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Evaluation of the behaviour of the lateral boreholes in the Gorm chalk field


*Danish Hydrocarbon Research and Technology Centre, Kgs. Lyngby, Denmark
bGeo, Kgs. Lyngby, Denmark
cISAMGEO Italia, Angera, Italy
dDepartment of Physics, Technical University of Denmark, Kgs. Lyngby, Denmark

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ABSTRACT

Open hole wellbores present cost efficient completion solution; however, instability of such boreholes in weak formations such as chalk in the course of production raises a concern. This paper presents a method for predicting the stability of the radial jet drilling laterals in the Danish Gorm chalk field. The method is based on four main parts: (1) rock mechanics testing in the triaxial cell; (2) the single lateral hole testing consisting of loading and fluid flow test; (3) utilizing CT imaging for identifying the damaged zone and its extension; and (4) numerical simulations, utilizing a chalk model that takes into account the post-peak softening and the rate dependency of the pore collapse stress. Simulation of bottom hole and reservoir pressure decline over a year by 17 MPa and 9 MPa, respectively, showed a small development of the plastic strain at the borehole wall. Further simulation up to four years with constant stress (creep) resulted in a change in the cross-sectional area of the borehole, where shear cracks developed at the wall and some distance away from the hole stress concentrations associated with pore collapse was observed. By the end of the simulation, the borehole was likely to change its geometry by removal of the plastified area. The single lateral hole test with flow carried out with differential pore pressure, drawdown, of about 2.5 MPa within five hours and one hour, respectively, provided insignificant permeability change during the flow test.

1. Introduction

Radial Jet Drilling (RJD) technology represents a well stimulation solution for hydrocarbon and geothermal wells, which is cost-effective compared to standard stimulation methods[1–3]. It consists of running the jet drilling nozzle in the coiled tubing down into the formation and creating extensive (up to 100 m) lateral open holes with a focused jet of a high-velocity fluid on the rock surface. In case of the cased hole completion, jet drilling is conducted preceding the milling of the casing. In spite of the lack of support from a casing, the open hole may sustain collapse in very competent formations; however, in soft or unconsolidated formations like chalk and sandstone, the open holes may be susceptible to instability during the course of production[4–11].

In chalk fields, open hole completions are difficult to design due to a complex combination of geomechanics and multiphase flow. The simulated stress path around the wellbore indicate that the most severe shear stress loads are experienced during drilling and in the first period of production[12]. However, the risk of failure increases with continued production, when reservoir depletion poses a high risk of instability and solid/fines production. There are three basic mechanisms identified as a source of fines production[13]; however, in field condition, it is highly possible that these mechanisms are interchangeable: (1) shear failure caused by an excessive drawdown magnitude that can lead to a catastrophic solids production; (2) tension failure resulted from a high gradient of pore pressure at the wall of borehole; and (3) fines migration, where fine particles in the porous space are carried by the fluid flow. According to Durrett et al.[14], solids are carried if the drag forces exerted on a surface particle exceeds the (apparent) cohesion between surface particles.

High porosity chalk, and so other soft or unconsolidated formations, experiences pore collapse in the course of production, which results in irreversible plastic deformation near the borehole. Chalk in the near wellbore also weakens...
as a result of production shut in or stimulations. This induced damage reduces the permeability, creating an increasing skin that results in steeper pressure gradient near the wellbore area. Moreover, test data on chalk show that with plastic deformation, the tensile strength reduces\(^{[15]}\). Several authors have reported the importance of the permissible well drawdown scheme for mitigating the solid production, thus the bean-up process is one of the effective means of controlling the solids production\(^{[16–19]}\). Beaning-up refers to the process of increasing the choke size to open up a well for production and it is a frequent event following a well shutdown. For weakly cemented formations, initial stages of bean-up after the rapid shut-down may affect the extent of elastic rebound of the grains, which is impacted directly by the magnitude of drawdown\(^{[17]}\). If bean-up (also shut-down) rate is aggressive, as a consequence, the potential for damaging the well increases, where generated transient gradients of pressure may mobilize fines.

When considering less costly stimulation techniques\(^{[20]}\), such as radial jet drilled laterals, the cost reduction and potential increase in productivity may be outweighed by the risk of borehole failure. Thus, evaluating the viability and stability of openhole laterals by detailed geomechanics analysis under in-situ operational conditions is of an essence. Oil bearing Ekofisk chalk formation in the Gorm field, located in the Danish part of the North Sea, is the focus area of a feasibility study for the RJD stimulation technique.

This study presents a series of rock mechanics tests, including advanced single lateral hole experiments and X-ray visualisations of the hole after the tests, on the Ekofisk formation reservoir chalk. The first aim is to establish a failure envelope and describe the behaviour of the studied chalk material under various stress paths. We particularly focus on how the in-situ reservoir conditions affect the stability of lateral holes. The results are directly useful for stimulation of wells in hydrocarbon and geothermal chalk reservoirs, and they also provide useful insights into the behaviour of lateral holes under various stress conditions.

2. Methods

2.1. The Chalk Model

Various visco-plasticity models have been proposed to simulate time-dependent deformation of soft rocks, like chalk\(^{[21–24]}\). These models together with many other models widely used for soft rocks are formulated based on overstress concept using different formulations relying on the relative position of stress point and reference or yield surface in stress space. However, these models often lack physical meanings for the model parameters and consistency of yield surfaces. In an alternative way, visco-plasticity can also be modeled via the consistent condition of non-stationary yield surfaces in which the plastic hardening rule is formulated with the consideration of both strain rate and viscoplastic deformation\(^{[21,25–28]}\).

