigate the effect of cementation on the behaviour of this weak formation while drilling exploration boreholes for discovering new potential mines in South Australia. To conduct the numerical studies a cube of 8 m³ made up of spherical particles with diameters ranging from 5 mm to 70 mm was constructed and analysed in the PFC 3D. It is a discontinuum code used in the analysis of the granular materials where the interaction of discrete grains is considered. In the considered model a cylindrical opening with the diameter of 0.3 m runs along the central vertical axis of the cube simulating the presence of a borehole. The stresses applied to the cube simulate the underground conditions around an exploration borehole at the depth of 80 m. The effects of the in-situ stresses around the borehole, strength of particle bonding and fluid flow pressure on the stability of the formation around the borehole have been investigated.

The second chapter presents a series of newly designed laboratory tests conducted on thick-walled hollow cylinders and solid cylindrical specimens, and involving real-time monitoring of the development of borehole failure. Specimens were designed to fit into a HQ Hoek triaxial cell of 63.5 mm diameter and 127 mm length. Various cement contents and fine to coarse sand weight ratios (δ) were considered to achieve the grading and strength most closely resembling formation properties at the drilling site. The whole process of laboratory studies (including designing and manufacturing of the Hoek cell modifications and specimen preparation facilities, specimen preparation and conducting the tests) was labour intensive and time consuming, and took more than 14 months to complete. It aims to provide a more accurate representation of the actual behaviour of poorly cemented sands, which is a key aspect in designing appropriate borehole support systems. The tests were conducted on specimens of poorly cemented sands prepared in the laboratory and the effects of different mixture characteristics (i.e. proportion of sand, cement and water) on their mechanical behaviour were studied by conducting compression tests on solid and hollow cylindrical specimens.

In the third chapter the principal stresses at the borehole wall have been calculated for an unsupported borehole, and the relationships between the given failure criterion parameters
and the principal stresses have been derived according to the theory of elasticity for different failure criteria. Three well-known failure criterion domains; the Coulomb, Drucker-Prager and Mogi, were considered versus the laboratory data from the hollow cylinder tests to investigate their ability to predict the shear failure of a borehole. The normal faulting stress condition, i.e. $\sigma_z \geq \sigma_\theta \geq \sigma_r$, was experimentally studied on the TWHC specimens to identify the failure criterion which best describes the shear failure behaviour of the specimens. The values obtained for $\tau_{oct}$, $\sigma_{oct}$ and $\sigma_{m,2}$ based on the data from the laboratory tests on the TWHC specimens were introduced into the Coulomb, Drucker-Prager and Mogi failure domains. Then, after a series of analytical calculations and using the results of experiments, a new failure criterion was introduced which can be widely used for predicting the failure of weakly bonded sandy formations. This method allows predicting the possibility of borehole failure in case of drilling a borehole in poorly cemented formations and thus, it will reduce the drilling costs for mining and drilling companies.

Servo-controlled loading systems with a triaxial Hoek cell were used for the experimental study. The apparatus were equipped with a micro camera to observe the sand debonding from the internal walls of the sample. To identify the failure criterion which best describes the shear failure behaviour of the specimens a possible stress path conditions, i.e. $\sigma_z \geq \sigma_\theta \geq \sigma_r$, was considered in laboratory tests on the TWHC specimens. The suitability of the three well-known failure criteria, i.e. the Coulomb, Drucker-Prager and Mogi domains for describing the compressive failure of the borehole under general stress states and drilled through poorly cemented sandy formation was assessed. A linear elastic constitutive model based on the stresses around a borehole, which was previously derived by Kirsch, has been considered for both the Drucker-Prager and Mogi failure domains. The octahedral shear stress ($\tau_{oct}$), octahedral normal stress ($\sigma_{oct}$) and effective mean normal stress ($\sigma_{m,2}$) were calculated based on two different approaches; (1) the TWHC specimen was considered as a single element, and (2) the stresses of an element on the borehole wall near the top platen of the specimen was calculated based on the applied stresses.

In chapter four the effects of compaction and shear zones on the borehole failure were investigated through a series of micromechanical studies on the localised breakout areas. The micromechanical behaviour of the specimens when inducing a localised zone on the borehole wall under different stress conditions was studied using the scanning electron microscope (SEM) imaging method. This study aims to provide a more realistic and comprehensive view
on the behaviour of poorly cemented sands and can be helpful when designing an adequate supporting system to keep the borehole open during the service period. Applying higher stresses than a certain threshold for the considered TWHC specimens led to the breakage of grain bondings within a narrow localised band which was normal to the maximum principal stress.

Chapter five focuses mainly on identifying the point of bonding breakage at the borehole wall and introducing a particle debonding criterion based on the total absorbed strain energy at the borehole wall. In order to determine the point of first particle bonding breakage, which leads to the particle dislodgement from the borehole wall the status of the borehole wall was visually monitored by a real-time camera and recorded. The total potential and dissipative absorbed strain energy (modulus of toughness) was derived for different stress paths and cementation strengths and a new borehole failure criterion was introduced. This criterion is more precise than the previously suggested criteria based on the maximum strength of the TWHC specimens, because the onset of the borehole failure may take place either prior or after the peak strength of the specimen is reached. The results from this study give a more realistic insight into the actual failure behaviour of poorly cemented granular formations, and will help to design an enhanced support system to avoid borehole collapse both during drilling and after its completion. Also, the effect of different stress regimes on the failure of boreholes in poorly cemented sandy formations was investigated and is presented in chapter five. Various stress paths were designed based both on the far-filed and an element on the borehole wall and the results were compared. It was found, that for any stress path the effect of the supporting stress on $\varepsilon_1$ was more significant for smaller borehole sizes. Also, a new failure quadrilateral was determined based on the considered stress paths for poorly cemented sands.

Chapter six of this thesis consists of the concluding remarks of this research as well as suggestions for future research.
CHAPTER 1

Background

Chapter 1 presents the first manuscript ‘Investigation of borehole stability in poorly cemented granular formations by discrete element method’ which provides a background to the discrete element method (DEM) developed for simulating a borehole drilled through poorly cemented sands and under confining hydrogeological condition. In the DEM, the interaction between rigid particles which constitute the whole model, is treated as a dynamic process with states of equilibrium developing whenever the internal forces get balanced. The contact forces and displacements of a stressed system of particles are found by tracing the movements of individual particles within that assembly. The calculations performed in the DEM alternate between the applications of the Newton’s second law to particles and the force-displacement law at the particle contacts. Newton’s second law is used to determine the motion of each particle arising from the contact and the body forces acting upon it, while the force-displacement law is used to update the contact forces arising from the relative motion at each contact. A series of numerical models have been simulated in the DEM using the three dimensional Particle Flow Code (PFC 3D). To verify these codes, a problem for a tunnel which has been solved analytically and numerically by Einstein et al. (1979) has been considered. In the main numerical model a $2 \times 2 \times 2$ m cube has been developed in the PFC 3D. The top layer is clay and the bottom layer is a cemented sandy formation. Both layers have the height of 1 metre. The model was developed for the porosity of $n=0.3$ and a cubic volume of grains was generated with more than 60000 particles. Furthermore, irregular assembly of sand grains (model with arbitrary particle size) was used to achieve a more realistic numerical simulation. Various models (more than 70) have been simulated to study the effects of different parameters on the behaviour of sandy formation around a borehole, namely, the effects of the in-situ stress, fluid flow pressure due to confined aquifer and normal bonding strength between the sand particles.

List of Manuscripts

# Statement of Authorship

<table>
<thead>
<tr>
<th>Title of Paper</th>
<th>Investigation of borehole stability in poorly cemented granular formations by discrete element method</th>
</tr>
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<tbody>
<tr>
<td>Publication Status</td>
<td>✅ Published, ☐ Accepted for Publication, ☐ Submitted for Publication, ☐ Publication style</td>
</tr>
</tbody>
</table>

## Author Contributions

By signing the Statement of Authorship, each author certifies that their stated contribution to the publication is accurate and that permission is granted for the publication to be included in the candidate’s thesis.

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Performed numerical simulation by the Discrete Element Method, interpreted data, wrote manuscript and acted as corresponding author.

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**Date** 27/03/2015

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Helped in data interpretation and contribute to research

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**Date**
Investigation of Borehole Stability in Poorly Cemented Granular Formations by Discrete Element Method

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Abstract

Behaviour of poorly cemented formations in case of drilling a vertical exploration borehole will be studied to achieve an in-depth understanding of borehole stability problem. Analysis of the granular formation behaviour has a significant importance in identifying stability issues, designing adequate borehole supports and choosing an efficient drilling method. This paper presents numerical investigations on the behaviour of poorly cemented formations in the vicinity of an unsupported vertical cylindrical borehole. Due to poor cementation and therefore granular behaviour of these formations, Discrete Element Method (DEM) was identified as well suited for developing realistic models. To conduct the numerical studies a cube of 8 m\textsuperscript{3} made up of spherical particles with diameters ranging from 5 mm to 70 mm was constructed and analysed in three-dimensional Particle Flow Code (PFC 3D). It is a discontinuum code used in analysis of the granular materials where the interaction of discrete grains is considered. A cylindrical opening with the diameter of 0.3 m runs along the central vertical axis of the cube simulating the presence of a borehole. The stresses applied to the cube simulate the underground conditions around an exploration borehole at the depth of 80 m. The effects of in-situ stresses around the borehole, strength of particle bonding and fluid flow pressure on the stability of the formation around the borehole have been investigated. It has been shown that the development of in-situ stresses in the ground due to drilling a borehole results in the formation of a plastic zone around that borehole. When there is lack of sufficient bonding between the sand grains, the interaction between them results in their movement towards the borehole opening and thus eventuates the collapse of the borehole wall. Furthermore, the presence of high pressure water flow expedites the process of the borehole collapse.

Keywords: borehole failure; numerical simulation; particle bonding; fluid flow pressure; sand production
1. Introduction

Exploration borehole is one of the most frequently used geotechnical data collection methods in mining, petroleum and civil engineering and the greatest yield of geotechnical data is obtained when both the borehole walls and the cores are investigated. Drilling a borehole is an excellent method not only for monitoring and petrographic assessment of borehole walls, but also for measuring spatial position of underground layers and for obtaining fissure characteristics. Hence, borehole instability is one of the most important geomechanical problems to be addressed in mining, petroleum and geotechnical engineering.

Extensive theoretical and experimental studies have been conducted on borehole stability in different rock formations. Gough and Bell (1981) were first to realize that the orientation of consistent breakouts in a large number of vertical oil wells in the province of Alberta, Canada, coincided with the direction of the regional least horizontal principal in-situ stress, as was revealed by the four-arm diameter logs. Haimson and Herrick (1985) and Maloney and Kaiser (1985) showed a clear correlation between breakout dimensions and the in-situ stress.
magnitude. It has been observed in the laboratory conditions that borehole breakouts grow mainly through radial penetration into the rock mass without any circumferential extension (Guenot, 1979).

The typical hollow cylinder approach is not well suited for investigating the stability of a poorly cemented formation, as the stability of a weak sandstone arch is more likely to be sensitive to fluid erosion, while the hollow cylinder tests on an intact rock sample mainly focus on rupture phenomena such as compressive and tensile failure (Hall and Harrisberger, 1970; Tippieand and Kohlhass, 1973; Cleary et al., 1979). Being a soft rock, poorly cemented sandstone has high potential for resulting in borehole instability. On the other hand, currently large number of oil, gas and water reservoirs worldwide are located in formations involving geologically young and poorly consolidated or unconsolidated sands or sandstone where grains are either lightly cemented, or even unbounded (Monus et al., 1992). Main problems associated with these weak formations are borehole instability while drilling, casing failure and sand production.

Haimson and Song (1998) tested two varieties of Berea sandstone; with 17% and 22% porosity, and revealed two distinct breakout shapes directly related to the microstructures of these rocks and attributing to their differing modes of failure.

Haimson and Kovacich (2003) and Haimson and Lee (2004) have studied borehole instability in high-porosity Berea, Tablerock and Mansfield sandstones. They found that a narrow zone ahead of a fracture-like breakout tip underwent apparent localized grain debonding and compaction and when the borehole size is considerably larger, fracture-like breakout can extend to sizable distances, creating a sand production hazard. Microscopic observation by (Fischer et al. 1977) revealed that the fundamental mechanism behind the sand production is the growth of small, opening-mode and splitting cracks oriented parallel to the tangential stress, starting very close to the borehole wall and occurring deeper in the matrix with increasing stress. Nouri et al. (2006) showed that in soft rocks pure shear failure or a combined shear-spalling process seems to occur, while in strong rock formations a combination of shear and extension fractures were observed. In the case of weak rock, material was totally plastified, while in some cases when the rock material was stronger the zone was relatively intact. Papamichos (2010) has employed finite element method to simulate borehole failure in Red Wildmoor sandstone under various axial and radial loading conditions and considering the pore pressure effect. He found that the lateral failure mode
prevails when the tangential stress at the borehole is the largest compressive principal stress and the axial failure mode prevails when the axial stress at the borehole is the largest compressive principal stress. He also showed that the borehole deformation increases and the failure stress decreases with the increase in external pore pressure.

The aim of this paper is to investigate borehole instability in high-porosity sandstones under confining hydrogeological condition. For this purpose a series of numerical models have been simulated in discrete element method (DEM) using the three dimensional Particle Flow Code (PFC 3D).

2. Site investigation

The main purpose of drilling exploration boreholes is to investigate the suitability of a given site for potential mining activates. Such boreholes are being drilled throughout Australia. In some cases the exploration boreholes are being drilled through poorly cemented or weak formations as is the case for the exploration boreholes drilled by Gold Fields at Burra, in South Australia. The majority of these boreholes are 25 cm to 30 cm in diameter with lengths varying from 100 m – 250 m depending on the underground condition.

At the considered area the subsurface investigations of sediments and borehole surveys have shown that the sediment above the bedrock is heterogeneous and irregular, shallower layers of the sediment are composed of silt and fine sand, deeper layers of the sediments change to a dark grey plastic clay, and the problematic poorly cemented sandstone comes after this clayey layer (Fig. 1). Hydrogeological conditions show that the groundwater table is at the depth of 53 m and there are both confined (porous sandstone aquifer) and unconfined (near-surface aquifer) aquifers present in the region.

![Fig. 1. Geological underground vertical cross section near Burra, South Australia](image)
Air core drilling method has been used to drill exploration boreholes at this site. This is a dry drilling method, which conventionally has been used for drilling through soft ground in Australia (Fig. 2). Air core drilling and related methods use hardened steel or tungsten blades to bore a hole into a soft ground. The drill cuttings are removed by injecting pressurized air which pushes 425-550 l/s of air at 2000-2400 kPa down the hole through the annular opening between the inner tube and the drill rod. The cuttings are then conveyed to the surface up the inner tube where they pass through the sample collection system and are collected if needed. Drilling continues with the addition of rods to the top of the drill string.

When the drilling string reaches the porous sandstone layer, there is a considerable potential for the borehole to collapse. Also, occasionally the actuator is unable to restart and rotate the rod if the gap between the drilling rod and the borehole wall is completely filled with sand grains and thus the drilling rods get stuck in the boreholes. These problems are often encountered at depths of 70 m - 150 m. The company losses at least AU$ 50,000 because of each borehole collapse, that happens before the completion of sampling. Geological conditions, stress state around the borehole, strength of the poorly cemented sandstone and presence of a high speed fluid flow due to a confined aquifer at the borehole collapse zone can be identified as the main factors driving the borehole instability.

Fig. 2. Air core drilling, Drilling site (Gold Fields’ drilling site near Burra, SA)

2.1. Stress distribution:

A common feature of all openings in the ground, ranging from small diameter boreholes to large excavations, is the redistribution of pre-existing stresses upon creation of the opening. This causes increase in the tangential stress and decrease in the radial stress near the opening wall thus resulting in the elastic deformation of the formation in the vicinity of the opening at the very least Ewy and Cook (1990 a, b). The stress distribution around a circular hole in an
infinite plate in linear elastic rock (see Fig. 3) was initially introduced by Kirsch (1898). Kirsch equations can also be generalised to calculate stresses around vertical and deviated boreholes with anisotropic far-field stresses. In fact, induced tangential stress can lead to debonding of sand grains, thus creating a damaged zone around the borehole as shown in Fig. 4, if there is no adequate cementation present between the sand grains. If the stresses around the opening are high enough they can cause inelastic deformation of the area around the borehole, which usually involves sand production and associated reduction of the area’s integrity and load-bearing capability Ewy and Cook (1990 a, b). Further growth of the damaged zone or the resulting inelastic deformations can render the opening useless for its original purpose or can require considerable amount of extra effort and expenses in order to make the opening functional and safe again. If this situation is simulated in a laboratory model, the size of the failed zones may be influenced not only by the final stress state but also by stress path, strain rate and test boundary conditions (Mavko and Jizba, 1991). In deep explorations, deeper boreholes have been drilled and such failures are becoming increasingly common due to the high stresses at those depths. For those conditions failures of the borehole wall have been documented by many observers (Fischer et al. 1977). Also, it has been found that the ratio of calculated tangential stress at failure to uniaxial compressive strength decreases rapidly with increasing borehole diameter size (Fischer et al. 1977).

![Diagram](image)

**Fig. 3.** In-situ stresses on an element at a radial distance \( r \) from a drilled borehole, in polar coordinates
2.2. Fluid flow and confined aquifer effect:

Because of the conduit effect in the borehole and high hydraulic gradient around the borehole, water flows rapidly from the aquifer to the borehole. In a borehole, which is excavated in a confined aquifer, the water level will rise to hydrostatic level, which is higher than the ground water level (Fig. 1). The seepage plays two main roles in causing instability of the borehole; firstly, the seepage increases the pore pressure by $i.\gamma_w$ in comparison by that of an unconfined aquifer and thus decreases the effective stress ($\sigma'$) between the particles, and secondly, it has been observed that the flow of groundwater is accompanied with a presence of a friction force between the water and the particles. In fact, as the water flows through the soil and loses head, its energy is being transferred to particles passed, which it is moving, and this in its turn creates a drag effect on the particles (Bell, 1993). The force (per unit volume) that the particles exert on the water is:

$$f = \gamma_w \frac{\partial h}{\partial x}$$

(1)

where $\gamma_w$ is unit weight of water.

The seepage force is especially important when considering local equilibrium in a soil since some particles may become locally unstable because of a high flow rate. To test these hypotheses, numerical study by using discrete element method has been conducted.
3. Numerical modelling using DEM

Numerical modeling has been widely used to simulate soil and rock behaviour under different loading conditions (Eberhardt, 2001; Tang et al., 1998; Wange et al., 2004; Quecedo et al., 2004; Jiang and Murakami, 2012). The most commonly applied numerical methods for rock mechanics problems are continuum methods, which include finite difference method, finite element method and boundary element method, and discontinuum methods, which include discrete element method and discrete fracture network method. Because of the weak bonding and consequently granular behaviour of poorly cemented formations, it is reasonable to study them through a discontinuum model, rather than an integrated medium model which is considered in finite element and finite difference methods (O’ Sullivan, 2011; Zhu et al., 2007; Duran, 2000). The strength of the poorly cemented formation is influenced by normal and shear bonds between the particles. In this paper the effect of bonding between sand grains that includes both shear and normal components on the borehole stability has been investigated. Series of numerical models have been simulated by discrete element method (DEM) built-in software, Particle Flow Code (PFC 3D), to achieve a better understanding of the borehole stability in weak formations.

3.1. General Formulation of particle motion

Cundall (1971) introduced the DEM to analyse the rock mechanics problems. Then Cundall and Strack (1979a) applied this method to soil and granular material. In the DEM, the interaction between rigid particles which constitute the whole model is treated as a dynamic process with states of equilibrium developing whenever the internal forces get balanced. The contact forces and displacements of a stressed system of particles are found by tracing the movements of individual particles within that assembly (Itasca, 2004). The calculations performed in DEM alternate between the applications of the Newton’s second law to particles and the force-displacement law at the particle contacts. Newton’s second law is used to determine the motion of each particle arising from the contact and the body forces acting upon it, while the force-displacement law is used to update the contact forces arising from the relative motion at each contact (Itasca, 2008). The overall governing equation, which has been used for the set of particles, is similar to the standard equation for dynamic analysis in continuum finite element of finite difference:

\[ M \ddot{u} + C \dot{u} + K(u) = \Delta F \]  \hspace{1cm} (2)
Where $M$ is the mass matrix, $C$ is damping matrix, $u$ is the incremental displacement, $K$ is the global stiffness matrix (i.e. depends upon the system geometry) and $\Delta F$ is the incremental force. The particles in a DEM model are analogous to the nodes in a FEM or FDM model. However particles in a DEM model are free to rotate. For instance, a particle has six degrees of freedom in a three-dimensional DEM analysis (O’Sullivan, 2011). According to Zhu et al. (2007) the most common equation which expresses the translational dynamics equilibrium of a particle, $p$, with mass $m_p$ is:

$$m_p \ddot{u}_p = \sum_{c=1}^{N_{cp}} F_{pc}^{con} + \sum_{j=1}^{N_{ncp}} F_{p j}^{non-con} + F_p^f + F_p^g + F_p^{app}$$

(3)

where $\ddot{u}_p$ is the acceleration vector, $F_{pc}^{con}$ are defined as contact forces due to contact $c$ in case of $N_{cp}$ contacts between particle $p$ and the rest of particles and even boundaries, and $F_{p j}^{non-con}$ are the non-contact forces (e.g. capillary forces). $F_p^f$ is the fluid interaction force, $F_p^g$ is the gravitational force and $F_p^{app}$ is a specified applied force on particle $p$. Also, the moment generated at each particle contact point can be derived by the cross-product of the contact force and a vector which connect the centre of the particle to the contact point. The dynamic rotational equilibrium is:

$$I_p \frac{\partial \omega_p}{\partial t} = \sum_{i=1}^{N_{mom}} M_{pi}$$

(4)

where $\omega_p$ is the angular velocity vector and $M_{pi}$ is the moment applied by the $i$th moment and totally there are $N_{mom}$ moments transferring contacts. It worth to mention that tangential forces produce a moment; however, the normal forces only imparts a moment if their line of action does not pass through the centre of the particle.

### 3.2. Initial conditions

Since the response of the system at discrete points in time is determined based on the model state at earlier stage, specification of the initial conditions is very important in determination of the results. Bagi (2005) showed a review of different methods to generate the particles. He also showed that the macro and particle-scale responses are sensitive to the particle size distribution. In many cases the initial location of the granular particles has an essential influence on the mechanical outcome of the system. In assemblies of granular materials there is a minimum number of particles and contacts or minimum amount of density must be attained that can transmit the stress from one side of the model to the other side. In the
geotechnical laboratory the particle can be deposited by falling toward the ground under the action of gravity. In a DEM model this process can be simulated by generating particles at some height above the final analysis domain and under vertical gravity force, allowing them to fall downwards. Then the particle packing was reached to desired density and the required initial stress state (i.e. in-situ stresses). In most DEM models employing random number generation is common in specimen generation (Jiang et al., 2003). Typically, a random number generator is a complex function that takes the time as a seed. A number of random numbers will be generated to describe each particle. For instance, in a three-dimensional model, 4 numbers are required for each particle; where the values for the \( x \), \( y \), and \( z \) coordinate of the particle centre and \( r \), the particle radius. Jiang et al. (2003) suggested a simple relation to determine the number of particles required to reach a given particles size as follows:

\[
N_{ri} = \frac{P_{ri}}{r_i^d} N_p
\]  

(5)

where \( N_{ri} \) is the total number of particles with radius \( r_i \), \( P_{ri} \) is the percentage of particles with radius \( r_i \), \( d \) is the dimension and \( N_p \) is the total number of particles in the model. Also, they introduced \( P \) as:

\[
P = \sum_{i=1}^{n} \frac{P_{ri}}{r_i^d}
\]  

(6)

where \( n \) is the total number of particles. Cheung (2010) and Ferrez (2001) described the approach to generate the particle size distribution to match the physical particle size distribution. They firstly placed some large particles in the model. Then the remaining space was filled with medium sized balls and finally iteration was used to get the needed density using increasingly smaller particles to fill the voids. Another method to reach the desired density is to gradually increase the particle sizes by expanding the particles (Itasca, 2008). This approach uses multiplying all the radii values for the particles in the model by a factor \( \alpha \) as follows:

\[
\alpha = 1 + \frac{\beta}{n^\gamma}
\]  

(7)

where, \( \beta < 1 \), \( n \) is the step and will increase until the expansion phase is terminated and \( \gamma \geq 1.0 \) is an integer. In the current study \( \beta \) and \( \gamma \) are considered 0.2 and 1 respectively. Also, Potyondy and Cundall (2004) derived the \( \alpha \) value with respect to the contact configuration and desired isotropic stress (\( \Delta p \)) state as follows:
\[
\alpha = \frac{3V \Delta p}{\sum_{b=1}^{N_b} \sum_{c=1}^{N_{c,b}} R_{b,c} K_{n,c} L_c} \quad \text{(3D system)}
\] 

where \( V \) is the volume of the system, \( N_p \) is the total number of particles, \( N_{c,b} \) is the number of contacts, \( \hat{R} \) is the distance from the particle \( b \) to contact \( c \), \( K_{n,c} \) is the contact stiffness for contact \( c \) and \( L_c \) is the sum of the radii of the contacting particles.

It worth to mention that once the particle sizes have been increased in each step, a series of calculations should be applied to bring the system of particles into a state of equilibrium. Also, the program should check the density in each time-step to immediately stop the program when it reached the required density. Otherwise there is a potential to induce significant particle overlaps, resulting in very large accelerations. After achieving the desired density, a series of calculation cycles should be used until the system comes into state of equilibrium. Assessment of equilibrium state can be verified by measuring the ratio of resultant force acting on each particle (i.e. out-of-balanced force) to the particle mass. When the maximum ratio is smaller than a user-defined value, the assembly is in equilibrium. Alternatively, there is a possibility to judge the equilibrium state with monitoring the stress state and the total number of contacts in the system at which these parameters comes into constant value.

3.3. Boundary condition

In a DEM simulation the choice of boundary condition plays a key role to receive a proper response from the model. Displacement boundary condition in continuum modelling is common to restrict or define the boundary displacement and traction boundary conditions along which stresses are specified. Similarly, in a DEM model displacement and force boundary conditions can be achieved by fixing or defining the coordinates of selected particles and applying specified force to a range of particles. There are four well-known types of boundary conditions in DEM; rigid wall, periodic boundary conditions, membrane boundaries and axisymmetrical boundaries. Rigid walls are more common to simulate machinery interacting (e.g. triaxial cell) with the granular material and have been used in this study. The contact forces determined at boundary-particle contact are used to update the particle position. Thus, they are similar to displacement boundary condition used in Finite Element Method analyses. Wall movement control can be performed by specifying a wall velocity or by developing an algorithm to move the walls based on a specific criterion. For instance, a planar rigid wall may be defined by its vertices coordinate and the normal vector
describing its orientation. The normal contact forces are calculated according to the distance from the centre of the particle to the rigid wall in the direction normal to the wall. The distance between a particle centroid \((x^p, y^p, z^p)\) and a wall with the standard equation (i.e. \(ax + by + cz + d = 0\)) will be calculated (in 3D) to be:

\[
d = \frac{ax^p + by^p + cz^p + d}{\sqrt{a^2 + b^2 + c^2}}
\]  

(9)

The shear forces are calculated with respect to the relative displacement of the wall and the particle at the contact point and orthogonal to the normal contact.

3.4. Controlling the stress state in the particulate system

At the end of particle creation procedure there will most probably be a sequence of DEM cycles carried out to gain the system of particles to the required stress level. Generally, applying stress on the particles of an assembly is by gravity and/or wall movement. In this study servo-controlled rigid boundaries are used to simulate element tests. The concept of servo-controlled rigid walls is illustrated in Fig. 5. A representative volume of the specimen will be considered to measure the stress. Then if this measured internal stress, \(\sigma_{ii}^m\), differs from the user-defined stress, \(\sigma_{ii}^r\), the walls normal to the direction \(i\) will be moved gently. In other words, if the \(\sigma_{ii}^m\) exceeds \(\sigma_{ii}^r\) the walls will be moved outwards and if the \(\sigma_{ii}^m\) is less than \(\sigma_{ii}^r\) the walls are moved inwards. The contact forces along the relevant boundaries can be integrated and divided by the boundary area to measure the stress, \(\sigma_{ii}\). Therefore the wall velocity is proportional to the magnitude of stress difference:

\[
V_{i}^{wall} = \alpha|\sigma_{ii}^m - \sigma_{ii}^r|
\]  

(10)

where \(\alpha\) is a user-defined constant. \(\alpha\) value is a problem dependent coefficient and should be small enough to ensure there is no dynamic stress wave through the system. Otherwise the stress will be reduced by ceasing the wall movement and stress condition will not be maintained at the end of compression.
3.5. Time-stepping

At the start of each time-step, the set of contacts (Fig. 6) is being updated issuing from the known particle and wall positions. These walls constitute the model boundaries and they are defined as being elastic to act as a membrane. During the deformation of a granular material the particle position and forces applying on the particles (balls) evolve in each time-step. In most DEM codes (e.g. PFC 3D) time integration method is similar to central-difference approach with a time increment $\Delta t$ has been used. Thus, particle acceleration, $\ddot{u}_p^t$, at time, $t$, can be calculated as:

$$\ddot{u}_p^t = \frac{1}{\Delta t} \left( v_p^{t+\frac{\Delta t}{2}} - v_p^{t-\frac{\Delta t}{2}} \right)$$

(11)

where $v_p^{t+\frac{\Delta t}{2}}$ and $v_p^{t-\frac{\Delta t}{2}}$ are the particle velocity vectors at $t + \frac{\Delta t}{2}$ and $t - \frac{\Delta t}{2}$ respectively. Then the velocity at time $t + \frac{\Delta t}{2}$ can be given as:

$$v_p^{t+\frac{\Delta t}{2}} = v_p^{t-\frac{\Delta t}{2}} + \Delta t m_p^{-1} (F_p^t)$$

(12)

and from the above equation we can derive the updated position of particle $x_p^{t+\Delta t}$ as:

$$x_p^{t+\frac{\Delta t}{2}} = x_p^t + \Delta t \times v_p^{t+\frac{\Delta t}{2}}$$

(13)
where the vector $x$ gives the particle position in Cartesian coordinates. According to Eq. 13 the central-difference time integration method may be applied to incrementally solve this equation.

The force-displacement law is then applied to each contact to update the contact forces based on the relative motion between the two entities at the contact and the contact constitutive model. According to Fig. 7, the law of motion is applied to each particle again to update its velocity and position based on the resultant force and moment arising from the contact and the body forces acting on the particle. Also, the wall positions are updated based on the specified wall velocities (Itasca, 2008). This iteration will be continued to reach the state of equilibrium (Fig. 7).
3.6. Fluid-Particle coupled DEM

There are many applications where the fluid-particle interaction is important to consider. For instance, in some cases the variation in the total head in the fluid causes particle motion (e.g. sand production in oil reservoir sandstones or triggering slope instabilities). According to Zeinkiewicz and Taylor (2000a) when the independent solution of the response of one phase or system is impossible without simultaneous solution of the other phases the system is expresses to be “coupled”. Simulations with coupled particle and fluid flow are considered in various studies (Zhu et al., 2008; Zeinkiewicz and Taylor, 2000a; Cundall and Strack, 1979).

In general, for incompressible fluid flow the equation for momentum is derived by Navier-Stokes as:

\[ \rho g - \Delta u_f + \mu \Delta^2 v_f = \rho \frac{\partial v_f}{\partial t} \]  \hspace{1cm} (14)

where \( \rho \) is the fluid density, \( v_f \) is the fluid velocity, \( g \) is the gravitational force and \( u_f \) is the fluid pressure. In geomechanics, Eq. 14 may not be used explicitly. Instead, it is assumed that Darcy’s law in considered in most studies. Eq. 15 shows the Darcy’s law which is one-dimensional empirical formula that relates the gradient in total head in direction \( j \) to the fluid velocity in direction \( j \).

\[ v_f^j = -k h_f = -k i_j \]  \hspace{1cm} (15)

where \( k \) is the permeability, \( h \) is the total head and \( i_j \) is the hydraulic gradient in direction \( j \).

For three-dimensional flow, combining considerations of continuity with Eq. 15 and assuming homogeneity, flow is given by:

\[ K_1 \frac{\partial^2 u_f}{\partial x_1^2} + K_2 \frac{\partial^2 u_f}{\partial x_2^2} + K_3 \frac{\partial^2 u_f}{\partial x_3^2} = 0 \]  \hspace{1cm} (16)

where \( K_1 \), \( K_2 \) and \( K_3 \) are the velocities in \( x_1 \), \( x_2 \) and \( x_3 \) directions respectively. When simulating a system of particles and fluid flow interacting, the fluid flow can be modelled by numerical solution of Navier-Stokes equation and then the motion of particles can be derived by the DEM. The challenge is that due to large number of particles in a DEM model, plenty of voids with complex geometries will be created. The solution procedure for Navier-Stokes equation would need to consider some kind of mesh with a very fine discretization which can properly record the geometry of voids. Different types of forces act on a particle when it is submerged in a fluid. Zhu et al. (2007) pointed out these forces can be classified either as
hydrostatics or hydrodynamics. The hydrostatic force is the buoyancy force due to pressure gradient around a particle and hydrodynamic forces include drag force, virtual mass force and the lift force. The drag force is the dominant fluid-particle interaction mechanism and for a single isolated particle moving through the fluid, drag force is given by Kaufi et al. (2002):

\[ f_d = C_d \pi \rho_f d_p^2 \left| v_f - v_p \right| \frac{|v_f - v_p|}{8} \]  

(17)

Where \( C_d \) is a drag coefficient, \( \rho_f \) is the fluid density, \( d_p \) is the particle diameter, \( v_f \) and \( v_p \) are the fluid and particle velocities respectively. The effect of the presence of other particles which cause increasing in shear stress on the particle surface, on the drag force should be considered in fluid-particle couple systems. Therefore, a corrective function, which depends on the porosity (\( n \)), should be used to be account for effect of other adjacent particles. Tsuji et al. (1993) calculated the drag forces for an assembly of particles as:

\[ f_d = \beta \frac{v_f - v_p}{\rho_f} \]  

(18)

If the porosity is less than 0.8 then \( \beta \) is given as follows:

\[ \beta = 150 \mu \left( \frac{1-n}{n^2} \right)^2 + 1.75 \left( \frac{(1-n)\rho_f|v_p-v_f|}{n d_p} \right) \]  

(19)

If the porosity exceeds 0.8 then \( \beta \) is given (Wen and Yu, 1966):

\[ \beta = \frac{3}{4} \frac{C |v_p-v_f|\rho_f(1-n)}{d_p n^{-2.7}} \]  

(20)

where \( C \) is a coefficient which depends on the Reynolds number. However, other various equations have been developed by different researchers to calculate the drag force in a DEM model (Xu and Yu, 1997; Kafui et al., 2002; and Zhu et al., 2007).

3.7. Contact model

In a DEM model the constitutive behaviour of the material is simulated by associating a contact model with each contact. Hogue (1998) showed that there are two stages to calculate the contact forces between particles and boundaries; contact detection and contact resolution. Initially, it is essential to define an algorithm to keep track of which particles are in contact for determining the inter-particle reactions or contact forces. A contact constitutive model is then defined to calculate the contact forces from the description of the contact geometry. Sutmann (2002) pointed out the most time-consuming aspect of DEM model (90%) is related to calculate the contact forces. According to Fig. 8 contact forces are usually calculated by
introducing a number of virtual springs at the contact points. Particles are assumed completely rigid in DEM simulations. In reality, with applying compression force, particles deform at the contact point. In a DEM model this deformation is simulated by a small amount of overlap at the contact points. The equations that determine the force-deformation relationship for the connecting spring are called contact constitutive model.

In continuum mechanics there are some phenomenological models that describe a constitutive response relating stress and strain. These models can be used to relate a contact displacement, \( U \), to a contact force, \( F \). The force-displacement response in different models can be expressed analytically as follows:

\[
F = KU \quad \text{Linear elastic} \tag{21}
\]

\[
F = f(U) \quad \text{Non-linear elastic} \tag{22}
\]

\[
F = \eta \dot{U} \quad \text{Viscous model (damping, } \eta \text{)} \tag{23}
\]

![Fig.8. Schematic view of Contact model method in DEM](image)

The above mentioned models can be combined to simulate the real behaviour of the geomaterial. Although there is indefinite number of possible combinations, there are some standard models which are derived from the basic models like Linear Maxwell model, linear Kelvin model and Burger’s model. Since viscous dashpot element is included in the models, these composite models can capture a response that varies with time. The easiest way to simulate the load-deformation response in the contact normal direction is the linear elastic spring. In the PFC 3D models, for two particles \( A \) and \( B \) in contact there are two stiffnesses \( k_{p,A}^n \) and \( k_{p,B}^n \) in normal direction and \( k_{p,A}^s \) and \( k_{p,B}^s \) in shear direction respectively. Therefore, effective normal and shear stiffnesses \( (K_{contact}^n, K_{contact}^s) \) at the contact point can be written as:
According to Itasca (2008), component behaviours consist of stiffness, slip and bonding. The contact stiffness relates contact forces and relative displacements in the normal and shear directions (Fig. 6). For ball-ball contact, the unit normal, \( n_i \), that defines the contact plane is given by:

\[
n_i = \frac{x_i^B - x_i^A}{d}
\]

(25)

where \( x_i^B \) and \( x_i^A \) are the position vectors of the centres of balls \( B \) and \( A \) and \( d \) is the distance between the ball centres. The contact force vector \( F_{i \text{tot}} \) which represents the action of ball \( A \) on ball \( B \) for ball-ball contact (see Fig. 6), and the action of the ball on the wall for ball-wall contact, can be resolved into normal and shear components with respect to the contact plane as:

\[
F_{i \text{tot}} = F_{i n} + F_{i s}
\]

(26)

The normal stiffness is the secant stiffness, since it relates the total normal force to the total normal deformation (Fig. 6):

\[
F_{i n} = K_{i n} U_{i n}
\]

(27)

\[
U_n = \begin{cases} 
R^A + R^B - d & \text{(ball–ball)} \\
R^B - d & \text{(wall–ball)}
\end{cases}
\]

(28)

where \( R \) is the particle radius, \( K^n \) is the contact stiffness in the normal direction, \( U^n \) is the overlap at the contact point and \( x_i^{[c]} \) is the location of contact point (Fig. 6). Movement of the contact is accounted for by updating \( n_i \) and \( x_i^{[c]} \) every time-step. The shear stiffness is a tangent stiffness, since it relates the increment of the shear force to the increment of the shear displacement:

\[
F^s = -\min \left( |\mu F^n|, F^s(U^s, \dot{U}^s) \right) \frac{\dot{U}^s}{|\dot{U}^s|}
\]

(29)

where \( F^s(U^s, \dot{U}^s) \) is the pre-sliding shear force calculated using the contact constitutive model. \( U^s \) and \( \dot{U}^s \) parameters depict the cumulative relative deformation and relative velocity at the contact point respectively. Referring to Itasca (2008), the tangential relative velocity of particle \( A \) relative to particle \( B \) at the sliding point, \( \dot{U}_s \), is calculated by:
\[ \dot{U}_s = (v_i^B - v_i^A) t_i - \omega_z^B (x_i^C - x_i^A) - \omega_z^A (x_i^C - x_i^B) \]  

where \( t_i \) is the unit vector in the direction of contact tangential vector, \( v_i^A \) and \( v_i^B \) are the translational velocities of particle a and b in direction i, the position of particle centres are given by \( x^A \) and \( x^B \) and the contact coordinate are defined by \( x^C \).

### 3.8. Slip

Slip behaviour is provided by defining a relation between shear and normal force components, such that the two contacting particles may slip relative to one another. This relation provides no normal strength in tension, and allows slip to occur by limiting the shear force. Slip behaviour should be active, unless a contact bond is exist between particles (Itasca, 2008).

