Brittle–ductile transition, shear failure and leakage in shales and mudrocks

Runar Nygård a, Marte Gutierrez b,*, Rolf K. Bratli c, Kaare Høeg a

a Department of Geosciences, University of Oslo, P.O. Box 1047, Blindern, N-0316 Oslo, Norway
b Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA 24061, USA
c Applied Rock Mechanics LLC, Abu Dhabi, United Arab Emirates

Received 14 January 2005; received in revised form 22 September 2005; accepted 1 October 2005

Abstract

This paper presents an experimental study on brittle–ductile transition, shear behaviour, the formation of shear fractures in shales and mudrocks, and the hydraulic properties of shales and mudrocks with shear induced fractures. The experimental study is based on triaxial tests on different mudrocks and shales from North Sea reservoirs and adjacent areas sheared at different effective confining stresses. It is shown that during burial, mudrocks behave as normally consolidated materials and exhibit ductile response to increased load. However, several processes like uplift, chemical diagenesis and overpressure build-up turn mudrocks into overconsolidated materials which exhibit brittle behaviour during loading. Triaxial tests with different loading conditions have been performed to establish the brittle-to-ductile transition of various mudrocks. Based on the results of the tests, it is shown that the brittle-to-ductile transition can be related to the overconsolidation ratio. A relationship between normalized undrained shear strength and overconsolidation ratio is established. In combination with this relationship, a correlation between compressional wave velocity and apparent pre-consolidation stress, which accounts for both mechanical and chemical diagenesis, may be used as a tool to evaluate possible leakage of hydrocarbon seals.

© 2005 Published by Elsevier Ltd.

Keywords: Britteness; Ductility; Fracturing; Leakage; Mudrocks; Shales; Shear failure

1. Background

When oil accumulates in reservoirs, there has to be a competent seal that keeps the oil in place and prevents its further migration. Mudrocks and shales are common seals in hydrocarbon accumulations worldwide (Corcoran and Dore´, 2002). Seals must be tight enough to trap oil from further movement. At burial depths of typical offshore reservoirs, shales have permeabilities in the range of $10^{-23}$ to $10^{-17}$ m$^2$ and pore sizes as small as 0.3–12 nm (Neuzil, 1994; Katsube and Williamson, 1994; Best and Katsube, 1995; Dewhurst et al., 1999). The low permeability, the high capillary entry pressures and the small pore throats, often smaller than the size of large oil molecules like asphaltenes and resins, make mudrocks efficient as seals. On the other hand, several field studies have reported that these sediments can act as conduits for channelized secondary migration of hydrocarbons (Hubbert and Rubey, 1959; du Rouchet, 1981; Mandl and Harkness, 1987; Roberts and Nunn, 1995). Since mudrocks have such low matrix permeability, which prevents oil from flowing through their matrix, there must be another source of enhanced permeability before leakage occurs. One explanation for leakage through seals and secondary migration is oil movement through fractures.

The main uncertainties in determining the possibility of fluid flow and leakage along fractures in hydrocarbon seals are: (1) the timing of fracture formation, (2) the permeability of the individual fractures, (3) the fracture network geometry and distribution, and (4) the changes in fracture permeability including the possibility of fracture closure. The timing of fracture formation is governed by a fracturing criterion. Fractures are formed as hydraulic fractures when the pore pressure exceeds the sum of the minimum total stress and tensile strength of the sediment, as shear fractures when the shear stress exceeds the shear strength, or as mixed-mode extensional-shear fractures. Fracture permeability is a function of fracture aperture, roughness and degree of alteration (Bandis et al., 1983). Geometrical properties of fractures which affect fluid flow include spacing (or frequency), persistence, length,
orientation and connectivity. Changes in fracture permeability during loading are functions of the fracture aperture and roughness, and asperity strength in relation to the normal and shear stresses applied across and along the fracture (Gutiérrez et al., 2000).