The model used in this study is based on shear failure and rate dependent pore collapse behaviour of chalk. The shear failure is given by the straight failure line in the \(p' - q\) diagram (mean effective stress and deviatoric stress space) and described by extending the Mohr-Coulomb criterion with the stabilizing effect of the intermediate principal stress:

\[
F = \sqrt{J_2^C (\cos \theta + \frac{\sin \phi \sin \theta}{\sqrt{3}} - \zeta (2 \cos 2\theta - 1)) - (p' \sin \phi + c \cos \phi_{\text{peak}})}
\]  

(1)

where: \(J_2^C\) - second invariant of deviatoric stress for a Cosserat model. In this chalk model, the Cosserat continuum approach\(^{[29]}\) is adopted in order to overcome numerical difficulties due to shear strain localisation and to model the breakout development, in which additional rotational degrees-of-freedom and an internal length parameter are incorporated. If no Cosserat formulation is used, \(J_2^C\) corresponds to the classical second invariant of deviatoric stress; \(\phi\) - friction angle, \(c\) - cohesion; \(\theta\) - Lode’s angle (\(\theta = -\pi/6\) for stress path as in standard triaxial test), \(\zeta\) - scales the impact of the intermediate principal stress, \(p'\) - mean effective stress.

The cohesion \(c\) and the angle of internal friction \(\phi\) vary in the course of hardening/softening. Hardening and softening in case of shear failure are functions of equivalent plastic strain determined via its rate:

\[
\dot{\varepsilon}_{pl} = \sqrt{2((\dot{\varepsilon}_{xx,pl} - \dot{\varepsilon}_{v,pl})^2 + (\dot{\varepsilon}_{yy,pl} - \dot{\varepsilon}_{v,pl})^2 + (\dot{\varepsilon}_{zz,pl} - \dot{\varepsilon}_{v,pl})^2 + \dot{\varepsilon}_{xy,pl}^2 + \dot{\varepsilon}_{yz,pl}^2 + \dot{\varepsilon}_{zx,pl}^2)}
\]  

(2)

In the case of the equivalent plastic strain being lower than the equivalent plastic strain at the peak strength, the material hardens by increasing the angle of friction from an initial to a peak value. Cohesion is assumed to be constant until the peak strength is reached. After reaching the peak strength, the material softens by a decrease of the angle of

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friction and the effective cohesion. The friction angle declines exponentially, while the effective cohesion may decline either linearly or exponentially.

A non-associated flow rule is applied on the shear failure surface. The plastic flow rule is controlled by the plastic potential that uses the same function as the yield surface, but the angle of internal friction $\varphi$ is replaced by the dilatancy angle $\psi$ and the parameter $\zeta$ is replaced by $\zeta_0$.

The yield surface for the pore collapse is given by an elliptical cap in the $p' - q$ diagram, similar as for a Modified Cam-Clay model$^{[30]}$:

$$F = \frac{2(p' + A_0)}{(p_{cc} + A_0)} - 1)^2 + \left( \frac{2q}{M(p_{cc} + A_0)} \right)^2 - 1$$

(3)

where $q$ is the deviatoric stress; $p_{cc}$ - hydrostatic pore collapse strength for the current volumetric plastic strain rate; $M$ - material parameter (similar in Modified Cam-Clay model which defines the slope of the critical state line) determines the shape of the elliptical yield surface; and $A_0$ - parameter which denotes a shift of the ellipse along the $p'$ axis.

The pore collapse yield surface is formulated with the associated flow rule. Generally, the rate independent model gives no strain or a small elastic rebound at constant stress condition, whereas the rate dependent model results in visco-plastic softening, where the onset of pore collapse is dependent on the rate. As the constant stress state cannot be beyond the yield surface, the strain softening is compensated by the strain hardening, resulting in volumetric plastic strain increase, i.e. creep strain$^{[27]}$. Chalk displays a susceptibility to compaction beyond the pore collapse state and similar to other rock types as unconsolidated sandstones$^{[31]}$ and diatomite and calcium carbonate$^{[32]}$, also shows rate dependency under constant stress$^{[33-35]}$. In this chalk model, the rate dependency of the pore collapse strength on the volumetric plastic strain rate is based on the de-Waal’s model$^{[36]}$:

$$p_{cc} = p_{c0} \left( \frac{\dot{\varepsilon}_{pl}}{\dot{\varepsilon}_{0}} \right)^b$$

(4)

where $\dot{\varepsilon}_{0}$ denotes the reference volumetric plastic strain rate and $b$ is a material parameter, representing the creep effect; $p_{c0}$ defines the standard hydrostatic pore collapse strength at the reference rate which depends on the volumetric plastic strain and it is governed by the hardening parameter. At the onset of pore collapse, the volumetric plastic strain rate increases, inducing an increase of $p_{cc}$. On the other hand, when the rate declines, $p_{cc}$ declines and the strain hardening is required to adjust $p_{cc}$ to the current state of stress, resulting in creep strain. It must be noted that if both shear and pore collapse criteria are violated, the preference is given to the shear failure.

The de Waal parameter $b$ translates the pore collapse strength determined in laboratory tests to the field conditions, as loading rate in the laboratory is much faster than a field compaction rate$^{[37]}$. For example, the reported compaction rate in the Valhall field is about 0.0001 %/h, whereas the typical loading rate in the laboratory is 0.1 %/h. According to de-Waal’s rate type model, the strain during the creep test has the form$^{[37]}$:

$$\varepsilon = c_{m0} \sigma \ln \left( 1 + \frac{\dot{\varepsilon}_{0}}{c_{m0} \sigma} \right)$$

(5)

where $c_{m0}$ is the uniaxial strain compaction compressibility.