The slip behaviour is defined by the friction coefficient at the contact, \( \mu \), where \( \mu \) is taken to be the minimum friction coefficient of the two contacting entities. The criterion of no-normal strength is enforced by checking whether the overlap given by Eq. 28 is less than or equal to zero. If it is, then both the normal and shear contact forces are set to zero. The contact will be checked for slip conditions by calculating the maximum allowable shear contact force:

\[ F_{\text{max}}^s = \mu |F_n| \]  

If \( |F_i^s| > F_{\text{max}}^s \), then slip is allowed to occur during the next calculation cycle by setting the magnitude of \( F_i^s \) equal to \( F_{\text{max}}^s \) via:

\[ F_i^s \leftarrow F_i^s \left( \frac{F_{\text{max}}^s}{|F_i^s|} \right) \]  

### 3.9. Bonding models

The contact-bond glue is of a vanishingly small size that acts only at the contact point, while the parallel bond glue is of a finite size that acts over either a circular or rectangular cross-section lying between the particles. The contact bond can transmit only a force, while the parallel bond can transmit both a force and a moment. The contact bond can be envisioned as a pair of elastic springs or a point of glue with constant normal and shear stiffness acting at the contact point. A contact bond is defined by the following two parameters: normal contact bond strength, \( F_{c,n} \), and shear contact bond strength, \( F_{c,s} \) (see Fig. 9). If the magnitude of the tensile normal contact force equals or exceeds the normal contact bond strength, the bond breaks, and both the normal and shear contact forces are set to zero. If the magnitude of the shear contact force equals or exceeds the shear contact bond strength, the bond breaks, but
the contact forces are not altered, provided that the shear force does not exceed the friction limit and the normal force is compressive (Itasca, 2008).

The force-displacement behaviour relating the normal and shear components of the contact force and the relative displacement for the particle contact occurring at a point is shown in Fig. 9. At any given time, either the contact-bond or the slip behaviour is active. In Fig. 9, $F^n$ is the normal contact force, where $F^n > 0$ indicates tension; $U^n$, is the relative normal displacement, where $U^n > 0$ indicates overlap; $F^s$, is the magnitude of the total shear contact force; and $U^s$, is the magnitude of the total shear displacement measured relative to the location of the contact point when the contact bond was formed (Itasca, 2008). Generally, maximum tensile and shear stresses acting on the bond periphery are calculated via beam theory to be:

$$\sigma_{max} = -\frac{F^n}{A} + \frac{|M^s|}{J} \bar{R}$$  \hspace{1cm} (33)

$$\tau_{max} = \frac{|F^s|}{A} + \frac{|M^n|}{I} \bar{R}$$  \hspace{1cm} (34)

Where $A$, $I$, $J$ denote the area, the polar moment of inertia of the disk and the moment of inertia of the bond disk respectively. Also $\bar{R}$ and $M$ are the bond radius and moment vector in parallel bond. If the maximum tensile stress exceeds the normal strength ($\sigma_{max} \geq \sigma_n$), or the maximum shear stress exceeds the shear strength ($\tau_{max} \geq \tau_s$), then the parallel bond breaks (O’ Sullivan, 2011).

![Fig. 9. Force-deformation behaviour for contact occurring at a point (a) normal component of contact force, (b) shear component of contact force](image_url)
4. Material and methods

4.1. Borehole description

In this study to investigate the borehole stability; a numerical model for a $2 \times 2 \times 2$ cube has been developed in PFC 3D. In this numerical model the top layer is clay and the bottom layer is a sandy formation. Both layers have the height of 1 metre. Model parameters are listed in Table 1. The model was developed for the porosity of $n=0.3$ and a cubic volume of grains was generated with more than 60000 particles. Furthermore, irregular assembly of sand grains (model with arbitrary particle size) was used to achieve a more realistic numerical simulation. Six unlimited walls were defined to keep the particles together and 0.5 was assigned for lateral earth pressure coefficient ($K_0$).

<table>
<thead>
<tr>
<th>Properties</th>
<th>Clay</th>
<th>Sand</th>
<th>Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness (N/m)</td>
<td>$10^8$</td>
<td>$10^8$</td>
<td>$10^7$</td>
</tr>
<tr>
<td>Shear Stiffness (N/m)</td>
<td>$10^8$</td>
<td>$10^8$</td>
<td>-</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.3</td>
<td>0.3</td>
<td>-</td>
</tr>
<tr>
<td>Density (kg/m$^3$)</td>
<td>700</td>
<td>2000</td>
<td>-</td>
</tr>
<tr>
<td>Normal bonding (N/m$^2$)</td>
<td>$10^6$</td>
<td>$10^3$</td>
<td>-</td>
</tr>
<tr>
<td>Shear bonding (N/m$^2$)</td>
<td>$10^3$</td>
<td>$10^0$</td>
<td>-</td>
</tr>
<tr>
<td>Friction coefficient</td>
<td>0</td>
<td>0.2</td>
<td>-</td>
</tr>
</tbody>
</table>

4.2. Numerical Calculations

After applying the stresses to the boundary, a borehole of 30 cm diameter was drilled in the model. Then, model was performed a number of calculation steps which compute the movement of the balls as they approach a mechanical equilibrium state based upon the prescribed radii and stiffness parameters, and model has reached equilibrium with gravitational body forces. Codes have been developed for applying stresses in both vertical and horizontal directions. As elaborated before, a servo-control in the program to keep the stress at the required magnitude while iteration was taking place. Due to clay characteristics there is considerable cohesion present between clay particles which prevents the material from collapsing after drilling the borehole. Linear contact bonding was used to study the behaviour of layers in this model. In the first step, to study the effect of in-situ stresses on the stability of the borehole, stresses were applied to the model to simulate the depth of 80 metres underground. In the second step, the effect of confined aquifer has been investigated by
applying fluid pressure of 100 kPa. In the final stage, to evaluate the influence of normal bonding between the particles, a model with different values of the bonding ranging from 0.1 MPa - 0.17 MPa was coded.

4.3. Code validation

When using numerical simulations to investigate the borehole stability, it is important to validate the codes which are written to calculate stresses and displacements underground. To verify these codes, a problem for a tunnel which has been solved analytically and numerically by Einstein et al. (1979) has been considered. A two dimensional model was considered with a tunnel radius of 5m, width and height of 100 m and depth of 0.4 m (Fig. 10). Normal and shear stiffness were assigned to the model to control the deformation of the tunnel crown.

One of the most important parameters for particle creation in PFC 3D is porosity, because it affects the number of particles in the model. Walls were defined to keep the particles within a limited boundary. It should be noted that in this modelling the function of walls is similar to thin latex membrane in a triaxial compression strength test. In this case walls should be able to transfer the load to the model and have minimum effect in absorbing the energy. In the next stage according to the given porosity, particles were generated in different sizes from 3 mm - 70 mm. To accelerate the process of reaching the equilibrium, large size particles were considered to be far away from the tunnel. Finally, density, shear and normal bonding, and friction were assigned to the model.

Fig. 10. Tunnel model a) model dimension, b) PFC3D model
4.3.1. Parameters of the model

Table 1 presents the parameters used in PFC 3D. According to Table 1, shear and normal bonding of clay particles is ten times higher than that for the sand grains. It should be noted that the parameters used in the FLAC 3D were Young’s modulus of 48 MPa, shear modulus of 17.91 MPa and Poisson’s ratio of 0.34.

4.3.2. Simulation procedure

To simulate the underground in-situ stresses a special code was written for applying loads to the particles. The procedure of applying forces to the model is designed in terms of an induced velocity of a given magnitude acting on the walls surrounding the model. By this, the program moves the walls in an iterative loop and evaluates stresses inside the model. Once particle stresses near the wall reach the desired value, program stops moving the walls. After reaching the given horizontal and vertical stresses due to walls’ pressure, strong repulsive forces were generated between the balls and caused a few particles to escape the confines of the walls, thus taking longer to reach the state of equilibrium. To address this issue a code was added to the program to remove escaping particles.

5. Results and discussions

5.1. DEM Model verification

The deformation of the tunnel crown has been calculated. It is obvious that if the bonding between sand grains is not high enough, tunnel collapses immediately after excavation. After several trials and after modifying the normal and shear bonds, 10.01 cm displacement for the tunnel crown was achieved. Fig. 11a shows the vertical deformation of the tunnel crown which has been derived by Einstein et al. (1979) and then simulated by FLAC 3D as well. The maximum vertical displacement is 9.6 cm. Fig. 11b shows the particles’ displacement in PFC 3D model. According to Fig. 11b maximum deformation in the PFC 3D model is 10.01 cm which shows good agreement with the analytical model.
5.2. In-situ stress effect

After applying stresses in vertical and horizontal directions, magnitudes of stresses were measured to confirm that steady state of stresses in the entire model has been achieved. To simulate drilling of the borehole the particles in the borehole area were removed from the box. When the borehole was excavated, because of the redistribution of the in-situ stresses around the borehole wall, tangent stress exceeded the bonding strength between the grains of the sandy formation and borehole wall collapsed. Fig. 12 shows the process of collapse of the borehole wall. It is obvious that after 110000 time-steps from excavating, process of collapsing accelerated and after 2.31 seconds or 140000 time-steps borehole had collapsed completely. According to Fig. 12 borehole walls remained stable in the clay layer due to cohesion forces between the clay particles. In fact, the sand layer couldn’t tolerate the new stress distribution and collapsed completely due to poor cementation and low strength of grain bonding. This phenomenon is common and is accompanied by sand production in deep exploration boreholes (Ewy and Cook, 1990a, b). It worth to mention that normal and shear stiffness of the cemented granular material will be reduced after increasing stresses when they reach to plastic or nonlinear behaviour. It means that the user defines an initial value as stiffness matrices in the DEM code (i.e. Table 1). Then, these matrices will be evolved in each time-step based on the material behaviour and applied stresses. In the next time-step of the iteration code, displacement of the particles will be based on new achieved forces and stiffness. Each principal run took about one week to complete. The time effect (creep) was not considered in this study because in case of exploration boreholes there is no need to keep the borehole open for a long period of time.
5.3. Fluid flow effect

Initially to compare the horizontal displacement of sand and clay particles, a two layer model was created with properties according to Table 1. Afterwards, a fluid pressure of 100 kPa was applied to the model (Fig. 13a). After running the program the sand layer started collapsing and sand particles moved into the borehole (sand production) but the clay layer remained stable. Fig. 13b shows the displacement of particles in each layer. It shows that the clay particles have little displacement (about 5 mm) in comparison to the sandy layer particles (180 mm). This difference in movement is justified since the particle bondings are different in sand and clay layers.

To study the effect of the presence of a confined aquifer layer as discussed earlier, sand layer situated between two clayey layers has been simulated. In the first phase the model was run with zero fluid flow pressure (Fig. 14a). After 40000 time-steps or 0.028 second from the
running the program, the movement of sand particles were limited to about 2 cm. It should be noted that clay layer stayed stable during the numerical simulation. In the second phase to compare the results, water flow pressure of 100 kPa was applied to the model (Fig. 14b). Fig. 15 shows that after the same number of time-steps, sand particles’ displacement was 160 mm approximately. Also according to Fig. 14b the borehole completely collapsed due to the applied water pressure. Thus, these studies confirmed that the presence of high pressure water will accelerate the collapse of the borehole wall.

![Fig. 13.](image13.png)  
Fig. 13. a) Horizontal displacement of particles in sand and clay b) sand and clay layer

![Fig. 14.](image14.png)  
Fig. 14. a) Layers with zero fluid flow pressure b) layers with 100 KPa fluid flow pressure
5.4. Bonding effect:

Because of the uncertainty in the bonding values between the particles, it is necessary to perform sensitivity analysis on this parameter. For this, the numerical model has been run for three different normal bonding strengths ranging from $0.1 \text{ MPa}$ - $0.17 \text{ MPa}$. Horizontal displacement of particles in the sand layer has been studied for various values of normal bonding as demonstrated in Fig. 16. Two phases can be indentified in the Fig. 16; the accelerating phase (up to $10000$ time-steps) and the steady phase which starts after the first phase is completed. In the first phase, with increase in normal bonding to reach $0.17 \text{ MPa}$, displacement of particles decreases by more than $40\%$. This clearly emphasizes the importance of sand grain bonding in maintaining the stability of a borehole in poorly cemented formations.
6. Conclusions

In the current research, borehole stability in poorly cemented formations has been studied by discrete element method using PFC 3D. Various models (more than 70) have been simulated to study the effects of different parameters on the behaviour of sandy formation around a borehole, namely, the effects of the in-situ stress, fluid flow pressure due to confined aquifer and normal bonding strength between the sand particles. The following conclusions were drawn from the presented study:

1- Borehole failure mechanism which was reported by drilling site engineers of Gold Field’s company is similar to the result of current numerical model failure mechanism. In DEM, it is possible to simulate the cemented sand and capture the real behaviour of this weak formation after breaking the cementation between sand particles. Then the model allows the particles to have large displacements and rotations as independent discrete grains with respect to their density. Therefore, the results confirmed the previous studies by Catherine O’Sullivan (2011), Zhu et al. (2007) and Cundall and Strack (1979a) and showed that the DEM is highly suitable for simulating the behaviours of granular formations such as poorly cemented sandstones. This method is based on the use of an explicit numerical scheme in which the interaction between the particles is monitored contact by contact and the motion of particles is modelled particle by particle and also allows for large displacements and rotations for particles.

2- After drilling a borehole, the state of equilibrium of underground stresses will be violated and stress concentration will occur around the borehole wall. The studies have shown, that because of the poor cementation of grains in the sandy formation, the induced tangential stress around the borehole will exceed the strength of the weak formation and results in borehole instability.

3- Comparison of behaviours of the clayey layer and the sandy formation in the presence of the fluid flow has shown that after 60000 time-steps the clay particles had only 5 mm of horizontal displacement but sandy layer particles were displaced for about 180 mm. This difference in movement is justified since the strength of particle bonding is different in sand and clay layers.

4- Presence of confined aquifers applies a considerable fluid pressure to the borehole wall. Applying a fluid flow pressure of 100 kPa indicated that collapse happened immediately (0.028 s) after the borehole excavation. However, the borehole breakout without the fluid
pressure occurred after 2.31 seconds. This clearly shows the important effect the confined aquifer has on borehole stability in sandy formation.

5- Sensitivity analysis conducted to study the influence of normal bonding strength showed that with the increase of the normal bonding from 0.1 to 0.17 MPa (e.g. clay or other cohesive agents), the displacement of particles decreases for more than 40% and leads to the increase in the borehole stability. The study of the importance of the bonding between the sand grains revealed that cementation plays a key role in the stability of boreholes in poorly cemented formations.

6. Acknowledgements

This work has been supported by the Deep Exploration technologies Cooperative Research Centre whose activities are funded by the Australian Government’s Research Programme. This is DET CRC Document 2012/064.

7. References


CHAPTER 2

Background

Chapter 2 presents the second manuscript titled ‘The failure behaviour of poorly cemented sands at a borehole wall using laboratory tests’. Laboratory investigations play a key role in acquiring a better understanding on the behaviour of a granular formation under different loading conditions and are necessary for determining the parameters required for the borehole stability analyses. The unconfined compressive strength (UCS), triaxial and thick-walled hollow cylinder (TWHC) tests are the most popular laboratory experiments for this purpose. Thick-walled hollow cylinder (TWHC) test is a common approach for simulating the stress and strain states adjacent to underground excavations in order to study the failure behaviour of geomaterials under different stress paths. The specific shape and loading paths that can be applied to these specimens make them more popular than any other available experimental test for simulating the in situ stress conditions around underground openings like boreholes, wellbores and tunnels and reproducing various combinations of stress paths.

A modified Hoek cell with a fitted micro camera was designed and manufactured at the University of Adelaide. The fitted camera allowed real-time monitoring and video recording of the borehole walls and of the process of borehole failure during the tests on the TWHC specimens. The dimensions of the TWHC specimens used in these tests were 63mm × 127mm. The facilities used for the tests were synchronised with a precise system of applying confining pressure at low level stresses (maximum 6 MPa) with no leakage or intrusion. Various mixtures of sand particles, cement and water were casted into prefabricated specific cylindrical moulds to prepare suitable specimens for laboratory tests. The main TWHC tests were conducted for three different cement contents (w_c), i.e. 6%, 7% and 8%, and two borehole diameter sizes, i.e. 10 mm and 20 mm. All procedures including compaction, grading size distribution and curing time were kept unchanged for both borehole size specimens.

List of Manuscripts

Statement of Authorship

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The failure behaviour of poorly cemented sands at a borehole wall using laboratory tests

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Abstract:

In several mineral exploration drilling sites in Australia weakly consolidated formations mainly consist of sand particles that are poorly bonded by cementing agents such as clay, iron oxide cement or calcite. These formations are being encountered when drilling boreholes to the depth of up to 200 m. To study the behaviour of this material thick-walled hollow cylinder (TWHC) and solid cylindrical synthetic specimens were designed and prepared by adding Portland cement and water to sand grains. The effects of different parameters such as water and cement content, grain size distribution and mixture curing time on the characteristics of the samples were studied to identify the mixture closely resembling the formation at the drilling site. The Hoek triaxial cell was modified to allow the visual monitoring of grain debonding and borehole breakout processes during the laboratory tests. The results showed the significance of real-time visual monitoring in determining the initiation of the borehole breakout. The size-scale effect study on TWHC specimens revealed with increasing the borehole size the ductility of the specimen decreased, however the axial and lateral stiffness of the TWHC specimen remained unchanged. Under different confining pressures the lateral strain at the borehole breakout initiation point was considerably lower in a larger size borehole (20 mm) versus a smaller one (10 mm). Also, it was observed that the level of peak strength increment in TWHC specimens decreases with increasing the confining pressure.

Keywords: Borehole stability; Experimental study; Thick-walled hollow cylinder; Poorly cemented sand; Mechanical properties

1. Introduction

Borehole stability analysis is an important challenge for researchers in the field of geotechnical, mining and petroleum engineering. Several borehole instability problems during or after the completion of drilling, have been reported by a number of exploration companies in Australia. Many of these problems are reported in drilling projects in poorly cemented sand formations at depths of up to 200 m beneath the ground. The sand production problem, as it is known, has also been observed in weakly bonded sandstones where the debonding of sand grains can be triggered by fluid pressure and induced stresses leading to the failure of the sandstone at the borehole wall (Geertsma 1985; Perkins and Weingarten...
The strength of a granular material formation is generated mainly by a natural cementing agent that bonds sand grains together (Al-Awad et al. 1999). In recent decades, a number of numerical and experimental studies have been conducted on borehole stability in different rock formations. Gough and Bell (1982) showed that, in a large number of vertical oil wells, the orientation of consistent breakouts coincides with the direction of the regional least horizontal principal in situ stress. Numerical studies on poorly cemented sand formation by Hashemi et al. (2014) revealed that breakage of weak bonding between sand particles causes instability and sand grains remain intact in the case of a borehole failure. In laboratory conditions it was shown that borehole breakouts grow mainly through radial penetration into the rock mass without any circumferential extension (Lee and Haimson 1993). Haimson and Song (1993) conducted laboratory tests on two varieties of Berea sandstone, with 17% and 22% porosity, and revealed two distinct breakout patterns directly related to the microstructures of these rocks and their differing modes of failure. Microscopic observation by Hagin and Zoback (2004) revealed that the fundamental mechanism behind sand production is the growth of small, opening-mode and splitting cracks oriented parallel to the tangential stress, starting very close to the borehole wall and propagating deeper into the matrix with increasing stress. Similar borehole instabilities have also been documented by many authors (Al-Ajmi and Zimmerman 2006; Chang et al. 1997, Ewy and Cook 1990; Ewy and Cook 1990; Fischer et al. 1978).

Unconfined compressive strength (UCS) triaxial and thick-walled hollow cylinder (TWHC) tests are among the most useful laboratory experiments (Adeyanju OA 2011; Hoskins 1969; King 1912; Robertson 1955). The geometry of the TWHC allows the application of various load path combinations to simulate stress conditions around boreholes. The classical hollow cylinder approach is not well suited to investigating the stability of poorly cemented formations because, during or subsequent to these tests, the instability of the weak sandstone arch and the grain debonding process in sandy formations cannot be captured. The hollow cylinder tests on an intact rock sample focus mainly on rupture phenomena such as shear and tensile failure (Cleary et al. 1979; Hall and Harrisberger 1970; Tippieand and Kohlhaas 1973).

In this study a series of newly designed laboratory tests involving real-time monitoring of the development of breakout in an unsupported borehole was conducted. The paper aims to provide a more accurate representation of the actual behavior of poorly cemented sands, which will be invaluable in designing appropriate borehole support systems. The tests were conducted on specimens of poorly cemented sands prepared in the laboratory and the effects
of different mixture characteristics (i.e. proportion of sand, cement and water) on their mechanical behavior were studied by conducting compression tests on solid and hollow cylindrical specimens.

2. Drilling field investigation

Exploration boreholes are usually drilled to uncover potential future mine sites. In many cases drilling is undertaken through poorly cemented sand formations. Generally, the boreholes are 250 – 300 mm in diameter and 50 – 200 m in length, depending on the underground conditions in South Australia. This study focuses on solid and TWHC laboratory test specimens based on disturbed samples collected from a problematic drilling site at Burra, South Australia. At this site the sediment above bedrock is heterogeneous, with the shallower layers composed of silt and fine sand and the deeper layers transition to dark grey plastic clay. The problematic, poorly cemented sandstone underlies this clayey layer and consists of sand particles with a weak cementation due to the presence of iron dioxide, clay and calcite. Quartz grains are mostly fine and sub-angular with random orientations. The pale, yellowish-grey specimens were prepared in the laboratory so that their fabric closely resembles that of the poorly cemented sands at the Burra site.

To drill a borehole at this site, different drilling methods have been trialed to minimize the risk of borehole failure. In some cases drilling mud has been used to maintain an open borehole during drilling. However, to facilitate further investigation, some boreholes need to remain open for several months after drilling. The air core drilling and reverse circulation drilling methods were used in these cases. These are dry drilling methods and have been conventionally applied to drilling through soft ground in Australia. The drill cuttings are removed by injecting pressurized air of 425 – 550 l/s at 2,000 – 2,400 kPa down the hole through the annular opening between the inner tube and the drill rod. The cuttings are conveyed to the surface up the inner tube and pass through the sample collection system from where they are collected if needed. Drilling continues with the addition of rods to the top of the drill string. When the drilling string reaches the porous poorly cemented sand layer, the borehole may collapse should the bonding between sand particles not be strong enough to provide stability. In addition, occasionally the actuator is unable to restart and rotate the rod if the gap between the drilling rod and the borehole wall is completely filled with sand grains and thus the drilling rods jam in the borehole. According to reports from the drilling company the main factors affecting borehole instability include the low strength of poorly cemented sands which cannot sustain the existing in-situ stress after drilling, and, in few cases, fluid flow due to a confined aquifer near the borehole collapse zone. However, there are other
factors that account for borehole instability in exploration boreholes such as erosion and poor drilling practice, and these are not considered in the current study.

3. Thick-walled hollow cylinders (TWHC)
Hollow cylinder specimens were first used in early 20th century when it was the importance of adopting a realistic model was identified for an underground opening at a depth of 9.5 km that was susceptible to collapse due to high in situ stresses (Hoskins 1969). Since then, a wide range of experimental investigations involving hollow cylindrical specimens has been conducted. (Robertson 1955) studied the effect of the inner-to-outter diameter ratio on the strength of various rocks. King (1912) analysed the system of fractures that might develop during compressive testing on hollow cylinder specimens under different stress states. Bridgman (1952) performed hollow cylinder tests under different loadings. Pomeroy and Hobbs (1962) examined the strength of coal hollow cylinder specimens. Mazanti and Sowers (1966) studied the behaviour of granite hollow cylinder specimens and the effect of the intermediate principal stress ($\sigma_2$) on their strength. Ewy et al. (1988) studied the deformation and fracture development in a hard rock around a borehole using TWHC tests. These and other works involving TWHC tests show that the TWHC configuration tests are well suited for identifying and investigating both the macro and micro properties of different rock types.

The stresses developed in a hollow specimen walls due to the application of uniform stresses can be analysed using two different methods depending on the thickness of the specimen. In a TWHC specimen the wall thickness, $t$, is larger in comparison to the specimen’s inner diameter, $D_i$ (i.e. $D_i < 20t$) and the stress distribution across the specimen wall cannot be considered homogeneous nor uniform. Closed form solutions for calculating stresses and strains in TWHC specimens can be found in different texts (Jaeger et al. 2009; Obert and Duvall 1967).

4. Experimental study
4.1. Laboratory test facilities and arrangements
Laboratory test facilities that were used in the current experimental testing comprised the following components;
Specimens that were both of a reasonable diameter and of a borehole wall thickness that satisfies the TWHC theory condition (i.e. $R_i < 10t$) were used. Hence, a HQ Hoek triaxial cell of 63.5 mm diameter and 127 mm in height was utilized.
A servo-controlled axial loading system of 100 kN loading capacity with 0.1 N accuracy was used for applying vertical stress to the specimen.
Although the Hoek cell was originally designed to apply high confining pressures to hard rock specimens, the hydraulic pressure gauge was modified to allow measuring the confining pressure at very low amounts. An automatic hydraulic machine was used in conjunction with a relief valve and a pressure gauge for applying and maintaining the external confining pressure. Since the maximum confining pressure adopted in the current tests was low (= 6 MPa), a pressure gauge with an accuracy of 0.01 MPa was used.

The TWHC specimens, consisting of poorly cemented sands, cannot be retrieved from the cell after the destructive tests due to the development of a large number of macro- and micro-cracks and the debonding of sand particles. In addition, the specimen attaches to the membrane and completely crumbles when moved. Thus, it was not possible to retrieve poorly cemented sand specimens and investigate the failure mechanism of the borehole after the test. To address this issue, the triaxial cell was modified to allow simultaneous capturing of the borehole failure mechanism and the process of sand grain debonding at each time step and at different stress paths. A micro camera with a 225 pixel per inch (ppi) resolution was installed inside the hollow platen to record the process of sand debonding and borehole breakout. The micro camera was connected to a personal computer to record the borehole conditions throughout the test.

A 60-channel data acquisition system was connected to two additional personal computers for recording and storage of data.

Initially three PVC moulds with a slot on the circumference were manufactured to prepare the specimens. However, during the compaction process, the body of the mould bulged and this deformation prevented the specimens from obtaining a uniform cylindrical shape. An additional problem was observed during de-moulding. Since the specimens were very weak, deformation of the mould to release the specimen imparted damage to it. To address this issue, steel moulds were designed and manufactured for this purpose (Fig. 1(a)). The dimensions of the moulds were 127 mm in length and 62.5 mm in diameter. Two removable steel dowels with the diameters of 25 mm and 10 mm were used together with the moulds. These dowels were embedded in the mould to create the borehole in the specimens. As shown in Fig. 1a, to avoid damaging the specimen during de-moulding, the moulds comprised two half-cylinders that are joined together by two screws. In addition, the dowels were wrapped in a plastic film (Mayla plastic) and the inner surface of moulds was lubricated with grease (containing a petroleum jelly mixture and stearic acid) which would not penetrate the
mixture. The bottom platens of the moulds were coated with an anti-rust paint to diminish the effect of lubrication. It was observed that unlike the concrete, the strength of the poorly cemented sand specimens were strongly influenced by the mould conditions.

(Kongsukprasert 2003) showed that the strength of poorly cemented sand samples is a function of the density of the mixture. Since there was insufficient space between the mould’s inner wall and the internal dowel to facilitate compaction, a tool was designed and manufactured to uniformly compact the mixture, as shown in Fig. 1(b). Each specimen was compacted in three separate layers of equal thickness (42 mm). The compaction energy for each impact was maintained constant and equal to 0.35 Nm/cm\(^3\), which was calculated based on the applied force of the manufactured compactor. Before placing the next layer, the surface of the previous compacted layer was scarified to increase the interlocking between successive layers. To minimise the bedding error effect for the very top layer a collar was used allowing this layer to achieve conditions similar to those of the lower layers (see Fig. 1a). To avoid the initial setting of the cement for all specimens the compaction time was strictly maintained to between 20-30 minutes. The time of compaction began from when water was first added to the mixture and completed when the final layer was compacted.

Two sets of cylindrical platens were manufactured from hardened steel and were hardened prior to grinding and lapping. The platens were designed using the commercial finite element analysis software ABAQUS 6.11 and loading steps similar to those applied during test conditions were applied to the model (Fig. 2). According to the results of simulated platens, the strain of the platens was less than 0.01% with the application of the maximum load of 100 kN. This is far greater than the predicted strength of the specimens. Each platen was designed...
with a tapered hole, 35 mm at the top and 25 mm or 10 mm at the bottom based on the TWHC specimen hole size. In addition, a small base was fitted to the platen to fix the camera into position a small distance above the specimen. The bottom platens were fitted with a uniform cylindrical hole of 20 mm and 10 mm in diameter, which can be fitted to the bottom ram of the loading machine. These platens were also simulated in ABAQUS. Sand particles, which were de-bonded from the borehole walls during the test, were allowed to fall onto the bottom platen.

Since the specimens were weak enough to simulate the poorly cemented sand at the drilling site they were vulnerable to disturbance prior to loading. To avoid applying the weight of the triaxial cell to the specimen during the test, a wooden base was manufactured to support the weight of the triaxial cell. Thus, there was no need to apply an external pressure on the specimens to hold the triaxial set before transferring it to the loading machine.

![Platen designed using ABAQUS](image)

Fig. 2. Platen designed using ABAQUS.
4.1.1. Test procedure and setup

The cell was placed on the wooden base and the bottom hollow platen mounted by the loading machine manually ensuring that it can move freely inside the Hoek cell. A thin black membrane was used at the end of the bottom hollow platen to ensure that the LED lights of the camera are not reflected during video recording. Both ends of the specimen were then levelled by applying a thin layer of dental paste and, after it had set, lubricated with a special grease to reduce the friction between the platen and the specimen and thereby limiting stress concentration and bedding error. During the setting of the dental paste the specimen was wrapped in a plastic film to avoid excessive drying of its surface, which could result in the loosening of particles. After the dental paste had set, precise measurements of the specimen’s weight, height and diameter were taken at three different points by means of a calliper.

Measuring the dental paste deformations at the top and bottom surfaces of the specimen during the test revealed that the dental pasted could withstand, without any noticeable deformation, a force of up to 100 kN, which is far greater than the strength of the poorly cemented specimen. The results from the preliminary tests on the specimens with no capping were not reproducible due to the significant inconsistency in the results.

The top hollow platen was positioned on the specimen and two spherical platens were placed on top of it to ensure that the vertical stress was uniformly applied to the specimen. Pairs of axial and lateral strain gauges were used to measure local deformations on the specimen. Two linear-variable differential transformers (LVDTs) were installed between the top and bottom rams of the loading machine to measure axial displacement externally. Prior to commencing the test, the upper machine ram was brought to the edge of the top platen to set the offsets and 5 N was applied to ensure contact between the top ram and the hollow platen. The captured image of the micro camera was checked to ensure that the focal length of the lens was on the middle of the specimen hole and the position of the LEDs was controlled to ensure the borehole illumination was suitable for recording. In the first stage of loading the vertical and confining stresses were increased simultaneously up to a certain stress level, which simulates the hydrostatic condition on the specimen boundary. Then, in the second stage, the sample was subjected to vertical compression at a constant displacement rate of 0.07 mm/min. The effects of various strain rates (0.02 – 0.1 mm/min) were also examined. No significant change was observed in the strength and strain behaviour of the specimens within this stain range. Data were recorded at 0.5 s time intervals.

As has been reported by various drilling companies, time is a key factor in predicting the borehole stability after drilling or when withdrawing the drilling rods from the borehole to
change the drilling bit. The video capture software was synchronised with the data acquisition system to facilitate the observation of the sand particle dislocation process in parallel with the recording of stress and strain measurements.

Several sets of synthetic mixtures of poorly cemented sands were prepared and tested with different cement-to-sand ratios (by weight), \( w_c \), coarse-to-fine sand particle ratios, \( \delta \), and grain size ranges.

4.1.2. Grain size distribution

To determine typical grain size distributions, samples were collected from the depth of up to 100 m at the drilling site in Burra, South Australia, and sieve analyses were performed using ASTM C-136 calibrated sieves plus pan. The particle size distribution was found to be almost uniform fine-grained. Based on these sieve analysis results natural silica sands (99.6% of silica) of two different grain size ranges, closely resembling the ones at the drilling site, were selected for preparing the synthetic mixtures in the laboratory. For these sand grains the mean diameter \( (D_{50}) \) for particle sizes between 0.425 – 1.4 mm (termed ‘coarse’) was 0.56 mm and, for grain sizes between 0.125 – 0.355 mm (termed ‘fine’), was 0.20 mm. The coefficient of uniformity \( (C_u = D_{60}/D_{10}) \) for the coarse and fine sand grains was 1.452 and 2.268 respectively. The sands were sieved and packed into plastic bags, with about 3 kilograms in each, to preserve the natural moisture content of the sands. The density of the fine and coarse sand particles was 1.47 g/cm\(^3\) and 1.59 g/cm\(^3\) respectively.

4.1.3. Water content

According to Gueguen and Palciauskas (1992), water content should be kept to a minimum to avoid segregation of the cement and sand grains. Different water contents were considered to create the most suitable mixture. To determine the optimum water content for the mixture standard Proctor compaction tests were conducted. These tests were performed on mixtures both with and without cement. The optimum water ratio evaluated by using compactor hammer with energy of 0.55 Nm/cm\(^3\) was achieved at 9.7 – 10.3%. There was no significant change in optimum water content and density in the case of the sand and cement mixture or for the sand grains only.

4.1.4. Cement content

Portland cement type II (specific gravity, \( G_s = 3.15 \, g/cm^3 \)) was used for preparing the TWHC specimens based on previous studies on the mechanical properties of weakly cemented sands (Saidi et al. 2003). The cement powder used in the current study was from a single bag and it was kept in a sealed and airtight container throughout the laboratory studies.
For preparing poorly cemented sands in laboratory conditions a wide range of $w_c$ values have been suggested by different researchers (Alsayed 1996; Kongsukprasert 2003; Saidi et al. 2003; Saidi et al. 2005). Kongsukprasert (2003) used a maximum of 2.5% of Portland cement, whereas (Saidi et al. 2003) used 9 – 18%. In addition, Gueguen and Palciauskas (1992) stated that the minimum $w_c$ is reached at $\delta = 1.5$. Saidi et al. (2005) showed that a very small amount of Portland cement will increase the strength and stiffness of cemented-sand mixtures if deposited at grain-to-grain contacts. However, based on the authors’ observations, having used sands of the above mentioned grain size distribution, for $w_c \leq 2.5\%$, the specimens could not be successfully de-moulded after the curing time (2 - 12 days). A number of mixtures with different $w_c$ and $\delta$ were prepared and examined to achieve the apparent mechanical behaviour of the drilling site samples. For the mentioned particle size distribution range, values of $w_c = 6\%$, 7% and 8% were selected. According to the test results, since the grain size distribution significantly affects the strength of the specimens, it is not possible to create poorly cemented sand samples by using a fixed $w_c$ value for different particle size distributions.

4.1.5. Coarse-to-fine sand particle ratio ($\delta$)

The cement content, $w_c$, and the coarse-to-fine sand particle ratio, $\delta$, were varied to obtain the desirable mixture. In this study, since the cement powder particle is finer than the sand grains, the cured Portland cement is assumed to be a continuous phase despite the fact that it includes micro-granular and micro-porosity (Saidi et al. 2005). The specimen contains three separate phases sand, cement and macro-porosity larger than the sand grain size (Ashby and Jones 2014). Scanning electron microscope photographs confirmed that macro-porosities were often larger in size than the sand particles themselves (Fig. 3a). According to (Saidi et al. 2005), minimum macro-porosity is achieved when $\delta$ is around 1-3. Also, based on the collected samples from the drilling site, the cured mixture should be weak enough to allow sand particles to debond when the surface is scratched with a finger nail. Therefore, different values for $\delta$ were examined for a constant $w_c$ to determine the most suitable $\delta$. Finally, $\delta = 1$ was chosen based on the test outcomes which are discussed later.
4.1.6. Curing time

Various curing times have been suggested in previous studies (e.g. Kongsukprasert 2003; Saidi et al. 2003). In the current research, ranges between 2 – 12 days were examined. The curing time includes the curing of the mixture, both in the mould and after de-moulding. In the case of less than 5 days curing the specimens could not be successfully de-moulded. Preliminary tests were performed on TWHC specimens with total 5 days curing time. Careful examination of the video taken during testing and of the cross-sections of the tested specimens (Fig. 3b) revealed that all boreholes had failed in a zone that seemed to have retained some moisture and hadn’t yet completely dried. After further studies, and based on the uniaxial compressive strength (UCS) test results, 5-day curing was deemed to be the optimal time for curing inside the mould when compacted at atmospheric pressure. After removal from the mould, each specimen was wrapped in plastic film, placed in an airtight plastic container and left to further cure for another 3 days under atmospheric pressure and at a constant water content. To ensure that 26 – 29% porosity was achieved no external pressure was applied to the specimens whilst curing.

5. Results and discussion

5.1. UCS tests on solid specimens

In order to determine the properties of the mixture, various preliminary tests were conducted on solid and TWHC specimens. The results of these tests can be used to compare other drilling fields and ground conditions to those examined in the current study. Preparation of
the mixture began with \( w_c = 1\% \) using only fine-grained sands. Table 1 presents the details of the mechanical properties that were observed from the uniaxial and triaxial tests. To determine the mechanical properties of the specimens, a number of solid cylindrical specimens with different sand grain size distributions, Portland cement and water contents were prepared.

Table 1 Properties of the prepared poorly cemented sand specimens

<table>
<thead>
<tr>
<th>Mechanical properties</th>
<th>Porosity ( n )</th>
<th>Tangent elastic modulus ( E_{tan} ) (GPa)</th>
<th>Uniaxial compressive strength UCS (MPa)</th>
<th>Poisson’s ratio ( \nu )</th>
<th>Coulomb parameters ( c ) (MPa)</th>
<th>( \phi ) (°)</th>
<th>Bulk density ( \rho \left( \frac{kg}{m^3} \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_c = 6% )</td>
<td>26 ± 3%</td>
<td>2.262</td>
<td>0.62</td>
<td>0.294</td>
<td>1.11</td>
<td>29.67</td>
<td>1954</td>
</tr>
<tr>
<td>( w_c = 7% )</td>
<td>26 ± 2%</td>
<td>2.465</td>
<td>1.05</td>
<td>0.250</td>
<td>1.38</td>
<td>30.06</td>
<td>1969</td>
</tr>
<tr>
<td>( w_c = 8% )</td>
<td>26 ± 2%</td>
<td>3.360</td>
<td>1.90</td>
<td>0.247</td>
<td>1.59</td>
<td>29.67</td>
<td>1974</td>
</tr>
</tbody>
</table>

5.1.1. Effect of particle size distribution in UCS tests

As mentioned above, two sets of sands, namely fine (0.125 – 0.355 mm) and coarse (0.425 – 1.4 mm) grained sands were examined in this study. Three sets of UCS tests were performed on solid specimens with the same values of \( w_c = 8\% \) and \( w_w = 10\% \), but with different coarse-to-fine sand particle ratios \( (\delta) \) (i.e. 0.14, 0.33 and 1.0). The specimens were left to cure for 5 days at atmospheric pressure with the same curing method that was discussed earlier in section 4.1.7.

Fig. 4a shows that the influence of \( \delta \) and \( w_c \) on the peak strength and pre-peak stiffness is considerable. It is evident that, whilst maintaining \( w_c \) constant (i.e. at 8\%) the strength and stiffness of the specimens increases as the proportion of coarse sand particles (i.e. \( \delta \)) rises and the specimens become more brittle in the post-peak regime. The strength of the mixture increases significantly as \( \delta \) grows because the coarse sand particles may exhibit greater resistance to rotation in the specimen matrix than the fine particles. Therefore, when the number of coarse grains increases, the rotation of particles becomes more difficult due to the improved level of interlocking between the particles after the compaction and the global compressive strength of the mixture is increases. In the case of the higher \( \delta \) the other reason behind the mixture strength growth is the decrease in the special surface of sand particles in the whole matrix which enhances the influence of cementation in the specimen. Therefore, by
increasing the sand grain size it is possible to obtain a higher strength mixture for a certain $w_c$ value. This confirms the findings by other researchers (Kongsukprasert 2003; Saidi et al. 2003; Saidi et al. 2005) who used different ranges of $w_c$ to create cemented-sand specimens. Also, the level of compaction improved with the increase of the $\delta$ up to a certain level and thus the cementing agent could suture more sand particles in a unit volume.