Shear failure occurs when loading creates shear stresses that exceed the shear strength. However, shear failure does not always result in shear fracturing for rocks. In addition to the failure criterion, fracturing is controlled by the ductility or brittleness of the mudrock. The deformation can be brittle or ductile depending on the properties of the mudrock and the effective confining stress level. Ductile behaviour is characterized by contractive response and gradual deformation to failure. Brittle deformation is characterized by dilative response and sudden failure at a well-defined peak shear strength followed by strain softening down to residual shear strength. Brittle response can be accompanied by the formation of distinct shear failure surfaces. Ductile response usually produces more diffused deformation, as typically evidenced by lateral expansion of cylindrical specimen of mudrocks loaded in shear, with less distinct shear failure surface. Fig. 1 illustrates the different types of failure that rocks can undergo depending on the level of effective confinement.

Once formed, shear fractures in brittle mudrocks will dilate at low effective normal stresses and have increased permeability with increasing shear deformation. Shear fractures created in ductile mudrocks will contract at high effective stresses and thereby have reduced permeability with increasing shear deformation (Ingram and Urai, 1999; Gutiérrez et al., 2000). The hydro-mechanical behaviour of an extensional fracture in a mudstone has been investigated experimentally by Gutiérrez et al. (2000). Natural extensional fractures were subjected to normal and shear loading using a direct shear apparatus. Their results show the important effects of ductility and brittleness on changes in permeability of fractures subjected to normal and shear loading.

There have been extensive studies of hydraulic fracturing as a leakage mechanism for hydrocarbon migration. For instance, there are several models for predicting hydraulic fracturing and leakage particularly in overpressured sedimentary basins (e.g. Ungerer et al., 1990; Bredehoeft et al., 1994; Pedersen and Bjorlykke, 1994; Roberts and Nunn, 1995; Wang and Xie, 1998). Such overpressured sediments, where the pore fluid pressures exceed hydrostatic pressure, are common in shaly sections of sedimentary basins (Bethke, 1986; Hunt, 1990).

In comparison, there are fewer studies on shear fracturing as a mechanism for leakage in cap rocks than on hydraulic fracturing. As noted by Lorenz et al. (1991), the role of shear failure due to anisotropics stresses, in comparison to hydraulic fracturing, has not been accorded its full significance in the study of fracturing and leakage in horizontally layered rocks. Studies on shear fracturing as a leakage mechanism have been presented by Dewhurst and Hennig (2003), and Finkbeiner et al. (2001).

The objective of this paper is to study experimentally the shear behaviour and the formation of shear fractures in mudrocks and the hydraulic properties of mudrocks with shear induced fractures. The experimental study is based on triaxial tests on different mudrocks and shales from North Sea reservoirs and adjacent areas sheared at different effective confining stresses. The main goal of the experimental study is to develop a procedure to predict the brittle-to-ductile transition of mudrocks, and how brittleness affects shear failure, fracturing and leakage in mudrocks.

2. Failure criteria for sedimentary rocks

For hydraulic fractures, fracturing will occur when the following condition is met (Fig. 1):

$$\sigma'_3 = -\sigma_T$$

(1)

$$\sigma'_3$$ is the minimum effective horizontal stress, and $$\sigma_T$$ is the tensile strength of the sediment. Shear fracturing may occur if the following condition is met

$$\sigma'_3 = N_\phi \sigma'_1 + \sigma_c$$

(2)

where $$\sigma'_1$$ is the maximum effective principal stress. In the above equation, $$N_\phi$$ is the failure coefficient and $$\sigma_c$$ is the unconfined compressive strength, respectively. These parameters are related to the effective friction angle $$\phi$$ and the cohesion $$c$$ of the sediment as follows:

$$N_\phi = \frac{1 + \sin \phi}{1 - \sin \phi}, \quad \sigma_c = \frac{2c \cos \phi}{1 - \sin \phi}$$

(3)

Eq. (2) is the linear Mohr–Coulomb failure criterion, and it can also be written in terms of the effective mean stress $$\rho'$$ and shear stress $$q$$ as

$$q = \rho' \sin \phi + c \cot \phi$$

(4)
where the effective mean and shear stresses are defined as:

\[ p' = \frac{\sigma_1' + \sigma_3'}{2}; \quad q = \frac{\sigma_1' - \sigma_3'}{2} \]  

(5)

The effective stresses \( \sigma_i' \) correspond to differences between the total stress \( \sigma_i \) and the pore fluid pressure \( p_p \) according to Biot–Terzaghi's effective stress principle (Terzaghi, 1925; Biot, 1941)

\[ \sigma_i' = \sigma_i - \alpha p_p \]  

(6)

where \( \alpha \) is the Biot poroelastic constant.