Differentiation of Equation 5 yields the following relation for the creep strain rate:

$$\dot{\varepsilon} = \frac{\dot{\varepsilon}_{0}}{1 + \frac{\dot{\varepsilon}_{0}}{c_{m0} \sigma}}$$

(6)

2.2. Wellbore stability analysis

The workflow for the advanced wellbore stability analysis is presented in Figure 1. The workflow is based on four main parts: (1) the conventional rock mechanics testing; (2) an advanced rock mechanics testing method, called the single lateral hole (SLH), in which the specimen was subjected to boundary conditions imposed by the mechanical stress and hydraulic force; (3) utilizing CT imaging for identifying the damaged zone and its extension; and (4) backward/forward numerical simulations of the test data e.g.$^{[38]}$. In this paper, numerical analyses are performed using the ISAMGEO simulator, employing the chalk model$^{[27]}$, which is adopted to take into account the post-peak shear softening and the rate dependency of pore collapse deformation.
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Fig. 1: Workflow for the advanced wellbore stability analysis

Fig. 2: Triaxial test stress directions for building failure envelope (shear failure and end-cap) for the chalk depicted in the $p' - q$ diagram.

2.2.1. Experimental setup

The test program for the conventional triaxial test was designed in order to establish a model that captures the pore collapse and the shear failure behaviour in the $p' - q$ diagram (Figure 2). For that reason, hydrostatic compression (hydrostatic), two-stage triaxial compression (shear failure) tests, and uniaxial strain compaction test were performed in accordance with standard ISRM test procedures on cylindrical core specimens [39]. In the hydrostatic compression test, the specimen was loaded under isotropic stresses until pore collapse was achieved, while in the triaxial two-stage compression tests, the specimen was loaded under axial compression at two consecutive confining pressures. In the uniaxial strain compaction test, the specimen with zero lateral strain condition was maintained (disregarding the lateral strain from strain gauges) until pore collapse was achieved. Pore collapse points were determined from the uniaxial strain compaction test and standard hydrostatic compression test, thereby establishing the end-cap surface. Three sets of triaxial two-stage compression tests were carried out to establish a shear failure surface. Triaxial compression and compaction tests included consolidation with effective axial stress to effective radial stress ratio of 0.4 up to effective axial stress of 20 MPa. Moreover, all the tests included constant stress phase (creep) of at least about one day. Throughout the tests, the specimens were loaded at a constant strain rate of 0.05 %/h.
2.2.2. Modelling in the Single Element technique

Once the stress and strain curve was obtained from the triaxial tests, back analysis of the stress and strain curve was carried for each test, using the Single Element technique. In this technique, the rock specimen is represented by a single element and the stress and/or strain boundary conditions are applied at the element boundary. The single element model is a good first approximation, however, due to the homogeneous stress and strain condition within the specimen, the Single Element approach fails to model the localisation of deformation along the discrete shear plane, as it smears the strains over the whole element representing the rock specimen. Thus the shear failure parameters with softening and dilatancy can vary from the refined finite grid model where strain localisation is a product of the simulation.

2.2.3. SLH imaging

X-ray computed microtomography (CT) was performed using a Nikon XT H 225 ST system with a cone beam setup. For each sample 1571 radiographic projections (2 frames per projection) were recorded by a flat panel detector (2000 x 2000 pixels, binned to 1000 x 1000 pixels) with an exposure time of 1.4 second (2 sec for the sample shown in Figure 14 a), while rotating the sample 360 degrees. X-ray radiation was generated using a tungsten filament with an acceleration voltage of 185 kV and power of 83.5 W. The X-ray radiation was passed through a 1.0 mm Sn filter. The 3D images were reconstructed using the cone beam filtered back projection method implemented in the X-Tek CT Pro 3D software (Nikon Metrology Inc.). The final voxel sizes were 142.3 um (Figure 9), 126.224 um (Figure 14 a), 88.849 um (Figure 14 b) and 89.130 um (Figure 14 c).

2.2.4. SLH testing

The SLH test was carried out on a test specimen with a diameter of 98.6 mm and height of 198 mm (Figure 3). A ‘horizontal wellbore’ hole with a diameter of 2.044 cm was drilled laterally at the center of the specimen and to a depth corresponding to 5.74 cm. Afterwards, a slightly bigger hole (plug hole) was drilled to a depth of 2.23 cm and a diameter of 2.552 cm. The reason for this was that the ‘horizontal wellbore’ had to be sealed inside the test specimen, otherwise the outside rubber membrane supplying the confining pressure would enter the borehole when the specimen was being loaded. Then the borehole was sealed by a plug of similar chalk material. The plug was carefully shaped to fit the SLH test specimen. A tiny hole was made at the center of the plug and a steel tubing connecting to the inner wellbore was installed. After installation of the plug, the tubing was sealed with epoxy rubber to prevent leakage from the inner hole. The rim of the plug was also sealed with epoxy rubber, to prevent the membrane entering the borehole due to confining pressure and to prevent the plug to fail in tension due to axial loading. The strain gauges were installed at selected locations (Figure 3 a), and the wires and tubing were aligned and fixed before the specimen was enclosed by the inner epoxy membrane. Finally the specimen was installed in the Hoek cell and mounted in the load frame, and the LVDT’s were aligned. In total 5 axial strain gauges and 2 LVDT’s were used for measuring the axial deformation, while 3 radial strain gauges were used for measuring the radial deformation.

The SLH test was carried out in two ways: (1) loading phase, in which the specimen with a hole was tested under hydrostatic compression with a fixed stress ratio of 0.4 up to 21 MPa effective axial stress followed by a constant stress phase (creep) at drained condition; (2) flowing phase, in which the same procedure applied, except after the creep phase, the fluid flow from the end boundaries of the specimen to the borehole was allowed, simulating the production condition in the reservoir. The chalk saturated tap water was flushed at five levels of constant rate until 2.5 MPa drawdown pressure was achieved, over 5 hours and one hour. Figure 3a presents the schematic of the single lateral hole test carried out with flow. As can be seen, the fluid was supplied from the top and bottom of the specimen via four interconnected pressure actuators (GDS). The fluid flown to the hole was collected on a balance.