It should be noted that increasing $\delta$ by more than 1.0 was not considered in this research, because the aim was to test particle sizes similar to those observed at the Burra drilling site.

**Fig. 4.** Results of UCS tests: (a) Effect of different $\delta$ (coarse/fine) values on the strength of poorly cemented sand specimens (b) Effect of different $w_c$ values on the stress-strain diagrams and (c) on the volumetric strain versus axial strain.

### 5.1.2. Effect of cement content in UCS tests

To study the effect of cement content ($w_c$), three different values of $w_c$ were considered, namely 5%, 6% and 8% under the same testing conditions. To examine reproducibility of the
results, specimens with 6% and 8% of cement content were tested twice. Fig. 4b, c presents the stress-strain diagram of the specimens after 7 days curing and with a dry density of 1.81 g/cm$^3$.

From the UCS test results, the following trends can be observed;

The maximum strength, $\sigma_{\text{max}}$, and pre-maximum stiffness increase as $w_c$ rises. This outcome is in agreement with Saidi et al. (2003) and Kongsukprasert et al. (2005) who used other ranges for $w_c$ and $\delta$.

The post-peak stress-strain trend of the specimens with lower values of $w_c$ exhibits greater ductile behaviour. As shown in Fig. 4a and Fig. 4b, the UCS is affected to a greater extent by $w_c$ than $\delta$.

The effect of $w_c$ on the volumetric strain is presented in Fig. 4c. It can be observed that as $w_c$ increases the specimens experience greater axial and volumetric strain prior to failure. In addition, for lower $w_c$ values, dilation is evident in the specimens after a limited amount of compression. However, with increasing cementation the lateral strain decreases and the volumetric strain is affected mostly by the axial compression strain.

Furthermore, the effect of water required for cement hydration was considered by increasing the water content in small increments within the range of 5% to 12%, covering the dry, optimum, and wet sections of the optimum water diagram. Kongsukprasert et al. (2005) showed that, for the cemented-sand mixture, the added water cannot be entirely used for cement hydration. The total water content of the mixture comprises of a part that serves to hydrate the cement and another part that is absorbed by the grains at the time when water is added to the mixture. In addition, Chen and Wu (2013) suggested that the degree of hydration increases as the curing time and water-to-cement ratio of the mixture rise. However, they showed that excessive water content increases the total porosity and results in strength reduction. When the sand grains are not in the saturated-surface dry (SSD) mode, part of the water content will be absorbed by the grains, while the remaining part covers the particles’ surfaces. Specimens with less than 6% water content could not be de-moulded after even 10 and 12 days of curing time, which inferred that, due to the high specific surface of the sand particles, a minimum of 6% water content was required to achieve hydration. The results showed that, for a specific density of a compacted mixture, the highest strength is obtained with the water content at or around the optimum, and the strength significantly reduced when the water content was less than the optimum value. As mentioned earlier, the
optimum water content was determined by standard compaction tests and was found to be almost 10%.

5.1.3. Young’s modulus

The Young’s modulus, \( E \), of the specimens was determined from the UCS tests. Mogi (1967) showed that in porous rocks the elastic modulus decreases with the increase in the strain because of the formation of micro-cracks.

Saidi et al. (2005) performed a series of UCS tests on synthetic cemented sand specimens and showed that unloading-reloading cycles will not coincide with the main stress-strain curves even during the initial stages of deformation. This behaviour implies that the material shows plastic behaviour even at very low stresses in the UCS test.

The tangent Young’s modulus of the solid specimens has been plotted against normalised deviator stress (\( q/q_f \)) in Fig. 5. It can be seen that continuous degradation of stiffness during shearing, which implies an intrinsic non-linear behaviour for the mixture. Guyer et al. (1997) showed that non-linear elasticity is an intrinsic characteristic of granular rocks. Also, near the peak stress, \( E \) reached its minimum value, which suggests that damage accumulates in the specimen. Previous studies have also shown that for poorly cemented sands there is a good agreement between the static and dynamic Young’s modulus (Hilbert Jr et al. 1994).

Lateral strain gauges were used to measure the elastic modulus in the lateral direction perpendicular to the loading axis. Fig. 5b illustrates the Young’s modulus of the solid specimens versus \( q/q_f \) in the lateral direction. It shows that the solid poorly cemented specimens behave non-linearly even in the early stages of loading. This is consistent with the findings of Hilbert Jr et al. (1994) and Saidi et al. (2003) where static and dynamic Young’s modulus was considered.
5.2. Triaxial test results on solid specimens

Triaxial tests on solid specimens were performed to determine the shear failure properties of the specimens under different stress conditions. As mentioned in the previous section, based on several preliminary laboratory tests on different mixtures, it was decided to perform TWHC tests on specimens with three different \( w_c \) values. Table 2 shows a summary of the final \( \delta \), \( w_c \) and \( w_w \) (water content) values which were identified by preliminary tests.

<table>
<thead>
<tr>
<th>Type of specimens</th>
<th>( w_c )</th>
<th>( \delta )</th>
<th>Borehole size (mm)</th>
<th>( \sigma_{conf} ) (MPa)</th>
<th>Displacement rate (mm/min)</th>
<th>( w_w )</th>
<th>Curing time (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid cylinder</td>
<td>6%, 7%, 8%</td>
<td>1</td>
<td>-</td>
<td>1.5</td>
<td>0.07</td>
<td>10%</td>
<td>8 days</td>
</tr>
<tr>
<td>TWHC</td>
<td>6%, 7%, 8%</td>
<td>1</td>
<td>10 &amp; 20</td>
<td>1.5</td>
<td>0.07</td>
<td>10%</td>
<td>8 days</td>
</tr>
</tbody>
</table>

Fig. 6a-c presents the results of the triaxial tests on the solid specimens with different \( w_c \) values. As shown in Fig. 6, an increase in the confining pressure results in an increase in the peak strength. However, confining pressure has a minimal effect on the stiffness of the specimens. Also, as it can be seen from Fig. 6, this material exhibits strain hardening.
behaviour, i.e. continuous increase in the deviatoric stress with axial strain. Thus, the strain corresponding to 1% was considered as the maximum strength for the specimens. Paterson (1967) showed that ductility increases with increasing $\sigma_3$. Fig. 6 shows that the ductility of the specimens at lower confining pressures is less significant and increasing the confining pressure results in the transition from brittle to ductile behaviour. According to the results, for $w_c = 7\%$ and 8\% the elastic strain remains relatively constant due to an increase in the confining pressure. Higher confining pressures (e.g. > 6 – 6.5 MPa) were examined for specimens in each of the three categories and the results showed that strain gauges on the specimens failed at these higher confining pressures. This problem often occurs due to the sandy surface and weak matrix of this material.

Fig. 6. Stress versus axial and lateral strain behaviour of solid specimens subjected to triaxial testing (a) $w_c = 6\%$ (b) $w_c = 7\%$ (c) $w_c = 8\%$. 

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5.2.1. Mohr-coulomb strength parameters

One of the most common failure criteria is the Mohr-Coulomb criterion, which has been widely used in borehole stability problems (Jaeger et al. 2009). Coulomb suggested that shear failure will take place when the shear stress is equal to the sum of cohesive shear strength and the product of the coefficient of internal friction and normal stress across the fracture plane. Later, Mohr asserted that the relationship between the normal and shear stresses along the failure plane is non-linear,

\[ \tau = f(\sigma_n) \]

where, \( f \) is a function that can be derived empirically.

To explain the behaviour of the material by the Coulomb failure criterion, the cohesion, \( c \), and the angle of internal friction, \( \phi \), can be determined. As expected, increasing the cement content produces higher cohesion between the sand grains and based on the Coulomb criterion, \( c \) (cohesion) changes for different cement values. The \( c \) and \( \phi \) values are presented in Table 1. For different \( w_c \) values the derived \( \phi \) values show that the angle of internal friction does not change dramatically due to the increase in the cement content. This is mainly due to the grain size distribution not being changed in the specimens tested. It should be mentioned that the application of different failure criteria, such as Drucker-Prager and Mogi-Coulomb (Al-Ajmi and Zimmerman 2006), on poorly cemented sands have been investigated in another study (Hashemi et al. 2014).

5.2.2. Volumetric strain results

Lateral strain was measured during the triaxial tests on solid and TWHC specimens to study the effect of confining pressure and cement content on the volumetric strain in the prepared specimens. Fig. 7 shows the average values of the volumetric strains calculated based on average values of lateral and axial strain gauges. As can be seen in this figure, at lower confining pressures (i.e. 1 and 2 MPa) the specimens begin to contract from the top and bottom sections first and after 0.55% axial compression strain, it diverts to lateral dilation. Increasing the confining pressure kept the specimen in a more contraction mode and dilation began at higher axial strain values. With an increase in the confining pressure the lateral strength of the specimens will rise and lateral dilation will be lower for the same axial strain. As mentioned earlier, after the elastic phase in the stress-strain diagram, the solid specimens exhibited strain hardening behaviour. Thus, for confining pressures of greater than 2 MPa the specimens remain in contraction mode and never conjugate the horizontal axis, (i.e. the zero
volumetric strain) until the maximum strength which was assumed equivalent to 1% of axial strain. The results also show that the effect of varying the cement content on the volumetric strain is less significant than altering the confining pressure. For instance, at confining pressures of 1 and 2 MPa, increasing the $w_c$ from 6% to 8% does not result in any divergence in the behavioural trend from contraction to dilation in the specimens.

Fig. 7. Volumetric strain versus axial strain results from triaxial tests on solid specimens (a) $w_c = 6\%$ (b) $w_c = 7\%$ (c) $w_c = 8\%$.

Fig. 6 also presents the deviator stress versus lateral strain behaviour. It shows that applying higher confining pressure slightly increases the stiffness of the material in the lateral
direction. In addition, the specimens exhibited more brittle behaviour at low confining pressures and, with increasing confining pressure, the material response became more ductile. It is worth mentioning that the specimens underwent lower lateral deformation by increasing the \( w_c \) at a constant \( w_w \), \( \delta \) and confining pressure.

5.3. Triaxial test results on TWHC specimens

Triaxial tests on TWHC specimens were conducted to investigate the borehole breakout and failure properties of the synthetic specimens under different stress conditions. Real-time video recording helped to determine the initiation and direction of borehole breakout and to locate the sand debonding on the borehole wall. The main TWHC tests were conducted for three different cement contents \((w_c)\) and for two different borehole diameters: 10 and 20 mm. Fig. 8 presents the results of the triaxial tests on 10 mm diameter borehole TWHC specimens for three different \( w_c \) values (6%, 7% and 8%) and various confining pressures. As shown in Fig. 8a, increasing the confining pressure from 1 to 3 MPa, significantly enhances the level of peak strength and strain energy in the specimens. It should be mentioned that the pre-peak stiffness does not change dramatically with an increase in the confining pressure for a certain \( w_c \). However, for \( \sigma_3 > 3 \text{ MPa} \), the level of the strength increment is lower than for the previous states. For confining pressures higher than 4.5 MPa, the first stage of the test (i.e. applying hydrostatic stress on the boundary of the specimens) could not be completed and the borehole collapsed immediately.
Fig. 8. Stress versus axial and lateral strain behaviour of 10 mm borehole TWHC specimens for (a) $w_c = 6\%$ (b) $w_c = 7\%$ (c) $w_c = 8\%$ and the effect of the confining pressure on the strength of the specimens.

Fig. 9 shows the process of the borehole failure prior to applying the deviatoric stress. However, the axial strain gauges continued to function even after failure. This emphasises the significance of using the micro camera during the borehole stability studies.

As shown in Fig. 8, unlike the solid specimen test results, the TWHC specimens do not exhibit strain hardening behaviour after the maximum strength and the peak strength can be determined in the applied stress ranges. Fig. 8 also shows that the ductility of the specimens increases with an increase in the confining pressure as is the case for the solid specimens.

Also, increasing the cement content by a small amount, otherwise the same sample preparation and test condition, slightly increase the stiffness of the specimens.
The results show that the confining pressure has a minimal effect on the stiffness as exhibited in the stress–strain curves. This result agrees with Mogi (2007) who showed that the yielding stress of ductile rocks does not increase with increasing confining pressure.

**Fig. 9.** Sand dislocation and borehole collapse processes due to the hydrostatic pressure with high confining pressure (6 MPa) prior to the application of the deviator stress on the TWHC specimen.

### 5.3.1. Effect of $w_c$ and $\sigma_{conf}$ on lateral and volumetric strain

Fig. 8 also shows the axial stress versus the average lateral strain for different values of $w_c$ and confining pressures. Analysis of the micro camera video recordings and Fig. 8 shows that, for a given $w_c$ with an increase in the confining pressure, the borehole breakout initiates at a lower lateral strain and the ductility in the direction of $\sigma_{conf}$ decreases. However, as mentioned in the previous section increasing the confining pressure results in greater ductility.

Fig. 10 shows that, at lower confining pressures (i.e. 1 and 2 MPa), the TWHC specimens transits to the dilation mode and with an increase in the confining pressure the borehole breakout initiated in a contraction mode as in solid specimens. This is due mainly to the fact that high confining pressures create borehole convergence and therefore, the size of the sample in the lateral direction reduces. Also, due to the presence of a borehole at low confining pressure, the volumetric strain curve conjugates the horizontal axis at higher axial strain value in compare with solid specimens in the same confining pressure which suggests that microcracks are developing on the borehole wall and sand grain dislocation occurs in the TWHC specimens versus solid specimens to release the applied stresses and return the
equilibrium condition to the specimen. In addition, as shown in Fig. 10a-c with an increase in $w_c$, the lateral strain increases when compared to the axial strain and prior to borehole breakout in the specimen, axial contraction of specimens with higher $w_c$ is less than for the specimens with lower cement content. In other words, an increase in the $w_c$ results in the decrease in the pore spaces of the specimens, and the application of the axial deviatoric stress causes less contraction in the axial direction. Therefore, for higher $w_c$ values the lateral strain in poorly cemented sand specimens is more dominant in comparison to the axial contraction.

Based on the real-time camera recordings it can be stated that, in the samples that failure happened in dilation, the transition from the volumetric strain contraction to the dilation occurred before the borehole breakout initiation.
Fig. 10. Volumetric strain versus axial strain measured by local strain gauges for different $w_c$ values and confining pressures for 10 mm borehole TWHC specimens.

5.3.2. Size-scale effect on TWHC specimens

As mentioned above, the TWHC specimens were prepared in two borehole sizes: 10 and 20 mm. The same process was adopted for the 20 mm borehole specimens, including grain size distribution, curing time, compaction force, and so on. Triaxial tests were performed on 20 mm borehole specimens for three different $w_c$ values and under different confining pressures. Fig. 11a,b present the deviator stress ($\sigma_1 - \sigma_3$) versus the axial strain ($\varepsilon_a$) for the TWHC specimens for two different $w_c$ values and different confining pressures. The figure illustrates the results of the specimens with 10 and 20 mm borehole sizes to compare their behaviours. As expected, the strength of the 20 mm size borehole TWHC specimens at failure is generally lower than that for the smaller borehole specimens especially at higher confining pressures due to size-scale effect which was shown by Carpinteri (2002). Also, it shows a considerable decrease in the ductility for the 20 mm borehole specimens versus that for the specimen with a 10 mm borehole. Fig. 11 shows that the stiffness of the TWHC specimens does not change with increased borehole size. This is in agreement with Mogi (2007) who suggested that the stiffness depends on the rock material and, since the same mixtures were used for both specimens with different borehole sizes, the same stiffness was observed from the tests. Fig. 11c shows the deviator stress versus lateral strains for 10 and 20 mm borehole specimens. It shows that, with increasing confining pressure, the lateral strain decreases for the same borehole size and $w_c$ values. In addition, it also shows that under different confining pressures the lateral strain at the borehole breakout initiation point is considerably lower in the 20 mm borehole specimens when compared with the 10 mm borehole specimens.
However, the lateral stiffness remains unchanged and is unaffected by the increase in borehole size from 10 to 20 mm. Also, the lateral ductility significantly decreased in the specimens of larger borehole size. In other words, the breakout in the borehole with larger diameter (20 mm) occurred at a lower strain compared to that for a smaller diameter borehole (10 mm).
6. Conclusions

This study has examined the stability of boreholes in poorly cemented sand by a series of newly developed laboratory tests. It was observed that the strength of the poorly cemented sand specimens is largely influenced by the mould conditions. Real-time monitoring of preliminary tests on TWHC specimens showed that in the case of curing the specimens for less than 8 days with a water content of 10%, borehole breakout initiated in a zone which was not fully dry.

At a given \( w_c \), the peak strength and stiffness of the specimens increased with an increase in the weight of coarse sand grains. Also, the specimens showed more brittle behaviour for higher levels of \( \delta \). According to the current study, since the particle size distribution significantly affects the strength of the specimens for different cement values, it is not possible to suggest a specific range of \( w_c \) for different particle sizes for creating synthetic poorly cemented sand specimens.

The post-peak stress-strain behaviour of the specimens with lower \( w_c \) shows more ductile trend and effect of \( w_c \) is more considerable than that of \( \delta \) in the UCS tests. In lower \( w_c \) values dilation occurred after a limited compression in the specimens and with increasing the cementation the lateral strain decreased and the volumetric strain was mostly dominated by the axial compression strain. Also, the results showed that the absolute magnitude of the elastic modulus decreased with increasing the strain.
Solid specimens exhibited strain hardening behaviour and no peak strength was observed in their stress-strain diagram both in axial and lateral direction. Unlike the solid specimens, TWHC did not exhibit strain hardening behaviour after the maximum strength was reached and the peak strength could be determined in the applied stress ranges.

In lower confining pressures (i.e. 1 and 2 MPa) solid specimens started contracting initially from the top and bottom sections and after 0.5%-0.6% of axial compression strain, it diverted to the lateral dilation. Increasing the confining pressure keeps the specimen in contraction mode and dilation starts with delay at a higher axial strain. For confining pressures of more than 2 MPa specimens remained in contraction mode and never diverted to the dilatation until the maximum strength was reached.

The TWHC specimens with confining pressures higher than 4.5 MPa, first stage of the test could not be completed and the borehole failed before the application of the deviator stress. Also, in the TWHC specimens the volumetric strain curve conjugated the axial strain axis at a higher magnitude which suggests that microcracks were formed on the borehole wall and sand grain dislocation was taking place in TWHC versus solid specimens to release the applied stresses. Based on the observations from the real-time camera recording it can be stated that the transition from the volumetric strain contraction to the dilation occurred before the initiation of the borehole breakout.

The failure strength of the 20 mm diameter borehole TWHC specimens was less than that for the 10 mm especially at the higher confining pressures due to the size-scale effect. Also, ductility was less in 20 mm borehole specimens in comparison to that for the 10 mm ones. However, the stiffness in axial and lateral directions did not change with an increase in the borehole size. Also, it was observed that the lateral ductility significantly decreased in the specimens with larger borehole size.

For further investigation on the behaviour of poorly cemented sands, it is suggested to upgrade the laboratory tests to polyaxial stress condition in order to simulate anisotropic horizontal in situ stresses adjacent to a drilled borehole.

**Acknowledgements**

This work has been supported by the Deep Exploration Technologies Cooperative Research Centre whose activities are funded by the Australian Government’s Research Programme. This is DET CRC Document 2014/262.
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Chapter 3 presents the third manuscript titled ‘Shear failure analysis of a shallow depth unsupported borehole drilled through poorly cemented granular rock’. The principal stresses at the borehole wall for 20 mm and 10 mm borehole sizes were estimated based on the theory of elasticity. All specimens were subjected to hydrostatic stress state by applying confining pressure and vertical stress at the same magnitude and rate. Afterwards, the vertical displacement was increased at a constant rate (0.07 mm/min) until the breakout took place. Since no supporting system was deployed inside the borehole, $P_w$ was considered to be zero. The stress values form the laboratory test results performed on synthetic thick-walled hollow cylinder (TWHC) specimens were introduced into the equations which were derived analytically. Then, the calculated values for $\tau_{oct}$, $\sigma_{oct}$ and $\sigma_{m,2}$ based on the data from the tests were introduced into the Coulomb, Drucker-Prcker and Mogi failure domains in order to identify the best suited failure domain for predicting the behaviour of the poorly cemented sands adjacent to the borehole wall. Test results were evaluated versus the elaborated failure criteria to investigate whether or not these criteria can predict the behaviour of the poorly cemented sands in case of drilling a borehole through them. Implementing the Mogi criterion in a borehole stability model showed that all obtained results can be represented with a single straight line. If the stress values lie on this line the borehole failure takes place, while the points above and beyond this line do not have any physical meaning. This failure envelope is valid for the poorly cemented sandy formations with the particle size distribution corresponding to the one at the drilling site in Burra, South Australia and within the range of $wc$ values considered in this study (i.e. 6%, 7% and 8%). Considering that a sandy formation can be encountered at the depth of up to 200 m, this criterion can be applicable for exploration boreholes which are being drilled through poorly cemented formations. Then, stabilisation systems such as the mud pressure can be designed based on the far-field stress configuration.

List of Manuscripts

# Statement of Authorship

<table>
<thead>
<tr>
<th>Title of Paper</th>
<th>Shear failure analysis of a shallow depth unsupported borehole drilled through poorly cemented granular rock.</th>
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<td>Publication Status</td>
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## Author Contributions

By signing the Statement of Authorship, each author certifies that their stated contribution to the publication is accurate and that permission is granted for the publication to be included in the candidate's thesis.

### Name of Principal Author (Candidate)

<table>
<thead>
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<th>Seyed Saeid Hashemi</th>
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**Contribution to the Paper**

Designed and performed laboratory tests, interpreted data, developed analytical method, manuscript and acted as corresponding author.

**Signature**

**Date** 27/03/2015

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**Contribution to the Paper**

Contributed to research, supervised and proofreading.

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**Date**
Shear failure analysis of a shallow depth unsupported borehole drilled through poorly cemented granular rock

S.S. Hashemi\textsuperscript{a}\textsuperscript{*}, A. Taheri\textsuperscript{a}, N. Melkoumian\textsuperscript{a}

Abstract

Adopting an appropriate failure criterion plays a key role in the borehole stability analysis. In this paper the induced stresses on a vertical borehole wall were calculated based on the elastic theory. Then, to predict the stability of a borehole drilled through a poorly cemented sand formation, failure envelopes in different failure criterion domains were derived using the results from a series of precise laboratory tests conducted on solid and hollow cylinder specimens. The mixture used in specimen preparation was designed to simulate the properties of the samples collected from depths up to 200 m at a drilling site in South Australia. The hollow cylinder test apparatus was developed by modifying a Hoek triaxial cell. These modifications allowed observing the process of debonding of sand grains from the borehole wall during the test and consequently, acquiring a better understanding on the failure mechanisms of a borehole drilled through poorly cemented sand formations. Three well-known failure criterion domains; Coulomb, Drucker-Prager and Mogi, were considered versus the laboratory test data to investigate their capability to predict the shear failure of a borehole using the data from hollow cylinder tests. The obtained results showed the significance of selecting an appropriate failure domain for predicting the shear failure behaviour of poorly cemented sands near the borehole wall. The results also showed that the Coulomb criterion is not well suited for predicting the borehole failure when there is no pressure acting inside the borehole. A failure envelope based on the Mogi domain was developed which can be used for the far-field stress states. The introduced failure envelope allows predicting the stability of a borehole drilled in poorly cemented sands.

Keywords: Borehole stability; Triaxial test; Thick-walled hollow cylinder; Failure criterion; Analytical method
Nomenclature

<table>
<thead>
<tr>
<th>Nomenclature</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWHC</td>
<td>thick-walled hollow cylinder</td>
</tr>
<tr>
<td>$w_c$</td>
<td>ratio of cement to sand grains weight</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>mean grain diameter (mm)</td>
</tr>
<tr>
<td>$C_u$</td>
<td>coefficient of uniformity</td>
</tr>
<tr>
<td>$\sigma_{\theta\theta}$</td>
<td>tangential stress (MPa)</td>
</tr>
<tr>
<td>$\sigma_{conf}$</td>
<td>confining stress (MPa)</td>
</tr>
<tr>
<td>$\sigma_{rr}$</td>
<td>radial stress (MPa)</td>
</tr>
<tr>
<td>$\phi$</td>
<td>angle of internal friction (deg)</td>
</tr>
<tr>
<td>$\sigma_c$</td>
<td>vertical stress (MPa)</td>
</tr>
<tr>
<td>$c$</td>
<td>cohesion (MPa)</td>
</tr>
<tr>
<td>$\tau$</td>
<td>shear stress (MPa)</td>
</tr>
<tr>
<td>$\sigma_n$</td>
<td>normal stress (MPa)</td>
</tr>
<tr>
<td>$p_w$</td>
<td>in-hole pressure (MPa)</td>
</tr>
<tr>
<td>$\sigma_{m,2}$</td>
<td>mean effective normal stress (MPa)</td>
</tr>
<tr>
<td>UCS</td>
<td>uniaxial compressive strength (MPa)</td>
</tr>
<tr>
<td>$\tau_{oct}$</td>
<td>octahedral shear stress (MPa)</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$\sigma_{oct}$</td>
<td>octahedral normal stress (MPa)</td>
</tr>
<tr>
<td>$k_o$</td>
<td>$\sigma_n/\sigma_v$</td>
</tr>
<tr>
<td>$\delta$</td>
<td>ratio of fine to coarse sand weight</td>
</tr>
<tr>
<td>$D_i$</td>
<td>internal diameter of a hollow cylinder (mm)</td>
</tr>
<tr>
<td>$I_i$</td>
<td>$i^{th}$ normal stress invariant</td>
</tr>
<tr>
<td>$\gamma_d$</td>
<td>dry density</td>
</tr>
<tr>
<td>$w_{opt}$</td>
<td>optimum water content</td>
</tr>
</tbody>
</table>

1. Introduction

Exploration drilling is one of the most frequently used data collection methods in mining, petroleum and civil engineering. Drilling a borehole is an excellent method not only for monitoring and petrographic assessment of borehole walls but also for measuring spatial position of formations and for obtaining characteristics of faults and joints. Hence, borehole stability is one of the most important challenges to be addressed in geomechanics. A number of drilling companies in Australia have reported several borehole instability issues in poorly cemented formations in last few years. Thus, it is important to conduct a comprehensive research on borehole failure at shallow depths (up to 200 m) where unconsolidated/poorly cemented sand formations are present.

There are many closed form solutions (Zoback, 2007; Kemeny, 2003; Fairhurst, 1968) for analysing boreholes via thick-walled hollow cylinder samples. Most of the borehole stability analyses are based on the assumptions that the rock is isotropic and the directions of regional stresses are known. However, in many cases these assumptions may not be valid. Since the initial stress state at a point in a certain depth underground determines the stress concentration around an opening, it is desirable that this initial state to be known in advance, before the development of an underground structure (Fairhurst, 1968). When a borehole is drilled through a weak material, such as poorly cemented sand formation that is present in sedimentary geological formations, the release of the pre-existing in situ stresses underground...
may result in borehole instability. Tangential stress will be generated around the borehole wall and the radial stress on the borehole wall is zero in case of an unsupported borehole. If the strength of the borehole wall is not sufficient, the borehole may collapse. This happens due to re-establishment of the equilibrium to the ground after the borehole excavation of the borehole. To assess the stability of a borehole, the stresses redistributed at the borehole wall must be determined and compared to the critical stress condition using an appropriate failure criterion. If there is a need for a support system, it can be designed with respect to the limits of the failure criterion.

2. Underground in situ stress state

Prior to drilling a borehole, the underground formations are usually subjected to vertical compressive stress and horizontal stresses caused by the overlying strata and lateral pressure respectively. The vertical principal far-field stress can usually be estimated as the weight of the overburden (Fairhurst, 2003). The minimum horizontal stress can be measured by hydraulic fracturing and leak off test (Amadei and Stephansson, 1997). However, determining the in situ maximum horizontal stress is a matter of debate and it can be guessed only based on specific considerations (Della Vecchia et al., 2014; Aadnoy et al., 2013; Zoback et al., 1985). The in situ stress state can be defined in terms of the principal stresses, $\sigma_v$, $\sigma_H$, and $\sigma_h$. The ratio of minimum horizontal stress to the vertical stress ($k_o = \sigma_h/\sigma_v$) ranges from 0.3 to 1.5 and the ratio of maximum horizontal stress to minimum horizontal stress ($\sigma_H/\sigma_h$) varies from 1 to 2 in most of oil fields and exploration boreholes (Herget, 1988; Tan et al., 1993; Chen et al., 2002). Based on the tectonic and faulting condition, Anderson (1951) suggested three different stress regimes for the underground stress conditions. These three stress regimes are the permutation of $\sigma_v$, $\sigma_H$ and $\sigma_h$ in higher to smaller order as follows,

\[
\begin{align*}
\sigma_v \geq \sigma_H \geq \sigma_h \\
\sigma_H \geq \sigma_v \geq \sigma_h \\
\sigma_H \geq \sigma_h \geq \sigma_v
\end{align*}
\]

(1)

Different faulting conditions that introduce in situ stress regimes are presented in Fig. 1. According to the placement of stresses in the above-mentioned three inequalities (Eqs. 1), the maximum, intermediate and minimum stresses can be defined. The most likely conditions at drilling fields are given by the first and second inequalities in Eqs. 1 (Al-Ajmi and Zimmerman, 2006).
This formation system is in a static equilibrium stress state assuming no movement exists due to any seismic activity nearby. Once a borehole is excavated, the balanced stress condition will be disturbed eventually causing instability in the adjacent rock formation. This causes an increase in the tangential stress and a decrease in the radial stress near the opening wall thus resulting in the elastic deformation of the formation near the opening at the very least (Ewy and Cook, 1990a, b). To re-establish the state of equilibrium the load, which was tolerated earlier by the removed material, must now be carried by the formation around the borehole. This new stress arrangement imposes a different set of stresses that can extend from the excavated area to a distance of up to few times the borehole diameter.

Depending on the conditions and purpose of drilling (e.g. air pressure or filled with a drilling mud) the walls of the excavated opening are usually supported by a corresponding supporting system exerting a pressure, $P_w$, on the borehole wall. The borehole support system must be designed to prevent the onset of borehole shear collapse which is revealed by the failure of rock material at the borehole wall. On the other hand, the high in-hole pressure exerted by the support system can result in the tensile failure of the borehole, as is in the case of hydraulic fracturing due to high drilling mud pressure. Zoback et al. (2003) showed that the applied pressure cannot be exactly equal to the in situ stresses that have existed before drilling the borehole.

2.1. State of stress around a vertical borehole

The equations for calculating stress distribution around a circular hole in an infinite plate in linear elastic rock were initially introduced by Kirsch (1898). Kirsch equations can also be
generalised to calculate stresses around vertical and deviated boreholes with anisotropic far-field stresses. After the completion of drilling operations in a formation under pre-existing anisotropic in situ stresses, total induced stresses acting on an element at the vertical borehole wall can be derived by Kirsch equations as follows,

\[ \sigma_{rr} = P_w \]  
\[ \sigma_{\theta \theta} = \sigma_H + \sigma_h - 2(\sigma_H - \sigma_h) \cos 2\theta - P_w \]  
\[ \sigma_{zz} = \sigma_v - 2\nu(\sigma_H - \sigma_h) \cos 2\theta \]  
\[ \tau_{r\theta} = 0 \]  
\[ \tau_{rz} = 0 \]  
\[ \tau_{\theta z} = 0 \]

where \( \sigma_{rr}, \sigma_{\theta \theta}, \sigma_z \) and \( \tau \) are the radial, tangential, vertical and shear stresses respectively. The angle \( \theta \) was measured from the direction of maximum horizontal stress.

It is worth to mention that from theoretical point of view tangential and vertical stresses can be assigned negative values corresponding to the tensile stress (Ljunggren and Amadei, 1989) and the negative tangential stress can result in a vertical hydraulic fracture (Haimson, 1968; Haimson, 1978; Haimson, et al., 2003). Number of researchers have presented different mathematical models for predicting rock failure under a given stress state based on the rock material characteristics. Elasto-plastic models to calculate the stress distribution around a borehole were suggested by Westergaard (1940), Mitchell et al. (1987) and Anthony and Crook (2002). Linear elastic constitutive models were presented by Fairhurst (1965b), Bradley (1979) and Aadnoy (1989b). These models are among the most common methods used to analyse the stresses around a borehole because of their simplicity and fewer input parameters compare to more sophisticated models. Although some researchers state that non-linear approaches can deliver more accurate results, they have acknowledged that some input parameters in such models are hardly available (Fleming et al., 1990; Woodland, 1990; Garrouch and Ebrahim, 2001). On the other hand, Aadnoy (1988), Tan et al. (1999) and Chen et al. (2002) showed that in ordinary oil-filled wellbores, the minimum (critical) support pressure required to keep the wellbore stable is not significantly influenced by the elastic
anisotropy. In this paper for a vertical borehole the stresses, which were generated on an element at the borehole, wall due to far field stresses were calculated based on the elastic theory. To predict the stability of a borehole drilled in poorly cemented sandy formation failure envelopes were drawn in different failure criterion domains using the results of a series of precise laboratory tests on solid and hollow cylinder specimens.

3. Drilling site investigations

The main purpose of drilling exploration boreholes is to investigate the suitability of a given site for potential mining activates. In a mining exploration drilling site in Burra, South Australia the exploration boreholes are drilled through unconsolidated poorly cemented formations. The majority of these boreholes are 25 cm to 30 cm in diameter with lengths varying from 80 m – 250 m depending on the mine exploration plan.

The subsurface investigation of sediments and borehole surveys showed that the sediment above the bedrock is heterogeneous and irregular. From the ground surface up to 30-40 m depth there is a layer of silt and fine sands. This layer of sediment is underlain by a dark grey clayey layer and below that a poorly cemented sandy layer was identified (Fig. 2). Air core and reverse circulation (RC) drilling methods were adopted to drill exploration boreholes (Fig. 3). The drill cuttings are removed by injecting pressurized air by pushing 425-550 l/s of air at 2000-2400 kPa down the hole through the annular opening between the inner tube and the drill rod. When the drilling string reaches the poorly cemented sandy layer there is a considerable potential for the borehole to collapse due to weak bonding between the sand particles. Occasionally the actuator is unable to restart and rotate the rod, and the drilling rod gets stuck in the borehole if the gap between the drilling rod and the borehole wall fills up with sand. Such problems are often encountered at depths of 70 m - 150 m. Each collapsed borehole costs at least AU$ 50,000 due to loss of rods and the bit. Unfavorable stress states around the borehole, poor cementation of sandy layer and the presence of a high groundwater inflow discharged from a confined aquifer into the borehole are known to be responsible for the borehole instability. In fact, in poorly cemented sands, where there is no adequate cementation between grains, the induced tangential stress can lead to debonding of sand grains thus creating a damaged zone around the borehole as shown in Fig. 4. Also, based on the field observations the friction between the drilling rod and borehole wall causes particle debondings as well. According to Fig. 4 when there is no $P_w$, the effective tangential stress
(\(\sigma'_\theta\)) at the borehole wall is higher than \(\sigma'_r\) and with increasing the distance from the borehole \(\sigma'_\theta\) decreases gradually. Failures of the borehole wall under these conditions have been documented by many researchers (Fischer et al. 1977). Mavko and Jizba (1991) showed that if this situation is simulated in a laboratory model, the size of failed zones may be influenced not only by the final stress state but also by the stress path, strain rate and test boundary conditions.

Fig. 2. Geological underground vertical cross section near Burra, South Australia

Fig. 3. RC drilling site near Burra, South Australia
4. Experimental study

Laboratory investigations play a key role in acquiring a better understanding on the behaviour of a granular formation under different loading conditions and are necessary for determining the parameters required for borehole stability analysis. Unconfined compressive strength (UCS), triaxial and thick-walled hollow cylinder (TWHC) tests are the most popular laboratory experiments for this purpose.

Site investigations showed that drilling through shallow formations results in extensive sand grain debondings at the borehole wall due to the lack of sufficient cementation and consolidation. Various mixtures of sand particles, cement and water were casted into prefabricated specific cylindrical moulds to prepare suitable specimens for laboratory tests. Specimens were designed to fit into a HQ Hoek triaxial cell of 63.5 mm diameter and 127 mm length. Various cement contents and fine to coarse sand weight ratios ($\delta$) were mixed to achieve the particle size distribution and texture closely resembling the properties of sandy layer at the drilling site. The whole process of experimental studies including Hoek cell modification, developing specimen preparation facilities, specimen preparation and conducting the tests was labour intensive and time consuming, and took more than 14 months to complete.

4.1 Thick-walled hollow cylinder (TWHC) test

The hollow cylinder specimen test is a common method used in a wide range of studies (Adam, 1912; King, 1912; Robertson, 1955) and has different applications ranging from simulating stress and strain conditions around underground openings such as boreholes and
tunnels to investigating the behaviour of geomaterials under various stress paths (Adeyanju and Olafuyi, 2011). In many literatures (e.g. Jaeger et al., 2007; Obert and Duvall, 1967) closed form solutions based on the linear theory of elasticity for calculating stresses and strains in thick-walled hollow cylinder specimens are presented. For a TWHC, with an inner diameter of $D_i$, outer diameter of $D_o$ and length of $L$, subjected to axial force ($f$), uniform internal stress ($S_i$) and external stress ($S_o$) the principal stresses at any point at a radial distance $r$ from the centre of the specimen can be presented in cylindrical coordinates as follows (Jaeger et al., 2007):

$$\sigma_{\theta\theta} = \frac{S_o D_o^2 - S_i D_i^2}{D_o^2 - D_i^2} + \frac{(S_o - S_i) D_i^2 D_o^2}{4r^2(D_o^2 - D_i^2)} \quad (3a)$$

$$\sigma_{rr} = \frac{S_o D_o^2 - S_i D_i^2}{D_o^2 - D_i^2} - \frac{(S_o - S_i) D_i^2 D_o^2}{4r^2(D_o^2 - D_i^2)} \quad (3b)$$

$$\sigma_z = \frac{4F}{\pi(D_o^2 - D_i^2)} + \frac{S_i D_i^2}{(D_o^2 - D_i^2)} \quad (3c)$$

where $\sigma_{\theta\theta}$, $\sigma_{rr}$ and $\sigma_z$ are the tangential, radial and axial principal stresses respectively which are assumed to be uniformly distributed over the specimen on the top and bottom surfaces.

4.2. Sample preparation

4.2.1. Moulds

Since the poorly cemented specimen is very weak in nature, deforming the mould to release the specimen after the curing time imparts damage to it. Also, moulds should be hard enough to allow compaction of the mixture without any deformation of the mould. Specific moulds were manufactured to produce specimens, which would fit into a HQ Hoek cell (Fig. 5). Each layer of mixture was compacted in 25 impacts with a specific compactor made for this purpose (Fig. 6). Compaction energy for each impact was kept constant and equal to 0.35 $Nm/cm^3$ that was calculated based on the applied force of the manufactured compactor. Two types of specimens, i.e. Solid and hollow were prepared according to the test plan.
4.2.2. Mixture

As discussed in the previous section, sample preparation is one of the most important stages of the laboratory tests aimed to simulate the actual behaviour of poorly cemented sands at the drilling site. The elements of synthetic poorly cemented mixed sand specimens prepared in this study, including texture and sand grading size distribution were designed similar to the properties of the samples collected from the drilling field in blanch town, south Australia. This is a problematic drilling site where the weak formation comprises of quartz grains (≥ 96%) with weak bonding interface of clay and calcite acting as cementing agents. Most of the sand grains are fine and sub-angular with random orientations. Pale yellowish-grey specimens with a fabric and grading similar to the poorly cemented sand formation at the drilling site were produced in the laboratory. It is a near-uniform mixture of fine-grained sand, Portland cement and water. Care was taken to produce homogenous specimens; a small mixer was used to ensure that the sand grains, cement and water mixed perfectly. The total
period from the start of mixing until sealing the moulds was strictly maintained between 30-40 minutes.