The strength parameters \( \sigma_T, \phi \) and \( c \) may be determined by laboratory strength testing of rock specimens. If testing is done under a wide range of effective confining stresses, a failure criterion can be established. A typical composite tensile and shear failure surface for sedimentary rocks is shown in the effective normal stress vs. shear stress diagram in Fig. 1. There are three different types of fractures shown in the diagram, namely, extensional or tensile fractures (point A), shear fractures (B to C), and mixed-mode tensile-shear fractures (A to B). Hydraulic fractures are created when the fluid pressure exceeds the total minimum stress plus tensile strength of the rock, i.e. when the effective minimum stress becomes negative and exceeds the tensile strength (Eq. (1)). The tensile strength of mudrock seals is typically only a few percent of the total minimum stress plus tensile strength of the rock. The effective stresses \( \sigma_i' \) correspond to differences between the total stress \( \sigma_i \) and the pore fluid pressure \( p_p \) according to Biot–Terzaghi’s effective stress principle (Terzaghi, 1925; Biot, 1941).

3. Ductility and brittleness of sedimentary rocks

Brittle deformation is more likely to occur when a material is stiff and has high shear strength. Thus, a lithified mudrock is more brittle than young and uncemented sediments. Also, brittle response and shear fracturing is more likely to occur at low effective stresses than at high stresses as shown schematically in Fig. 1. Depending on the brittleness of the material, different types of discontinuities can form as shown in Fig. 1. For unconfined (i.e. no lateral effective stresses) condition, vertical or load parallel fractures can form for brittle rock even in the absence of tensile stresses. The style of failure changes dramatically with increasing rock ductility. At low confining stresses and high brittleness, local conjugated shear surfaces may form in combination with tensile surfaces. At intermediate confining stresses, distinct shear fractures become more dominant without the formation of vertical fractures. We further increase in effective confining stress causes the shear surface to be less distinct until the material becomes completely ductile and shows a more diffused rather than localized deformation. Note that the barrel shape typically observed on ductile specimens is caused by the restraining friction at the end caps that apply loads on laboratory specimens. With lubricated ends, specimens are expected to maintain a cylindrical shape.

An important type of shear-induced fracture is the sub-vertical fractures that are formed in a compressional regime under low effective confining stresses. Einstein and Dershowitz (1990), and Lorenz et al. (1991) have extensively discussed the origin and distribution of this type of fracture. These fractures are formed not due to extension or tensile boundary stress but by locally induced tensile stresses at the tips of microcracks or flaws. These flaws propagate parallel to the direction of maximum principal stress, which is often the vertical stress in the field, thereby creating sub-vertical fractures. In the presence of large lateral stresses, the propagating flaws may coalesce to form shear fractures instead of sub-vertical fractures. The importance of shear-induced fracturing as a source of permeability in reservoir and cap rocks was discussed by Lorenz et al. (1991). They showed that shear-induced fractures can be common in reservoirs even in the absence of bending of the strata. Einstein and Dershowitz (1990) argue that shear-induced fractures due to brittle failure of rocks are possibly the most common types of discontinuities, and they cover the middleground between faulting on the one hand, and tensile fracturing in an extensional regime on the other hand.

Different parameters have been proposed as measures of brittleness. Griggs and Handin (1960) related brittleness to the amount of rock deformation before failure, based on experimental observations that ductility generally increases with increasing strain at failure. Renshaw and Schulson (2001) developed a criterion for brittle-to-ductile transition based on the rate of loading. Petley (1999) proposed that the brittle-to-ductile transition for mudrocks corresponds to the intersection of the failure envelope corresponding to the peak shear stresses and the residual shear strength envelope.