2.2.5. Modelling the SLH test

As it was required to dismount the specimen for CT scanning, it was necessary to run a predictive model for the SLH test in order to determine at which stress state the breakouts begin developing around the horizontal hole. At stress ratio of around 0.4, uniaxial strain compaction condition was obtained from the test, thus modelling of the SLH test by means of a 2D simulation under plain strain condition can represent the results of the 3D simulation (Figure 4 a). The specimen had the following dimensions: diameter was 98.6 mm, height was 197.6 mm, and hole diameter was 20.44 mm. The model consisted of 3386 higher order finite elements. Assuming symmetry, one fourth of the full geometry was modelled. In the model, above and below the specimen, interface elements were used with a thickness of 1 mm.
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Fig. 3: a) Schematic of the single lateral hole testing with flow and a close look-up is a cross-section normal to the hole axis and illustrates the axial and radial stress boundaries at the specimen, as well as injected fluid pressure ($P_{inj}$) at the edge boundaries and expelled fluid pressure ($P_{exp}$) through the hole. Four pumps (GDS) are utilised to inject oil into the high pressure separator vessels to push the water into the specimen. b) Top view schematic of the specimen and positioning of the axial/radial strain gauges and LVDT's; c) Illustration of the Ekofisk formation chalk specimen sealed with the plug hole and glued with the strain gauges. The chalk specimen has height of 200 mm and a diameter of 100 mm. The lateral hole is drilled with a length of 57.4 mm and diameter of 20.44 mm. The end plug hole has a depth of 22.3 mm and a diameter of 25.52 mm.

2.2.6. Modelling the stability of the RJD laterals

For this study, the stability of the lateral borehole was investigated with the two-dimensional mesh illustrated in Figure 4 b. The 2D geometry with circular hole was modelled with 2772 higher order finite elements. The detailed view of the model shows that the borehole has 10 mm radius. Assuming symmetry, one fourth of the cross section was modelled. In the wellbore stability simulation, reservoir conditions corresponding to the Gorm chalk field at the reference depth of about 2070 m were used: the effective vertical stress of 11.8 MPa, effective (isotropic) horizontal stress of 5.8 MPa and initial reservoir fluid pressure of 29.5 MPa. The simulations were carried out in four steps:

1. During the first step, initial conditions (pore pressure, stresses) were applied, then the drilling phase of the wellbore was simulated. During drilling, the effective stresses normal to the borehole wall were reduced for elements inside the drilled area lasting for 12 hours followed by 2 days of constant stress phase. As in the Radial Jet Drilling the technique does not use drilling mud, the pressure change in the well was kept at zero.
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Fig. 4: a) 2D schematic of a specimen with single lateral hole (SLH) built in the finite element model; b) Finite element mesh for the stability analysis of a horizontal openhole lateral, assuming plane strain conditions. The model domain consists of the length 5 m and height of 5 m. A close up of the near borehole area shows that the diameter of the lateral is 10 mm. Applied initial conditions to the model: effective vertical stress of 11.8 MPa, effective horizontal stress of 5.8 MPa and reservoir fluid pressure of 29.5 MPa.

2. In the second step, for over 306 days, the production phase was simulated with incremental reduction of the wellbore pressure by 7 MPa relative to the pore pressure including a creep phase. The pressure is kept constant at the outer boundary.
3. The third step simulated the reservoir depletion, in which the pore pressure at the outer boundary was declined by 9 MPa and the wellbore pressure by 10 MPa with subsequent creep phase of about 50 days.
4. In the last step, long-term creep (up to four years) of the wellbore was simulated at constant boundary stresses set in the previous step.

2.3. Chalk material
The core material used in this study stems from the Gorm field’s Ekofisk formation, which is located in the Danish part of the North Sea. Prior to plugging, the core sections were wrapped in a plastic bag and cast in gypsum, in order to keep the core material correctly oriented and stable during plugging. Plugging was carried out using a diamond drill bit. Isopar-L oil was used as coolant. Finally, the specimens were end trimmed to the final length of about 7.5 cm and diameter of 3.8 cm for the triaxial tests and length of about 200 cm and diameter of 10 cm for the SLH tests. Five specimens were prepared for the triaxial tests (with labels 3.A1, 3.A2, 3.A3, 3.A4 and 3.B3) and two specimens for the SLH test (with labels SLH 1 and SLH 2). After plugging, the specimens were oven dried at 105° C until no loss of mass was seen. The porosity of the specimens for the conventional tests was in the range of 35.0-35.7% (estimated based on grain density of 2.71 g/cm³) and the dry bulk density was 1.74-1.76 g/cm³. The porosity of the specimens prepared for the SLH test was also about 35%.

3. Results
3.1. Interpretation of the lab data
In total five rock mechanics tests on standard cylindrical specimens were carried out. Hydrostatic compression test on specimen 3.A4, which included first consolidation phase up to effective axial stress of 20 MPa with a fixed stress ratio of 0.4, and then compaction phase until pore collapse was achieved. Two stage triaxial compression tests were carried out on specimens 3.A1, 3.A2 and 3.A3, which also included consolidation phase up to effective axial stress
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**Fig. 5:** The plots of a) mean stress $p'$ against axial strain; b) deviatoric stress $q$ against axial strain; and c) shear failure and pore collapse yield surface

of 20 MPa with a fixed stress ratio of 0.4, followed by shear test at two different confining pressures. The specimen 3.A1 was tested with 1.5 MPa and 3 MPa of confining pressure during the first and second shear phases, respectively. The specimen 3.A2 was tested with 6 MPa confining pressure during the first shear phase and with 1.5 MPa confining pressure during the second shear phase. The third specimen 3.A3 was tested with constant confining stress of 3 MPa and 6 MPa, during the first and second shear phases, respectively. The specimen 3.B2 was tested under uniaxial strain compaction condition until pore collapse was achieved.