4.2.3. Particle size distribution

To determine the grain size distribution of the samples collected (\( \rho = 1720 \, kg/m^3 \)) from the depths up to 100 m at the problematic drilling site in Burra, south Australia, sieve analysis based on calibrated ASTM C-136 sieves plus pan were performed. Based on the sieve analysis results, Australian natural well sorted silica sands of two different grain size ranges (i.e. “coarse” and “fine” sand) for the specimen mixture were chosen as the most similar particle size distribution and texture to the drilling site samples. Fig. 7 shows the grain size distribution for the “coarse”, “fine” and the drilling site samples. The mean diameter value (\( D_{50} \)) is 0.56 mm for silica dry sand with the grain size between 0.425 mm- 1.4 mm and is 0.20 mm for the grain size between 0.125 mm - 0.355 mm. The coefficient of uniformity (\( C_u = D_{60}/D_{10} \)) for coarse and fine sand grains is 1.452 and 2.268 respectively.

![Fig. 7. Sand particle size distribution curve for laboratory test specimens and collected samples from a drilling site at South Australia](image)

4.2.4. Water content

Standard proctor test was performed on the prepared cemented-sand mixture and the optimum water content was measured to be 9.7~10.3% by using standard compactor hammer with the energy of 0.55 Nm/cm\(^3\). It is worth noting that there was no significant change in
the optimum water content \((w_{opt})\) and dry density \((\gamma_d)\) in the case of cement-sand mixture and when considering only sand grains.

4.2.5. Curing time

Different curing times have been suggested in previous studies (Alsayed, 2002; Saidi et al., 2002; Younessi et al., 2013). Curing time ranging from 2-12 days was examined to identify the specimen most similar to the problematic sandy formation located at the drilling site. The curing time includes the time when the specimen was in the mould and when it was out of the mould in a plastic wrap. Finally, 8 days were identified as the optimal curing time for the specimen, which comprised of keeping the specimen for 5 days in the mould and 3 days outside of it in a plastic wrap at temperatures between 18 – 22° Celsius. After the given curing time specimens were completely dried and the hydration of cement powder was considered suitable for performing the tests.

4.2.6. Cement content

Since the cementing agent at the actual field was different from the one used for laboratory tests and preparing a specimen with the original cement takes a long time (e.g. Several years), Portland cement type II \((G_s = 3.15 \text{ g/cm}^3)\) was used as a cementing agent for preparing the TWHC and solid specimens. It should be noted that collecting undisturbed sample was extremely difficult. Saidi et al. (2003) showed that Portland cement is a suitable cementing agent for simulating poorly bonded specimens. A wide range of cement contents \((2.5\% \leq w_c \leq 18\%)\) for producing poorly cemented synthetic sandstone has been suggested in different works (Alsayed, 2002; Saidi et al., 2004; Saidi et al., 2002; Younessi et al., 2013; Kongsukprasert, 2003). For instance, Kongsukprasert (2003) used maximum of 2.5% of Portland cement to prepare poorly cemented gravel specimens, while Saidi et al. (2004) used 9-18%. Gueguen and Palciauskas (1992) stated that the minimum \(w_c\) is reached at \(\delta = 0.4\) (weight of fine/coarse sands). However, based on authors’ observations, for the given grading sizes of sand particles when \(w_c \leq 2.5\%\), the specimens could not be safely demoulded after 8 days of curing time.

Different mixtures with respect to various Portland cement content \((w_c)\) and various proportions of fine and coarse silica sands with different grading ratios \((\delta)\) were examined. Finally, for the above mentioned sand grain size ranges, \(w_c =6\%, 7\%\) and \(8\%\) were selected.
According to the current study, since the sand particle grading has a significant role in the strength of specimens, it is not possible to suggest a definite range of $w_c$ for various sand grain distributions to produce poorly cemented sand samples. Therefore, $w_c$ should be selected based on the chosen grain size distributions.

4.3. Testing set-up

A modified hollow cylinder test cell with a fitted micro camera was designed and manufactured at the University of Adelaide. The fitted camera allows real-time viewing and video recording of the borehole walls during the triaxial test on TWHC specimens. The modified hollow cylinder test cell was built by applying modifications to a large Hoek triaxial cell into which a TWHC specimen of 63 mm × 127 mm can be fitted. These facilities were synchronised with a precise system of applying confining pressure in low magnitude stresses (maximum 5 MPa) with no leakage or intrusion. According to the UCS test on the cell platens, the strain of the platens is less than 0.005% when subjected to a maximum of 100 kN loading force which is far more than the strength of TWHC specimens. The following laboratory facilities were used for conducting the tests discussed in this study:

- A servo controlled axial loading system of 100 kN capacity with the 0.1 n accuracy was used for applying vertical stress to the specimen. An automatic hydraulic machine in conjunction with a relief valve and a pressure gauge was used to apply and monitor the external confining pressure. Since the maximum confining pressure in the tests was low (maximum 5 MPa), using a precise pressure transducer was essential. Therefore, a pressure gauge was used to measure the confining pressure with the accuracy of 0.001 MPa.
- A micro camera with 225 PPI resolution was installed inside the hollow platen to record the process of borehole breakout.
- A 60-channel data acquisition system was connected to two personal computers in order to monitor and record stress, strain, load and displacement.

4.4. Testing procedure

The specimen loading process comprised of several stages. First, vertical and confining stresses acting on the specimen were simultaneously increased up to a certain stress level to simulate the hydrostatic stress condition. Then, the vertical load was increased at the constant
displacement rate of 0.07 mm/min. Confining pressure was varied in each test to induce a predetermined stress at the external wall of the specimen. Table 1 presents the schedule of the main conducted tests. Various strain rates (0.02-0.10 mm/min) were considered as well. Results showed that for the stress ranges considered in this test program, there was no significant change in strength and stress-strain behaviour of the specimens for the strain rate up to 0.08 mm/min.

<table>
<thead>
<tr>
<th>Type of specimen</th>
<th>Cement content</th>
<th>δ</th>
<th>borehole size</th>
<th>Confining pressure (MPa)</th>
<th>Displacement rate (mm/min)</th>
<th>Water content (weight)</th>
<th>Curing time (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid cylinder</td>
<td>6%, 7%, 8%</td>
<td>1</td>
<td>-</td>
<td>1-5</td>
<td>0.07</td>
<td>10%</td>
<td>8</td>
</tr>
<tr>
<td>TWHC</td>
<td>6%, 7%, 8%</td>
<td>1</td>
<td>10 mm, 20 mm</td>
<td>1-5</td>
<td>0.07</td>
<td>10%</td>
<td>8</td>
</tr>
</tbody>
</table>

It should be mentioned that all tests were conducted in a dry boundary condition and the effect of pore pressure is not considered in this study.

5. Failure criteria used for TWHC and solid specimens

5.1. Coulomb failure criterion

Coulomb suggested that shear failure occurs when the shear stress is equal to the sum of the cohesive shear strength and the product of the coefficient of internal friction and normal stress across the fracture plane. Jaeger et al. (2007) rearranged the Coulomb criterion in terms of principal stresses as,

\[ \sigma_1 = C + q\sigma_3 \]  

(4)

where \(\sigma_1\) and \(\sigma_3\) are the maximum and minimum principal stresses, respectively. \(C\) is the uniaxial compressive strength of intact rock and is given by,

\[ C = \frac{2c \cos\phi}{1-\sin\phi} \]  

(5)

and \(q\) expresses the friction properties of the material and is defined as follows,
\[ q = \frac{1 + \sin \phi}{1 - \sin \phi} \]  

Later, Mohr asserts that the relationship between the normal and shear stresses along the failure plane is non-linear,

\[ \tau = f(\sigma_n) \]  

where, \( f \) is a function that can be derived empirically. According to this, the linear form of the Mohr criterion is equivalent to the Coulomb criterion.

### 5.2. Drucker-Prager failure criterion

Since the distortional energy is proportional to the octahedral shear stress, Drucker and Prager (1952) suggested that the failure of the geomaterials would occur when the distortional strain energy meets a specific value that increases with the mean normal stress, \( \sigma_{oct} \). It is possible to use the concept of this failure criterion and analyse the obtained test data in \( \tau_{oct} - \sigma_{oct} \) domain. The main difference between the Drucker-Prager and the Coulomb criteria is that the strengthen effect of the intermediate principal stress has been considered in the Drucker-Prager criterion. In this domain, principal stresses are expressed in terms of the octahedral shear stress as a function of the octahedral normal stress as follows,

\[ \tau_{oct} = f(\sigma_{oct}) \]  

In the linear case the failure criterion can be written as,

\[ \tau_{oct} = a + b \sigma_{oct} \]  

where \( \tau_{oct} \) and \( \sigma_{oct} \) are the octahedral shear and normal stresses respectively as,

\[ \tau_{oct} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_3)^2 + (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2} \]  

\[ \sigma_{oct} = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \]  

\( a \) and \( b \) are material property constants and can be estimated empirically. The Drucker-Prager criterion was also written in a non-linear form (e.g. Wong et al., 1997; Klein et al., 2001).
Khan et al. (1991), suggested a parabolic form of Drucker-Prager criterion for weak rocks in terms of the stress invariants which can be rearranged in terms of \( \tau_{oct} \) and \( \sigma_{oct} \) as follows,

\[
\tau_{oct} = a' + b' \sigma_{oct} + c' \sigma_{oct}^2
\]  

(12)

where \( a' \), \( b' \), \( c' \) are material properties. Eq.12 can be considered as a non-linear version of the Drucker-Prager failure criterion.

5.3. Mogi failure criterion

Although the Drucker-Prager failure domain considers the effect of \( \sigma_2 \), it was initially developed based on the theory and had not been verified by experimental studies. Mogi (1971) proposed another criterion based on the extensive polyaxial compressive tests on various rocks. He proposed to consider effective mean normal stress, \( \sigma_{m,2} \), instead of \( \sigma_{oct} \) in the failure function, since the failure occurs in the direction of strike of the intermediate stress. Mogi’s results were in good agreement with subsequent observations by a number of researchers (Takahashi and Koide, 1989; Haimson and Chang, 2000). In general, Mogi (1971) suggested the following relationship as a failure function based on a large number of polyaxial tests,

\[
\tau_{oct} = f(\sigma_{m,2})
\]  

(13)

\[
\sigma_{m,2} = \frac{\sigma_1 + \sigma_3}{2}
\]  

(14)

where \( f \) is a monotonic function increasing along both axes. The failure envelope was not explicitly defined in \( \tau_{oct} - \sigma_{m,2} \) by Mogi (2007) and the function is supposed to be obtained empirically.

In polyaxial compression tests, there are three different variables (i.e. principal stresses) which cannot be shown explicitly in two-dimensional space to analyse the principal stresses. Therefore, the function should be reasonably reduced to two dimensions in order to show them in the \( \tau_{oct} - \sigma_{m,2} \) or \( \tau_{oct} - \sigma_{oct} \) space. Al-Ajmi (2006) examined seven different sets of hard rock polyaxial test data to determine the failure envelope in the \( \tau_{oct} - \sigma_{m,2} \) and \( \tau_{oct} - \sigma_{oct} \) domains. Plotting the triaxial and polyaxial test data revealed that the Drucker-Prager criterion best-fit linear line is in a good correlation with the triaxial test data, but these models do not represent the polyaxial stress states. Also, Aadnoy et al. (1987) and McLean
and Addis (1990b) showed that the Drucker-Prager criterion overestimates the strength of the rocks.

6. Results and discussion

6.1. Solid cylindrical specimens

To identify shear strength characteristics of the designed mixture, initially triaxial tests were conducted on the solid specimens. Solid specimens showed strain hardening behaviour and the stress corresponding to the 1% of axial strain was considered as the peak stress. The results from these tests were evaluated versus different failure criteria as follows.

6.1.1. Mohr-Coulomb failure domain

Fig. 8 shows the Mohr circles at failure for the triaxial tests on solid specimens. To explain the behaviour of the material by the Coulomb failure criterion, the cohesion, \( c \), and angle of internal friction, \( \phi \), can be estimated from the graphs presented in Fig. 8. As expected, increasing the cement value produced higher cohesion between the sand grains and based on the Coulomb criterion \( c \) will change for different cement values. The obtained \( c \) and \( \phi \) values are presented in Table 2 based on Eq. 4-6 and Fig. 8a. For different \( w_c \) values the \( \phi \) did not change dramatically by increasing the \( w_c \). This is acceptable since the grading was not changed in the mixtures.
Fig. 8. Results from laboratory tests on solid specimens analysed by the Coulomb failure function: (a) $\sigma_1 - \sigma_3$ space, (b) Mohr circles $w_c=6\%$, $\sigma_{conf} = 1 - 4$ MPa (c) Mohr circles $w_c=7\%$, $\sigma_{conf} = 1 - 4$ MPa (d) Mohr circles $w_c=8\%$, $\sigma_{conf} = 1 - 4$ MPa

Al-Ajmi and Zimmerman (2005) showed that although the Mohr-Coulomb criterion can be used in triaxial test condition ($\sigma_2 = \sigma_3$), the results cannot be extended to polyaxial stress state. This failure criterion assumes that the intermediate principal stress has no strengthen effect on failure of the specimen.
Table 2 Constants of the Coulomb shear failure parameters equation (Eq. 7) based on laboratory tests on cylindrical solid specimens for different \( w_c \)

<table>
<thead>
<tr>
<th>Cement %</th>
<th>C (degree)</th>
<th>( q )</th>
<th>( c ) (MPa)</th>
<th>( \phi )</th>
<th>( r^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>3.826</td>
<td>2.961</td>
<td>1.11</td>
<td>29.67</td>
<td>0.9933</td>
</tr>
<tr>
<td>7</td>
<td>3.803</td>
<td>3.132</td>
<td>1.38</td>
<td>30.06</td>
<td>0.9953</td>
</tr>
<tr>
<td>8</td>
<td>5.348</td>
<td>2.804</td>
<td>1.59</td>
<td>29.67</td>
<td>0.9968</td>
</tr>
</tbody>
</table>

6.1.2. Drucker-Prager failure domain

The data from conducted tests were analysed in the \( \tau_{oct} - \sigma_{oct} \) space based on the Drucker-Prager domain to obtain the equation for \( \tau_{oct} \) as a function of \( \sigma_{oct} \) (Eq. 8). Fig. 9 shows the experimental data in the \( \tau_{oct} - \sigma_{oct} \) domain. According to Fig. 9, the relationship between the data in the \( \tau_{oct} - \sigma_{oct} \) space is relatively linear. \( a \) and \( b \) constants in Eq. 8 and corresponding correlation coefficients in Eq. 9 are presented in Table 3. Although it is possible to fit a polynomial equation to the obtained data, the Table 3 demonstrates that with a good approximation, a linear relationship between the octahedral shear stress and octahedral normal stress can be used for the mixture. Fig. 9 shows that increasing the \( w_c \) slightly shifts the failure envelope upward in the \( \tau_{oct} - \sigma_{oct} \) space. It should be mentioned that the results were verified by several tests (at least 3 tests for each \( w_c \) value) to ensure the reproducibility of the results.

Table 3 Constants of the Drucker-Prager failure domain equation (Eq. (15)) obtained from laboratory tests on cylindrical solid specimens with different \( w_c \)

<table>
<thead>
<tr>
<th>Cement %</th>
<th>Drucker-Prager constants</th>
<th>( a )</th>
<th>( b )</th>
<th>( r^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td></td>
<td>1.079</td>
<td>0.561</td>
<td>0.994</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>1.223</td>
<td>0.566</td>
<td>0.996</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>1.337</td>
<td>0.584</td>
<td>0.997</td>
</tr>
</tbody>
</table>
6.1.3. Mogi failure domain

For the test data plotted in the $\tau_{oct} - \sigma_{m,2}$ domain, an arbitrary linear function could be written as follows,

$$\tau_{oct} = a + b \sigma_{m,2}$$  \hspace{1cm} (15)

where $a$ is the intersection of the line with the vertical axis ($\tau_{oct}$) and $b$ is the slope of the line. Al-Ajmi (2006) showed that when using the Mogi stress domain (i.e. $\tau_{oct} - \sigma_{m,2}$), triaxial and polyaxial test data coincide into a single line for a specific type of rock. It means that if a set of triaxial test data is plotted together with their best fitting linear model, the constants $a$ and $b$ of the linear model can be derived. Therefore, it can be suggested that the shear failure under the polyaxial stress state ($\tau_{oct} - \sigma_{m,2}$ ) can be predicted from the triaxial test data using the linear Mogi criterion.

The Mogi failure domain was considered for plotting failure envelopes using the obtained test results. The octahedral shear stress and the mean effective normal stress were calculated and plotted in the $\tau_{oct} - \sigma_{m,2}$ space. Fig. 10 shows the results for three given $w_c$ values. Likewise, the relationship between $\tau_{oct}$ and $\sigma_{m,2}$ is almost linear. The constants in Eq. 15 were calculated from Fig.10 and are presented in Table 4.
Fig. 10. Results from the laboratory tests on the solid specimens analysed in the Mogi stress domain

<table>
<thead>
<tr>
<th>Cement %</th>
<th>a</th>
<th>b</th>
<th>r²</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>3.826</td>
<td>2.961</td>
<td>0.993</td>
</tr>
<tr>
<td>7</td>
<td>4.348</td>
<td>2.993</td>
<td>0.995</td>
</tr>
<tr>
<td>8</td>
<td>4.848</td>
<td>3.104</td>
<td>0.996</td>
</tr>
</tbody>
</table>

It is worth mentioning that the maximum confining pressure applied to the specimens (= 5 MPa) was based on the highest amount of stress that a TWHC specimen could take. Beyond 5 MPa, specimens with maximum $w_c = 8\%$ could not even meet hydrostatic stress condition which was the first step of testing.

6.2. Borehole stability analysis base on TWHC specimens

In order to investigate the stability of boreholes, an element on the borehole wall in a TWHC specimen must be studied. For the case of a solid specimen, the specimen was considered as a single element and the principal stress matrix was analysed. However, for TWHC specimens principal stresses should be calculated based on the applied stresses at the boundary of the specimen.
Two dowels having different sizes were manufactured for preparing specimens with 10 mm and 20 mm diameter boreholes. Similar procedure and specifications including grading, water content, compaction method and curing time were applied to all the specimens. Laboratory tests on TWCHC specimens were performed for both borehole size specimens under different confining pressures.

As mentioned earlier, the main borehole failure issues are the shear failure in the form of a borehole breakout accompanied by debonding of the sand grains, and the tensile failure observed as hydraulic fracturing at the borehole wall. Hashemi et al. (2013) showed that the main reason for borehole failure in a granular material is the debonding of particles in poorly cemented granular formations. It is essential to consider the change in the tangential stress and radial stress when changing the internal borehole pressure. Thus, two cases may take place, \( \sigma_\theta \geq \sigma_r \) or \( \sigma_r > \sigma_\theta \) (see Fig. 4) which are associated with shear failure or tensile fracturing. Considering \( \sigma_z \), the magnitude of vertical principal stress, there are six states of principal stresses that may occur based on the pressure acting inside the borehole and in situ stress state,

\[
\begin{align*}
\sigma_\theta & \geq \sigma_r \geq \sigma_z \\
\sigma_z & \geq \sigma_\theta \geq \sigma_r \\
\sigma_\theta & \geq \sigma_z \geq \sigma_r \\
\sigma_z & \geq \sigma_r \geq \sigma_\theta \\
\sigma_r & \geq \sigma_z \geq \sigma_\theta \\
\sigma_r & \geq \sigma_\theta \geq \sigma_z
\end{align*}
\]

borehole breakout (16)

\[
\begin{align*}
\sigma_z & \geq \sigma_r \geq \sigma_\theta \\
\sigma_r & \geq \sigma_\theta \geq \sigma_z \\
\sigma_r & \geq \sigma_z \geq \sigma_\theta
\end{align*}
\]

tensile fracture (17)

It should be noted that a further condition for tensile failure to occur is when \( \sigma_{\theta\theta} < 0 \) at the borehole wall.

Grain debonding occurs when there is no sufficient pressure inside the borehole. Therefore, case one may happen under this condition (\( \sigma_\theta \geq \sigma_r \)). According to the Kirsch equations, the tensile strength of rock may be exceeded at the angle of 0 and 180° where the tangential stress acquires its minimum value. On the other hand, in case of low pressure acting in the borehole, the tangential stress increases toward the compressive strength of the rock. In this case, debonding takes place and the compressive strength of the formation will be exceeded at the angle of ±90° in case of \( \sigma_\theta \geq \sigma_z \).
Test results from TWHC specimens were evaluated versus the elaborated failure criteria to investigate whether or not these criteria can predict the behaviour of the poorly cemented sands in case of drilling a borehole through it. The stress states were defined theoretically in case of applying the in-hole pressure, $P_w$. Having performed laboratory tests on TWHC specimens, one can evaluate the accuracy of the derived analytical solutions and modify them if required. Current experimental set-up aimed to simulate the second stress state of compressive failures in Eq. 16 (i.e. $\sigma_z \geq \sigma_\theta \geq \sigma_r$).

As stated in section 5.4, all specimens were introduced to hydrostatic stress state by applying confining pressure and vertical stress at the same magnitude and rate. Afterwards, the vertical displacement was increased at a constant rate ($0.07 \text{ mm/min}$) until breakout took place (Fig. 11). Since no supporting system was deployed inside the borehole, $P_w$ was considered to be zero. Based on Eqs. 2 the principal stresses at the borehole wall for 20 mm and 10 mm borehole sizes were estimated as,

$$
\sigma_1 = \sigma_{zz} = \sigma_v - 2v(\sigma_x - \sigma_y) \cos 2\theta = \sigma_v + 2v(\sigma_h - \sigma_h) = \sigma_v \quad (18)
$$

$$
\sigma_2 = \sigma_{\theta\theta} = \sigma_x + \sigma_y - 2(\sigma_x - \sigma_y) \cos 2\theta - P_w = 3\sigma_{conf} - \sigma_{conf} - P_w = 2\sigma_{conf} \quad (19)
$$

$$
\sigma_3 = \sigma_{rr} = P_w = 0 \quad (20)
$$

It should be mentioned that $\sigma_{\theta\theta}$ was also calculated based on Eq. 2a for two different borehole sizes (i.e. 10 mm and 20 mm). The results showed that the range of changes in $\sigma_{\theta\theta}$ value calculated by these two different methods did not affect the results associated with the failure criteria for the borehole sizes used in the current test program. Thus, in order to reduce the number of expressions Eqs. 18-20 were used for subsequent calculations.
6.2.1 Coulomb failure domain

Different general expressions have been phrased to use the Coulomb criterion for a drilled borehole. In order to investigate the capability of the Coulomb criterion for the performed laboratory tests conditions (second stress state in Eq. 16), induced stresses on an element at the borehole wall were applied to this criterion. Introducing Eqs. 18-20 into the Coulomb criterion,

\[ \sigma_z = C_0 + q \sigma_r \]  \hspace{1cm} (21)

Since \( \sigma_r = P_w \) is zero in an unsupported borehole,

\[ \sigma_1 = \sigma_v = C_0 = \frac{(2C \cos \phi)}{(1 - \sin \phi)} = \text{constant value} \]  \hspace{1cm} (22)

According to the Eq. 22, the maximum principal stress (\( \sigma_1 \)) takes a constant value in the \( \sigma_1 - \sigma_3 \) domain when there is no pressure acting inside the borehole. While the test results showed that increasing the intermediate stress (\( \sigma_2 = \sigma_{\theta \theta} \)) imposes larger magnitude for \( \sigma_z \), which is not the case in the Coulomb criterion. Likewise, for the third stress state in Eq. 16 (i.e. \( \sigma_\theta \geq \sigma_z \geq \sigma_r \)), \( \sigma_1 = \sigma_{\theta \theta} \) will also take a constant value if there is no pressure inside the borehole. However, it is possible to derive a relationship between \( \sigma_1 \) and \( \sigma_3 \) using the Coulomb criterion expression for the first stress state in Eq. 16 since both \( \sigma_1 \) and \( \sigma_3 \) take non-zero values.
Thus, the Coulomb criterion could not properly predict this specific condition that may take place due to drilling a borehole before applying mud pressure. It is worth to mention that when applying an in-hole support pressure, $P_w$, the principal stress states will transform from one case to another in Eq. 16 and 17 and obviously the Coulomb criterion may not results in a constant value in the $\sigma_1 - \sigma_3$ space for other cases.

### 6.2.2. Drucker-Prager failure domain

As mentioned in the previous section, the second case from Eq. 16 was experimentally studied by conducting TWHC tests. The effective shear stress and octahedral normal stress were re-calculated based on an element at the borehole wall as follow,

$$\sigma_{oct} = \frac{\sigma_v + 2\sigma_{conf}}{3}$$  \hspace{1cm} (23)

$$\tau_{oct} = \frac{1}{3} \sqrt{2\sigma_v^2 + 8\sigma_{conf}^2 - 4\sigma_v\sigma_{conf}}$$  \hspace{1cm} (24)

The data from experimental tests were used to calculate the $\tau_{oct}$ and $\sigma_{oct}$ on the borehole wall in order to represent the trend of data in the $\tau_{oct} - \sigma_{oct}$ stress domain. Fig. 12 shows the derived octahedral shear stress and the mean normal stress on the borehole wall in different cement contents ($w_c$) and confining pressures. As can be seen in Fig. 12, the effect of borehole size and cement content are considerable and, therefore, it is not possible to present a single equation as a failure envelope based on the $\tau_{oct} - \sigma_{oct}$ space for different $w_c$ values. Table 5 presents the constants for the linear failure envelopes in the $\tau_{oct} - \sigma_{oct}$ space together with the corresponding correlation coefficients.

**Table 5** Constants of the $\tau_{oct} - \sigma_{oct}$ domain (Drucker-Prager) shear failure equation for an element on a borehole wall obtained from laboratory tests on TWHC specimens with different $w_c$.

<table>
<thead>
<tr>
<th>Hole size (mm)</th>
<th>10</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>6</td>
<td>0.8008</td>
<td>0.6581</td>
</tr>
<tr>
<td>7</td>
<td>0.8644</td>
<td>0.7078</td>
</tr>
<tr>
<td>8</td>
<td>1.0401</td>
<td>0.7028</td>
</tr>
</tbody>
</table>
6.2.3. Mogi failure domain

According to Mogi (2007), to study the failure behaviour of the borehole in a TWHC, the stress components in the $\tau_{oct} - \sigma_{m,2}$ domain on an element on the borehole wall should be calculated. Fig. 13 represents the experimental data in the $\tau_{oct} - \sigma_{m,2}$ space. It demonstrates that by using the Mogi hypothesis almost all data obtained for different borehole sizes and cement contents can be located onto a single line, unlike the Drucker-Prager stress domain. Table 6 shows the constants for six lines corresponding to three different $w_c$ values for 10 mm and 20 mm borehole specimens. This shows that, it is possible to present a single line equation in terms of octahedral shear stress and effective mean normal stress for poorly cemented sands having three different cement contents used in this study. Fig. 13 shows that under a given confining pressure with an increase in the cement content the failure points move up while they are still placed on the same line. Table 6 also shows that the correlation coefficients are closer to 1 when using the Mogi criterion.
Fig. 13. Stresses on an element of a borehole wall in TWHC specimens analysed in the Mogi stress domain

Table 6 Constants of the $\tau_{oct} - \sigma_{m,2}$ domain (Mogi) shear failure equation for an element on a borehole wall obtained from laboratory tests on TWHC specimens with different $w_c$

<table>
<thead>
<tr>
<th>Hole size (mm)</th>
<th>10</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement %</td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>6</td>
<td>-0.0409</td>
<td>0.832</td>
</tr>
<tr>
<td>7</td>
<td>0.1107</td>
<td>0.804</td>
</tr>
<tr>
<td>8</td>
<td>0.17</td>
<td>0.798</td>
</tr>
</tbody>
</table>

In order to compare the results from the TWHC specimens with 20 mm and 10 mm diameter boreholes, the test data in the $\tau_{oct} - \sigma_{m,2}$ space for both borehole size specimens were superimposed in Fig. 13. As expected, due to size-scale effect suggested by Carpinteri (1996), decreasing the size of the borehole diameter elevates the failure stress of the specimen. Thus, the specimen with 10 mm diameter borehole could tolerate more stress before its failure than the one with the 20 mm diameter borehole. However, as presented in Fig 13, results from tests on specimens having two different borehole sizes show similar failure envelopes. It should be mentioned that although changing the borehole size and $w_c$ affect the response of the specimens in the Mogi failure domain, the amount of change observed in this criterion is less significant compared with those obtained by the Drucker-Prager failure domain.
Fig. 14 compares the results for solid and TWHC specimens in the Mogi failure domain. It shows that the results for solid and TWHC did not align on a single line and solid specimens show more dependency on $w_c$. This difference in the behaviour could be due to the following,

1- Since the solid specimens showed strain hardening behaviour, the maximum vertical stress was calculated based on 1% of axial strain, but in TWHC specimens this was not the case and exact peak stress was captured during the test.

2- The stress conditions in solid and TWHC specimens were different. In TWHC, stress condition was $\sigma_1 > \sigma_2 > \sigma_3 = 0$, but in solid specimens $\sigma_1 > \sigma_2 = \sigma_3 > 0$ and this may cause the difference in behaviour.

![Fig. 14. Comparison between the results from the solid specimens versus the TWHC specimens in the Mogi failure domain](image)

According to Table 6, a single average line for the poorly cemented sand mixture, which was investigated in the current study, can be presented as follows,

$$\tau_{oct} = 0.82566 \sigma_{m,2} + 0.01296$$ \hspace{1cm} (25)

On the other hand, the trend line of all the data associated with 10 mm and 20 mm borehole size specimens can be plotted together and the best fitted equation is graphically found to be the following,

$$\tau_{oct} = 0.8186 \sigma_{m,2} + 0.0422$$ \hspace{1cm} (26)
The correlation coefficient corresponding to Eq. 26 is 0.997, which shows a good estimation of the obtained experimental data.

According to Eq. 25 and Eq. 26, if the stress magnitudes on the borehole wall are below this line, the borehole will stay stable without any in-hole pressure ($P_w = 0$). If the stress values lie on this line the borehole failure takes place, while the points above and beyond this line do not have any physical meaning. This failure envelope is valid for the poorly cemented sand formations with particle size distribution corresponding to ones at the drilling site in Burra, South Australia and within the range of $w_c$ values considered in this study. However, considering the presence of a sandy formation at up to the depth of 200 m, this criterion can be applicable for exploration boreholes which are being drilled through poorly cemented formations. Then, stabilisation systems such as the mud pressure can be designed based on the far-field stress configuration.

### 6.2.4. Parameters of the Coulomb criterion based on the Mogi failure domain

In order to estimate the Coulomb shear strength parameters based on the Mogi failure domain in the case of an unsupported borehole, results from laboratory tests were used. According to the definition of $\tau_{oct}$ and stress invariants, Eq. 15 can be rearranged as follows,

$$(I_1^2 - 3I_2)^{0.5} = A + B(I_1 - \sigma_2) \tag{27}$$

where A and B are material property constants. Generally, one of the following three conditions may apply to the intermediate stress as,

$$\begin{cases} 
\sigma_\theta = \sigma_2 \\
\text{or,} \\
\sigma_r = \sigma_2 \\
\text{or,} \\
\sigma_z = \sigma_2
\end{cases} \tag{28}$$

Stress invariants remain unchanged in all three cases. It is inferred that the only term in Eq. 27 that varies based on the stress conditions is the intermediate principal stress. Similar to the previous section and considering the laboratory test boundary and initial conditions, the principal stresses are similar to Eq. 18-20. Thus, stress invariants can be written as,

$$I_1 = \sigma_v + 2\sigma_{conf} \tag{29}$$
Introducing Eq. 30 into Eq.29,

\[ I_2 = \frac{1}{2} \left[ I_1^2 - \left( \sigma_v^2 + 4\sigma_{conf}^2 \right) \right] \]  

According to Eq. 27,

\[ S = A + B(X) \]  

where \( S = \sqrt{\sigma_v^2 + 4\sigma_{conf}^2 - 2\sigma_v\sigma_{conf}} \)  

\[ X = I_1 - \sigma_2 \]  

It is worth to mention that by using the experimental data to plot the \( S \) versus \( X \), the Mogi criterion can be related to the constant parameters in the Coulomb criterion (i.e. \( c \) and \( \phi \)). Fig. 15 shows the data in the \( S \)-\( X \) space.

**Fig. 15.** The Mogi failure function in terms of \( S \) and \( X \) (Eq. 33 and Eq. 34) for obtaining the Coulomb shear parameters based on the experimental data from the specimens with 10 mm and 20 mm diameter boreholes.

In this case according to the linear Mogi criterion one can calculate \( \tau_{oct} \) as Eq. 24 and then,

\[ \sigma_{m2} = \frac{\sigma_v + \sigma_{rv}}{2} = \frac{\sigma_v}{2} \]  

(35)
However, for the second case in Eq. 16 (\(\sigma_\theta \geq \sigma_z \geq \sigma_r\)) \(\tau_{oct}\) and \(\sigma_{oct}\) remain unchanged and only \(\sigma_{m,2}\) changes as,

\[
\sigma_{m,2} = \frac{1}{2} (2\sigma_{conf}) = \sigma_{conf}
\]

(36)

Therefore once the stress condition varies, it is essential to apply changes to the mean effective stress. Also, the comparison between \(\tau_{oct}\) and \(S\) from Eq. 33 reveals that these two terms are proportional and it is possible to estimate the Coulomb constants from Eq. 13 when considering the stresses on the borehole wall in different stress conditions.

6.2.5. \(\tau_{max}\) in the Coulomb criterion based on the Mogi failure domain

In order to relate the Mogi constants to the Coulomb criterion, the relationship between the maximum shear stress on the borehole wall and the principal stresses must be identified. According to the Coulomb criterion,

\[
\tau_{max} = \frac{1}{2} (\sigma_1 - \sigma_3)
\]

(37)

As mentioned earlier for the linear Mogi domain in Eq. 15 it can be written,

\[
\sqrt{\frac{2}{3}} \sqrt{(\sigma_v - 2\sigma_{conf})^2 + 2\sigma_v \sigma_{conf}} = a + b \left(\frac{\sigma_2}{2}\right)
\]

(38)

Solving the \(\tau_{max}\) equation in terms of \(\sigma_2\) gives,

\[
\tau_{max} = \frac{-18ab + 4\sigma_2 \pm \sqrt{(18ab + 4\sigma_2)^2 - 4(9b^2 - 8)(9a^2 - 2\sigma_2^2)}}{2(9b^2 - 8)}
\]

(39)

This equation shows that it is possible to find the maximum shear stress in the Coulomb criterion by applying the Mogi failure domain for an unsupported borehole. Thus, it is possible to derive the \(\tau_{max}\) based on the laboratory test data and compare them with the Coulomb equation under different stress conditions.

7. Conclusion

Number of laboratory tests was performed on synthetic solid and thick-walled hollow cylinder (TWHC) specimens to investigate the failure behaviour of poorly cemented sands
adjacent to the borehole wall. Different cemented-sand mixtures were considered to prepare poorly cemented sand specimens closely resembling grading and textural properties to the samples collected from a problematic drilling site in South Australia. Triaxial tests at different confining pressures were performed on solid specimens to find the shear strength properties of the mixtures.

The normal faulting stress condition, i.e. $\sigma_z \geq \sigma_\theta \geq \sigma_r$, was experimentally studied on TWHC specimens to identify the failure criterion which best describes the shear failure behaviour of the specimens. The values obtained for $\tau_{oct}$, $\sigma_{oct}$ and $\sigma_{m,2}$ based on the data from the laboratory tests on TWHC specimens were introduced into the Coulomb, Drucker-Prager and Mogi failure domains. In order to verify the derived failure equation, the data obtained from the triaxial tests on TWHC specimens were used. The results showed that:

- Although the Coulomb criterion can be used as a rock failure criterion under triaxial stress state, applying this criterion to borehole wall analysis could not represent the failure behavior of the TWHC specimens under specific stress conditions. This criterion showed a constant value of $\sigma_1$ under different confining pressures which was not observed during the tests.
- The Drucker-Prager criterion could not fully represent the failure behavior of an element on the borehole wall. When using this domain all failure points for different borehole sizes and cement contents could not be fitted into a single failure envelope within the range of tests performed in this study.
- Implementing the Mogi criterion in a borehole stability model showed that all the results can be represented with a single straight line. Therefore, influence of size-scale effect on boreholes and different cement contents were not significant within the test condition practiced in this study.
- It was shown that for an unsupported borehole the Coulomb shear strength parameters can be calculated based on the Mogi stress domain. To achieve this, the principal stresses on the borehole wall should be calculated based on the stress state condition.
- Two borehole sizes, 10 mm and 20 mm diameters, were considered. The results showed that due to the size-scale effect, the failure stress in specimen with 10 mm borehole was higher than the one with 20 mm diameter borehole. However, the results showed that if the data from the 10 mm borehole specimen are superimposed into the
\[ \tau_{oct} - \sigma_{m,2} \] domain together with the result from the 20 mm borehole specimen, all the data would lie on a single linear equation graph.

Acknowledgements

This work has been supported by the Deep Exploration Technologies Cooperative Research Centre whose activities are funded by the Australian Government’s Research Programme. This is DET CRC Document 2014/390.

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CHAPTER 4

Background

Chapter 4 presents the fourth manuscript titled ‘Effect of localised zones in borehole failure in poorly cemented sandy formations’. The development of localised breakout zones and compaction bands at the borehole wall was identified by simultaneous real-time monitoring and deformation measurement during the tests. Real-time monitoring showed that the cracks were formed on the borehole wall with no signs of the formation of compaction bands at the initial stage of the crack development. Applying higher stresses than a certain threshold for the considered TWHC specimens led to the breakage of grain bondings within a narrow localised band which was normal to the maximum principal stress. This allowed the detached and loosened grains to rearrange to form a lower porosity compaction band and this initial breakage intensified the stress concentration behind the breakout tip. As microcracks extended, the stress concentration moved toward the specimen’s outer boundary releasing more sand particles and progressing the breakout zone. This process continued until the state of equilibrium was established between the formed sand arch and induced stresses acting on the localised band tip.

Scanning electron microscopy (SEM) imaging was performed on the corresponding sections of the tested TWHC specimens in order to determine the geometry of localised zones at the borehole wall. SEM imaging showed that microcracks extended into the borehole wall perpendicular to the axis of the vertical stress and then reoriented outward into the specimen following trajectories of the confining pressure. Sand particles which were debonded at the localised bands due to the intense microcracking fell into the borehole. Moreover, SEM studies showed that when drilling a hole in a cylindrical solid specimen prior to applying the far-field stresses, microcracks could develop on the borehole wall and an area around the borehole will be disturbed. According to the SEM analyses, no crushed grains were observed in the localised zone and the sand particles remained intact under the range of applied stresses and along the border of the breakout zone located on the boundary of particles. This allows suggesting that stresses that caused debonding and repacking were at levels lower than those required to induce grain damage. Results showed that the depth and width of the breakout zone increased with increasing the confining pressure ($\sigma_h = \sigma_H$) and the $w_c$ values affect the shape of the breakout zone. Also the depth and width of the breakout zone for the specimens with borehole diameter of 20 mm were generally almost 20% and 70% larger than those for
the 10 mm ones respectively. Investigations of the grain size distribution for sand particles retrieved during testing showed that they contain more of the fine grain sands versus the grain size distribution used in the specimen preparation which contained 50% of coarse sand grains.

**List of Manuscripts**

## Statement of Authorship

<table>
<thead>
<tr>
<th>Title of Paper</th>
<th>Effect of localised zones in borehole failure in poorly cemented sandy formations</th>
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<tr>
<td>Publication Status</td>
<td>○ Published, ○ Accepted for Publication, ○ Submitted for Publication, ○ Publication style</td>
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</table>

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By signing the Statement of Authorship, each author certifies that their stated contribution to the publication is accurate and that permission is granted for the publication to be included in the candidate’s thesis.