A parameter used in soil mechanics to quantify the ductility or brittleness of clays is the overconsolidation ratio (OCR). Soil mechanics makes a clear distinction between materials that have been loaded previously to a higher effective stress level (e.g. due to burial) and then unloaded to its current in situ effective stress (e.g. due to uplift). These materials are called overconsolidated (OC) materials and the maximum effective vertical stress it has been subjected to in its geologic history is called the pre-consolidation stress \( \sigma_{v_{\text{max}}} \). If the current effective vertical stress in the material is as high or higher than at any time in its history, then the material is normally consolidated (NC). The overconsolidation ratio is defined as the ratio between the maximum effective vertical stress \( \sigma_{v_{\text{max}}} \) and the present effective vertical stress \( \sigma_v \), that is:

\[ \text{OCR} = \frac{\sigma_{v_{\text{max}}}}{\sigma_v} \]  

(7)

The above definition of OCR is only valid if the effective vertical stress \( \sigma_v \) is equal to the maximum effective principal stress \( \sigma_{1'} \) and the effective horizontal stress \( \sigma_{3'} \) is equal to the minimum effective principal stress \( \sigma_{3'} \). These conditions apply only in horizontally layered rocks where compaction is vertical and one-dimensional and where there are no tectonic stresses. However, the results and procedures to be presented here are also applicable to more complicated cases where the effective
principal stresses are not oriented in the horizontal and vertical directions. Application to the general case, where the maximum effective principal stress deviates from the vertical, would require knowledge of the stress situation including the orientations of the principal stresses in the field. As will be shown below, the above definition of OCR is applicable to both isotropic (i.e. $\sigma'_h = \sigma'_v$) and anisotropic (i.e. $\sigma'_h \neq \sigma'_v$) stress conditions.

Overconsolidation has an important effect on the compressibility and shear behaviour of clays. During compaction, OC clays are 5–15 times stiffer than NC clays and the compaction curve, expressed in terms of porosity vs. effective stress, of OC clays is much flatter than NC clays (Johnston and Novello, 1994). It is thus possible to detect the pre-consolidation stress from the shape of the compaction curve. The pre-consolidation stress corresponds to the sharpest point or bend in the compaction curve indicating yielding and an abrupt change in compressibility. The pre-consolidation stress is also considered as a yield stress in Critical State Soil Mechanics (Schofield and Wroth, 1968). Strongly overconsolidated clays exhibit brittle response with well-defined peak and residual shear strengths and stiffer shear stress–strain response than normally consolidated clays. At failure, OC clays develop well-defined shear surfaces. Since OCR increases with increasing pre-consolidation stress and decreasing vertical stress, one should be able to relate OCR to brittleness which also increases with increasing strength and decreasing stress level. Based on the difference in the behaviour of NC and OC materials, Ingram and Urai (1999) developed the following empirical brittleness index BRI for mudrocks

$$\text{BRI} = \frac{(\sigma'_c)_{OC}}{(\sigma'_c)_{NC}}$$  \hspace{1cm} (8)

where $(\sigma'_c)_{NC}$ and $(\sigma'_c)_{OC}$ are the unconfined compressive strength at NC and OC states.

Although OCR is widely used to quantify the ductility or brittleness of clays (see, for example, Ladd and Foott, 1977), there is still a need to determine the applicability of using OCR as a measure of brittleness for shales and mudrocks. This is because shales and mudrocks are materials intermediate between soft clays and hard cemented rocks. Overconsolidation in lithified mudrocks is caused not only by mechanical loading and unloading due to erosion or overpressuring, but also by diagenetic processes such as aging, cementation, mineral transformation, and long-term secondary compression or creep. The increase in overconsolidation due to non-mechanical processes is called pseudo-overconsolidation (Bjerrum and Wu, 1967; Burland, 1990). Bjørlykke and Hoeg (1997) postulated that for young and shallow sediments, the pre-consolidation stress is equal to the mechanical pre-consolidation stress, while at great depths, where other diagenetic processes are significant, the apparent pre-consolidation stress may become much higher than the mechanical pre-consolidation stress. It is one of the objectives of this paper to investigate the use of OCR as a measure of the brittleness of mudrocks.