Stress-strain curves describing the geomechanics behaviour from the triaxial tests were interpreted in order to evaluate the deformation and strength properties. The Young’s modulus was estimated for specimens 3.A1, 3.A2 and 3.A3 tested in triaxial compression, from the axial loading path under constant confining pressure, as the effective axial stress increase divided by the axial strain increase. Similarly, the Poisson’s ratio was determined from the ratio of radial strain to the axial strain, under the test path with constant confining pressure. The bulk modulus was estimated for specimen 3.A4 tested under hydrostatic compression, from the effective mean stress increase divided to the volumetric strain increase. The ratio of horizontal to vertical stress under uniaxial strain condition can help us to give the desired ratio of stress in which plane strain condition is satisfied at vertical plane in elastic range. This ratio is selected as the desired value of stress ratio for loading at constant stress ration in SLH test to keep the stress status close to plain strain condition in vertical plane. The elasto-plastic modulus in uniaxial strain compaction test was determined from the axial loading path, as the tangent slope of the effective axial stress versus axial strain in the plastic region, while the inverse of this modulus provided the estimate of the uniaxial strain compressibility. One must note that the uniaxial strain compaction modulus is an elastic parameter, since the creep phase was performed post pore collapse, the elasto-plastic modulus in uniaxial strain compaction was used in the calculations.

For the reservoir chalk with about 35% porosity, the magnitude of the Young’s modulus (estimated from from LVDT measurement, $E^*$, and strain gauge measurement, $E$) was in the range of 4000-4900 MPa from LVDT, while strain gauge measurement provided the range of 3600-6100 MPa. An explanation for this is that strain gauges provide a local measurement of strain and chalk can exhibit local variation of porosity inside the specimen, so more scattering in Young’s modulus is estimated from strain gauge measurements while the overall porosity of the specimen is rather homogeneous. The Poisson’s ratio ($v$) for this reservoir chalk was in the range between 0.2 and 0.27. The calculated bulk modulus was 3000 MPa from the expelled to the balance volumetric strain curve, while from the volumetric strain with strain gauge measurements the estimate was 3500 MPa. The stress ratio at which plain strain condition can be achieved was estimated to be 0.35. The yield surface was constructed as a combination of shear stresses and pore collapse strength illustrated in the $p' - q$ diagram in Figure 5. With failure and residual strength on stress-strain curves from triaxial compression tests, shown in Figure 5, shear failure parameters are estimated. To define the shape of the pore collapse yield surface, the onset of pore collapse should be determined in $p' - q$ diagram from the stress-strain curve of uniaxial strain compaction and hydrostatic compression tests at experimental rate. A procedure to define the pore collapse can be found in Amour et.al.[41].

Figure 6 illustrates the evolution of the compressional and shear wave velocities under hydrostatic compression, compression and uniaxial strain conditions. The general trend observed from these plots was that, under hydrostatic compression loading, the velocity (for the reservoir chalk) at a stress state approached the pore collapse (approximately
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Fig. 6: Compressional and shear wave velocity development for specimens tested under a) hydrostatic compression (3.A4), b) uniaxial strain compaction (3.B2), and c-e) triaxial compression load (3.A1, 3.A2 and 3.A3); f) Cross plot of the compressional and shear wave velocity for all the tested specimens. 3.A3 is tested at 1.5 and 3 MPa confining stresses during the first and second compressive loading, 3.A2 is tested at 6 and 1.5 MPa confining stresses during the first and second compressive loading, 3.A3 is tested at 3 and 6 MPa confining stresses during the first and second compressive loading.

at 36 MPa), then again increased and slowed down at a steady velocity throughout the rest of loading. Similarly, in uniaxial strain compaction test the velocities slowed down when approaching a pore collapse, but increased again after the onset of pore collapse. This suggests that at a micro scale, pore collapse is likely occurring. During triaxial compressive loading, velocities increased with applied loading; however, closer to a stress state where shear failure developed and micro-cracks were generated, velocities slowed down noticeably. Figure 6 d provides cross plot of the compressional and shear wave velocities on the horizontal and vertical axis, respectively. It can be observed that the difference between the compressional and shear wave velocity was approximately a factor of two.

Figure 7 provides information on the creep response of the reservoir chalk at constant 60 MPa effective axial stress under uniaxial strain boundary condition for the specimen 3.B3. It shows that under constant stress, the chalk continued
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Fig. 7: Creep response at constant effective axial stress of 60 MPa from uniaxial strain compaction test (for specimen 3.B2): a) Fitting of the strain rate versus logarithmic time from model with experiment; b) Fitting of the strain versus logarithmic time from model with experiment. The estimated creep parameter $b$ is 0.9.

Table 1
Summary of the rock mechanics properties from conventional hydrostatic compression (3.A4) and uniaxial strain (3.B2) tests.

<table>
<thead>
<tr>
<th>ID</th>
<th>$\phi$</th>
<th>$K$</th>
<th>$\sigma_{1,\text{onset}}$</th>
<th>$\sigma_{3,\text{onset}}$</th>
<th>$K_0$</th>
<th>$\epsilon_{m0}$</th>
<th>$K^*$</th>
<th>$\chi$</th>
<th>$b_{\text{creep}}$</th>
</tr>
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<td>3.A4</td>
<td>35.3</td>
<td>0.6</td>
<td>36</td>
<td>36</td>
<td>3000</td>
<td>3500</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3.B2</td>
<td>35.8</td>
<td>0.5</td>
<td>43.5</td>
<td>20.5</td>
<td>0.35</td>
<td>800</td>
<td>$1.25\times10^{-3}$</td>
<td>-</td>
<td>0.09</td>
</tr>
</tbody>
</table>

to compact with increasing strain, but with decreasing rate. Andersen et al.\textsuperscript{[37]} explain this rate-dependent effect as rate-dependent friction between grain contact points (asperites) of the chalk, which has a typical size of 1-2 micron. At the initial stage of the constant stress phase, the asperites slide along under the imposed load and continue to be in motion until the static friction is overcome. Even if the imposed strain rate stops, the asperites either slow down sliding or stop sliding transferring the load to nearby contact points. According to the logarithmic relation given in Equation 5, the process eventually slows with time when there is not enough shear stress to overcome the static friction between the contact points.