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Effect of localised zones in borehole failure in poorly cemented sandy formations

S.S. Hashemi*, A.Taheri*, N. Melkoumian*

Abstract

Poorly cemented sands are mainly located in areas where layers of unconsolidated formations exist. Drilling a borehole in the ground causes stress perturbation and induces tangential stresses on the borehole wall. If the cohesion between sand particles generated by existing cementation is not high enough, the tensile stress concentration may cause grain debonding and, consequently, borehole breakout. In this study a series of solid and thick-walled hollow cylinder (TWHC) laboratory tests was performed on synthetic poorly cemented sand specimens. The applied stresses were high enough to generate breakout on the borehole wall. The development of localised breakout zones and compaction bands at the borehole wall was identified by simultaneous real-time monitoring and deformation measurement during the tests. The results from the video recording of the tests showed that a narrow localized zone develops in the direction of the horizontal stress, where stress concentration causes the full breakout in specimens. Dilation occurred at lower confining pressures in TWHC specimens and contracting behaviour was observed during the onset of shear bands at higher pressures. Scanning electron microscopy (SEM) studies showed that sand particles stayed intact under the applied stresses and micro- and macrocracks develops along their boundaries. The SEM imaging was also used to investigate and characterize pre-existing microcracks on the borehole wall developed due to the specimen preparation. It showed that boring the solid specimen in order to produce a TWHC specimen can generate microcracks on the borehole wall prior to testing which affects the process of borehole failure development during the test.

Key words: borehole stability; experimental study; thick-walled hollow cylinder; poorly cemented sand; localised zones

Nomenclature

<table>
<thead>
<tr>
<th>TWHC</th>
<th>thick-walled hollow cylinder</th>
<th>(D_{50})</th>
<th>mean grain diameter (mm)</th>
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<tbody>
<tr>
<td>(w_c)</td>
<td>weight ratio of cement to sand grains</td>
<td>(C_u)</td>
<td>coefficient of uniformity</td>
</tr>
<tr>
<td>(\delta)</td>
<td>weight ratio of coarse to fine sand</td>
<td>(q)</td>
<td>deviator stress (MPa)</td>
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<tr>
<td>(w_w)</td>
<td>weight ratio of water to sand grains</td>
<td>(\sigma_{conf})</td>
<td>confining stress (MPa)</td>
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<td>(\sigma_{\theta\theta})</td>
<td>tangential stress (MPa)</td>
<td>(\phi)</td>
<td>angle of internal friction (deg)</td>
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<td>(\sigma_{rr})</td>
<td>radial stress (MPa)</td>
<td>(\sigma_{m,2})</td>
<td>mean effective normal stress (MPa)</td>
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<td>(\sigma_z)</td>
<td>vertical stress (MPa)</td>
<td>(\tau_{oct})</td>
<td>octahedral shear stress (MPa)</td>
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<tr>
<td>(\tau)</td>
<td>shear stress (MPa)</td>
<td>(\sigma_{oct})</td>
<td>octahedral normal stress (MPa)</td>
</tr>
<tr>
<td>(D_i)</td>
<td>internal diameter in a hollow cylinder specimen (mm)</td>
<td>(\gamma_d)</td>
<td>dry density (gr/cm(^2))</td>
</tr>
</tbody>
</table>
1. Introduction
Drilling a borehole in the ground is a common method for measuring the spatial position of underground formations, characteristics of faults and fractures, extracting oil and gas and also for monitoring the petrographic assessment of borehole walls. Several borehole instability problems during and after borehole completion have been reported by drilling companies in Australia. Most of these reports are related to fields containing poorly cemented sands at depths up to 200 m. When a borehole is drilled through such a weak formation, instability or grain bonding breakage at the borehole wall may occur due to the relaxation of the pre-existing in situ stresses underground. Tangential stresses develop around the borehole and the radial stress tends to become zero on the borehole wall in case no supporting system is available (Hagin and Zoback 2004, a,b). If the strength of the borehole wall is not high enough, the borehole may collapse. This is due to the spontaneous re-establishment of the equilibrium in the ground after the borehole excavation. A large number of mines, as well as unconventional oil and gas reservoirs are located in formations containing geologically young and unconsolidated sands or sandstones where the particles are either weakly bonded, or even un-cemented all around the world (Musaed et al., 1999). Mavko and Jizba (1991) showed that the presence of poorly cemented sandstone increases the potential of borehole instability.

A well designed experimental study can be a valuable approach for acquiring in-depth knowledge on the failure behaviour of granular materials. Conducting laboratory studies on the development of localized damaged zones in poorly cemented sand specimens can help to better understand the structures of shear, compaction and dilation zones that influence the borehole instability in such formations. These localised zones develop due to grain dislocation and crushing which may leads to formation of compaction bands and porosity reduction or dilatant zones. Haimson and Herrick (1986) and Maloney and Kaiser (1989) presented a clear correlation between the borehole breakout dimensions and the in situ stress levels. Haimson and Kovacich (2003) and Lee (2005) studied borehole instability in high-porosity Berea, Tablerock and Mansfield sandstones. They found that a narrow zone ahead of a fracture-like breakout tip underwent apparent localized grain debonding and compaction. They showed that when the borehole size is considerably larger, a fracture-like breakout can extend to sizable distances, creating a failure hazard. Nouri et al. (2006) suggested that in soft rocks a pure shear failure or a combined shear-spalling process seems to occur, while in hard rock formations a combination of shear and extension fractures was observed. In the case of weak rocks, material was totally plastified, while in some cases when the rock material was
stronger the zone remained relatively intact. Haimson and Klaetsch (2007) showed that in natural high porosity sandstones grains are bonded along narrow contact areas while in low porosity sandstones particles’ bonds are all around them. Therefore, in high porosity rocks grains will be dislodged when the in situ stresses reach the strength of the narrow cementation zone (e.g. Navajo and Aztec sandstones). A number of researchers have performed laboratory tests aimed to produce compaction bands by applying high triaxial compressive stresses (Haimson, 2007; Olsson and Holcomb, 2002; Vajdova and Wong, 2003; and Olsson, 1999). Haimson (2006) and Vajdova and Wong (2003) reported the formation of compaction bands in laboratory tests on hard rocks. Vajdova and Wong (2003) performed triaxial tests on slotted cylindrical sandstones and found a high stress concentration at the notch edge, which resulted in compaction bands. Katsman and Aharonov (2006) simulated the compaction bands with a network of springs and stated that compaction bands can be generated around the borehole. It is worth noting that the localised compaction bands are important factors in actual field conditions, as they can induce local permeability reduction in underground layers. Also, Desrues et al. (1996) suggested that the localisation phenomenon causes failure in porous soft rocks especially at low stresses and temperatures.

The thick-walled hollow cylinder (TWHC) test is a common method for simulating stress and strain states adjacent to underground excavations in order to study the failure behaviour of geomaterials under different stress paths. The specific shape and loading paths that can be applied to these specimens make them more popular for simulating in situ stress conditions around underground openings such as boreholes, wellbores and tunnels and for reproducing various combinations of stress paths, in comparison to any other available experimental method (Daemen and Fairhurst, 1971). Hoskins (1969) investigated the strengths of five different rocks in the form of TWHCs. Alsayed (2002, 1996) utilised TWHC specimens to study the effect of different loading conditions on the behaviour of hard rocks. Santarelli and Brown (1989) and Perie and Goodman (1988) investigated the macroscopic failure mechanisms of synthetic rocks made of gypsum cement by conducting TWHC tests.

In this paper a series of TWHC laboratory tests was designed and conducted under fully controlled conditions. Real-time monitoring of sand debonding was carried out during the test in order to study the failure mechanism of poorly cemented sands around the borehole. Micromechanical behaviour of the specimens when inducing a localised zone on the borehole wall under different stress conditions was investigated using scanning electron microscopy (SEM). This study aims to provide a more realistic and comprehensive view on the
behaviour of poorly cemented sands and can be helpful when designing an adequate supporting system to keep the borehole open during the service period.

2. Stress distribution around a TWHC

Depending on the thickness of a TWHC specimen, stresses developed at its wall due to applied uniform stresses can be analysed using different approaches. Unlike thin-walled hollow cylinders, stress distribution in TWHC specimens is not homogenous. There are closed-form solutions for calculating stresses and strain in the TWHC specimens using the theory of elasticity which can be found in different literatures (e.g. Jaeger et al., 2007; Obert and Duvall, 1967). For a TWHC with an inner diameter of $D_i$, outer diameter of $D_o$ and length of $L$ subjected to an axial force ($F$) and to uniform internal stress ($S_i$) and external stress ($S_o$), the principal stresses in a cylindrical coordinates system at any point of radial distance $r$ can be defined as (Jaeger et al., 2007):

\[
\sigma_{\theta\theta} = \frac{S_oD_o^2 - S_iD_i^2}{D_o^2 - D_i^2} + \frac{(S_o - S_i)D_i^2D_o^2}{4r^2(D_o^2 - D_i^2)} \quad (1a)
\]

\[
\sigma_{rr} = \frac{S_oD_o^2 - S_iD_i^2}{D_o^2 - D_i^2} - \frac{(S_o - S_i)D_i^2D_o^2}{4r^2(D_o^2 - D_i^2)} \quad (1b)
\]

\[
\sigma_z = \frac{4F}{\pi(D_o^2 - D_i^2)} + \frac{S_iD_i^2}{(D_o^2 - D_i^2)} \quad (1c)
\]

where $\sigma_{\theta\theta}$, $\sigma_{rr}$ and $\sigma_z$ are the tangential, radial and axial principal stresses respectively.

According to Eq. 1 the tangential stress concentration can lead to the debonding of sand grains and result in the development of an inelastic damaged zone around the borehole if there is no adequate cementing agent present at the interface of sand grains (Ewy and Cook, 1990a).

3. Exploration drillings

Exploration boreholes are being drilled throughout Australia to discover new mineral reserves leading to potential mining activates. One of these drilling sites is located in Burra, South Australia where exploration boreholes are being drilled through a poorly cemented sandy formation. The majority of these boreholes are 25 cm to 30 cm in diameter with lengths varying from 80 m – 250 m depending on the mine exploration plan. The subsurface investigations of sediments and borehole surveys show that the sediment above the bedrock is
heterogeneous and irregular, and shallower layers of the sediment are composed of silt and fine sand. This layer is underlain by dark grey plastic clay, and then by a poorly cemented sandy layer (Fig. 1). Air core and reverse circulation (RC) drilling methods were used to drill exploration boreholes at this site. These are dry drilling methods that conventionally have been used for drilling through a soft ground in Australia (Fig. 2). The drill cuttings are being removed by injecting pressurized air. This is done by pushing 425-550 l/s of air at 2000-2400 kPa down the hole through the annular opening between the inner tube and the drill rod. The cuttings are then conveyed to the surface up the inner tube where they pass through the sample collection system and collected if required. As drilling proceeds rods are added to the top of the drill string. When the drilling string reaches the poorly cemented sandy layer there is a considerable potential for the borehole to collapse due to the weak bonding between the sand particles. Occasionally, the actuator is unable to restart and rotate the rod, and the drilling rod gets stuck in the borehole if the gap between the drilling rod and the borehole wall gets filled up with sand. Such problems are often encountered at depths of 70 m - 150 m. Further growth of the damaged zone or the resulting inelastic deformations can render the opening useless for its original purpose and, therefore, considerable amount of extra work and expenses are required in order to make the opening functional and safe again.

Fig. 1. Geological cross section near Burra, South Australia (Hashemi et al. 2014)
4. Material and methods

4.1. Laboratory experimental studies

Laboratory studies on the damage development in poorly cemented sand specimens allow us to develop a better understanding on the mechanical behaviour of such materials under in situ stresses in the vicinity of a drilled borehole. Uniaxial compressive strength (UCS) and triaxial tests on solid and thick-walled hollow cylinders (TWHC) were performed under various stress conditions. According to the drilling site investigations, drilling through a shallow formation results in relatively extensive sand grain debonding at the borehole wall due to lack of strong particle cementation and consolidation. Various mixtures of sand, cement and water were casted into specifically designed and prefabricated cylindrical moulds to prepare suitable specimens for laboratory tests. Fig. 3 shows the facilities that were designed and manufactured to perform the tests on synthetic TWHC samples. Specimens were designed to fit into a HQ Hoek triaxial cell of 63.5 mm diameter and 127 mm length. Various cement and water contents \((w_c, w_w)\) and coarse to fine sand weight ratios \((\delta)\) were considered to achieve the grading and texture most closely resembling the formation properties at the drilling site. The whole process of experimental studies including the Hoek cell modification, developing specimen preparation facilities, specimen preparation and conducting the tests was labour intensive and time consuming, and took more than 14 months to complete.
Fig. 3. Facilities manufactured for laboratory tests; (a) special moulds for preparing TWHC specimens, (b) special device for compacting the mixture in the mould uniformly, (c) wooden base to carry the triaxial cell weight, (d) Platen designed to deploy the micro camera.

4.2. Specimen Preparation

4.2.1. Mixture

Most of samples collected from the drilling site were of fine and sub-angular sand grains with random orientations. The pale yellowish-grey specimen was produced at the laboratory and its grading size and fabric closely resembles that of poorly cemented sands in top 200 m of the exploration field formation. It is a near-uniform mixture of fine-grained sand, Portland cement and water. Care was taken to produce homogenous specimens; a small mixer was used to ensure that the sand grains, cement and water mixed perfectly. The total period from the start of mixing until sealing the moulds was strictly maintained between 30-40 minutes.

Natural silica sands of two different size ranges were used for reproducing poorly cemented sandy layer at the drilling site. The mean diameter value ($D_{50}$) is 0.56 mm for the silica dry sand with grain sizes between 0.425 mm - 1.4 mm and is 0.20 mm for grain sizes between 0.125 mm - 0.355 mm. Coefficient of uniformity ($C_u = D_{60}/D_{10}$) for coarse and fine sand grains is 1.452 and 2.268 respectively.
The optimum water ratio for cement-sand mixture was evaluated by using the standard Proctor test compactor hammer; with the energy of 0.55 $Nm/cm^3$ was achieved 9.7–10.3%. Fig. 4 shows the compaction test results for the prepared mixture both for the cases when cemented powder was used and without it. The results demonstrate that considering a cement-sand mixture or only sand grains does not have a significant effect on the optimum water content and dry density. Based on the previous studies on poorly cemented sands (Hashemi et al., 2013; Hashemi et al., 2014; Saidi et al., 2003) 8 days were identified as the optimal curing time for the specimen, which comprised of keeping the specimen for 5 days in the mould and 3 days outside of it in a plastic wrap at temperatures between $18 - 22^\circ C$.

Fig. 4. Proctor compaction test of silica sand with and without Portland cement powder ($c/s=6\%$) Water content, $w_t$ (% by dry weight of grains).

4.2.2. Cement content

Saidi et al. (2003) showed that Portland cement is a suitable substitute for the natural cement agent to simulate poorly cemented specimens. A wide range of cement content ($w_c$) values have been suggested in different studies for creating a poorly cemented synthetic sandstone (Saidi et al., 2004; Saidi et al., 2003; Younessi et al., 2013; Kongsukprasert, 2003). For instance, Kongsukprasert (2003) used a maximum of 2.5% Portland cements to prepare poorly cemented specimens, while Saidi et al. (2004) used 9-18%. Gueguen and Palciauskas (1992) have stated that the minimum $w_c$ is reached at $\delta = 2.5$. However, based on our observations, for the considered grading sizes of sand particles the specimens cannot be safely demoulded even after 8 days of curing time when $w_c \leq 2.5\%$. 
Various mixtures with respect to different $w_c$ and $\delta$ values were prepared and examined. Finally, for the above mentioned sand grain size ranges, $w_c = 6\%, 7\%$ and $8\%$ were selected. It should be mentioned that since the grading size distribution plays a key role in the strength of specimens, $w_c$ should be selected based on used grain size distribution for different investigations.

4.3. Testing set-up

A modified Hoek cell with a fitted micro camera was designed and manufactured at the University of Adelaide. The fitted camera allowed real-time monitoring and video recording of the borehole walls and of the process of particle bonding breakage during the tests on TWHC specimens. The modified hollow cylinder test cell was built by applying modifications to the HQ Hoek triaxial cell into which a TWHC specimen of $63\text{mm} \times 127\text{mm}$ can be fitted. These facilities were synchronised with a precise system of applying confining pressure at low level stresses (maximum $6 \text{ MPa}$) with no leakage or intrusion. According to the UCS tests on cell platens, the strain of the platens is less than $0.005\%$ when subjected to a maximum of $100 \text{ kN}$ loading force which is quite higher than the strength of the prepared TWHC specimens.

The specimen loading process comprised of several stages. First, vertical and confining stresses acting on the specimen were simultaneously increased up to a certain stress level to simulate hydrostatic stress condition. Then, the vertical load was increased at constant displacement rate of $0.07 \text{ mm/min}$. The level of the confining pressure was different for each test to induce a predetermined stress at the external wall of the specimen. Table 1 presents the schedule of the main conducted tests. It should be noted that different displacement rates ($0.02$-$0.1 \text{ mm/min}$) were considered to identify the best suited one for capturing the process of grain debonding.

<table>
<thead>
<tr>
<th>Type of specimens</th>
<th>$w_c$</th>
<th>$\delta$</th>
<th>$\sigma_{\text{conf}}$ (MPa)</th>
<th>Displacement rate (mm/min)</th>
<th>$w_w$</th>
<th>Curing time (days)</th>
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<tr>
<td>Solid cylinder</td>
<td>$6%, 7%, 8%$</td>
<td>1</td>
<td>1-5</td>
<td>0.07</td>
<td>10%</td>
<td>8 days</td>
</tr>
<tr>
<td>TWHC</td>
<td>$6%, 7%, 8%$</td>
<td>10 &amp; 20</td>
<td>1-5</td>
<td>0.07</td>
<td>10%</td>
<td>8 days</td>
</tr>
</tbody>
</table>

Table 1 The plan of triaxial tests performed on solid and TWHC poorly cemented sand specimens
5. Test Results

5.1. Stress-strain behaviour in solid cylindrical specimens

Triaxial tests were conducted on the final designed solid specimens with different \( w_c \) values (i.e. 6%, 7% and 8%) to identify the shear strength properties under various stress conditions. Fig. 5 a-c presents the stress-strain diagram for specimens with different \( w_c \) and at various magnitudes of confining pressure. According to Fig. 5, increasing the confining pressure causes an increase in the peak strength. However, changes in the confining pressure have a minimal effect on the stiffness of the specimens. Also, as it is inferred from Fig. 5 the behaviour of the material is strain hardening as deviatoric stress, \( q \), increases with an increase in axial strain, \( \varepsilon_a \), within the ranges of strains applied to each sample. Bésuelle et al., (2000) showed that in Vosges sandstone with 22% porosity, the deviatoric strength increases up to a certain level with increasing the confining pressure and then decreases in 50-60 MPa of confining pressure, while the global volumetric strain is continuously compacting up to the onset of localisation. However, our test results showed that the deviatoric strength continuously increases with increasing the confining pressure within the range of the applied confining pressures. The strain hardening trend of the poorly cemented material may cause this behaviour, while the observed behaviour of the Vosges sandstone was a strain softening after the peak stress. It is worth mentioning that applying confining pressure, more than 5MPa to the solid specimens was not possible in the first stage of the testing process due to the weakness of specimens. Table 2 represents the mechanical properties of poorly cemented sand specimens.

![Stress-strain diagram](image-url)
**Fig. 5.** Stress versus axial strain behaviour of solid specimens subjected to triaxial testing (a) $w_c = 6\%$ (b) $w_c = 7\%$ (c) $w_c = 8\%$.

<table>
<thead>
<tr>
<th>Cement content</th>
<th>Porosity ($\alpha$)</th>
<th>Tangent elastic modulus $E_{tan}$ (GPa)</th>
<th>Uniaxial compressive strength UCS (MPa)</th>
<th>Poisson’s ratio $\nu$</th>
<th>Coulomb parameters $c$ (MPa)</th>
<th>$\phi$ (°)</th>
<th>Bulk density $\rho$ ($kg/m^3$)</th>
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</thead>
<tbody>
<tr>
<td>$w_c = 6%$</td>
<td>26 ± 3%</td>
<td>2.262</td>
<td>0.62</td>
<td>0.294</td>
<td>1.11</td>
<td>29.67</td>
<td>1954</td>
</tr>
<tr>
<td>$w_c = 7%$</td>
<td>26 ± 2%</td>
<td>2.465</td>
<td>1.05</td>
<td>0.250</td>
<td>1.38</td>
<td>30.06</td>
<td>1969</td>
</tr>
<tr>
<td>$w_c = 8%$</td>
<td>26 ± 2%</td>
<td>3.360</td>
<td>1.90</td>
<td>0.247</td>
<td>1.59</td>
<td>29.67</td>
<td>1974</td>
</tr>
</tbody>
</table>

5.2. **Stress-strain behaviour of TWHC specimens**

Triaxial tests were conducted on the synthetic TWHC specimens using modified Hoek cell to study the effect of cement content, confining pressure and diameter size on borehole breakout.
and failure mechanisms under different stress conditions. Since retrieving intact specimens after the tests was not possible, real-time camera monitoring was implemented. This allowed to identify the initiation and direction of borehole breakout and to locate the damaged zone on the borehole wall. Principal TWHC tests were conducted for three different cement contents \( w_c = 6\%, 7\%, 8\% \) and two borehole diameter sizes, 10 mm and 20 mm (see Table 1). Otherwise, sample preparation procedure including compaction, grading size distribution and curing time were kept unchanged for both borehole size specimens.

Fig. 6 illustrates the results of the triaxial tests on the TWHC specimens with 10 mm diameter borehole, \( w_c \) values of 6\%, 7\% and 8\% and for three different external confining pressures. As shown in Fig. 6, unlike the solid specimens, the TWHC samples exhibit a strain softening behaviour after the peak strength and, therefore, the peak strength could be determined within the applied stress ranges. Fig. 7 shows the process of borehole failure for the applied confining pressure of more than 4 MPa prior to exerting the deviatoric stress. This could be identified by the micro camera installed inside the borehole and indicates that the first stage of the test (hydrostatic stress condition) could not be completed.

![Deviatoric stress vs. axial strain of 10 mm borehole TWHC specimens for (a) \( w_c = 6\% \) (b) \( w_c = 7\% \) (c) \( w_c = 8\% \) (d) effect of cement content on the axial and lateral strength of the TWHC for \( \sigma_{\text{conf}} = 3 \) MPa.](image)

**Fig. 6.** Deviatoric stress vs. axial strain of 10 mm borehole TWHC specimens for (a) \( w_c = 6\% \) (b) \( w_c = 7\% \) (c) \( w_c = 8\% \) (d) effect of cement content on the axial and lateral strength of the TWHC for \( \sigma_{\text{conf}} = 3 \) MPa.
Fig. 7. The process of borehole failure prior to exerting the deviatoric stress in case of applying confining pressure of more than 4.5 MPa to TWHC specimens.

Fig. 8 presents the results for the specimens with 10 mm and 20 mm diameter boreholes and shows the effect of the borehole size on the development of a localized damage zone. It shows the deviatoric stress versus axial strain for the TWHC specimens with \( w_c = 7\% \) and for various external confining pressures. As expected, the strength of the TWHC specimens with 20 mm diameter borehole is generally lower than for specimens with a smaller borehole especially in higher confining pressures due to scale effect which is suggested by Carpinteri (1996) and Aifantis, (1999). Although some variations between the results from the three test series were observed, the level of variations is trivial and appears to be considered by scatter rather than trend.

Fig. 8. Stress–strain behaviour of 10 mm and 20 mm diameter boreholes for \( w_c = 7\% \).

5.3. Volumetric strain

5.3.1. Solid cylindrical specimens
During the tests lateral and axial strains on solid and TWHC specimens were measured to study the effect of confining pressure and cement content on the volumetric strain in the prepared specimens. Fig. 9 presents the average values of the volumetric strains. It shows an initial contractancy in all confining pressures. Then, up to the maximum measured level of the deviatoric stress, either dilatancy (at $\sigma_{conf} = 1, 2$ MPa) or contractancy (at $\sigma_{conf} = 3, 4$ MPa) were observed depending on the level of the confining pressure. With increasing the confining pressure, the lateral strength of the specimens increased and the lateral dilation was lower in the same axial strain. This result is consistent with the results from Cornet and Fairhurst (1974) and Bésuelle and Rudnicki (2004). However, due to the weaker particle bondings in poorly cemented sand specimens in comparison with hard rocks, the range of confining pressures which causes the volumetric strain switches from contractancy to dilatancy is lower than those observed in previous studies conducted for hard rocks.

Fig. 9 also demonstrates the effect of different $w_c$ values on the volumetric strain. It shows that changing the cement content has a minor effect on the volumetric strain as compare to confining pressure change. For instance, for 1MPa and 2 MPa of confining pressures increasing the $w_c$ from 6% to 8% does not result in any trend divergence (from contraction to dilation) in the behaviour of the specimens.

![Fig. 9. Volumetric strain versus axial strain measured by local strain gauges for different $w_c$ values and confining pressures in solid specimens.](image)

### 5.3.2. TWHC specimens

Fig. 10 shows that at lower confining pressures (=1 and 2 MPa) the volumetric strain enters the dilatancy mode and with increasing the confining pressure the borehole breakout goes...
into the contractancy mode similar to the solid specimens. However, unlike the solid specimens, due to the presence of a borehole in the specimens, the volumetric strain curve conjugates the horizontal axis at a higher axial strain. This implies sand grain dislocations and generation of microcracks on the borehole wall in the TWHC specimens occur to release the applied stresses and re-establish the state of equilibrium. Also, according to Fig. 10 with increasing the \( w_c \) the lateral dilatancy of the specimens increased in comparison with the axial strain. According to the camera recordings, before the initiation of borehole breakout the axial contraction of the specimens with higher \( w_c \) was lower than that for the specimens with lower \( w_c \). Thus, increasing the \( w_c \) leads to a decrease in the pore spaces and applying the axial deviatoric stress induces lower contractancy in the axial direction. Therefore, in higher \( w_c \) values the lateral strain increment is more pronounced than the reduction in axial strain in poorly cemented sand specimens. Real-time borehole monitoring showed that transition from contractancy to dilatancy occurred before the initiation of particle debonding and dislocation.

![Fig. 10. Volumetric strain versus axial strain measured by strain gauges for different \( w_c \) values and confining pressures for TWHC specimens with 10 mm diameter borehole.](image)

5.4. Defects induced on the borehole wall due to drilling

Field observations during drilling operations show that sand dislocation or running sand phenomenon in poorly cemented sand layers occurs during the drilling process or a short time (up to an hour) after the borehole drilling is completed. Haimson and Klaetsch (2007) showed that if the specimens are bored prior to applying the far field stresses, debonded sand grains may be detached from the borehole wall and a rather small breakout will be generated. It should be noted that if a borehole is drilled through a solid cylindrical specimen and after...
applying the far-field stresses no instability issues are observed. This suggests that the apparent mechanical properties of the specimen are not the same as those for the poorly cemented formation at the considered drilling site. However, size-scaling effect should be considered in laboratory tests as well.

In experimental studies all test and specimen conditions must be under control before other properties are examined. Drilling a borehole through a synthetic specimen before applying the stresses (e.g. Younessi et al., 2013) may generate cracks and disturb the surface of the borehole wall. It is not possible to control the formation of random and chaotic microcracks and defects before applying the far-field stresses on the specimen. Therefore, stress concentration may take place on the borehole wall due to drilling induced microcracks and after applying the stresses the borehole breakout may occur because of these pre-existing defects. This may result in acquiring an unrealistic view on the borehole stability. This problem can be addressed either by drilling a borehole through the specimen after applying the in situ stresses which is very difficult and costly. Another option is to prepare a specimen with an intact pre-existing borehole. This method was practised in this study.

In order to support the above mentioned arguments, three solid specimens with different cement contents were prepared. Boreholes were drilled through them without applying external stresses. Then specimens were cut into different sections for the SEM imaging. Fig. 11a shows the microcracks on the borehole wall generated due to drilling a hole in the specimen before the test. Fig. 11b shows the wall of a prefabricated borehole specimen. In this figure microcracks with smaller width that have formed due to the cement shrinkage can be observed on the borehole wall. This confirms that this method creates less macro- and microcracks on the borehole wall as compare with the method which requires drilling of a borehole into a solid specimen.
Fig. 11. (a) Microcrack developed on the borehole wall due to drilling a borehole in the solid specimen (magnification=117), (b) Borehole wall without any visible cracks in the prefabricated TWHC (magnification=2382), (c) Shrinkage microcrack developed on the borehole wall (magnification=2382).

5.5. Geometry of localised zone

As a general view of the localised band, it was a narrow zone with a few grains wide, in which compactant volumetric strain was dominant. As mentioned earlier, in order to determine the location of the initial localised damage zone on the borehole wall during the test, a micro camera was deployed inside the top platen. Once the testing of the specimen was completed, the damaged specimen was unloaded and removed gently from the cell to investigate the dimension of the initial localised bands (i.e. width, length and depth) which were induced due to the applied stresses. Afterwards, scanning electron microscopy (SEM) imaging was carried out for post-test inspections. Klein et al. (2001) and Haimson and Kovacich (2003) suggested that the formation of the apparent compaction band in the sandstone under different stress paths was due to its homogeneous mineralogy and weak grain bonding (95% quartz). Micromechanical studies on sandstones conducted by Katsman et al. (2009) showed that the breakouts are formed by episodic spalling of thin rock flakes separated by dilatant extensile inter- and intra-granular microcracks simultaneously.

Applying stresses to the TWHC specimens that exceeded a critical value induced sand particle debonding and dislocation on the borehole wall. According to Fig. 12 the breakout geometries on the borehole wall are almost rectangular to oval shape and V shape in depth.
similar to limestone and Tablerock as tested by Haimson (2003) and high porosity sandstones (18-29\%) tested by Haimson and Song (1998) and Cuss et al. (2003). Specimens were dissected the centre of the borehole to visually observe the localised bands. Based on the recorded videos during the tests, these bands are the areas that the first grains were debonded on the borehole circumference. Detached particles in the breakout zone were removed from the borehole wall by blowing very low air pressure to avoid disturbing the breakout zone.

![Images of specimens and breakout geometries.](image)

**Fig. 12.** Breakout geometries on the borehole wall are almost rectangular to oval in shape.

### 5.5.1. Depth of the localised zone

The depth of the breakout zone was measured by dissecting the specimens at just above or below the breakout zone based on the sand debonding area which was captured by camera. Fig. 13a and Fig. 13b show that depth of the localised zone is greater for the 20 mm diameter borehole. Also, as presented in Fig. 13c with increasing the $w_c$ the depth of the breakout zone decreases.

![Images of depth measurements.](image)
According to Fig. 13 the depth of the localised zone ranges from 1-6 average grain diameters (i.e. 0.576 mm) for the confining pressure range of 1-4 MPa. Menéndez et al. (1996) observed 5 intact grain diameters for the localised band in Berea sandstone tested at 40 MPa confining pressure. Bésuelle et al. (2000) reported localised bands of 1-4 intact grain sizes for Vosges Sandstone at the confining pressure of 30 MPa. It should be noted that with increasing the confining pressure the tangential stress increases and subsequently the number of microcracks on the borehole wall elevates as well. After the initiation of the localised damaged zone and with an increase in the deviatoric stress, damage zone spreads throughout the borehole circumference rather than propagating away from the borehole. Also, under lower tangential stresses, the localised damaged zone developed ahead of the borehole wall until it reached the state of equilibrium and formed a stable arch, because the stress on the borehole wall was not high enough to break the adjacent particle bondings and extend the width of the localised zone. It should be mentioned that due to the limited size of specimens in laboratory tests the depth of the localised zone increased while elevating the deviator stress, which is not the case in actual stress condition. Thus, all the measurements were done in the strain corresponding to the 1.05 times of the peak stress.

5.5.2. **Width of the localised zone**

In the first stage of the test with the application of higher confining and axial stresses, specimens experienced higher pressures until reaching the hydrostatic stress state. Naturally considerable number of microcracks generated in the stress concentration area which is located adjacent to the borehole wall. In the next stage of the test, where the deviatoric stress gradually elevated, the number of affected contact bonds and particles prone to dislodge grew
and was more than for the case with lower confining pressures. However, since the range of change in the confining pressure was not high enough (i.e. 1-5 MPa) due to the material with low strength, the width of localised bands (Fig. 12) did not change dramatically. It may be inferred that the isotropic confining pressure ($\sigma_2 = \sigma_3$) has a minimal influence on the expansion of the localised zone width. However, with an increase in the maximum principal stress ($\sigma_1$) the width of the breakout zone grows. Also, Fig. 14 shows that with increasing $w_c$, the width of the localised zone decreases for a certain borehole size. This may be due to the strengthening effect of the cement agent that provides more sutures between grains and thus hindering the propagation of the breakout zone. As expected, it was observed that the width of the localised zone is considerably larger in borehole with 20mm diameter than in the one with the 10mm due to the size-scale effect which is discussed in Carpinteri (1996) and Aifantis (1999) (see Fig. 14). Real-time monitoring of boreholes showed that the rate of decreasing the breakout zone width grew with increasing the cement ratio. In other words, increasing the $w_c$ transforms the geometry of the breakout zone, namely from ‘square’ to a ‘rectangular’ shape in the dissected specimen. This may be due to the transition from ductile to brittle behaviour of the specimens with increasing the $w_c$ value.
5.5.3. Length of the localised zone

Lengths of the localised damage zones were measured in terms of the angle from the centre of the borehole. Results showed that in most of the cases in 20 mm diameter boreholes and in 80% of 10 mm diameter boreholes the length of the localised zone extended all around the borehole wall (i.e. 360°) (Fig. 15). Younessi et al. (2013) and Papamichos et al. (2010) showed that with increasing the far-field stress anisotropy in the horizontal direction the length of localised zone on the borehole wall shrinks. Since in triaxial tests the far-field stress anisotropy in the horizontal direction is negligible, in the conducted tests the maximum breakout length was observed on the borehole wall.

5.6. Scanning electron microscopy (SEM) studies

Scanning electron microscopy (SEM) imaging showed that microcracks extended into the borehole wall (Fig. 16) perpendicular to the axis of the vertical stress and then reoriented outward into the specimen following trajectories of the confining pressure. Sand particles which were debonded at the localised bands due to the intense microcracking fell into the
borehole (see Fig. 16). As the microcracks propagate, the stress concentration moves toward the specimen’s outer boundary releasing more sand particles and further developing the localised zone. The localised band remains narrow possibly due to the effect of some adjacent locked grains which keep the boundary locked in place and hinder the further bond breakage beyond them. The other reason could be related to the stress release in the localised area. Naturally, the localised zone will fail first, because the applied stresses have already overcome the static inertia here, generated a certain momentum and the grain movements have started. To trigger the movement of particles outside the localised zone more forces may be required for overcoming the resistance in the static mode. Thus, an increase in the confining stress will result in further development of fragmentations in the damaged zone and the localised zone will start to propagate.

![Stress induced cracks in the localised zones which are perpendicular to the maximum principal stress.](image)

Also, the recorded test videos (Fig. 17a) showed that grain debonding in the 20 mm borehole often occurs through detachment of flake-like formations of particles from the borehole wall, while the particle debonding in the 10 mm borehole was usually in the form of individual grain detachment similar to the sanding phenomenon in oil reservoirs (Fig. 17b). According to Klaetch and Haimson (2002), occurrence of such breakouts in oil and gas reservoirs is the source of sand production.

It is important to note that no preferred horizontal breakout direction was observed in the conducted tests due to the application of an isotropic confining pressure to the specimens and
breakout directions were controlled by the heterogeneity of the specimens due to mixture preparation.

![Fig. 17. Sand debonding process in (a) 20 mm diameter borehole in the form of flakes (b) 10 mm diameter borehole in the form of grain by grain.](image)

5.7. Compaction bands detection

Charalampidou et al. (2010) showed that the localised band zone consists of a shear deformation vector and its development in the specimens is mostly accompanied by compaction or/and dilation. Microscopic studies conducted by Haimson (2003) revealed that fracture-like breakouts were preceded by the formation of a very narrow band of compacted grains ahead of the breakout tip, leading to the significant reduction in porosity. The analysed results from a few strain gauges in TWHC specimens’ tests showed that after reaching the maximum stress, an instant drop occurs in stress-strain curves (see Fig.18). It implies that a substantial stress reduction has occurred while no significant changes were observed in the strain values. However, the external LVDTs measurements showed that the specimen deformation continued until the completion of the test. It should be noted that before the peak stress was reached, the results of the LVDTs and strain gauges followed a similar trend (Fig. 18). The videos that were recorded during several tests were analysed with respect to the time of that occurrence. It was observed that until the peak stress, there was not any sand particle debonding from the borehole wall and the specimen underwent uniform deformation. Shortly after the peak stress and during strain softening behaviour, the first sand particle separated from the borehole wall and subsequent debonding continued from one side of the borehole wall which was apparently the weakest point based on specimen heterogeneity. Thus, stress concentration occurred at the localised zone and the majority of specimen deformation happens in that area, after the sample passes the peak point and drops into softening state. Therefore, it can be elaborated that in the damaged area, a band of
compaction zone was localized which was account for the deformation after the peak stress. When this area fell outside of the strain gauge’s measurement domain, the gauge could not capture the overall deformation of the specimen, whereas the stress measurements showed that it kept decreasing until the completion of the test. Therefore, it is reasonable to state that, the strain gauges could not register the deformation of the specimen after the development of the localised zone which fell outside of the strain gauges measurement zone.

![Graph](image)

**Fig. 18.** Instant drop in stress in some of strain gauges results versus LVDT strain results (especially in 20 mm boreholes) indicated that compaction zone was out of measured area and most of deformation in the specimens was in the localised zone.

Also, SEM analyses showed that in synthetic poorly cemented sand specimens, the main reason of borehole breakout is the breakage of bonding between the grains similar to Tablerocks in which the Feldspar grains cracked first and more extensively than the quartz grains (Haimson and Klaetsch, 2007). According to Fig. 19, all the microcracks which were observed in the localised zone passed through cement sutures and between particles and no crushed particles were observed in the localised zone for the considered cementation values (i.e. $w_c = 6 - 8\%$). The compaction bands appeared to initiate at the borehole wall, with more extensive development in the vicinity of the wall than away from it and into the specimen. Thus, it can be stated that the process of the microcrack development is associated with the either dilatancy or porosity reduction around the shear band and can be related to the grain movement.
Fig. 19. All the captured micro and macro cracks were on the cementation areas and on the boundary of grains in TWHC specimens. No crushed grain was observed in the range of the applied stresses for $w_c = 6$–$8\%$.

It is worth mentioning that unlike previous tests on Berea sandstone which were conducted by Haimson (2006) and Haimson and Song (1998), there was no need to extract the particles from the compaction zone by circulating a drilling fluid. When the first group of particles got debonded and fell into the borehole, the stress concentration moved to the next set of grains causing more particle debonding and dislodgment from the borehole wall. This results in further development of the breakout zone.

5.8. Failure pattern

As mentioned in the previous section, the onset of the borehole breakout in all TWHC specimens occurs with the appearance of a few shear bands on the borehole wall. Also, shear bands were observed on the external surfaces of the solid specimens subjected to triaxial testing. Fig. 20 shows the typical patterns of shear bands observed after the tests in solid specimens on the cylinder surface. Fig. 21a–c represents that axial and inclined shear bands on the surface of solid specimens at low (1–2 MPa) confining pressures. With increasing the confining pressure, more inclined shear bands formed on the surface of specimen and the bands became thicker. This result agrees with Ord et al. (1991) and Bésuelle et al. (2000) who showed that shear band directions depend on the value of the confining pressure. Also, the number of shear bands increases with increasing the confining pressure and decreases with an increases in $w_c$. For confining pressure of 4 MPa and 5 MPa, conjugate shear bands were observed on solid specimens and bands were closer to each other. The trajectories of shear bands became more deviated from the direction of the maximum principal stress with increasing the confining pressure (Fig. 21d–e). However, with increasing the $w_c$, once the confining pressure elevated the deviation from the direction of $\sigma_1$ became less pronounced.
Charalampidou et al. (2010) showed that under high confining pressures (130–190 MPa) horizontal compaction bands are dominant in the deformation of porous Vosges sandstone.

**Fig. 20.** Failure pattern on the surface of solid specimens under different confining pressures (a) $\sigma_{\text{conf}} = 1$ MPa (b) $\sigma_{\text{conf}} = 2 – 3$ MPa (c) $\sigma_{\text{conf}} = 4 – 4.5$ MPa.