### 4. Experimental undrained shear behaviour of mudrocks

In order to illustrate typical features of the shear behaviour of mudrocks, triaxial test results from samples of Kimmeridge clay taken from two locations in the UK are presented. Kimmeridge clay is an Upper Jurassic sediment found in large areas in the North Sea as source or seal rock in several fields (Doré et al., 1985; Campbell and Ormaesen, 1987). The first batch of materials, hereafter referred to as KWC (Kimmeridge Westbury Clay), consisted of undisturbed block samples of un lithified/uncemented Kimmeridge clay taken from an open pit in the Westbury Quarry in Wiltshire, UK in the Mutabilis zone (Birkelund et al., 1983). This locality is within the Pewsey basin, which has a maximum burial depth of 0.75 km (Penn et al., 1987). At the sampling site, Kimmeridge Westbury Clay had an estimated burial depth of about 0.5 km before being uplifted (Kvilhaug, 1995). The second batch, referred to as KBC (Kimmeridge Bay Clay), consisted of samples of lithified/cemented Kimmeridge ‘shales’ from the Kimmeridge Bay, Isle of Purbeck, Dorset, UK. Samples were obtained from 100 mm diameter coreting from the foreshore in the Eudoxus zone (Cox and Gallois, 1981). Kimmeridge Bay is located within the Portland-South Wight Basin, and had a maximum burial depth of 1.7 km before being uplifted (Penn et al., 1987; Stoneley, 1982; Selley and Stoneley, 1987). Details of the triaxial tests and the mineralogical and physical properties of Kimmeridge clay are given in Nygård and Gutierrez (2002). In addition to KWC and KBC, test results on several intact mudrocks from the North Sea and adjacent areas are also used in the subsequent discussions.

The laboratory investigations on KBC and KWC specimen were performed using consolidated undrained (CU) triaxial tests following the procedures developed by Berre (1981). In CU tests, rock specimens are isotropically consolidated at a certain consolidation pressure and then sheared in a displacement-controlled loading by increasing axial deformation under undrained conditions and while keeping the confining pressure constant. During shearing, changes in pore pressure were measured. For each rock type, three or more specimens are tested at different levels of initial effective vertical consolidation stress which corresponds to typical temperatures at depths of 3–4 km. Sample size is also an important factor to consider as large rock samples are known to have lower shear strength than small samples. However, it has been shown theoretically and experimentally by Gutierrez et al. (1996a,b) that scale effects are significant only for samples tested under low or no confining stresses, while it was shown that scale
effects are insignificant at high confining stresses. Thus, for the high confining pressure test results presented in this paper, scale effects are considered not to affect the test results. Although the triaxial tests were done at a very slow axial straining rate of 0.1%/h, the loading rates that can be achieved in the laboratory are much faster than loading rates in the field which occur in geologic time scales. Procedures are available to correct laboratory data for loading rate effects (e.g. Berre et al., 1996). However, these procedures require knowledge of field loading rates which can vary significantly from field to field. Loading rate effects are neglected in the interpretation of the laboratory results presented below.

The results of the CU triaxial tests are shown in terms of the shear stress $q$ vs. axial strain $\varepsilon_1$, and the effective stress path in the effective mean stress $p'$ vs. shear stress $q$. Six samples of KWC were tested with effective vertical consolidation stresses $\sigma_0^{ve}$ ranging from 1 MPa (sample C1) to 48 MPa (sample C6). Also, six samples of KBC were tested with consolidation stresses ranging from 1 MPa (sample M1) to 58 MPa (sample M6). All samples were consolidated isotropically except for samples C6 and M6 which were consolidated anisotropically following a uniaxial strain consolidation.

The results of the undrained shearing of KWC are shown in Figs. 2–4. The stress–strain curves, shown in terms of shear stress normalized with respect to the effective vertical consolidation stress $q/\sigma_0^{ve}$ vs. axial strain $\varepsilon_1$ (Fig. 2), display an approximately linear elastic initial deformation followed by non-linear deformation before failure is reached. Samples tested at low initial consolidation stress (samples C1–C3) show well-defined peak shear stresses followed by sudden reduction in shear strength and distinct strain softening down to a residual shear stress. On the other hand, samples C5 and C6 which were tested at high initial confining stresses show less defined peak shear and less strain softening in comparison to samples C1–C3. Sample C4 exhibited a transitional brittle–ductile behaviour with continued straining at an almost constant shear stress followed by a slight strain softening to a residual shear stress.