The practical application of Figure 7 is that the de Waal’s creep parameter ($b$) can be estimated from Equations 5 and 6 by fitting the two curves from the experiment: (1) strain rate versus logarithmic time and (2) additional strain versus logarithmic time. The model best fitted to the experiment data with the value of 0.08 %/h of the applied strain rate before the experiment at time zero ($\dot{\epsilon}_0$) and with value of 1.13 $10^{-4}$ MPa$^{-1}$ of the multiplication of the creep parameter and uniaxial compressibility ($bc_{m0}$). Considering the uniaxial compressibility in the plastic region has a value of 1.25 $10^{-3}$ MPa$^{-1}$, the model provided the estimate of the parameter $b$ as 0.09. This lies within the range of $b$ values inferred by other studies for chalk\textsuperscript{[42]}.

Tables 1 and 2 provides summary of the estimated elastic ($K^*$) bulk modulus from expelled fluid into the balance, $K$ bulk modulus from strain gauge measurement, $E^*$ Young’s modulus from LVDT, $E$ Young’s modulus from strain gauge measurement, $\nu$ Poisson’s ratio, $K_0$ stress ratio, $M$ elasto-plastic modulus in uniaxial strain compaction and $\epsilon_{m0}$ uniaxial strain compressibility, as well as stresses ($\sigma_{1,\text{onset}}$ and $\sigma_{3,\text{onset}}$ are onset estimations of effective axial and radial stresses; $\sigma_{1,\text{fail}}$ and $\sigma_{3,\text{fail}}$ failure and residual stress estimations of effective axial stress; $\sigma_{3,\text{fail}}$ and $\sigma_{3,\text{res}}$ failure and residual stress estimations of effective radial stress) for the reservoir chalk tested under hydrostatic compression, compression and uniaxial strain compaction conditions. A detailed back analysis of the test results presented in this section utilising the Single Element approach is provided in the supplementary material (see also Table S1).

3.2. Single lateral hole test

Figure 8 presents the results of the single lateral hole test for the specimen SLH 1. The time history of the effective axial and radial stresses is illustrated in Figure 8 a. The test was carried under hydrostatic compression with a fixed stress ratio (effective radial stress to effective axial stress) of 0.4. The specimen was loaded to the effective axial stress of 15.7 MPa followed by a constant stress (creep) phase, and then the effective axial stress was increased to 21 MPa
Table 2

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Fig. 8: Single lateral hole test on specimen SLH 1: a) Time history of effective axial (SigA) and radial (SigR) stresses; b) Time history of axial strain from LVDT and volumetric strain from balance measurement; c) Time history of axial and radial strains from strain gauge measurement.

with a subsequent constant stress phase. The axial strain from strain gauge measurement is the average of the three measurements: two located at the sides of the specimen circumference, and one located at the end face of the horizontal hole. The radial strain gauge measurement is also the average of the three strain gauges (recall the positioning of the axial/radial strain gauges from Figure 3). Figure 8 b and c present the time history of the strains from LVDT, strain gauge and expelled fluid on the balance. It can be observed that about 0.1% axial strain developed during the first creep phase from LVDT, whereas 0.07% axial strain measured from the strain gauge. The volumetric strain from expelled fluid on the balance showed higher straining (about 0.2%). With the increase of the axial strain from strain gauge during the creep phases, corresponding decline in the radial strain was observed. This volume change may indicate that locally the chalk in some distance close to the hole is undergoing a pronounced deformation.
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Fig. 9: CT image visualisation of the breakout development for the specimen SLH 1: a) top view of the lateral hole and b) plane view of the cross section of the hole in the middle of the specimen (red line), where shear cracks developed at the sides of the wall. Scale 47.12 mm.

Borehole breakout occurs as fracturing or spalling of the rock adjacent to the walls of the borehole that is drilled into rock subjected to stress. When the internal pressure is zero, the rock close to the boundary of the wall is subjected to unconfined, plane strain compression, resulting in a concentration of the tangential stress in the direction of the least principal stress; however, the rock further away from the borehole behaves as confined, with reduced tangential stress with distance from the borehole wall [43].

Figure 9 presents a visualisation of the stress induced breakout developed at the cross section of the borehole of the tested SLH 1 specimen. X-ray computed microtomography (CT) was performed using a Nikon XT H 225 ST system with a cone beam setup. From the image, the failure cracks were initiated at the wall. This unstable cracks separated as thin rock flakes from the bulk rock. Ewy et al. [44] have observed that at the breakout base, electrical conductivity of the rock enhances, suggesting an increase in porosity caused by incomplete spalling and dilation of the rock in those regions. In addition to those thin flakes, two large shear cracks at each side of the borehole subparallel to the wall were formed. Haimson and Song [45] suggest that in carbonate rocks, where the grains are much stronger than the cementing material, the micromechanism of failure is intergranular shear. The open state of these two large cracks from the CT image suggest the intergranular shear dislodging and crushing of the grains [45,46].