**Fig. 21.** Axial and inclined shear bands on the surface of the solid specimens (a) $\sigma_{\text{conf}} = 1$ MPa (b) $\sigma_{\text{conf}} = 1.5$ MPa (c) $\sigma_{\text{conf}} = 2$ MPa (d) $\sigma_{\text{conf}} = 3$ MPa (e) $\sigma_{\text{conf}} = 4$ MPa (f) $\sigma_{\text{conf}} = 4.5$ MPa.
Fig. 22 shows that in the TWHC specimens most of the immobilised shear bands are parallel or sub-parallel to the direction of the confining pressure at the borehole wall. However, in specimens with lower confining pressures (1-2 Mpa) inclined compaction bands (as characterized by Eichhubl et al. (2010) developed at the angle of 25-40° to the vertical axis.

Fig. 22. Shear bands are parallel or sub-parallel to the direction of confining pressure on the borehole wall in TWHC specimens.

5.9. Shear band orientation

Shear failure angle can be predicted by the Mohr-Coulomb relations as; \( \theta = 45^\circ - \phi / 2 \) where \( \phi \) is the angle of internal friction of the material. This orientation corresponds to the plane that undergoes no extension or the pure shear strain plane in the Mohr-Coulomb criterion (Bésuelle et al., 2000). The Mohr prediction for the average shear band angles was derived as 30.10°. This calculated value of \( \theta \) was lower than the one from the actual test which was \( \theta_{ave} \approx 33^\circ \) for the range of confining pressures applied in this study. Bésuelle and Rudnicki (2004) showed that the Mohr-Coulomb relation for calculating the shear failure angle is more precise in high confining pressures.

5.10. Debonded grain size distribution

Once the test on the TWHC specimens was completed, the detached sand particles were carefully collected from the lower platen. SEM study was performed on the debonded grains to identify the size range and ratios of the detached particles versus the initially used particle sizes for preparing the mixtures. Fig. 23 shows the SEM photos of the detached particles for three different \( w_c \) values at the same confining pressure. Investigations of the grain size distribution for sand particles retrieved during testing shows that they contain more of the fine grain sands versus the grain size distribution used in the specimen preparation which
contained 50% of coarse sand grains. This agrees with the findings by Adeyanju and Olafuyi (2011) who studied the effect of the fluid flow on the sand production in boreholes for higher stress ranges. Similar to the results of our study, their results showed that particles that were detached from the borehole wall under the fluid pressure are in finer range in comparison with the total particle size distribution of the specimen. Also, the result showed that the number of detached coarse grains increases with an increase in $w_c$. This may be due to the more extensive development of the microcracks at the borehole wall and because the mode of particle detachment for higher $w_c$ values is in the form of flakes rather than grain by grain dislodgment.

Fig. 23. The detached particle size distribution in three different $w_c$ values at the same confining pressure (a) $w_c = 6\%$ (b) $w_c = 7\%$ (c, d) $w_c = 8\%$.

6. Discussion

6.1. Pore collapse in localised bands

In low-porosity sandstones bondings fill the space between the grains leaving smaller room for pore spaces, but in poorly cemented sands the volume of pore spaces is relatively higher than that of the total bonding. The pore collapse that occurred within the compaction bands is the result of intense bonding breakage and crack development in the specimens. Haimson and Klaetsch (2007) showed that the bonding variation is a key factor in understanding the differences between the breakout micro-mechanisms in high and low porosity rocks. In the
current tests, even at the lowest confining pressure (= 1 MPa) and a given $w_c$, specimens exhibited nonlinear stress-strain relations with a certain level of compaction based on the applied stress. At higher stress levels, the specimens experienced more compaction. This high compressibility potential is attributed to the pore collapse and the material compaction can be divided into 2 phases; elastic behaviour and pore collapse as stated by Nouri et al. (2006). If the sand grains are not cemented, the average stress at the time of pore collapse is assumed to be the pre-consolidation stress (Vardoulakis et al., 1996). However, in a cemented material it is believed that cementation delays the pore collapse to some extent higher than pre-consolidation stress. Nouri et al. (2006) showed that the difference between the pre-consolidation and pore collapse stresses depends on the level of cementation in the material. As showed earlier, increasing the $w_c$ value changed the level of compaction at the same confining pressure up to the peak stress. This implies that the $w_c$ value affects the compaction level and reduces the pore collapse up to a certain amount in poorly cemented sands.

6.2. Compaction band formation

Compaction bands are formed in the zone of weak particle contacts where the bonding sutures can break and let the particles rearrange to form a denser matrix. Thus, borehole breakout is accommodated by debonding of those loosely cemented particles. In the case of strong bonding between grains, the particles within the compaction band are typically cracked or even crushed due to the high stress concentration. This form of breakout is often dilatant like limestone or Tablerock sandstone (Haimson, 2003). In this form of breakout, extensile cracking will happen and even volume increment may occur by rolling of some crushed particles on the others. Using the acoustic emission method, Charalampidou et al. (2010) showed that shear band evolution is identified by breakage of the grain contacts or the grains themselves. The SEM photos revealed that in the considered synthetic specimens the localised bands are characterized by only sliding and rolling of sands on each other due to the disintegration of bonding between them. According to the breakout observations, the paucity of grain shattering suggests that concentrated stress magnitude near the compaction band tip along the confining pressure orientation was not large enough to cause quartz grains breakage, but it sufficed for breaking up the particle bondings. When the bonding was broken, it allowed particle movements and repacking into compacted and less porous arrangement. This confirms that the breakout only occurs within the cementing matrix. Even inside the compaction bands there were no signs of grain shattering and crushing.
6.3. Mechanism of the localised zone expansion

Recorded videos showed that the breakout zone grows as the debonded sands fragments/particles are falling off from the borehole wall (Fig. 24). Due to relocation of the stress concentration on the edge of breakout line, compaction band extends and results in detachment of more particles. Olsson (1999) and Klein et al. (2001) performed a series of triaxial tests on sandstone specimens and regenerated the cracked and crushed grains within compaction bands resembling the intra-granular microcracks we observed at the borehole wall. In actual field stress conditions this process diminishes after reaching the state of equilibrium due to the development of a stable arch across the plastic zone and the compaction band transmits the stress to the adjacent area. However, in laboratory test conditions this was not the case and the process of breakout development continued until the application of the stress increment was stopped.

Fig. 24. Breakout zone generation process subsequent to the falling of debonded sands fragments.

Homogeneity of specimens diminished in the strain-hardening section of the stress-strain graph before the maximum stress due to decreasing the material stiffness. Ord et al. (1991) suggested that this loss of homogeneity be recognised as the localisation phenomenon. However, videos recorded during the tests showed that the macroscopic shear bands were immobilised on the borehole wall only in the softening mode section of the stress-strain graph. Thus, the formation of the visible localisation bands did not coincide with the loss of homogeneity and it occurred after the peak stress.

6.4. Sand particle debonding stages
Having conducted laboratory experiments for disparate stress magnitudes and investigating the post-peak behaviour of TWHC specimens, several stages for the development of debonding phenomenon were identified. Haimson (2006) showed that the breakout in granite begins before failure of the borehole wall. The cracks were dilatant and extensile as they did not exhibit any shear movement. Current test results on poorly cemented sands showed that the crack development on the borehole wall did not seem to be dilatant and the mechanism of failure consisted of localized debonded and dislocated grains which led to the formation of an area with lower porosity or a compaction band in the direction of the latter crack and it moved between particles, toward the orientation of weakest bonding. Also, unlike in the case of hard rocks, cracks in the considered materials were mostly intra-granular because the cementation matrix is practically weaker than the grains. The damaged zone comprised of debonded particles and the next sets of intact grains were ready to fall into the borehole as it advanced.

6.5. Real-time camera observations

Real-time borehole monitoring showed that for $6\% \leq w_c \leq 7\%$ sand grain dislocations continued under constant vertical and confining stresses from a certain point onward. That is, when the process of particle movement started and the stresses were kept constant ($\varepsilon_a > 1.05 \varepsilon_{peak}$), the sand debonding process continued until the state of stable equilibrium was established. However, in some cases after around 30 minutes borehole failure occurred without leading to the state of stable equilibrium. According to the real-time monitoring during tests, grain debondings were randomly oriented and typically placed at the middle one third of the specimen. This may be due to time dependent behaviour of cemented sands which was reported by the drilling companies earlier. Also, it was observed that the sand particle dislocation process was strongly affected by increasing the confining pressure. For confining pressures higher than 3MPa a stable arch was formed in rare cases in TWHC specimens. For instance, only in 10% and 7.5 % of the specimens a stable arch was formed for 4 MPa and 4.5 MPa of confining pressure respectively.

Also, recorded videos showed that under considered stress ranges for the current test program, there were no significant changes in the borehole breakout onset until up to the displacement rate of 0.09 mm/min. Applying the stresses to the TWHC specimens in displacement rate between the rate of 0.07 mm/min which was considered for the main tests and the maximum rate of 0.09 mm/min resulted in the debonding of grains from the borehole wall towards a certain rate. Applying the stress at a displacement rate higher than the 0.09
mm/min resulted in the excessive particle debonding which shows the material’s dependency on displacement rate. However, this needs further studies for other cases to draw a more generic conclusion.

7. Conclusions

In this study a series of solid and thick-walled hollow cylinder (TWHC) laboratory tests was performed on synthetic poorly cemented sand specimens. Applying higher stresses than a certain threshold for the considered TWHC specimens led to the breakage of grain bondings within a narrow localised band which was normal to the maximum principal stress. This allowed the detached and loosened grains to rearrange to form a lower porosity compaction band and this initial breakage intensified the stress concentration behind the breakout tip. As microcracks extended, the stress concentration moved toward the specimen’s outer boundary releasing more sand particles and progressing the breakout zone. This process continued until the state of equilibrium was established between the formed sand arch and induced stresses acting on the localised band tip. Results showed that applying the confining pressure beyond a specific magnitude transformed the mode of failure from dilation to the pure contraction in TWHC. According to the test results, a volumetric strain dilatancy was observed at the confining pressure 1-2 MPa and it transformed to contractancy under higher pressures. The volumetric strain results illustrated that the localised bands were in general compactant for the confining pressures greater than 2 MPa in poorly cemented sands. It should be noted that the dilatancy was observed before the formation of the localisation damage zone especially at low confining pressures.

Scanning electron microscopy (SEM) observations revealed that the fundamental mechanism of bonding breakage was the extension of small, opening-mode and splitting cracks which developed in the vicinity of the borehole wall and propagated deeper into the matrix with increasing the stress.

Real-time monitoring showed that the cracks were formed on the borehole wall with no signs of the formation of compaction bands at the initial stage of the crack development. During this process no considerable change was observed in the volumetric strain. Once the volumetric strain reduced dramatically the compaction bands formed and could be observed. Also, a localized compaction zone formed along a narrow band on the borehole wall, the direction of which was sub-parallel to the confining pressure direction and perpendicular to the maximum principal stress.
Moreover, SEM studies showed that when drilling a hole in a cylindrical solid specimen prior to applying far-field stresses, microcracks would develop on the borehole wall and an area around the borehole will be disturbed.

The geometry of failed zones was influenced by the stress state and borehole size. Results showed that the depth and width of the breakout zone increased with increasing the confining pressure \( \sigma_h = \sigma_H \) and the \( w_c \) values affect the shape of the breakout zone. Also the depth and width of the breakout zone for the specimens with borehole diameter of 20 mm were generally almost 20% and 70% larger than those for the 10 mm ones respectively.

Near the borehole wall intra-granular cracks were observed after the test. According to SEM analyses, no crushed grains were observed in the localised zone and the sand particles remained intact under the range of applied stresses and along the border of the breakout zone located on the boundary of particles. This allows suggesting that stresses that caused debonding and repacking were at levels lower than those required to induce grain damage. In some cases released grains rolled at the localised band and generated dilatancy under low confining pressures. At localised zones, debonded intact grains usually repacked and accommodated a lower porosity narrow compaction band until the completion of the test at \( \sigma_{conf} \geq 3 \) MPa.

Results showed that the grain size distribution of debonded grains contained higher amount of fine grain sizes especially in lower confining pressures (i.e. 1-2 MPa), compared with the particle size distribution of the prepared specimen.

Real-time camera recordings also showed that sand debonding continued under constant vertical and confining stresses after a certain point. This may be due to the limited thickness of specimens unlike actual borehole conditions.

**Acknowledgements**

This work has been supported by the Deep Exploration Technologies Cooperative Research Centre whose activities are funded by the Australian Government’s Research Programme. This is DET CRC Document 2014/486.

**References**


CHAPTER 5

Background

This chapter consists of two manuscripts and describes the behaviour of poorly cemented sands under different stress paths.

In the first manuscript ‘Effect of different stress paths on the behaviour of weakly cemented sands adjacent to a drilled borehole’ the TWHC specimens were subjected to five different stress paths and the principal stresses at the borehole wall were calculated. The aim of the presented laboratory test program is to study the mechanical behaviour of poorly cemented sands under different stress paths that simulate the in-situ stresses in the field.

Two approaches were considered for applying the stresses to the specimens including the far-field stresses (first and second paths) and an element at the borehole wall (third, fourth and fifth paths) and the results for different stress paths were compared. The borehole behaviour was monitored by a real-time video camera and recorded in order to determine the borehole failure along with measuring the stress-strain for poorly cemented sand specimens. Two borehole diameter sizes (10 mm and 20 mm) and three different cement contents (6%, 7% and 8%) were considered versus five different stress paths. A new failure quadrilateral was determined based on the conducted tests studying the effects of stress paths on the behaviour of poorly cemented sands. Results showed that with increasing the cement content, similar quadrilaterals with almost parallel sides will be established. This finding can largely enhance the designing of the supporting systems and be utilised for predicting the possible borehole instability prior to drilling.

The second manuscript is titled ‘A new sand dislocation criterion at boreholes drilled through poorly cemented sandy formations’. The sand particle debonding usually precedes the borehole failure and it can be considered as a sign that the onset of the borehole collapse is imminent. Detecting the bonding breakage point and introducing an appropriate failure criterion plays a key role in the borehole stability analysis. The total potential and dissipative absorbed strain energy per volume of material up to the point of the observed particle debonding was calculated. The bonding breakage points at the borehole wall were detected for various stress paths, cement values and borehole sizes. The results showed that the particle bonding breakage point at the borehole wall was reached both before and after the peak strength of the thick-walled hollow cylinder specimens depending on the stress path and
cement content. The data from the laboratory tests were used to calculate $\sigma_{m,2}$ at the borehole wall in order to represent the trend of obtained data in the $U - \sigma_{m,2}$ domain.

Modulus of toughness ($U$) for different stress paths was derived and plotted versus $\sigma_{m,2}$ as a borehole instability criterion. The relationship between the data in the $U - \sigma_{m,2}$ domain was relatively linear.

According to the presented shear strength properties of the synthetic poorly cemented sands, it is now possible to estimate and compare shear properties of the materials at the drilling field. When drilling through a poorly cemented sandy formation at a new drilling field, an ordinary triaxial test can be performed on an undisturbed sample collected from the drilling site and the possibility of the borehole failure can be evaluated by comparing the test data with the results presented in the current paper.

**List of Manuscripts**


Statement of Authorship

<table>
<thead>
<tr>
<th>Title of Paper</th>
<th>Effect of different stress paths on the behaviour of weakly cemented sands adjacent to a drilled borehole</th>
</tr>
</thead>
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<tr>
<td>Publication Status</td>
<td>○ Published, ○ Accepted for Publication, ○ Submitted for Publication, ○ Publication style</td>
</tr>
</tbody>
</table>

**Author Contributions**

By signing the Statement of Authorship, each author certifies that their stated contribution to the publication is accurate and that permission is granted for the publication to be included in the candidate's thesis.

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**Contribution to the Paper**

Designed and performed laboratory tests, developed model and theory, data analysis, interpreted data, prepared manuscript and acted as corresponding author.

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Contributed to research, Supervised and performed proofreading

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Date:
Effect of different stress path regimes on borehole instability in poorly cemented formations

S.S. Hashemi\textsuperscript{a*}, N. Melkoumian\textsuperscript{a}

Abstract

Boreholes are drilled for different purposes such as discovering potential new deposits of underground minerals, extraction of petroleum, underground strata investigation, etc. Although no specific significant problems have been reported on drilling through hard rocks and strong formations, considerable problems have been observed in areas consisting of a sandy formation where particles are not strongly cemented by natural cement agents such as clay, iron oxide or calcite. In this study a series of solid and thick-walled hollow cylinder (TWHC) laboratory tests were conducted on synthetic poorly cemented sand specimens in which the applied stresses were at levels of generating breakout on the borehole wall. Five different stress paths were designed and applied to the specimens to investigate the effect of stress paths on the borehole failure. Two borehole sizes (10 mm and 20 mm) and three different cement contents (6%, 7% and 8%) were considered to evaluate the effect of scaling and bonding strength on the borehole failure in poorly cemented sandy formations. The results showed that in this weak formation $\sigma_\theta$ has a more significant effect on the instability of the borehole than the cement agent content. It was found, that for any stress path the effect of the supporting stress on $\varepsilon_1$ was more significant for smaller borehole sizes. Also, a new failure quadrilateral was determined based on the conducted stress paths for poorly cemented sands. Results showed that with increasing the cement content, similar quadrilaterals with almost parallel sides will be established. These outcomes can contribute to improving the design of the supporting systems and be utilised to predict the borehole instability prior to drilling.

Keywords: Borehole stability; Experimental studies; Effect of stress paths; Poorly cemented sands; Thick-walled hollow cylinders

Nomenclature

<table>
<thead>
<tr>
<th>$TWHC$</th>
<th>thick-walled hollow cylinder</th>
<th>$\sigma_{conf}$</th>
<th>confining stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_c$</td>
<td>cement to sand grains weight ratio</td>
<td>$\phi$</td>
<td>angle of internal friction (degrees)</td>
</tr>
<tr>
<td>$\sigma_\theta$</td>
<td>tangential stress (MPa)</td>
<td>$\sigma_n$</td>
<td>normal stress (MPa)</td>
</tr>
<tr>
<td>$\sigma_r$</td>
<td>radial stress (MPa)</td>
<td>$q$</td>
<td>deviator stress</td>
</tr>
<tr>
<td>$\sigma_z$</td>
<td>vertical stress (MPa)</td>
<td>$D_i$</td>
<td>Diameter of TWHC specimen</td>
</tr>
<tr>
<td>$UCS$</td>
<td>uniaxial compressive strength (MPa)</td>
<td>$P$</td>
<td>perimeter of TWHC specimen</td>
</tr>
<tr>
<td>$c$</td>
<td>cohesion (MPa)</td>
<td>$D_{50}$</td>
<td>mean grain diameter (mm)</td>
</tr>
</tbody>
</table>
1. Introduction

Drilling a borehole into the earth crust and removing the underground geomaterials cause stress concentration around the borehole. The magnitude of stresses depends on the borehole trajectory, value and orientation of in situ stresses and mud pressure inside the borehole (Bradley, 1979). After a borehole is drilled, tangential stress will be induced on the borehole wall based on the magnitude of the in situ stress, while the radial stress will tend to zero if there is no supporting pressure acting inside the borehole. Since there is no adequate cement agent presents on the interface of sand grains in a poorly cemented sand formation, the induced extensile stress leads to the breakage of bonding between the particles and creating an inelastic damaged zone around the borehole (Ewy and Cook, 1990b, Ewy and Cook, 1990a). The further growth of the damaged zone may lead to the borehole failure and render the borehole useless for exploration purposes. Failure of the borehole can cause problems such as stuck pipes, lost circulation, etc. It has been estimated that about 70% of the world's oil and gas reserves are found within this category of weakly-consolidated or non-consolidated strata (Bellarby, 2009). Usually, depending on the ground conditions and method of drilling (e.g. air pressure or filled with a drilling mud) the excavated volume is supported by a specific supporting system exerting a pressure, $P_w$ to the borehole wall (Zoback et al., 2003). The required in-hole pressure is calculated via borehole stability analysis with respect to the underground formation behaviour that can be assumed as being either elastic, homogenous and isotropic or inelastic and heterogeneous (Bradley, 1979). Tan et al. (2013) showed that the safe drilling fluid density window is wider when the borehole is drilled along the minimum principal horizontal stress. Hashemi et al. (2014a) introduced a numerical model of the borehole failure in granular materials and showed that the main reason for the borehole failure in such formations is the dislocation of particles due to fluid pressure and in situ stresses. Likewise, sand production occurs in oil wellbores due to the combination of extensile stress concentration and fluid flow pressure at the wellbore wall (Ranjith et al., 2013). Sand production control is one of the most necessary ways of increasing well production in oil and gas reservoirs. According to Ramos et al. (1994), companies reported an increase in the sand free rate of up to 44% after the sand production control.

Laboratory tests are one of the best approaches for studying the borehole stability in both hard and weak rocks. The thick-walled hollow cylinder (TWHC) test is a common approach for simulating stress and strain states around a borehole in order to investigate failure.
mechanism of an underground formation under different stress paths through independent internal and external pressures, and transference of the external boundary to an infinite distance (Ewy and Cook, 1990b). Alsayed (2002) stated that different stress paths can be applied to TWHC specimens to simulate the underground in situ stresses. Various studies have been performed by using TWHC specimens to investigate the failure behaviour of boreholes and tunnels. Alsayed (2002) and Alsayed (1996) conducted a series of TWHC tests on rock samples to study the effect of anisotropic stress conditions on the behaviour of hard rocks. (Pomeroy and Hobbs (1962) examined the strength of coal hollow cylinder specimens. (Mazanti and Sowers (1966) studied the behaviour of granite hollow cylinder specimens and the effect of the intermediate principal stress ($\sigma_2$) on their strength. Perie and Goodman (1989) investigated the macroscopic failure mechanism of synthetic rocks made of gypsum cement by conducting TWHC test. Ewy and Cook, (1990 b), Ewy and Cook (1990a) carried out a valuable experimental study on the behaviour of Indiana limestone and consolidated Berea sandstone in the form of TWHC. Younessi et al. (2013) conducted laboratory tests on synthetic sandstone specimens and studied the properties of borehole breakouts due to stress anisotropy.

The main purpose of this study is to investigate the effect of different stress regimes on the failure of boreholes in poorly cemented sandy formations. In the current research a number of new TWHC laboratory tests were designed and performed under controlled laboratory conditions. Various stress paths were designed based on both far-field and an element on the borehole wall, and the results were compared for different stress paths. The borehole behaviour was monitored by a real-time video camera recording in order to determine the borehole failure along with measuring the stresses and strains for poorly cemented sand specimens. The outcome of this paper presents more realistic understandings of the actual failure behaviour of poorly cemented sandy formations under different stress paths.

2. Exploration borehole drilling site observations

Borehole instability and stuck-pipe issues were reported on a drilling site located near Burra, South Australia. At this site boreholes were drilled with a purpose to discover mineral deposits for potential new mines. According to the in situ geological tests, borehole failures took place in a poorly cemented sandy layer. Conventional dry drilling methods, including air core and reverse circulation (RC) drilling, were used to drill the boreholes. Subsurface investigations of sediments showed that the layers above the bedrock are not homogeneous,
the shallower layers of the sediment are composed of silt and fine sand, the deeper layers of the sediments change to dark grey plastic clay, and the problematic poorly cemented sandstone comes after this clayey layer. The drilled boreholes were generally 25 cm - 30 cm in diameter with 50 m – 200 m in length depending on the underground conditions. During the drilling process, when the drilling rod encounters the poorly cemented sandy layer, there is a considerable potential for the borehole instability. Although in previous studies (Nouri et al., 2006, Durrett et al., 1977) the fluid velocity has been identified as an important factor for wellbore instability, as per site investigations the effect of induced stresses on the borehole wall instability dominates in shallow depth boreholes where no fluid velocity exists and low pore pressure has a trivial effect on the instability of drilled boreholes.

Samples were collected from each metre of unconsolidated sandy layer and subsequently solid and TWHC specimens were prepared for laboratory tests based on the size and geometry of sand particles collected from the site. Geochemistry tests showed that the formation consists of quartz sand grains with a weak cementation interface of iron dioxide, clay and calcite as cementing agents. The yellowish-grey grains were mostly fine and sub-angular with random orientations.

3. Principal stresses at the borehole wall

In the cylindrical coordinate system the tangential, radial and vertical stresses are the stress states at a borehole wall. These stresses are induced due to the presence of in situ stresses and can be calculated using different equations such as Kirsch equations based on the theory of elasticity (Obert and Duvall, 1967, Jaeger et al., 2009). According to the Kirsch equations, tangential and radial stresses will change across the cylinder wall with the radial distance, $r$. Fairhurst (2003) indicated that the maximum principal vertical stress is calculated as the weight of the overlying layers at a certain depth. The minimum horizontal stress can be measured by hydraulic fracturing and leak off test (Amadei and Stephansson, 1997). It should be noted that determining the in situ maximum horizontal stress is not straightforward and it can be estimated only based on certain assumptions (Della Vecchia et al., 2014; Aadnoy et al., 2013; Zoback et al., 1985). Not always the principal stresses are in the horizontal and vertical orientation. This can be cleared by analyzing image logs where the deviations may occur. In these cases the in-situ stresses need to be transformed to horizontal and vertical principal orientations (Aadnoy and Looyeh, 2011).
In different studies, stress-strain diagrams have been used to present the mechanical behavior of TWHC specimens. Although “engineering” stresses can be calculated based on the initial cross section of the specimens and can be easily obtained from the test results, in this study, the “true” stresses were considered for deriving the precise stress-strain diagrams. In this approach, the actual diameter of the specimen in each step of the test was considered for calculating the stress as follows:

\[ P_i = \pi D_i \]  
\[ P_{i+1} = \pi D_{i+1} \]  
\[ \Delta P = \Delta L = \pi (D_{i+1} - D_i) \rightarrow \frac{\Delta P}{30} = \frac{\Delta L}{30} = \frac{\pi}{30} (D_{i+1} - D_i) \]  
\[ \rightarrow \varepsilon_{i+1} = \frac{\pi}{30} (D_{i+1} - D_i) \]

where \( P \) and \( D \) are the perimeter and diameter of the TWHC specimens respectively. It should be noted that the length of the strain gauges is 30 mm and thus, \( \varepsilon_{\text{gauge}} = \frac{\Delta P}{30} \). The area of the specimens at each step of loading during the tests can be calculated by the following equation:

\[ A_{i+1} = \frac{\pi}{4} \left( D_i - \frac{30 \varepsilon_{i+1}}{\pi} \right)^2 \]

where \( \varepsilon \) is the average strain at each step of loading which was measured by strain gauges. Thus, true stress was derived at each step of loading during the tests.

4. Laboratory testing program

Specimen preparation is one the most important stages of the test to simulate the actual behaviour of poorly cemented sands, which exist at the drilling site. Most of the samples collected from the drilling site consisted of fine and sub-angular sand grains with random orientations. The specimens were prepared in the laboratory and their grading size and fabric closely resembled that of the poorly cemented sands in top 200 m of the drilling site formation. It was a near-uniform mixture of fine-grained sand, Portland cement and water. Care was taken to produce homogenous specimens; a small mixer was used to ensure that the sand grains, cement and water mixed perfectly. 30-40 minutes was considered as the total period from the start of mixing until sealing the moulds. To avoid sticking the mixture to the
inner dowel, dowels were covered by a plastic film (i.e. Mayla plastic). This plastic film helped to avoid damaging the borehole when extracting the dowel in the demoulding process. Dental paste was used for levelling the ends of specimens. Since this material is a very fine powder, it can fill the void even better than gypsum (water-dissolved). Also, the compression strength of dental paste is more than that of gypsum. According to the initial results, using dental paste helped to reproduce the test results and reduce the bedding error to the minimal value than was in the tests for the specimens without dental paste at both ends.

4.1. Sand grain size distribution

Sieve analysis was conducted based on the calibrated ASTM C-136 sieves. The chemical analysis of the grains showed that the formation comprises of quartz grains (≥ 96%) with a weak cementation including clay and calcite between sand grains. Australian natural well sorted silica sands of two different grain size ranges were identified as the best suited materials for reproducing samples collected from poorly cemented sandy layer at the drilling site ($\rho = 1720 \text{ kg/m}^3$). The mean diameter value ($D_{50}$) was 0.56 mm for the silica dry sand with grain sizes between 0.425 mm - 1.4 mm (coarse) and was 0.20 mm for grain sizes between 0.125 mm - 0.355 mm (fine) (Hashemi et al., 2014b). A remarkable analogy was found between the laboratory specimens and those collected from the drilling site in terms of grain seize and material.

4.2. Water content

The optimum water ratio for cement-sand mixture was derived by using the standard Proctor test compactor hammer; with the energy of 0.55 Nm/cm$^3$ was achieved 9.7~10.3%. It should be mentioned that to avoid the formation of separate layers, the top of each layer was scratched by spatula. The preliminary results demonstrated that considering a cement-sand mixture or only sand grains does not have a significant effect on the optimum water content and dry density.

4.3. Curing time

Different curing time applications can be found in previous studies (Saidi et al., 2005, Saidi et al., 2003, Alsayed, 2002). Curing time ranging from 2 days to 12 days was examined to determine the most suitable option for preparing specimens with a texture similar to the
problematic sandy formation at the drilling site and with the UCS between 2 MPa to 4 MPa. This curing time included both the time when the specimen was in the mould and when it was out of the mould in a plastic wrap. Finally, 8 days were identified as the optimal curing time for the specimen, which comprised of keeping the specimen for 5 days in the mould and 3 days outside of it in a plastic wrap at temperatures between $18 - 22^\circ$ Celsius.

4.4. Cement content

Portland cement has been suggested as a suitable substitute for the natural cement agents for making cemented-sand specimens in different studies (Saidi et al., 2005, Saidi et al., 2003, Kongsukprasert et al., 2005). However, the range of Portland cement contents ($w_c$) used for creating poorly bonded synthetic specimens in previous studies is broad. Kongsukprasert et al. (2005) applied maximum of 2.5% Portland cement to prepare poorly cemented specimens, while (Saidi et al. (2005) used 9-18%. As per authors’ investigations, for the mentioned grading sizes of sand grains the specimens could not be de-moulded with a sound integrity even after 8 days of curing time when $w_c \leq 2.5\%$.

A wide range of specimens with respect to different amount of $w_c$ and $\delta$ were prepared and tested. Eventually, for the considered sand sizes, $w_c = 6\%, 7\%$ and $8\%$ were selected. It should be noted that preliminary test results showed that grading size distribution plays an important role in strength of specimens and $w_c$ value for any testing program must be chosen based on the certain particle size distribution. In a more tangible observation, it was assumed that sand grains should be debonded in case of scratching with normal pressure by finger after the cured time.

4.5. Testing set-up

The moulds were designed to include two different inner dowels with 10 mm and 20 mm diameters to create a hole in the specimens. TWHC specimens of 63 mm $\times$ 27 mm were prepared to be fitted into a modified HQ Hoek cell. The cell with a deployed micro camera was designed for monitoring the borehole wall status and the process of borehole failure during the tests. Two tapered shape hollow platens were designed by ABAQUS software package with 35 mm on the top and 25 mm and 10mm on the bottom which were manufactured and used for 20 mm and 10 mm borehole size specimens respectively. The UCS tests on cell platens showed, that the strain of the platens was less than 0.005% when a maximum of 100 kN loading force was applied which was quite higher than the strength of
the prepared sand-cement specimens. These pieces of equipment were synched with an automatic hydraulic maintainer of confining pressure which was capable to apply low confining stresses (maximum 6 MPa) with 0.001 MPa of accuracy. The specimens were adequately quite weak to be a representative for poorly cemented formations. In order to avoid applying the weight of cell apparatus on the specimens during the test, a wooden base was manufactured which carried the weight of the cell during the test (Fig. 1). With this arrangement, there was no need of external forces to hold the cell steel before transforming to the loading machine. Performance of the modified test cell was verified by testing several prepared specimens under different stress combinations to ensure the accuracy of the results. Fig. 2 shows a flowchart that summarizes the process of the testing program in the current study, and Table 1 represents the mechanical properties of the prepared specimens (Hashemi et al., 2014b).

![Fig. 1 Wooden base to carry the triaxial cell weight and prevent it from acting on the specimen before the test.](image)

**Table 1** Properties of the prepared poorly cemented sand specimens

<table>
<thead>
<tr>
<th>Mechanical properties</th>
<th>Porosity ($\omega_c$)</th>
<th>Tangent elastic modulus ($E_{tan}$) (GPa)</th>
<th>Uniaxial compressive strength (UCS) (MPa)</th>
<th>Poisson’s ratio ($\nu$)</th>
<th>Coulomb parameters ($c$ (MPa), $\phi$ (°))</th>
<th>Bulk density ($\rho$ (kg/m$^3$))</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_c = 6%$</td>
<td>26 ± 3%</td>
<td>2.262</td>
<td>0.62</td>
<td>0.294</td>
<td>1.11, 29.67</td>
<td>1954</td>
</tr>
<tr>
<td>$w_c = 7%$</td>
<td>26 ± 2%</td>
<td>2.465</td>
<td>1.05</td>
<td>0.250</td>
<td>1.38, 30.06</td>
<td>1969</td>
</tr>
<tr>
<td>$w_c = 8%$</td>
<td>26 ± 2%</td>
<td>3.360</td>
<td>1.90</td>
<td>0.247</td>
<td>1.59, 29.67</td>
<td>1974</td>
</tr>
</tbody>
</table>
Fig. 2 a typical Flowchart for the performed experimental study of borehole stability in poorly cemented sands
4.6. Stress Paths

To investigate the effect of stress paths on the behaviour of TWHC, five different stress paths (Eq. 2a-e) were designed and applied to the specimens. Solid cylindrical specimens were treated as an element and the applied boundary stresses were considered as principal stresses. But, for the TWHC specimens, the principal stresses at the borehole wall were calculated based on the applied stresses on the boundary.

Two approaches were used for analysing the stress paths; 1- considering the stresses on the boundary of the specimens, 2- considering certain stresses at the borehole wall. In the first approach (the first and second stress paths) the vertical stress ($\sigma_z$) and the confining pressure ($\sigma_{conf}$) were considered and the induced stresses at the borehole wall were calculated subsequently based on the applied stresses on the boundary of the TWHC specimens. In the second approach (the third, fourth and fifth stress paths) the principal stresses ($\sigma_\theta, \sigma_r, \sigma_z$) for an element on the borehole wall were considered and the stresses were applied to the boundaries of the specimen to induce the desired magnitude of tangential and vertical stresses on the borehole wall and on both ends of the specimen respectively.

In the first stress paths (Eq. 2a) both the vertical stress and the confining pressure were increased simultaneously at the same rate up to the pre-defined value which simulated hydrostatic condition acting on the boundary of the specimens. Then, in the second step, the specimen was subjected to a vertical loading increment (normal faulting) corresponding to a constant displacement rate of 0.07 mm/min. During the test the level of the confining pressure applied to the external surface of the specimens was kept constant by an automatic pressure gauge system. In the second stress path when the stress level reached to the hydrostatic condition, the confining pressure (reverse faulting) was increased corresponding to a constant pressure rate of 0.2 MPa/min. The lateral displacement rate was measured by four 30 mm lateral strain gauges which were connected to the data acquisition and linked to the hydraulic pump systems.

In the third and fourth stress paths $\sigma_\theta$ and $\sigma_z$ were increased simultaneously up to a certain value. Then, as per the experiment plan the specimen was subjected either to $\sigma_z$ or $\sigma_\theta$ increment. In order to achieve the same tangential and vertical stresses at the borehole wall in the initial step of the test the confining pressure value was derived in terms of the tangential stress and the rate of increasing the vertical stress was kept constant and was the same as the
rate of increase of the tangential stress at the borehole wall. It is worth mentioning that in an unsupported borehole, \( \sigma_r \) at the borehole wall is zero.

In the fifth path, \( \sigma_\theta \) and \( \sigma_z \) were increased simultaneously until the failure at the borehole wall was observed by the real-time camera recording. It should be mentioned that for each TWHC specimen \( \sigma_\theta \) and \( \sigma_z \) were calculated separately based on the stress application area and the borehole size. Experiments were repeated for different cement contents (i.e. 6%, 7% and 8%). Eq. 2a-e shows the principal stress status at the boundary of specimens and borehole wall for each stress path. The difference between the first and fourth stress paths is in the initial step of the experiment. In the first path, the magnitude of \( \sigma_z \) is considerably lower than that for the fourth path until the end of the first stage of the experiment. The same relationship applied to the second and third stress paths.

Fig. 3 shows the different stress paths applied to the TWHC specimens in terms of stress on the specimens versus time of loading. It typically represents the sequence of the induced stresses at the boundary and the borehole walls which were calculated based on the applied stresses to the specimens for different stress paths (Eq. 2). It should be mentioned that the recording of the measured test data was carried out at the time intervals of 0.5 second. The camera recorder program was paired with the data acquisition system to get a parallel recording with respect to the time in order to compare the measured data with the process of the borehole failure during the test.

\[
\begin{align*}
1^{st} \text{ stress path} & \quad \begin{cases} 
\sigma_1 = \sigma_2 = \sigma_3 = \sigma_{\text{conf}} = \sigma_z \text{ (first step)} \\
\sigma_1 = \sigma_2 > \sigma_2 = \sigma_3 = \sigma_{\text{conf}} \text{ (second step)}
\end{cases} \\
(2a) \\
2^{nd} \text{ stress path} & \quad \begin{cases} 
\sigma_1 = \sigma_2 = \sigma_3 = \sigma_{\text{conf}} = \sigma_z \text{ (first step)} \\
\sigma_1 = \sigma_2 > \sigma_3 = \sigma_{\text{conf}} > \sigma_3 = \sigma_z \text{ (second step)}
\end{cases} \\
(2b) \\
3^{rd} \text{ stress path} & \quad \begin{cases} 
\sigma_1 = \sigma_2 = \sigma_\theta = \sigma_2 > \sigma_3 = \sigma_r = 0 \text{ (first step)} \\
\sigma_1 = \sigma_\theta > \sigma_2 = \sigma_3 > \sigma_r = 0 \text{ (second step)}
\end{cases} \\
(2c) \\
4^{th} \text{ stress path} & \quad \begin{cases} 
\sigma_1 = \sigma_2 = \sigma_\theta = \sigma_2 > \sigma_3 = \sigma_r = 0 \text{ (first step)} \\
\sigma_1 = \sigma_2 = \sigma_\theta > \sigma_2 = \sigma_\theta > \sigma_r = 0 \text{ (second step)}
\end{cases} \\
(2d) \\
5^{th} \text{ stress path} & \quad \begin{cases} 
\sigma_1 = \sigma_\theta = \sigma_2 = \sigma_\theta > \sigma_3 = \sigma_r = 0 \text{ (first step)} \\
\sigma_1 = \sigma_\theta = \sigma_2 > \sigma_3 > \sigma_r = 0 \text{ (second step)}
\end{cases} \\
(2e)
\end{align*}
\]
5. Results

Laboratory tests were conducted on TWHC synthetic specimens to investigate the failure behaviour of poorly cemented sands in the vicinity of a drilled borehole under different stress paths. Video recording by an installed camera was employed to identify the instance of the borehole failure onset and to locate the damaged zone at the borehole wall. As mentioned in section 4.3 main tests were performed for three different cement contents ($w_c$) and for two borehole diameters of 10 mm and 20 mm.

5.1. UCS test results on TWHC specimens

Fig. 3 The loading sequences in five different stress paths corresponding to the stresses at the boundary of specimens (left) and stresses calculated at the borehole wall (right) (a) The first path (b) The second path (c) The third path (d) The fourth path (e) The fifth path.
The unconfined compressive strength (UCS) tests using foil strain gauges and external strain gauge for measuring the strains were conducted on TWHC specimens (Fig. 4) to study the borehole behaviour in the absence of $\sigma_\theta$. The specimens deformed until their maximum strength was reached and after the peak stress they demonstrated strain-softening behaviour with a steep slope in the stress-strain graph (Fig. 4b). According to the recorded videos, even after a large deformation (1.7 mm) there was no sand particle dislocated from the borehole wall and the borehole was still intact after the test. A number of long and inclined cracks sub-parallel to the maximum principal stress were observed on the surface of the specimens (Fig. 4a). This implies that when there is no tangential stress around the borehole even in the presence of high vertical stress the borehole instability will not occurs in the TWHC specimens.

![Fig. 4](image)

**Fig. 4** (a) The UCS test on TWHC specimens showed that there is no borehole breakout when $\sigma_\theta = 0$ (b) stress-strain diagram for TWHC specimen under uniaxial stress test for $w_c=6\%$, $7\%$ measured by foil and external strain gauges
5.2. The boundary analysed stress paths

5.2.1. Stress-strain relationship

As mentioned in section 4.4, in the first and second stress paths $\sigma_z$ and $\sigma_{conf}$ were applied to the boundary of TWHC specimens without considering the stress values at the borehole wall during the tests. Then the induced stresses at the borehole wall were calculated based on the applied stresses on the boundary. Fig. 5 represents the deviatoric stress in the direction of the main principal stress ($\sigma_1$) versus $\varepsilon_1$ for the 10 mm diameter borehole for different cement contents and lateral pressures.

The specimens showed more ductile behaviour when $\sigma_1 = \sigma_z$ and the borehole failure was observed at higher strains ($\varepsilon_1$) for the first stress path than for the second path. This suggests the important influence of the tangential stress as the maximum principal stress on the borehole failure in poorly cemented sands.

The stiffness in the initial stage of the tests (almost linear part) was similar in both stress paths which shows the homogeneity of the specimens in vertical and lateral directions in the first and second stress paths respectively. With increasing the cement content, the stiffness of the specimens in the vertical direction obtained higher values than in the lateral orientation for the second path. However, the rate of increase in stiffness was lower for the second path. Increasing the $w_c$ increases the lateral stiffness ($\varepsilon_2$) of the specimens in the second path and the same behaviour was observed in the vertical direction in the first stress path.

Borehole failure was observed in the strain-hardening section of the stress-strain diagram when the tangential stress was the maximum principal stress ($\sigma_1$), while in the case of $\sigma_z = \sigma_1$, borehole instability was observed in the strain-softening or residual stress condition (especially for lower $\sigma_2$ values). For $\sigma_2 = 1 - 2\, MPa$ the strength of the TWHC in the second path was higher for all cement contents. It means that with elevating $\sigma_2$ in the lateral and vertical directions in the first and second stress paths respectively, microcracks will be developed at the borehole wall and the effect of the supporting force to avoid borehole failure in the lateral direction (for the first path) was more considerable than in the vertical direction (for the second path).
Fig. 5 Comparison between the results from the first and second stress path for (a) $w_c = 6\%$ (b) $w_c = 7\%$ (c) $w_c = 8\%$. 
Fig. 6 shows that the effect of $w_c$ on stabilising a borehole is more pronounced than that of the vertical stress ($\sigma_2$) for the second path. For instance, a specimen with $w_c = 6\%$ and under $\sigma_z = 4\ MPa$ has a lower strength than a specimen with $w_c = 8\%$ and under $\sigma_z = 1\ MPa$. However, for the first stress path, this was not the case and the effect of $\sigma_{conf}$ was more remarkable as shown in Fig. 6a.

Unlike for the $\varepsilon_1$ direction the specimens experienced lower strain in the direction of $\sigma_2$ for the first path than for the second path as shown in Fig. 7. This figure shows that for a given $w_c$, with an increase in the lateral pressure, the borehole failure initiates at a lower lateral strain and the ductility in the direction of $\sigma_{conf}$ decreases for the first path. Also, increasing the confining pressure results in a larger ductility. This confirms the effect of the lateral support pressure on preventing the borehole instability in the first path versus the influence of the vertical support pressure in the second path. Also, with increasing the supporting force, the strain in its direction decreased in the first path. But, as shown in Fig. 3 a-b, the influence of the vertical stress ($\sigma_2$) on the strain in the corresponding direction is not straightforward for the second path. This may be due to the trivial influence of $\sigma_2 = \sigma_z$ on the borehole instability versus $\sigma_2 = \sigma_\theta$. Generally, for the second path with an increase in $w_c$, the vertical strain increased for a certain $\sigma_z$. 

(a)
Fig. 6 Effect of the cement content and $\sigma_2$ on the stress-strain behaviour of poorly cemented sand TWHC specimens for (a) the first stress path (b) the second stress path
5.2.2. Volumetric strain analysis

The lateral strains were measured during the tests on TWHC specimens to study the effect of stress and cement content on the volumetric strain. The volumetric strain versus the strain in the direction of the maximum principal stress is plotted in Fig. 8. It shows that, at lower confining pressures (i.e. 1 and 2 MPa), the specimens entered the dilation mode, and with an increase in the lateral pressure the borehole instability commenced in the contraction mode for the first stress path. This occurred because the high lateral pressures created borehole convergence and therefore, the size of the sample in the lateral direction was reduced. In the second path, the specimens did not reach the dilation mode and borehole instability was observed in the contraction mode. However, the trends of volumetric strains were toward the negative value or dilation mode. Also, comparison between the results from the first and second paths showed that in a certain $\sigma_2$ and $w_c$ the volumetric strain is higher for the second path.
Fig. 8 The volumetric strain versus the strain in the direction of the maximum principal stress ($\varepsilon_1$) for (a) $w_c = 7\%$ (b) $w_c = 8\%$.

Fig. 9a also shows the volumetric strain versus the average axial strain for different values of $w_c$ in $\sigma_{conf} = 1\ MPa$ for the first path. It illustrates that with an increase in $w_c$, the lateral strain increases when compared to the axial strain and prior to the borehole failure in the specimen the axial contraction of specimens with higher $w_c$ is less than for the specimens with a lower cement content. In other words, an increase in the $w_c$ results in the decrease in
the pore spaces on the borehole wall, and the application of the axial deviatoric stress causes less contraction in the axial direction. Therefore, for higher $w_c$ the lateral strain was more dominant in comparison to the axial contraction. Based on the real-time borehole observations, it can be stated that, in the samples that failed in dilation, the transition from the contraction to the dilation occurred before the borehole breakout initiation. However according to Fig. 9b, with an increase in cement content, the volumetric strain increased for a certain $\sigma_2$. Since $\sigma_1$ was the confining pressure in the second path, obviously the value of the lateral strain will increase when increasing the $w_c$ due to higher strength effect of the cement agent in the TWHC specimens. This confirms the key role of the lateral strain in determining the volumetric strain in the prepared specimens.

**Fig. 9** Effect of the cement content on the volumetric strain of TWHC specimens for (a) the first stress path (b) the second stress path.
5.2.3. Size-scale effect

As discussed in section 4.3, the specimens were prepared in two borehole sizes; 10 and 20 mm. The same process was used for the preparation of both borehole size specimens including the grading size, curing time and compaction force. Experiments were conducted on 20 mm borehole specimens for three different \( w_c \) values and under different stress paths. Fig. 10 presents the deviator stress versus the axial strain \( (\varepsilon_a) \) for three different cement contents and different lateral pressures. As expected, the strength of the 20 mm size borehole TWHC specimens at failure is generally lower than that for the smaller borehole size specimens especially at higher lateral pressures due to size-scale effect which was shown by (Carpinteri, 2002). Also, it shows a considerable decrease in the ductility of the 20 mm borehole specimen versus that for the specimen with a 10 mm borehole in the first stress path. It is worth to mention that the stiffness of the specimens does not significantly change with the increase in the borehole size. This is in agreement with (Mogi, 2007) who suggested that the stiffness depends on the rock material and, since the same mixtures were used for both specimens with different borehole sizes, the same stiffness was observed. Also, with an increase in the cement content and lateral pressure the strength increased and the effect of \( w_c \) was more pronounced in the 20 mm borehole size specimens.

![Fig. 10](image.png)

**Fig. 10** The deviator stress versus the axial strain \( (\varepsilon_a) \) for three different cement contents and different lateral pressures in the first path for 20 mm borehole specimens.
5.3. Stress paths based on an element at the borehole wall

5.3.1. The third stress path

As mentioned in section 4.4 $\sigma_\theta$ and $\sigma_z$ were calculated for an element at the borehole wall based on the lateral and axial stresses applied on the boundary of the TWHC specimens. The stresses were applied to the specimen at the same rate until a given value was reached. Then, $\sigma_\theta$ was increased until an instability at the borehole wall was observed, while $\sigma_z$ was kept unchanged. In fact, the main difference between the second and third stress paths was in the initial stage of the test, which the axial stress applied to the specimen was considerably higher in the third path for the corresponding borehole size and specimen outer diameter. Fig. 11 demonstrates the deviatoric stress versus the lateral strain ($\varepsilon_1$) for the third stress path for three different cement contents and $\sigma_z = \sigma_z = 1 - 4 \text{MPa}$ and $\sigma_3 = \sigma_r = 0$. It should be mentioned that the deviatoric stress was derived based on the principal stresses at the borehole wall due to the applied stresses on the boundary in each step as follows;

\[
q = \sigma_{ij} - \sigma_{Hyd} \tag{3a}
\]

\[
\Rightarrow q_i = \sigma_{\theta i} - \left(\frac{\sigma_{\theta i} + \sigma_x + \sigma_r i}{3}\right) \tag{3b}
\]

According to Fig. 11, with an increase in $\sigma_z$ the strength of the specimens at borehole failure was higher for all $w_c$ and the instability was observed in larger lateral strains. It shows the effect of $\sigma_z$ as a supporting pressure in increasing the ductility for a given $w_c$ and the specimens showed more ductile behaviour in higher vertical stresses. Also, with increasing $w_c$ the bonding breakage at the borehole wall was observed in lower lateral strains.

Fig. 11 also shows the deviatoric stress versus the vertical strains. The trends of graphs are similar to the lateral strains. However, generally the magnitude of strains in the direction of $\sigma_z$ was lower as expected. Since specimens are not solid and there is a hollow space in them, the pressures applied in the lateral direction will not be transferred in the vertical direction due to Poisson's effect, but will cause a borehole convergence. Also, the ductile behaviour of poorly cemented sands in both orientations results in debonding of sand particles at the borehole wall and a lower vertical strain was observed when compared with that for solid specimens.
Fig. 11 The deviatoric stress versus the lateral strain ($\varepsilon_1$) and axial strain ($\varepsilon_2$) in the third stress path for three different cement contents for (a) $w_c = 6\%$ (b) $w_c = 7\%$ (c) $w_c = 8\%$.

5.3.2. Volumetric strain for the third stress path

Fig. 12 shows the average volumetric strains versus the lateral and axial strains for the third stress path for $w_c = 6\%, 7\%$. As can be seen in this figure, at lower axial stresses (i.e. $\sigma_z = 1, 2 \text{ MPa}$) the specimens began contracting from the lateral orientation first and after 0.35%-0.5% of lateral compressive strain, they tended toward axial dilation. However, unlike in the first stress path in none of the conducted tests in the third path the specimens reached the negative volumetric strain. Increasing the vertical stress kept the specimen in a contraction mode more and the dilation began at higher lateral strains. With an increase in $\sigma_z$ the vertical strength of the specimens will increase and the axial dilation will be lower for the same lateral strain.

As mentioned in the last section, after the elastic phase on the stress-strain diagram, the specimens exhibited strain hardening behaviour. Thus, for vertical stresses of greater than 2 MPa the specimens remained in contraction mode and the volumetric strain was not decreased until observation of the borehole failure. The results also showed that with an increase in $w_c$ the volumetric strain will be increased for certain $\sigma_z$ in this path similar to that in the second stress path. This confirms that with an increase in $w_c$ the lateral strain increased in all the stress paths. However, the effect of changing the cement content on the volumetric strain was less significant than the effect of changes in the vertical stress. For instance, at
axial stress of 2 MPa, increasing $w_c$ from 6% to 8% did not result in any divergence in the behavioural trend from contraction to dilation in the specimens.

Fig. 12 The average volumetric strains versus the lateral and axial strains for the third stress path for (a) $w_c = 6\%$ (b) $w_c = 7\%$. 
5.3.3. 20 mm borehole for the third path

The experiments were conducted on TWHC specimens with 20 mm borehole sizes under the third stress path as well in order to compare the results with results from 10 mm borehole size specimens. Fig. 13 shows that the strength of the 10 mm borehole size specimens is considerably higher than that for 20 mm sizes as in the case of the first stress path. Also, for a 20 mm borehole size sample, borehole failure was observed after the peak stress. However, for the 10 mm borehole size specimens the camera recording captured the borehole collapse in the strain-hardening section of the stress-strain curve. Also, the lateral ductility was significantly lower in the specimens with larger borehole sizes. In other words, the instability in a borehole with 20 mm diameter occurred at a lower strain compared to that for a smaller diameter borehole (10 mm).

Fig. 13 also shows the deviator stress versus axial strain for 10 and 20 mm borehole size specimens. It shows that the effect of the vertical stress on the axial strain was more significant in 10 mm borehole size specimens than in specimens with larger boreholes. It shows that, with increasing $\sigma_z$, the axial strain decreases for the 20 mm borehole size and the same $\sigma_c$ values. However this was not the case for the 10 mm borehole size specimens. This may be due to the weaker influence of the vertical stress on inducing axial strain when the tangential stress is the maximum principal stress. However, $\varepsilon_1$ increased with increasing $\sigma_\theta$ for both borehole size specimens which shows the important role of $\sigma_1$ in borehole failure in this weak material. It is worth mentioning that the axial stiffness remained unchanged and was unaffected by the increase in the borehole size from 10 to 20 mm.
Fig. 13 The deviator stress versus the lateral strain ($\varepsilon_1$) and the axial strain ($\varepsilon_2$) for 10 mm and 20 mm borehole size specimens for $w_c = 6\%$ and different axial stresses ($\sigma_2$) for the third stress path.

5.3.4. The fourth stress path

Fig. 14a shows the stress-strain relationships for the fourth stress path for different cement contents in $\sigma_\theta = 4 \text{ MPa}$ at the borehole wall. In this path the tangential stress at the borehole wall was increased along with the vertical stress at the same rate of up to 4 $\text{MPa}$. Then $\sigma_z$ was increased while $\sigma_\theta$ was kept unchanged. It shows that in $w_c = 6\%, 7\%$ the axial strain is higher than the lateral strain at the instant of borehole instability onset and $w_c$ has a significant effect on increasing the strength of the specimens. Also, the pre-peak stiffness of the specimens increased with increasing $w_c$ at the same $\sigma_\theta$. The volumetric strain of the specimens decreased with increasing the cement content in certain $\sigma_\theta$ in this stress path. This result agrees with the results obtained from the first path.

The trend of the stress-strain relationships in the fourth path was similar to ones in the first path where the axial stress was much lower than that for the fourth path in the initial stage of the test. Fig. 14b shows the total vertical stress versus the axial strain for the first and fourth stress paths for $\sigma_\theta = 4 \text{ MPa}$. It shows that the borehole instability occurred in a higher stress and lower axial strain for the fourth path in different cement contents. Since the initial vertical stress was higher for the fourth path, the specimens showed a more brittle behaviour.
before the borehole failure. This phenomenon may be due to the effect of the loading rate on the strength of poorly cemented sands.

![Diagram](image)

**Fig. 14 (a)** The stress-strain relationships in the fourth stress path for different cement contents and for \( \sigma_0 = 4 \text{ MPa} \) at the borehole wall (b) total vertical stress (\( \sigma_1 \)) versus axial strain for the first and fourth stress paths in \( \sigma_0 = 4 \text{ MPa} \) and \( w_c = 7\%, 8\% \).

To compare the results for the third and fourth stress paths the deviatoric stresses were plotted versus the strain in the orientation of the maximum principal stress, which is in lateral and axial directions respectively, and for different cement contents were calculated and superimposed in Fig. 15. It shows that the strength of specimens in the third path was
significantly higher when increasing the tangential stress in comparison to the vertical stress increase. As can be seen from Fig. 15, the specimens showed more ductility with a less stiffness under fourth stress path. Thus, after around 1.3% of the axial strain in the vertical direction horizontal localised zone was generated at the borehole wall and debonding of sand particles was viewed by the camera unlike in the third stress path. For the third path due to the arching effect phenomenon at the borehole, specimens underwent lower lateral strain with a higher strength.

**Fig. 15** Comparison between the third and fourth stress paths in terms of the deviatoric stress and strain in the direction of the maximum principal stress and strain in $\sigma_2$ direction.

5.3.5. *Fifth stress path*

In the fifth stress path the TWHC specimens were subjected to $\sigma_\theta$ and $\sigma_z$ increment simultaneously until the borehole failure was captured by the micro camera. Surprisingly, it was observed that the specimens could tolerate high levels of stresses and after reaching 50 kN of axial force which was the maximum capacity of the usual load cell, the borehole instability was not even seen in a specimen with $w_c = 6\%$. Thus, the load cell was replaced with a 250 kN one and the tests were repeated for this stress path.

Fig. 16 represents the results for three different $w_c$ values (6%, 7% and 8%) in the fifth stress path. As shown in Fig. 16, increasing $w_c$ from 6% to 8%, significantly enhances the
level of the failure strength. Also, the pre-peak stiffness increased dramatically with an increase in the cement content and the specimens exhibited strain-hardening behaviour up to the maximum stress.

It is worth to mention that for the first and second stress paths where the confining pressure and axial stress were increased at the same time, for the lateral pressures higher than 4.5 MPa, the first stage of the experiment (i.e. hydrostatic stress status) could not be completed and the borehole instability was observed prior to commencing the second stage. But specimens could withstand more than 35 MPa of tangential stresses and axial stress simultaneously. The only difference was the magnitude of the tangential stress in different paths and it emphasizes the influence of the application of stress in different paths on the borehole failure. Moreover, the significance of using the micro camera during the TWHC tests was more obvious when the axial strain gauges continued working even after the failure, and the onset of borehole instability was identified only by the installed micro camera inside the top platen.

Fig. 16 Effect of $w_c$ values (6%, 7% and 8%) on the strength of TWHC specimens for the fifth stress path in axial and lateral directions.

Fig. 16 also shows that in case of elevating the tangential stress and vertical stress the magnitude of the lateral strain is lower than the vertical strain at the same $w_c$ respectively. This may be due to the larger size of specimens in the vertical direction and more strain allocation to the specimen in this stress condition. However the trend of graph for lateral and
vertical direction stayed unchanged which suggests the strengthening effect of the cementing agent in both directions. As expected the specimens were in contraction mode in axial and lateral directions during the tests and no sign of dilation was recorded by the strain gauges. Also, with increasing the cement content the volumetric strain increased. This was due to the higher stresses, which were tolerated by the specimens with higher \( w_c \) and therefore more volumetric strains were induced in the specimens.

5.4. Failure stress quadrilateral

Based on the conducted tests on the TWHC specimens under different stress paths, a failure stress quadrilateral was developed for estimating the other stress status that causes the borehole instability in poorly cemented sands. Eq. 4 shows the different stress conditions which were considered in the current testing program for an unsupported borehole \( (\sigma_r = 0) \).

\[
\begin{align*}
\sigma_\theta &> \sigma_z \\
\sigma_z &> \sigma_\theta \\
\sigma_\theta & = \sigma_z 
\end{align*}
\]

Fig. 17 shows the \( \sigma_z \) versus \( \sigma_\theta \) for different stress paths for an element at the borehole wall in \( \sigma_2 = 4 \text{MPa} \). It shows that for the third stress path where the \( \sigma_z \) was kept constant after the first stage of the experiment (point A), with an increase in \( w_c \), \( \sigma_\theta \) increased parallel to the x-axis. Also, for the fourth stress path where the \( \sigma_\theta \) was kept unchanged in the main part of the test (from point A), with increasing \( w_c \), \( \sigma_z \) increased parallel to the y-axis. In the fifth path the graph is the bisector of x and y axes and with increasing the cement content, the tangential and vertical stresses increased accordingly. Fig. 17 shows three different similar quadrilaterals, which have almost parallel sides for three different \( w_c \). This suggests that increasing the cement content generates a proportional increase in the strength of the specimens in different stress paths. Also, any combination of \( \sigma_z \) and \( \sigma_\theta \) at the borehole wall which is located inside this quadrilateral cannot induce borehole failure and if the point is located on the boundary the borehole instability will occur. It should be mentioned that for lower \( \sigma_2 \) values a smaller failure stress quadrilateral can be plotted.

In addition, the non-right slope of lateral sides of the developed failure stress quadrilaterals indicates the effect of the support pressure which can enhance the stability of a borehole. In other words, point B at the borehole wall can undergo maximum of \( \sigma_\theta = 28.60 \text{MPa} \) when \( \sigma_z = 4 \text{MPa} \). But, if the \( \sigma_z \) is increased to 10 MPa, the specimen can tolerate about \( \sigma_\theta = \)
30 MPa (point C). This can be applied to the fourth path as well which is the other inclined slope of the quadrilateral. However the higher slope of the right side versus the left side indicates that the effect of increasing $\sigma_\theta$ is more pronounced than that for $\sigma_z$ as the maximum principal stress. In other words, in case of increasing the $\sigma_\theta$ for 10%, $\sigma_z$ should be increased for 20%, but in point D, in case of increasing $\sigma_z$ for 10%, $\sigma_\theta$ should to be increased only for 12% to avoid the borehole instability.

Therefore, for the range of tested $w_c$, it is possible to predict the borehole instability in different stress conditions based on the introduced failure domain. The stresses at the borehole should be calculated based on the far-field stresses and the location of that point must be identified on the failure stress quadrilateral. If the point is inside the stress quadrilateral the borehole will remain stable under the existing in situ stresses. Also, it shows that the rate of increase in the tangential stress is not the same as that for the vertical stress. It means that the boreholes experienced instability in lower $\sigma_z$ when it was the maximum principal stress than when $\sigma_\theta = \sigma_1$.  

![Failure stress quadrilaterals for estimating the stress status that causes borehole instability in poorly cemented sands for $w_c = 6\%, 7\%, 8\%$](image)

**Fig. 17** Failure stress quadrilaterals for estimating the stress status that causes borehole instability in poorly cemented sands for $w_c = 6\%, 7\%, 8\%$
6. Conclusion

Five different stress paths including normal and strike slip faulting stress regimes were designed and applied to synthetic TWHC poorly cemented sand specimens. The borehole status was monitored by a micro camera installed in the top platen. Three different cement contents and 2 borehole sizes were investigated. Two approaches were considered for applying the stresses to the specimens including the far-field stresses (first and second paths) and an element at the borehole wall (third, fourth and fifth paths). The UCS tests on the TWHC showed that in the absence of the tangential stress no sign of instability was observed at the borehole wall.

The effect of the cement content on the borehole stability was more pronounced than the effect of the change in the vertical stress (σ₂) for the second path. However for the first stress path, this was not the case and the effect of σ₀ = σ₂ was more remarkable. This confirms that σ₀ has a major effect on the instability of a borehole and it is even more than the effect of the cementation level.

In the first path with an increase in wᵋ the volumetric strain decreased. However, in the second path the volumetric strain increased for a certain σ₂. This confirms the key role of the lateral strain in determining the volumetric strain in poorly cemented sands.

Comparison between the results in the first and second stress paths showed that in a certain σ₂ and wᵋ the volumetric strain is higher for the second stress path.

With an increase in σ₂ in the lateral and vertical directions in the first and second stress paths respectively, microcracks were developed at the borehole and the effect of the supporting force to avoid the borehole failure in the lateral direction (for the first path) was more considerable than in the vertical direction (for the second path).

The strength of the 20 mm size borehole TWHC specimens at failure is generally lower than that for the smaller borehole specimens especially at higher lateral pressures due to size-scale effect in all the tested stress paths. In addition, the effect of the supporting stress on ε₁ was more significant in 10 mm borehole size specimens in both first and third path.

For the third stress path the effect of σ₂ as a support pressure was significant in increasing the ductility for a given wᵋ and the specimens showed a more ductile behaviour in higher vertical stresses. Also, in none of the conducted tests, specimens met the negative volumetric strain unlike the first stress path. Increasing the vertical stress kept the specimen in a more contraction mode and dilation began at the higher lateral strain values. The effect of changing
the cement content on the volumetric strain was less significant than changes in the vertical stress.

In the fourth stress path, the axial strain was higher than the lateral strain at the onset of the borehole instability and \( w_c \) had a significant effect on elevating the strength of the specimens. Also, the pre-peak stiffness of the specimens increased with increasing \( w_c \) at the same \( \sigma_\theta \) and in general the confining pressure was observed to have a trivial effect on the stiffness. Comparing the third and fourth stress paths suggested that the supporting effect of the vertical stress was more remarkable than the effect of the lateral pressure and the specimens could tolerate higher tangential stresses before failure. Due to more than 1.3% of axial strain in the axial direction, a horizontal localised zone was observed at the borehole wall by the camera unlike in the case of the third stress path. For the third stress path due to the induced arching effect phenomenon around the borehole, specimens underwent lower lateral strain and exhibited a higher strength.

The results showed that with an increase in \( w_c \) the volumetric strain will increase for certain \( \sigma_z \) in both the second and third stress path. However, the volumetric strain of the specimens decreased with increasing the cement content in certain \( \sigma_\theta \) in the first and fourth path.

Increasing the cement content by a small amount, otherwise keeping the same sample preparation and test conditions, significantly increases the stiffness of the specimens in the fifth path.

A new failure domain was determined based on the considered stress paths for poorly cemented sands. Three different similar quadrilaterals, with almost parallel sides for three different \( w_c \) were defined. Any combination of \( \sigma_z \) and \( \sigma_\theta \) at the borehole wall which is located inside this quadrilateral cannot induce borehole failure and if the point is located on the boundary the borehole instability will occur.

Acknowledgements

This work has been supported by the Deep Exploration Technologies Cooperative Research Centre whose activities are funded by the Australian Government’s Research Programme. This is DET CRC Document 2015/446.

References


# Statement of Authorship

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<td>Publication Status</td>
<td>○ Published, ○ Accepted for Publication, Θ Submitted for Publication, ○ Publication style</td>
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A new sand dislocation criterion at boreholes drilled through poorly cemented sandy formations

S.S. Hashemi*, N. Melkoumian

Abstract

The breakage of bonding between sand particles and their dislodgment from the borehole wall are among the main factors resulting in a borehole failure in poorly cemented granular formations. The sand particle debonding usually precedes the borehole failure and it can be considered as a sign that the onset of the borehole collapse is imminent. Detecting the bonding breakage point and introducing an appropriate failure criterion will play a key role in borehole stability analysis. To investigate the influence of different factors on the initiation of particle debonding at the borehole wall, a series of new laboratory tests was designed and performed on synthetic poorly cemented sand specimens. The tests were devised to allow visual observation of the onset dislodgment of particles from the borehole wall under various stress paths and for different cement contents. This study resulted in a more realistic prediction of the actual behaviour of this formation in the vicinity of a drilled borehole. The total potential and dissipative absorbed strain energy per volume of material up to the point of the observed particle debonding was calculated. The results showed that the particle bonding breakage point at the borehole wall was reached both before and after the peak strength of the thick-walled hollow cylinder specimens depending on the stress path and cement content. Three different cement contents and two borehole sizes were investigated to study the influence of the bonding strength and scale on the particle dislocation. Test results showed that the stress path has a significant effect on the onset of the particle bonding breakage. Also, it was shown that for different stress paths there is a near linear relationship between the absorbed energy and the normal effective mean stress.

Keywords: Borehole stability; Experimental studies; Total absorbed strain energy; Poorly cemented sands; Particle bonding breakage

1. Introduction

Borehole stability analysis is one the most challenging topics in geomechanics. Shallow depth boreholes with diameters ranging from 100 mm-600 mm and at depths of 30 m-400 m are often drilled for exploration purposes or direct geothermal applications. In vertical loop systems for direct-use of geothermal energy, boreholes are usually no deeper than 150 m (Lund et al. 2005). When a borehole is drilled through an unconsolidated formation such as poorly cemented granular formations, it may collapse within a short period of time (i.e. in an hour to a few months) after drilling is completed depending on the magnitude and direction of the underground flow pressure and/or in situ stresses. If the strength of the cementation between the sand particles is not high enough to withstand the extensile stress concentration
the latter may cause grain debonding and this may lead to the borehole failure. This phenomenon tends to return the ground to the state of equilibrium following the stress release due to drilling. If a borehole instability develops, some perforations that were already formed on the borehole wall due to falling out of dislocated grains may cause drilling issues such as stuck-pipe, pack-off and lost circulation. Depending on the magnitude of the induced tangential stress around the borehole, the particle dislocation process may diminish after the formation of a stable arch. However, in some cases, especially in deeper boreholes and for reverse faulting stress regimes where $\sigma_H > \sigma_h > \sigma_v$, the arching effect cannot halt the process of particle dislocation and the borehole will collapse completely (Hashemi et al. 2014a). Bratli and Risnes (1981) identified the driving forces (e.g. in situ stresses and underground fluid flow), formation resisting forces (e.g. friction, cementation and arching effect) and operational approach (e.g. air core drilling and sonic drilling methods) as the main factors affecting the particle debonding at the borehole wall which causes instabilities. Hashemi et al. (2012) numerically simulated the borehole failure in granular materials and showed that the main reason for borehole failure in such formations is the dislocation of particles.

Lee et al. (2012) suggested a numerical model for borehole analysis in a formation which comprises of laminations and weak planes. They showed that borehole failure occurs along or across the rock planes depending on the relative orientation of the borehole and the rock plane trajectory.

In order to avoid borehole instability, different support systems are employed such as mud pressure, casings and polymers. They are used to provide enough support pressure against the borehole wall to keep it open for the service period which can vary depending on the borehole purpose. One of the main stability problems in formations with different strength layers is the considerable difference between the required mud pressures. Low leakage pressure requires a low drilling density to prevent lost circulation, while in hard rocks higher mud pressure is required to keep the borehole open (Tan et al. 2013). Therefore, it is essential to determine the differences in failure pressure for poorly cemented granular formation and other stronger strata.

Laboratory tests are among the best-suited methods for studying the borehole instability phenomenon. Number of researchers have utilised the thick-walled hollow cylinder (TWHC) specimens for studying the stability of man-made underground openings such as tunnels, boreholes and oil wells. Alsayed (2002) and Alsayed (1996) performed a series of TWHC tests on rocks to study the effect of anisotropic stress conditions on the behaviour of hard
rocks. Perie and Goodman (1989) investigated the macroscopic failure mechanism of synthetic rocks made of gypsum cement by conducting TWHC test. Ewy and Cook (1990a, b) carried out a valuable experimental study on the behaviour of Indiana limestone and consolidated Berea sandstone in the form of TWHC. Younessi et al. (2013) conducted laboratory tests on the TWHC synthetic sandstone specimens and studied the micromechanical properties of the borehole breakouts.

In this study, a number of TWHC laboratory tests were designed and conducted on poorly cemented sand specimens under controlled conditions. Status of the borehole wall was visually monitored by real-time camera recording in order to determine the point of first particle bonding breakage which leads to dislodging the particle from the borehole wall. The total potential and dissipative absorbed strain energy (modulus of toughness) was derived for different stress paths and cementation strengths and a new borehole failure criterion was introduced. This criterion is more precise than previously suggested criteria based on maximum strength of TWHC specimens, because the onset of borehole failure may take place either prior or after the peak strength of the specimen is reached. The results of this study give a more realistic insight into the actual failure behaviour of poorly cemented granular formations, and they will help to design an enhanced support system to avoid borehole collapse both during drilling and after its completion.

1.1. Induced stresses around a drilled borehole

The in situ stress state can be defined in terms of the principal stresses, \( \sigma_v \), \( \sigma_H \), and \( \sigma_h \). Fairhurst (2003) showed that the maximum vertical stress can be calculated as the weight of the overlying layers at a certain depth and the minimum horizontal stress is estimated by in-field tests such as hydraulic fracturing and leak off tests (Amadei and Stephansson 1997). Nevertheless, measuring the maximum horizontal stress (\( \sigma_H \)) is not straightforward and it can be estimated based on specific assumptions and considerations (Aadnoy et al. 2013; Della Vecchia et al. 2014; Zoback et al. 1985). The tangential, radial and vertical stresses around a borehole may be calculated by different equations such as Kirsch equations based on principal stresses. In literature (Jaeger et al. 2009; Obert and Duvall 1967), there are different closed form solutions for calculating stress and strain in a TWHC by the theory of elasticity. The principal stresses in a cylindrical coordinates system for a TWHC at any point of radial distance, \( r \), are defined as,
\[
\sigma_\theta = \frac{S_o D_0^2 - S_i D_i^2}{D_0^2 - D_i^2} + \frac{(S_o - S_i)D_i^2 D_0^2}{4\pi^2(D_0^2 - D_i^2)}
\]

(1a)

\[
\sigma_r = \frac{S_o D_0^2 - S_i D_i^2}{D_0^2 - D_i^2} - \frac{(S_o - S_i)D_i^2 D_0^2}{4\pi^2(D_0^2 - D_i^2)}
\]

(1b)

\[
\sigma_z = \frac{4F}{\pi(D_0^2 - D_i^2)} + \frac{S_i D_i^2}{(D_0^2 - D_i^2)}
\]

(1c)

where \(\sigma_\theta, \sigma_r,\) and \(\sigma_z\) are the tangential, radial and vertical principal stresses respectively (Fig. 1), and \(S_i\) and \(S_o\) are the uniform internal and external stresses acting on the TWHC. Stress-strain graphs were used to calculate the modulus of toughness. Although the “engineering” stress values can be easily obtained from the test results, in this study the “true” stresses were considered for deriving the precise strain energy values. In this approach, the actual diameter of the specimen in each step of the test was considered for calculating the stress.

**Fig. 1.** Tangential (\(\sigma_\theta\)) and radial (\(\sigma_r\)) stresses induced due to pre-existing far-field stresses around a drilled borehole.

1.2. Basic concepts of Thermodynamics on potential and dissipative energy

1.2.1. Potential Energy

(Li 2001) showed that the macroscopic deformation in materials can be categorized into elastic and inelastic deformations which correspond to different mechanisms at the microscopic level. In other words, material failure comprises of thermal and mechanical processes ranging from an atomic to macroscopic levels. In macroscopic level the elastic
deformation is a collective property of atomic displacements from their original positions (Li 1999). With increasing the far-field stresses, atoms may cross the established energy threshold and jump into a new equilibrium mode of free energy which will result in the bonding breakage and in establishing a new configuration of bonds. Borehole failure will initiate during the process of re-bonding or macroscopic inelastic deformations. In general, a failure criterion based on elastic energy density can be described as;

\[ U_s^e = U_{sc}^e (\phi_1, \dot{\phi}_1, T) \]  

(2)

where \( U_s^e \) is the elastic strain energy density defined on a given control volume. \( U_{sc}^e \) is the critical value, \( \phi_1 \) and \( \dot{\phi}_1 \) are the mechanical dissipation and its rate, and \( T \) is the material temperature. For a material that exhibits brittle failure, the effect of the mechanical dissipation, \( \phi_1 \), on \( U_{sc}^e \) will be ignored and Cottrell (1964) showed that the strain rate has no influence on the failure mode of brittle material. In linear elastic mechanics, the effect of volume change on the elastic strain energy might be ignored as stated by Li (1999). Due to various assumptions used for deriving the specified strain energy density, different failure criteria have been proposed for evaluating the material strength parameters. According to Freudenthal (1950) if the specified potential strain energy for a certain group of particles in a material exceeds the critical value, the failure occurs in the collection of material particles. Collins and Kelly (2002) assessed some well-known existing critical-state models for geomaterials and indicated their shortcomings. They showed that a few failure criteria, such as the modified Cam-Clay model, may be developed by using the intermediate dissipation stress space approach, which will satisfy the second law of thermodynamics. This will address the shortcomings of some other models, such as the original Cam-Clay model (Roscoe and Burland 1968), according to which the irreversible plastic deformations would happen without generating any dissipation, and this violates the second law of thermodynamics. Li (1999) showed that despite the substantial difference in the nature and structure of materials, there is a consistent similarity observed in their macroscopic behaviours such as density and elasticity, etc., which can be defined over a given control volume. However, the scale and size effects should be considered when determining material failure as Bazant and Chen (1997) indicated.

1.2.2. Dissipative energy
During an inelastic deformation, the energy dissipative irreversible process occurs along with damage in the material. The plastic strain energy density failure criterion has been suggested and implemented for predicting the failure behaviour of ductile materials by a number of researchers (Clift et al. 1990; Gillemot 1976). According to Lee (1969), the material failure may take place due to excessive plastic dissipation with small material damage from void development or due to large material damage by void coalescence mechanism. According to Truesdell and Noll (2004) mechanical and thermal dissipation can be de-coupled and the intrinsic mechanical dissipation due to heat conduction can be expressed as:

\[ \phi = P_{ij}\dot{e}_{ij}^p + A_k\dot{V}_k + Y_{ij}\dot{D}_{ij} \geq 0 \]  

(3)

where \( \phi \) is the intrinsic dissipative process in a mechanical system and expresses the material life expectancy before failure. \( P_{ij}, Y_{ij} \) and \( A_k \) are thermodynamic stresses and introduce a mechanical intrinsic dissipation. The mechanical dissipative energy density \( \phi \) is defined as;

\[ \phi = \int_0^1 \dot{\phi} \, dt \]  

(4)

\( \phi \) is an index to explain the strength of bonding at macroscopic scale in a thermodynamic framework. It should be mentioned that the \( U_{sc}^e \) decreases during the irreversible dissipative deformation. The non-brittle failure of a material is a progressive process and is accompanied by the reduction in the material strength and increase in the dissipation. Large inelastic deformations are the result of the ductile failure of a material. Such deformations take place due to large dissipation and are controlled by \( \phi \). Bridgman (1952) showed that the hydrostatic stress plays a key role on material failure while it has no effect on the plastic deformation.

It was shown in Eq. 3 that the dissipative energy can be expressed in a general form as;

\[ \phi = U^p + U^d - U^v \]  

(5)

where \( U^p \) is the plastic strain energy density, \( U^d \) is the damage dissipation density and \( U^v \) is the stored energy density of cold work as suggested by Lemaitre and Chaboche (1990). They showed that \( U^v \) is only 5% of \( U^p \) and can be neglected in ductile materials. The plastic flow of a material and the material damage are two different physical processes as was shown by Lemaitre and Chaboche (1990). They showed that plastic and damage dissipations can be
illustrated by two monotonously increasing functions and the effect of these two processes on materials might be different. If the material failure is controlled by the damage dissipation and the microscopic mechanism of failure is the coalescence of the adjacent porosities the continuum damage failure criteria can be applied. Otherwise, the failure of the control volume is mainly controlled by the elastic and plastic strain energy densities. The application of the total dissipated energy for defining a failure criterion was also suggested by Benallal et al. (1993).

2. Experimental study

Investigation of subsurface sediments at a drilling site in South Australia showed that the layers above the bedrock are not homogeneous, i.e. the shallower layers of the sediment are composed of silt and fine sand, while the deeper layers of the sediments change to a dark grey plastic clay, and the problematic poorly cemented granular layer comes after this clayey layer. Based on the site investigation conducted by Hashemi et al. (2014b), the stress concentration around a borehole, low strength of the sandy layer and in some cases groundwater flow due to an aquifer near the drilling zone were identified as the major factors accounting for the borehole instability. Samples were collected from each metre of the unconsolidated sandy layer for further investigation. The pale yellowish-grey sand grains were mostly fine and sub-angular with random orientations. Thus, the specimens used for the laboratory tests were designed and prepared based on the sand grain size distribution and geometry obtained from the samples collected from this drilling site.

A series of novel physical model experiments for the TWHC specimens was designed and conducted to investigate the failure mechanism of boreholes drilled through poorly cemented granular formations. Also, unconfined compressive strength (UCS) and triaxial tests were performed on solid cylindrical specimens in order to obtain the mechanical properties of prepared specimens. The Hoek cell, pressure automatic maintainer and dependent gauges were modified in order to apply and measure low range stresses (i.e. maximum 5 MPa of confining pressure with 0.5 kPa accuracy) on specimens. To determine the point of particle bonding breakage at the borehole wall with respect to the stress-strain status of the specimen, the real-time borehole condition was visually monitored while applying the stresses on the specimen. Specimens were designed to fit into a HQ Hoek triaxial cell of 63.5 mm diameter and 127 mm length. The whole process of experimental studies and data analysis was effortful and time consuming, and took more than 18 months to complete.
2.1. Specimen preparation

2.1.1. Mixture

Specific stainless steel moulds were designed and manufactured for specimen preparation. Sands, cement, and water were mixed into slurry, and then the mixture was poured into the mould in three separate layers and each layer was compacted in 25 impacts with a compactor that was manufactured specifically for this purpose. Two types of specimens, i.e. solid and TWHC were prepared as per test plan.