It must be mentioned that high and medium porosity chalk represent a special case where at some distance from the hole, pore collapse is encountered, with pronounced stress concentration. This indirectly causes extension of the damaged zone near the borehole due to stress redistribution as a result of the pronounced stress increase some distance away from the hole. Generally, the shape of the borehole breakout narrows in the direction of the minimum principal stress and stabilizes at some point, despite the very high shear stress concentrations. For soft rocks such as chalk, that exhibit strain softening post peak, the complete removal of the rock by spalling may be prevented by the residual strength of the material. Despite of this, if a slightly larger differential stress is applied on this specimen, then the large conjugate shear fractures would intersect, forming a v shaped (or dog-ear) breakout.

An evaluation of the shear breakout development at the borehole wall from experiment requires care in the interpretation (upscaling to the reservoir condition), as the observed breakouts are not only related to the state of the stress and strength of the rock, but also whether experiment was carried out with pre drilled borehole or drilling of the hole was carried out simultaneously with experiment. Zheng et al. [43] state that, generally, the stable breakout created instantly in rock already subjected to stress is much larger than the stable breakout created in the same rock with a preexisting borehole by subsequently increasing the stress increased to the same values.

3.3. Wellbore stability analysis

For the wellbore stability analysis, the reservoir chalk properties derived from the back analysis of the SLH test for the specimen SLH 1 with 2D modelling were used as input for the simulator (Table S2). Figure 10a presents the time history of the pore pressure at the borehole and at the boundary of the model, which was 5 m away from the hole. During the first step, in which the drilling of the lateral borehole was simulated, the bottom hole pressure (BHP) and reservoir pressure (Pres) were kept equal for 12 hours with a subsequent constant stress phase for 48 hours. Following this step, production started with an incremental decline of the BPH by 7 MPa over 306 days. In the third step, both BHP and reservoir pressure were declined by 10 MPa and 9 MPa, respectively, with a subsequent constant
Fig. 10: a) The time history of the pressure change at the bottom hole (BHP) and reservoir (Pres); b) Development of the equivalent plastic strain at the borehole wall in the course of increasing drawdown/depletion. The results are shown next to the hole in zone of length=0.1 m, height=0.1 m.

stress phase lasting about 50 days. Figure 10 b presents the equivalent plastic strain close to the hole as calculated after approximately 370 days. In our study, the instability of the borehole wall was assessed in a plasticity based manner, i.e. considering magnitude and spatial distribution (shape). If there were no plastic strain around the borehole, the risk of borehole instability associated with fines production would be zero. If pronounced plastic strain occurs, the risk of instability must be considered. After undergoing the drawdown and depletion schemes described above, the rock at the borehole wall developed approximately 20% plastic strain. Based on the quite limited extent of the plastified zone at the borehole wall, the well was regarded to be stable.

In order to assess the feasibility of open hole radial laterals in the long-run, the simulation continued further up to 4 years under constant stress conditions, with bottom hole pressure of 12.5 MPa and reservoir pressure of 20.5 MPa. Figure 11 presents the calculated equivalent plastic strain, porosity, and vertical stress after 1.5 years, 2 years, 3 years and 4 years of simulation of production at constant stress. After one and a half years of production, the magnitude of the plastic strain at the borehole wall increased from 20% to 30%. The cross-section of the borehole clearly changed, it shrunk and the plastified zone spread further into the chalk with shear breakouts. This plastic zone resulted in a decrease of the vertical stress at the vicinity of the borehole wall; however, due to stress deviation, at some distance from the wellbore wall, the stress concentration was observed to have a magnitude of about 46 MPa. In this stress concentrated area, the porosity decreased, suggesting that pore collapse was occurring. At the wall, the porosity increased, corresponding to a volume increase due to dilatancy.

In the course of continued production up to four years, the plastified zone at the borehole wall further developed into the chalk with a pronounced shear breakout. Correspondingly, expansion of the stress concentration further into the chalk and decrease of the vertical stress at borehole wall continued. At the end of the simulation time of four years, it was observed that the high porosity region, corresponding to the failed chalk and the low porosity region, corresponding to the pore collapse state were getting closer to each other, which can be regarded as a potential risk for borehole instability. It is likely that the cross-section of the hole will change by removal of the plastified zone. With this results, the importance of using a modelling tool capable of simulating shear strain localisation by incorporating Cosserat approach is demonstrated.

3.4. Single lateral hole test with flow

This section presents the test data for the specimen SLH 2. The SLH test incorporating fluid flow to be presented in this section aims at investigating the destabilizing effect of fluid drag forces due to transient pressure gradient in the small sized lateral boreholes. The experiment was carried out in three steps: (1) The specimen was loaded under hydrostatic compression with a fixed stress ratio of 0.4 to effective axial stress of 21 MPa, then the specimen was unloaded and investigated under CT scan for evaluating the extent of the damaged zone around the hole. Since the destabilization is mainly related to the rock’s damage, it was important to carry out the flow test on the specimen with a hole that has had a damage; (2) The same specimen was brought to the same level of effective axial stress and then maintained at a constant stress level before the flow test started. During the flow test, the load frame was fixed, hence total stresses at the end of the specimen maintained constant. In addition, radial strain was also maintained constant
Fig. 11: Long-term open hole stability analysis for a lateral with circular geometry. Left - Equivalent plastic strain; Middle - Porosity; Right - Vertical stress. The result is shown next to the hole in zone of length=0.1 m, height=0.1 m. Porosity plots show the development of lower porosity regions at some distance away from the borehole wall, suggesting the concentration of tangential stress in some distance away from the wall.

by the expelled fluid. The flow test was carried out with drawdown for 5 hours. After the 5 hours of flow test, the specimen was investigated under CT scan for evaluating the extent of the damaged zone around the hole. (3) In this step, the same procedure repeated for the specimen tested in earlier step, where flow test lasted for 1 hour.