As mentioned earlier in this section the components of the prepared poorly cemented sand specimens were designed to simulate the actual behaviour of the sandy formation at the considered site in South Australia. Thus, the fabric and grading size distribution of specimens resembled the average of those for the samples collected from the drilling site. Fig. 2 shows the grain size distribution of the collected samples ($\rho = 1720 \, kg/m^3$) from the drilling site up to the depth of 100 m. The sieve analysis based on calibrated ASTM C-136 sieves plus pan were performed. According to the chemical analysis of the grains the formation comprises of quartz grains ($\geq 96\%$) with a weak bonding interface of clay and calcite acting as cementing agents. As per the sieve analysis results for the site samples (Fig. 2), Australian natural well sorted silica sands of two different grain size ranges (i.e. “coarse” and “fine” sand) were selected for the mixture as the most similar material properties and grading size distribution to the site samples. The “course” and “fine” sand sizes are between $0.425 \, mm$ - $1.4 \, mm$ and $0.125 \, mm$ - $0.355 \, mm$ respectively.

![Fig. 2. Sand particle size distribution curve for laboratory test specimens and for collected samples from a problematic drilling site at South Australia.](image-url)
The optimum water content was measured to be 9.7~10.3% by performing a standard proctor test and using standard compactor hammer. It should be mentioned that the dry density of the cemented sand and only sand grains was measured separately and results showed no significant alteration between the results. A small delicate mixer was utilised to ensure that sand grains, cement and water mixed perfectly in order to produce a homogeneous mixture. The total time frame from the start of mixing the slurry until sealing the moulds was strictly maintained between 30-40 minutes. A wide range of curing time (2-12 days) has been suggested for creating poorly cemented sand mixtures in previous studies (Alsayed 1996; Saidi et al. 2003; Younessi et al. 2013). Finally, 8 days was considered as the optimal total curing time in this study based on several try and errors to reach the desired specimens. The curing time included both the time when the mixture was poured into the mould and the following step when it was demoulded and kept in a plastic wrap. Care was taken to maintain the specimens at temperatures between 18°—22° Celsius.

Based on the previous studies on cemented sands (Alsayed 2002; Kongsukprasert 2003; Saidi et al. 2005) the Portland cement type II ($G_s = 3.15 \text{ g/cm}^3$) was used as a cementing agent for preparing the TWHC and solid specimens. A wide range of cement contents ($2.5\% \geq w_c \geq 18\%$) for creating a poorly cemented synthetic sandstone have been suggested in different research works. Hashemi et al. (2014b) showed that $w_c$ should be selected based on the used sand grading ratios ($\delta$). In the current study $w_c = 6\%, 7\%$ and $8\%$ were chosen for the mentioned sand size ranges. Table 1 represents detailed mechanical properties of the prepared specimens.

**Table 1** Properties of the prepared poorly cemented sand specimens (Hashemi et al. 2014b).

<table>
<thead>
<tr>
<th>Mechanical properties</th>
<th>Porosity ($\alpha$)</th>
<th>Tangent elastic modulus $E_{tan}$ (GPa)</th>
<th>Uniaxial compressive strength UCS (MPa)</th>
<th>Poisson’s ratio $v$</th>
<th>Coulomb parameters $c$ (MPa) $\phi$ (°)</th>
<th>Bulk density $\rho$ ($kg/m^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_c = 6%$</td>
<td>$26 \pm 3%$</td>
<td>2.262</td>
<td>0.62</td>
<td>0.294</td>
<td>1.11</td>
<td>29.67</td>
</tr>
<tr>
<td>$w_c = 7%$</td>
<td>$26 \pm 2%$</td>
<td>2.465</td>
<td>1.05</td>
<td>0.250</td>
<td>1.38</td>
<td>30.06</td>
</tr>
<tr>
<td>$w_c = 8%$</td>
<td>$26 \pm 2%$</td>
<td>3.360</td>
<td>1.90</td>
<td>0.247</td>
<td>1.59</td>
<td>29.67</td>
</tr>
</tbody>
</table>
2.1.2. Laboratory test facilities

The modified hollow cylinder test cell was prepared by applying some changes to a large Hoek triaxial cell into which a TWHC specimen of 63 mm × 127 mm can be fitted. The fitted micro camera with 225 PPI resolution allowed real-time monitoring of the borehole walls and sand debonding process during the tests. A 60 channel data acquisition system was connected to two separate personal computers in order to record stress, strain and displacement. These facilities were synchronized with a precise system of applying confining pressure in low stresses (maximum 5 MPa) with no leakage or intrusion. According to the UCS test results on cell platens, the strain of the platens is less than 0.005% when subjected to a maximum of 200 kN loading force which is much higher than the strength of the prepared TWHC specimens.

2.2. Test procedure and plan

Five different stress paths were applied to the TWHC specimens to investigate the effect of stress paths on the borehole stability in poorly cemented granular formations. Loading schedule was designed based on two approaches; (1) principal far-field stresses and (2) principal stresses at the borehole wall (See Eq. 6). In the far-field based stress paths, the vertical stress ($\sigma_z$) and confining pressure ($\sigma_{conf}$) were considered and they both were increased simultaneously at the same rate up to certain level which simulated the hydrostatic condition acting on the specimens. Then, in the second step, depending on the experiment plan the specimen was subjected either to a vertical loading increment (normal faulting) corresponding to a constant displacement rate of 0.07 mm/min or to a confining pressure increment (reverse faulting) corresponding to a constant pressure rate of 0.2 MPa/min. In the first path (Eq. 6a) the magnitude of the confining pressure acting on the external surface of the specimens was kept constant during the second step of the test by an automatic pressure maintainer system. In the second approach (the third, fourth and fifth stress paths) the principal stresses ($\sigma_\theta, \sigma_r, \sigma_z$) for an element on the borehole wall was considered and the far-field stresses were applied to the boundaries of the specimen to induce the desired magnitude of tangential stress on the borehole wall. In the third and fourth stress paths $\sigma_\theta$ and $\sigma_z$ were increased simultaneously up to a certain value. Then, as per the experiment plan the specimen was subjected either to $\sigma_z$ or $\sigma_\theta$ increment. It is worth mentioning that in an unsupported borehole, $\sigma_r$ at the borehole wall is zero.

In the third stage of the test program (the fifth path), $\sigma_\theta$ and $\sigma_z$ were increased simultaneously until the bonding breakage between the sand grains at the borehole wall was
observed by the real-time camera recording. By adjusting the loading machine and pressure maintainer, equal tangential and vertical stresses were applied on the borehole wall. It should be mentioned that for each of the TWHC specimens \( \sigma_\theta \) and \( \sigma_z \) were calculated separately based on the area and the borehole size. Each experiment was repeated for three different cement contents (i.e. 6%, 7% and 8%). Eq. 6a-e shows the status of the principal stresses at the boundary specimens and borehole wall in the final step of experiments for each stress path respectively.

1\(^{st}\) stress path \[
\begin{align*}
\sigma_1 &= \sigma_z > \sigma_2 = \sigma_3 = \sigma_{\text{conf}} \ (\text{far-field}) \\
\sigma_1 &= \sigma_z > \sigma_2 = \sigma_\theta > \sigma_r = 0 \ (\text{at borehole wall})
\end{align*}
\] (6a)

2\(^{nd}\) stress path \[
\begin{align*}
\sigma_1 &= \sigma_2 = \sigma_{\text{conf}} > \sigma_3 = \sigma_z \ (\text{far-field}) \\
\sigma_1 &= \sigma_\theta > \sigma_2 = \sigma_z > \sigma_r = 0 \ (\text{at borehole wall})
\end{align*}
\] (6b)

3\(^{rd}\) stress path \[
\begin{align*}
\sigma_1 &= \sigma_2 = \sigma_{\text{conf}} > \sigma_3 = \sigma_z \ (\text{far-field}) \\
\sigma_1 &= \sigma_\theta > \sigma_2 = \sigma_z > \sigma_r = 0 \ (\text{at borehole wall})
\end{align*}
\] (6c)

4\(^{th}\) stress path \[
\begin{align*}
\sigma_1 &= \sigma_z > \sigma_2 = \sigma_3 = \sigma_{\text{conf}} \ (\text{far-field}) \\
\sigma_1 &= \sigma_z > \sigma_2 = \sigma_\theta > \sigma_r = 0 \ (\text{at borehole wall})
\end{align*}
\] (6d)

5\(^{th}\) stress path \[
\begin{align*}
\sigma_1 &= \sigma_z > \sigma_2 = \sigma_3 = \sigma_{\text{conf}} \ (\text{far-field}) \\
\sigma_1 &= \sigma_\theta = \sigma_2 = \sigma_z > \sigma_r = 0 \ (\text{at borehole wall})
\end{align*}
\] (6e)

The difference between the first and fourth stress paths is in the initial step of the tests. In the first path, the magnitude of \( \sigma_\theta \) is considerably higher until reaching the hydrostatic stress than that for the fourth path. The same condition applies to the second and third stress paths.

3. The total absorbed strain energy

In general, the elastic strain energy density can be calculated for a volume element subjected to arbitrary stress states as follows:

\[
U_s^e = \frac{1}{2} \left( \sigma_{xx} \varepsilon_{xx} + \sigma_{yy} \varepsilon_{yy} + \sigma_{zz} \varepsilon_{zz} + (\tau_{xy} \gamma_{xy} + \tau_{yz} \gamma_{yz} + \tau_{zx} \gamma_{zx}) \right)
\] (7a)

Also, with respect to the principal axes for an elastic and isotropic body;

\[
U_s^e = \frac{1}{2E} \left( \sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\nu(\sigma_1 \sigma_2 + \sigma_1 \sigma_3 + \sigma_3 \sigma_2) \right)
\] (7b)

On the other hand we have;

\[
U_s^e = U_{sv}^e + U_{sd}^e
\] (8a)

which;

\[
U_{sv}^e = \frac{1-2\nu}{6E} (\sigma_1 + \sigma_2 + \sigma_3)^2
\] (8b)

\[
U_{sd}^e = \frac{1}{12G} \left[ (\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_3 - \sigma_2)^2 \right]
\] (8c)
where $U_{sv}^e$ is the potential energy due to volume change and $U_{sd}^e$ is the potential energy due to distortion. For instance, according to the Von-Mises failure criterion the material failure occurs by the potential of the distortion strain energy density as;

$$U_s^e = U_d^e = \frac{4(1+\nu)}{6E} \tau_{oct}^2$$ \hspace{1cm} (9)

If the material failure is mainly controlled by the large plastic deformation process, the failure criterion should be defined as a plastic energy density failure criterion where the influence of porosities are negligible in a given failure volume. Thus,

$$U^p = \int_0^{\varepsilon^p} \sigma_{ij} \, d \varepsilon_{ij}^p = \int_0^{\varepsilon^p} q_{ij} \, d \varepsilon_{ij}^p + \int_0^{\varepsilon^p} \sigma_H \, d \varepsilon_{ii}^p = U_c^p$$ \hspace{1cm} (10)

where $q_{ij} = \sigma_{ij} - \sigma_H \delta_{ij}$ and $U_c^p$ is the critical value of the plastic strain energy density which can be determined by laboratory tests (Clift et al. 1990), and for incompressible materials $\sigma_H \, d \varepsilon_{ii}^p \approx 0$. On the other hand, if the effect of the void coalescence cannot be ignored due to the generation of an unbalanced flow between the adjacent voids and considerable elastic strain energy, the damage dissipation failure criterion can be written as;

$$U^d = U^e \left( \frac{\bar{E}}{E} - 1 \right) = U_c^d$$ \hspace{1cm} (11)

where $\bar{E}$ is the undisturbed Young’s modulus of the material, and $U_c^d$ is the critical value of the damage dissipation density. Hashemi et al. (2014b) showed that the volumetric strain is not negligible ($d \varepsilon_{ii}^p \neq 0$) in poorly cemented sand specimens. When considering the principal stresses at the borehole wall in a 10 mm diameter borehole sample for different stress paths in the current test plan we have;

$$\sigma_1 = \sigma_z, \sigma_2 = \sigma_\theta = 2.05 \, \sigma_{conf}, \sigma_3 = \sigma_r = 0$$ \hspace{1cm} (12)

Thus, Eq. 10 can be re-written as follows;

$$U^p = \int_0^{\varepsilon^p} \sigma_{ij} \, d \varepsilon_{ij}^p = \int_0^{\varepsilon^p} \sigma_z \, d \varepsilon_{z}^p + 2.05 \int_0^{\varepsilon^p} \sigma_{ij} \, d \varepsilon_{ij}^p$$ \hspace{1cm} (13)

where $\varepsilon_z$ and $\varepsilon_l$ are the vertical and lateral strains measured by strain gauges during the tests. As mentioned by Hashemi et al. (2014b), the poorly cemented sand specimens showed
inelastic behaviour from the start of the second step in the conducted tests. Therefore, $U^e$ in Eq. 2 has a minimal role when increasing the deviatoric stress.

After conducting further analysis of the camera recordings versus the obtained data, it was observed that the bonding breakage point did not occur at the peak stress. Since the debonding of sand particles at the borehole wall is a sign of the borehole failure initiation, these points were detected during the experiments. A failure criterion based on the total potential and dissipative strain energy densities (the modulus of toughness) was considered for each test on the TWHC based on the applied stresses. The total energy (i.e. potential and dissipative) was derived based on the results from the conducted tests. Fig. 3 shows the typical total potential and plastic dissipative strain energies for the first, second and fourth stress paths. According to Fig. 3 it can be written;

$$U^p + U^d + U^s_e = (A_1 + A'_1) + (A_2 + A'_2)$$ (14)

It should be mentioned that $A'_1$ and $A'_2$ were calculated based on an element at the borehole wall, because the principal stresses values at the borehole wall ($\sigma_0$) are different from the far-field stresses ($\sigma_{conrf}$). The modulus of toughness was calculated by the trapezoidal rule, which is a well-known numerical method. The trapezoidal rule works by approximating the region under the graph of a function as a trapezoid and calculating its area. Since during the test the data recording was performed for each 0.5 second, the area under the stress-strain graph was calculated based on a high number of strips and the average error was 0.012% (Atkinson 1991). As shown in Fig. 3, the stress-strain relationship was almost linear until reaching the hydrostatic status (the first step of the experiments).
4. Results

4.1. Bonding breakage point at the borehole wall

Most of failure criteria introduced for boreholes have been suggested based on the peak stresses which were recorded in stress-strain diagrams from laboratory tests or numerical models (Hashemi et al. 2014b). Since the process of borehole failure due to applied stresses was recorded by a deployed camera on top of the borehole, the instant of first bonding breakage could be recorded against the stress condition. After each test, the recorded video was analysed and the first bonding breakage point at the borehole wall was determined. Then, the corresponding point on the stress-strain diagram was identified. It should be mentioned that this process was performed by considering the time in the stress-strain diagram and the camera recording simultaneously. Fig. 4a shows the process of debonding of sand grains at the borehole wall in 20 mm diameter boreholes and the corresponding points on the stress-strain curve. The debonding started at point 1 and continuous falling of sand flakes intensified at point 4, which was considered as the borehole failure point in the performed
When continuing the experiment from the point 4 onward, the specimen collapsed completely under a constant vertical stress. Nouri et al. (2006) suggested that for the material to be dislodged, the debonding point must be located within the strain softening phase. However, based on more than 200 tests conducted on the TWHC specimens, the bonding breakage point at the borehole wall was observed both before (strain-hardening mode) and after the peak stress (strain-softening). Hashemi et al. (2014b) showed that in the case of using solid poorly cemented specimens, the material is not exhibiting a strain-softening behaviour after the peak stress in the stress-strain diagram, and the strain-hardening residual mode resumes. Since in some cases the particle breakage point was recorded after the peak stress in TWHC specimens, it was not possible to use only the stress in the failure criterion. According to Fig. 4b, two points (point “A” and “B”) can be identified on the stress-strain diagram having the same stress values but corresponding to two different strains. Although these two points have the same stresses, the general condition of the specimen is completely different in terms of stiffness and induced microcracks, which have developed adjacent to the borehole after the peak stress was reached, since the specimen experienced larger strain after the maximum stress.
Fig. 4. The process of particle bonding breakage at the borehole wall in 20 mm diameter borehole ($w_c = 7\%$ and $\sigma_{\text{conf}} = 1 \text{ MPa}$) and the corresponding points on stress-strain graph. Points “A” and “B” represent the two points on the stress-strain graph with the same stresses but different strains.

Fig. 5 a-c show the particle bonding breakage points at the borehole wall for the first stress path and different $w_c$ values in the 10 mm diameter borehole specimens. In the first stress path (Eq. 6a), with increasing $\sigma_\theta$ the bonding breakage often occurred at lower axial strains and the debonding points moved away from the peak stress. In all cases at $\sigma_{\text{conf}} = 4 \text{ MPa}$ the bonding breakage points were observed in the strain-hardening section of the stress-strain diagram. Fig. 6 represents the bonding breakage points in 20 mm diameter boreholes for the first stress path. It shows that for $\sigma_{\text{conf}} = 1, 2 \text{ MPa}$, the first particle debonding was observed after the peak stress was reached and with increasing the confining pressure, the bonding breakage point shifted backward on the stress-strain diagram. Fig. 7 shows the identified debonding points for 10 mm diameter borehole specimens for the second and third paths. In the second and third stress paths (Eq. 6b,c) with increasing $\sigma_2$, the first bonding breakage generally occurred at higher lateral strains. However, some exceptions were observed at $\sigma_2 = 4 \text{ MPa}$. It should be mentioned that the specimens in the second and third stress paths showed a strain-hardening behaviour and the last point in the stress-strain curve represents the onset of excessive sanding which was considered as borehole failure. However, the TWHC specimens were safely retrieved from the cell with no significant damage on their boundaries.
Fig. 5. The identified bonding breakage points at the borehole wall of 10 mm borehole size and for different confining pressures and cement contents (a) first stress path $w_c = 6\%$, (b) first stress path $w_c = 7\%$, (c) first stress path $w_c = 8\%$. 
Fig. 6. Detected bonding breakage points at the borehole wall of a 20 mm borehole size and for different confining pressures (a) first stress path \(w_c = 6\%\), (b) first stress path \(w_c = 7\%\).
Fig. 7. Detected bonding breakage points at the borehole wall of 10 mm borehole size and for different confining pressures and cement contents (a) second stress path, $w_c = 7\%$, (b) third stress path $w_c = 8\%$.

4.2. Bonding breakage point criterion based on strain energy per volume of material

Bonding breakage between sand particles at the borehole wall occurs if the formation adjacent to the borehole is directly failed in tension or the aggregation of microcracks due to shear failure undergoes tension. Mogi (1967) suggested that failure will occur when the distortional strain energy reaches a critical value which increases with the effective mean normal stress. In theory, the octahedral shear stress has a direct relationship with the strain energy. He proposed to consider the effective mean normal stress, $\sigma_{m,2}$, instead of $\sigma_{oct}$ in the failure function, since the failure occurs in the direction of strike of the intermediate stress.
The data from the laboratory tests were used to calculate $\sigma_{m,2}$ at the borehole wall in order to represent the trend of obtained data in the $U - \sigma_{m,2}$ domain.

As discussed in previous section, the bonding breakage points at the borehole wall were detected for various stress paths, cement values and borehole sizes. Then, based on the method which was elaborated in section 3, the total absorbed energy values per unit volume (ultimate toughness) were calculated for a single element at the borehole wall for each test. The principal stresses and the total absorbed strain energy were calculated based on the applied stresses at the boundary of the TWHC specimen. Since there was no supporting system deployed in the borehole, radial stress was considered to be zero in the current test program.

The total strain energy was derived based on the stress-strain diagram in the vertical and lateral directions. The effective mean normal stress, $\sigma_{m,2}$ at the borehole wall was calculated as follows:

$$\sigma_{m,2} = \frac{\sigma_1 + \sigma_3}{2} \quad (15)$$

4.2.1. First stress path

The data from the experiments were used to calculate the principal stresses based on each stress path (Eq. 6) applied to the specimens. The values of the total potential and dissipative energies per volume were plotted for the given value of $w_c$ and different values of the intermediate principal stress as a function of $\sigma_{m,2}$. This mode of presentation was selected to provide an insight into the behaviour of the total absorbed strain energy and with the thought that the criterion for the bonding breakage might have involved the value of the total strain energy to a linear function other than a power. Fig. 8a,b show the total strain energy versus the normal effective mean stress ($\sigma_{m,2}$) for each test conducted on 10 mm and 20 mm borehole size specimens for the first stress path, different cement contents ($w_c = \text{weight ratio of cement to sand grains}$) and confining pressures, respectively. According to Fig. 8, the relationship between the data in the $U - \sigma_{m,2}$ domain is relatively linear. Although it was possible to fit a polynomial equation to the obtained data with a good approximation, a linear relationship between the strain energy and $\sigma_{m,2}$ can be used for this material for different $\sigma_2 = \sigma_0$. All states of stress below such a line are permissible states for which the borehole will not fail and those above it are not permissible because bonding breakage occurs before they can be attained. Any state of stress represented by a point on the line of constant $w_c$
shows a limiting state of stress, from which the bonding strength, achieved by the value of $\sigma_{m,2}$ at that point. It is worth mentioning that the maximum confining pressure applied to the specimens (= 4 MPa) was based on the highest amount of stress that a TWHC specimen could withstand. In the case of applying $\sigma_{conf} > 5MPa$ the specimens with a maximum of $w_c = 8\%$ could not even meet the hydrostatic stress condition which was the first step of testing. As can be seen in Fig. 8, with an increase in the tangential stress the modulus of toughness increases for different cement contents. Also, increasing the $w_c$ in the same path slightly shifts the failure envelope upward in most of the cases. However, the effect of the cement content is less considerable than the effect of the tangential stress in elevating the toughness. Fig. 8c shows the toughness level versus the effective normal mean stress for 10 mm and 20 mm borehole size specimens for the first path. It shows that the relationship between the data in the $U - \sigma_{m,2}$ domain is almost linear for the 20 mm diameter borehole as well. Al-Ajmi and Zimmerman (2006) and Hashemi et al. (2014b) showed that there is also a linear relationship between $\tau_{oct}$ and $\sigma_{m,2}$ in poorly cemented sands at maximum stress for the first stress path. As can be seen from Fig. 8c, the effects of the borehole size and cement content are considerable and comparison between the results suggests that the level of strain energy absorbed by the material until reaching the bonding breakage point is considerably lower in the 20 mm diameter borehole specimens, than in the 10 mm ones. For the test data plotted in the $U - \sigma_{m,2}$ domain, an arbitrary linear function can be defined as follows,

$$U = a + b \sigma_{m,2}$$

(16)

where $a$ is the intersection of the line with the vertical axis ($U$) and $b$ is the slope of the line. The constants in Eq. 16 were calculated from Fig.8c and are presented in Table 2. It shows that, generally, the slope of trend lines for the 10 mm and 20 mm diameter borehole specimens are in a same range which suggests that the ratio of $\Delta U / \Delta \sigma_{m,2}$ is almost constant for both borehole sizes. As expected, due to size-scale effect suggested by Carpinteri (2002), decreasing the size of the borehole diameter increases the failure stress in TWHC specimens.
Table 2 Constants of the linear equation (Eq. 16) for the firth stress path for the TWHC specimens with 10 mm and 20 mm diameter boreholes.

<table>
<thead>
<tr>
<th>Cement %</th>
<th>Borehole size (mm)</th>
<th>a</th>
<th>b</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>10</td>
<td>5.35</td>
<td>1.25</td>
<td>0.995</td>
</tr>
<tr>
<td>7</td>
<td>10</td>
<td>2.68</td>
<td>1.72</td>
<td>0.961</td>
</tr>
<tr>
<td>8</td>
<td>10</td>
<td>3.35</td>
<td>1.75</td>
<td>0.935</td>
</tr>
<tr>
<td>6</td>
<td>20</td>
<td>-1.2616</td>
<td>1.15</td>
<td>0.993</td>
</tr>
<tr>
<td>7</td>
<td>20</td>
<td>-1.33</td>
<td>1.19</td>
<td>0.988</td>
</tr>
<tr>
<td>8</td>
<td>20</td>
<td>-3.15</td>
<td>1.52</td>
<td>0.981</td>
</tr>
</tbody>
</table>

(a)

(b)
Fig. 8. Calculated total strain energy absorbed per volume of material (modulus of toughness) versus normal effective mean stress based on the results from laboratory tests conducted on TWHC specimens for the first stress path and different $w_c$ in (a) 10 mm diameter borehole specimens, (b) 20 mm diameter borehole specimens, (c) 10 mm and 20 mm diameter borehole specimens.

4.2.2. Second stress path

Fig. 9 represents the experimental data in the $U - \sigma_{m,2}$ domain for the second stress path where $\sigma_\theta$ was the maximum principal stress at the borehole wall. As mentioned in section 2.2, the first and second stages of the experiments for the second path were based on the far-field stresses ($\sigma_z$ and $\sigma_{conf}$) and the principal stresses were derived based on applied stresses on the boundary of the TWHC specimens. In the second stress path the majority of the energy absorbed in the lateral direction as expected and the vertical stress played minimal role in total energy. Likewise, linear equations with a good approximation could be fitted to the obtained data in the $U - \sigma_{m,2}$ domain. As can be seen in Fig. 9, the toughness modulus level dramatically increases with increasing $\sigma_2 = \sigma_z$ in the second stress path for different $w_c$ values.
Fig. 9. Derived total strain energy absorbed per volume of material (modulus of toughness) versus normal effective mean stress based on the results from laboratory tests conducted on TWHC specimens for the second stress path and different w_c in 10 mm diameter borehole specimens.

4.2.3. Third stress path

In the third stress path, \( \sigma_\theta \) and \( \sigma_z \) were increased at the same rate until a certain value while keeping \( \sigma_z \) constant. In the second step \( \sigma_\theta \) was increased until a bonding breakage point was observed at the borehole wall. The difference between the second and third stress paths is in the first stage of the experiment, which in the third path the magnitude of the confining pressure acting on the boundary of the specimen is considerably higher. Fig. 10a shows the \( U - \sigma_{m,2} \) for the third stress path. It is obvious from this graph that the relationship between the toughness modulus and the effective mean normal stress is almost linear again. Although with increasing \( w_c \) the strength of the specimen increases dramatically, the level of increment in the strain energy value was not reflected in that order and the effect of the cement content was not straightforward. Fig. 10b demonstrates the effect of the borehole size for the third stress path. It shows that for the same \( w_c \) and \( \sigma_2 \), the amount of energy required for reaching the bonding breakage point in the 20 mm borehole size specimens is remarkably lower than for the 10 mm ones. Also, the slope of the trend line for both borehole sizes stayed almost unchanged which is consistent with the conducted experiments for the first stress path and a linear equation can be fitted for both borehole sizes.
Fig. 10. Derived total strain energy absorbed per volume of material (modulus of toughness) versus normal effective mean stress based on the results from laboratory tests conducted on TWHC specimens for the third stress path and different \( w_c \) in (a) 10 mm diameter borehole specimens, (b) comparison between 10 mm and 20 mm diameter borehole specimens.

In order to compare the results from the second and third stress paths, the test data in the \( U - \sigma_{m,2} \) domain were superimposed for both stress paths as is shown in Fig. 11. As can been seen from Fig. 11, the data are located on a limited boundary and the slope of trend lines stays almost unchanged for both stress paths except for \( w_c = 6\% \) in the second path which is considerably lower than the other results. For \( \sigma_{confr} \geq 3 \) MPa the specimens with the lowest
strength (i.e. $w_c = 6\%$) underwent high $\sigma_\theta$ values in the initial stage of the tests for the second path. This may induce damage to the specimens and led to the lower toughness modulus in this cement content. Table 3 represents the constants of the linear equation given in Eq. 16 and calculated from Fig.11. Due to initially higher $\sigma_\theta$ values for the second stress path one would anticipate to observe a higher energy magnitude for it, however the graph shows that all data are scattered within the boundary and no dominancy was observed for the second and third paths in terms of the total absorbed energy. Since the borehole failure occurs mainly due to high $\sigma_\theta$ values, Fig.11 shows that in the second and third stress paths the magnitude of $\sigma_z$ at the initial stages of the tests does not affect the final toughness modulus in poorly cemented sand specimens. Fig. 11 also shows that an increase in $w_c$ does not affect the strain energy level for these stress paths. This is an important outcome, since it shows that in this material the cement content will not affect the bonding breakage dramatically in unsupported boreholes when the $\sigma_\theta$ is the maximum principal stress.

![Fig. 11. Total strain energy absorbed per volume of material (modulus of toughness) versus normal effective mean stress based on the results from laboratory tests conducted on TWHC specimens for the second and third stress paths and different $w_c$ in 10 mm diameter borehole specimens.](image-url)
Table 3 Constants of the linear equation (Eq. 16) for the second and third stress paths for the TWHC specimens with 10 mm diameter borehole.

<table>
<thead>
<tr>
<th>Stress path</th>
<th>Cement %</th>
<th>a</th>
<th>b</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>6</td>
<td>1.88</td>
<td>0.74</td>
<td>0.847</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>-8.75</td>
<td>2.43</td>
<td>0.964</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>-9.41</td>
<td>2.80</td>
<td>0.963</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>-6.11</td>
<td>2.51</td>
<td>0.975</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>-9.80</td>
<td>2.70</td>
<td>0.965</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>-9.07</td>
<td>2.41</td>
<td>0.991</td>
</tr>
</tbody>
</table>

4.2.4. Fourth stress path

To compare the results of the first and fourth stress paths $U$ and $\sigma_{m,2}$ for different cement contents were calculated and superimposed in Fig. 12. It shows that in a given lateral pressure ($\sigma_{conf} = 2MPa$), the total energy was significantly higher for certain $w_c$ values for the first stress path in comparison with the fourth path. Although the second stage of the stress path is the same for both the first and fourth paths, the results for the first and forth paths did not align into a single line. However, the slope of the trend lines in both paths remained almost unchanged. In the first path, the lateral boundaries of specimens experienced a higher confining pressure when axial stress was applied in the initial stage of testing. In other words, $\sigma_\theta$ at the borehole wall was remarkably higher than $\sigma_z$ before reaching the hydrostatic state and more lateral support was applied to the specimen than in the case of the fourth path. Therefore, in the first stress path the specimens showed ductile behaviour and they were allowed to undergo higher strains in the vertical direction. Thus, in the first stress path the bonding breakage at the borehole wall occurred at a higher vertical strain. While in the fourth path, $\sigma_\theta$ and $\sigma_z$ were increased simultaneously and a lower confining pressure induced brittle behaviour to the specimen. Hence, in the fourth stress path the specimens could tolerate higher average principal stresses before the occurrence of bonding breakage in comparison with the first stress path. It is important to note that according to the micromechanical studies $\sigma_{conf} = 2MPa$ was not high enough to produce microcracks at the borehole wall before completing the first stage of the test, because high magnitudes of $\sigma_\theta$ may generate microcracks at the borehole wall in the first stage of tests and consequently, the bonding breakage will occur at the lower values of $\sigma_{m,2}$. 

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4.2.5. Fifth stress path

In the fifth stress path TWHC specimens were subjected to $\sigma_\theta$ and $\sigma_z$ increment at the same time until the bonding breakage point was observed. Most importantly, it was observed that the specimens could tolerate high levels of confining pressures and vertical stresses and even after reaching to 50 kN of vertical force, the bonding breakage point was not observed yet in a specimen with the minimal cement content ($w_c = 6\%$). The load cell was replaced with a 100 kN one and the tests were repeated. The strain energy levels at the particle debonding point were calculated for each $w_c$. Fig. 13 shows that an increase in $w_c$ the toughness modulus of the TWHC specimens considerably increased. Likewise, it is possible to fit a linear equation to this stress path with an acceptable coefficient of determination ($r^2 \geq 0.95$). Results showed that increasing the principal stresses enhances the stability of an unsupported TWHC specimen. This could be due to increasing the reciprocated support at the borehole wall and counteracting the applied stresses.
4.2.6. Bonding breakage criterion in $U - \sigma_{oct}$ domain

Since the distortional strain energy is proportional to the octahedral shear stress, the Drucker-Prager failure model suggests that the failure of the geomaterials would occur when the strain energy reaches a specific value that increases with the octahedral normal stress, $\sigma_{oct}$. It is possible to use the concept of this failure criterion and analyse the obtained test data in $U - \sigma_{oct}$ domain as well. The octahedral normal stress ($\sigma_{oct}$) in each experiment for the 10 mm borehole size specimens was calculated for the first and second stress paths. Fig. 14 shows the derived toughness and the octahedral normal stress for different cement contents ($w_c$). As can be seen from Fig. 14, representing the data in $U - \sigma_{oct}$ domain did not change the trend of failure envelopes in $U - \sigma_{m,2}$ domain dramatically which was already shown in Fig. 8 and Fig. 9. Moreover, the test data presented in $U - \sigma_{m,2}$ domain still demonstrated a better fit for a linear equation based on the coefficient of determination closer to one.
Fig. 14. Calculated total strain energy absorbed per volume of material (modulus of toughness) versus normal octahedral stress based on the results of laboratory tests conducted on 10 mm diameter borehole TWHC specimens for (a) the first stress path, (b) the second stress path.

5. Conclusion

Borehole stability analysis was performed based on the bonding breakage point at the borehole wall. This point was detected by a micro camera which was installed into the top platen of the Hoek cell to allow visual observation of the real-time status of the borehole during the test and based on this the corresponding point on the stress-strain graph was identified. Then, the total strain energy per volume of material up to reaching the bonding breakage point was calculated for different applied stress paths in TWHC specimens. Two
borehole diameter sizes (10 mm and 20 mm) and three different cement contents (6%, 7% and 8%) were considered versus five different stress paths.

Recorded videos showed that debonding points were located both before and after the peak maximum principal stress ($\sigma_1$) depending on the applied stresses and cement contents. When the $\sigma_z = \sigma_1$ (first and fourth stress paths) for $\sigma_{conf} \leq 2 \text{ MPa}$ the debonding points were observed after the maximum principal stress ($\sigma_1$) for 10 mm and 20 mm borehole size specimens. In these cases there are two points with similar stresses and different strains on the $\sigma_1 - \varepsilon_1$ graph.

Modulus of toughness ($U$) for different stress paths was derived and plotted versus $\sigma_{m,2}$ as a borehole instability criterion. The relationship between the data in the $U - \sigma_{m,2}$ domain was relatively linear.

For the first stress path, with an increase in the tangential stress the total strain energy (toughness) increased for different values of $w_c$, while the effect of the cement content was less considerable than the effect of the tangential stress in increasing the toughness. Test results on 20 mm borehole size samples showed that the effect of the borehole size is considerable and the comparison between the results suggested that for the same $w_c$ and $\sigma_2$, the level of strain energy absorbed by the material until reaching the bonding breakage point was much lower for the 20 mm diameter borehole specimens than for the 10 mm ones. However, the slope of trend lines in 10 mm and 20 mm borehole size specimens were similar.

In the second stress path, where $\sigma_\theta = \sigma_1$, the majority of strain absorbed energy was in the lateral direction as expected. Likewise, linear equations could be fitted to the obtained data in $U - \sigma_{m,2}$ domain. Also, the toughness dramatically increased with elevating the $\sigma_2 = \sigma_z$ in the second stress path for different $w_c$ values.

Despite of the remarkable influence of $w_c$ on the strength of the specimens, the calculations showed that $w_c$ will not dramatically affect the bonding breakage in the specimens when $\sigma_\theta$ was the maximum principal stress. The comparison between the second and third stress paths showed that the initial magnitude of $\sigma_z$, before achieving the hydrostatic status, did not affect the final toughness in poorly cemented sands while the final strengths for these paths varied.

Comparison between the first and fourth stress paths showed that for a given confining pressure, the specimens in the first path showed more ductile behaviour as they were exposed to a higher level of lateral support in the initial stage of the test. This pressure was not high
enough to induce damage to the borehole and the ultimate toughness was considerably higher for the first stress path than it was for the fourth path for a certain $w_c$.

Results from simultaneously increasing the principal stresses (the fifth stress path) showed that the stability of the TWHC specimens excelled significantly. This may be due to the increase of the reciprocated support at the borehole wall and counteracting effect of the applied stresses.

A near linear relationship was observed between the octahedral normal stress and the total absorbed energy as well. However, in terms of the obtained determination coefficients, $\sigma_{m,2}$ had a better linear regression equation than $\sigma_{oct}$ versus the total strain energy.

**Acknowledgements**

This work has been supported by the Deep Exploration Technologies Cooperative Research Centre whose activities are funded by the Australian Government’s Research Programme. This is DET CRC Document 2015/013.

**References**


CHAPTER 6

Concluding Remarks

This thesis has developed and introduced two different failure criteria for poorly cemented sandy formations adjacent to an unsupported borehole drilled through it. The first criterion is based on the Mogi failure domain and the second is based on the total potential and dissipative absorbed strain energy per volume of the material. For this research a mining exploration drilling site in South Australia was considered and site investigation was performed. At this site the exploration boreholes are drilled through poorly cemented formations. The majority of the boreholes are 25 cm to 30 cm in diameter with lengths varying from 80 m – 250 m depending on the mine exploration plan.

In this thesis, a stress failure quadrilateral has been introduced for different stress paths and cement contents for an unsupported borehole. A new experimental method has been designed and implemented to monitor the borehole status during the triaxial test in real-time. This system was synchronised with the data acquisition in order to detect the particle debonding point and identify the borehole failure during the test. Also, numerical simulation of the borehole instability by the discrete element method was performed in order to evaluate the influence of different parameters such as confined aquifer and in situ stresses on the borehole instability in the considered weak formation. Moreover, the micromechanical studies of the localised zones were performed by the scanning electron microscopy (SEM) to investigate the geometry of the damaged zone at the borehole wall.

Applying a fluid flow pressure in the numerical model showed that the failure of the borehole took place immediately after the borehole excavation. However, the borehole breakout without the presence of fluid pressure occurred within a short time after drilling. This clearly shows the important effect that a confined aquifer has on the borehole stability in sandy formation and that a borehole cannot be drilled through poorly cemented sands and maintained without any supporting system.

Modulus of toughness ($U$) for different stress paths was derived and plotted versus $\sigma_{m,2}$ as a borehole instability criterion. The results showed that the relationship between the data in the $U - \sigma_{m,2}$ domain was relatively linear.
The studies showed that the Drucker-Prager criterion cannot fully represent the failure behaviour of an element on the borehole wall. When using this domain the failure points for different borehole sizes and cement contents could not be fitted into a single failure envelope within the range of tests performed in this study. Implementing the Mogi criterion in a borehole stability model showed that all obtained results can be represented by a single straight line. Therefore, the influence of the size-scale effect and different cement contents on the boreholes were not significant within the range of test conditions practiced in this study.

SEM studies showed that the geometry of failed zones was influenced by the stress state and borehole size. The depth and width of the breakout zone increased with increasing the confining pressure \( (\sigma_h = \sigma_H) \), and the \( w_c \) values affected the shape of the breakout zone. Also the depth and width of the breakout zone for the specimens with borehole diameter of 20 mm were generally almost 20% and 70% larger than those for the 10 mm ones respectively.

For further summary on each major finding, please see the abstracts and conclusions of the papers included in this thesis.

While this thesis details the extensive progress that has been made towards the borehole stability analysis in poorly cemented sandy formation, there are still many aspects on which further research is required. This author would like to direct attention to the following topics for future research; (1) the introduced failure criteria can be extended by considering other cement contents, (2) The effect of other faulting systems can be simulated in the discrete element method, (3) The polyaxial testing machine can be modified and further developed for the cubic poorly cemented sand specimens with predefined boreholes in order to investigate the effect of the anisotropic horizontal stresses on the borehole failure.