Figure 12 presents the time history of the effective axial and radial stresses, the pore pressure, as well as the strain and ultrasonic velocity measurements for the test with flow phase for the SLH specimen 2. The plot shows loading to effective axial stress of 21 MPa with subsequent constant stress phase and flow test. The creep phase resulted in
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Fig. 12: Single lateral hole test with drawdown phase included for the specimen SLH 2: a) Time history of effective axial and radial stresses; b) Time history of axial strain from LVDT measurement; c) Time history of the axial and radial strain from strain gauge measurement; d) Plot of the compressive and shear wave velocities development in the course of increased effective axial stress with a fixed stress ratio of 0.4.

insignificant change of the axial strains recorded from LVDT’s and strain gauges; however, during the flow test, the radial and volumetric strain from expelled fluid to the balance measured significant change in the strain. During the flow test, the pore pressure increase resulted in the effective stresses decrease; which in turn caused unloading of the specimen, thereby the decrease in the radial/volumetric strain. After the flow test, the pore pressure again decreased, so that radial strain reached its previous level prior to the start of the flow test. The increase in the compressive wave velocity throughout the test was observed, suggesting that at a micro scale, pore collapse is likely occurring, where grain boundary slip, grain rotation and calcite twinning micromechanism are active. The trend of the shear wave velocity at the initial stage of loading may be linked to the closure of the randomly oriented microcracks, while further increase of the loading reduced or halted the velocity development, where grain breakage and destruction of cementation was likely occurring.

As mentioned earlier, the flow test was carried out at two different drawdown rates: (1) the slow drawdown rate lasted for about five hours and (2) the fast drawdown rate lasted for about one hour. Four pumps with capacity of about 80 cm$^3$/h were used for flushing water during 5 hours and 1 hour of drawdown test. The fluid was injected from the top and bottom of the specimen at five levels of constant rates until almost 2.5 MPa pore pressure difference was achieved between the hole and the two ends of the sample (Figure 13 a). Figure 13 b presents a plot of the permeability and drawdown pressure versus the flow rate during the flow test of 5 hours and 1 hour. Prior to applying any load on the specimen, the fluid flushing showed a permeability of about 0.33 mD for this specimen. During the actual flow test, both cases showed constant permeability, of approximately 0.23 mD. The fast drawdown case had a slightly lower permeability value, compared to the slow drawdown phase. We relate this permeability reduction not to the flow test, but to compaction, where loading of the specimen prior to each flow test causes reduction of pore connectivity. Based on the results, we suggest that tensile failure is unlikely to occur for the given drawdown rates applied to the Gorm field.

Figure 14 presents the CT imaging of the specimen SLH 2. Figure 14 a shows stress induced shear microcracks
were emanated at the borehole wall (mostly concentrated at the left side of the borehole) when effective axial stress level reached 21 MPa. As can be seen, the extent of the damaged zone is insignificant compared to the specimen SLH 1 in Figure 9. Figure 14 b and c show the development of the shear breakouts induced not only by stresses, but also due to fluid flushing during the 5 hours and 1 hour of flow test, respectively. Figure 14 b illustrates several fractures sheared off forming thin slabs at the wall. The breakout depth further increased in depth into the chalk at the end of the last flow test (Figure 14 c), where thin, but more elongated fractures aligned subparallel to the borehole wall were formed. The presented results from the flow test provide some insights on how the behaviour of the open lateral hole would be under dynamic operational conditions in the reservoir. Although the results of the flow test did not suggest significant permeability alteration due to the drawdown rate, future tests with much higher drawdown rate will enable to design safer operational conditions for the chalk reservoir producing from an open hole completed well.

4. Conclusion

Oil-bearing Ekofisk chalk formation in the Gorm field, located in the Danish part of the North Sea, was the focus area of feasibility study of the RJD stimulation technique. The main objectives were (1) to assess the sustainability of the laterals under simulated pressure drawdown and reservoir depletion schemes; and (2) to evaluate influence of the transient fluid pressure gradient, generated during shut-down and bean-up operations on the stability of the laterals. Advanced experimental rock mechanics tests were carried out following a workflow for RJD hole stability analyses utilising actual reservoir material. These resulted in estimation of material properties of Gorm field’s Ekofisk formation, visualisation of breakout and damage zones, as well as prediction of the wellbore stability. The key findings of this study are as follows:

- The lateral hole remained stable with a small development of the plastic strain at the borehole wall during the early stage of the fluid production under the applied drawdown schemes.

- During creep phase, the development of shear failure breakouts expanded further into the chalk in time. At some distance from the borehole wall, the simulations indicate stress concentration and porosity reduction in this region, suggesting a pore collapse. At the wall, the porosity increased, corresponding to the volume increase due to dilatancy.

- Further, the results of the SLH flow tests with two different injection rates showed insignificant permeability change under the induced transient pressure gradient.

- In terms of stability of the hole in the long-run, based on the simulation analyses for the Gorm field, the lateral can serve without instability concerns up to a year. However, with time, depending on the operational conditions of the reservoir, such as drawdown rate, depletion, how aggressive the shut-in and shut-down processes, among
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Fig. 14: Breakout development for the specimen SLH 2: a) After effective axial loading to 21 MPa with a fixed stress ratio of 0.4 with subsequent constant stress phase; b) After effective axial loading to 21 MPa with a fixed stress ratio of 0.4 with subsequent constant stress phase, including flow test lasted for 5 hours; c) After effective axial loading to 21 MPa with a fixed stress ratio of 0.4 with subsequent constant stress phase, including flow test lasted for 1 hour. The final voxel sizes were a) 126.224 μm, b) 88.849 μm and c) 89.130 μm.

others, will affect the performance of the lateral hole. Simulation results suggest that the hole is likely to change its geometry significantly by removal of the plastified area after three years of the fluid production.

5. Acknowledgements

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References

Evaluation of the behaviour of the borehole extensions in the Gorm chalk field


Evaluation of the behaviour of the lateral boreholes in the Gorm chalk field