The effect of consolidation stresses can also be seen on the effective stress paths (Fig. 3). For low consolidation stresses, samples C1 and C2 dilated as evidenced by the tendency of the stress paths to lean to the right as the shear stress is increased indicating an increase in mean effective stress and reduction in pore pressure. Samples C3 to C6 contracted as indicated by the stress path tending to the left when the shear stress is increased. After peak stress is reached in the contractive samples, the effective mean stress reduces as the shear stress is reduced. This indicates that the samples continue to contract during strain softening. On the other hand, the brittle samples showed very little change in effective mean stress during strain softening. The dilative response of samples C1 and C2, and the contractive response of samples C3–C6 can be clearly seen in the stress paths normalized with respect to the effective vertical consolidation stress in Fig. 4.

Fig. 5 shows photographs of samples C1 and C5 after testing. In sample C1, which was tested at low consolidation
stresses, a distinct shear plane with an open fracture plane formed (Fig. 5a). Due to the low confining stress, wing cracks, representing local tensile fractures, can be observed at the ends of the shear fractures. In comparison, sample C5 (Fig. 5b) showed only minor signs of developing a shear plane, and instead the sample bulged by shortening in the vertical direction and expanding in the horizontal direction. Sample C5 continued to bulge even after being subjected to axial strains larger than those applied to sample C1. Based on the failure modes of the samples, the stress–strain response and the effective stress path, it can be concluded that samples tested at low effective confining pressure exhibited brittle response, while samples tested at high effective confining pressure exhibited ductile response.

Figs. 6 and 7 show results of the undrained shearing of the lithified KBC mudrock. These results should be compared to those shown in Figs. 2 and 3 for the uncemented KWC samples. The normalized shear stress vs. axial strain plots from the undrained shearing of the lithified mudrock KBC are shown in Fig. 6. As can be seen, the stress–strain response becomes stiffer with decrease in confining stress. Sample M1, which was isotropically consolidated at $\sigma_{vo} = 1$ MPa, experienced a brittle response and a sudden failure and the triaxial equipment was not able to follow the rapid post failure response. With increasing confining stress, the difference between peak and residual shear strength becomes smaller, and the axial strain measured at the peak shear stress increases. Fig. 7 shows the stress paths for KBC. KBC has significantly higher shear strength than KWC, but similarities in the effective stress paths can be observed, particularly in the transition from contractive response at high consolidation stresses to dilative response at low consolidation stresses. As shown in the figures, samples M1–M3 clearly exhibit dilative response, while samples M4 and M6 reveal contractive response. Fig. 8a shows a photograph of sample M1 after shearing, which reveals a distinct shear fracture indicating a brittle response. In comparison, the photograph of sample M5 (Fig. 8b), which was loaded at high confining pressure, shows only uniform shortening and lateral expansion indicating a ductile response with no sign of localized deformation.

Nygård et al. (2004) have performed an extensive study on the compaction behaviour of KWC and KBC. They concluded that the apparent pre-consolidation stresses of KWC and KBC are about 6 and 22 MPa, respectively. The $\sigma_{vo}$ values of 6 MPa for KWC and 22 MPa for KBC correspond to the maximum burial depth of about 0.5 km for KWC and 1.7 km for KBC. It appears that the pre-consolidation stresses for both KWC and KBC are mainly due to mechanical compaction. Based on the $\sigma_{vo}$ value of 6 MPa, it can be concluded that samples C1 and C2 are overconsolidated with OCR of 6 and 1.2, respectively, while samples C3–C6 are normally consolidated as they have been loaded above 6 MPa before shearing. For KBC, samples M1–M3 are overconsolidated with OCR of 22, 4.4 and 1.5, respectively, and samples M4–M6 are...
normally consolidated. Since the OC samples C1, C2, M1, M2 and M3 exhibited brittle response, and the NC samples C3–C5 and M4–M5 exhibited ductile response, it can be postulated that brittle and ductile behaviour for shales directly correlate with OCR as is the case for clays (Ladd and Foott, 1977). Note that judging whether the response is ductile or brittle is based not only on the stress–strain response, but on the effective stress path and the observed failure mode of the sample.

A third set of experimental results is shown in Figs. 9 and 10 to further elucidate the effects of overconsolidation on the undrained response of mudrocks. The results shown are for a shale taken from the Barents Sea. Four samples were tested at slightly anisotropic consolidation stresses ranging from $\sigma'_v = 8$ MPa (sample S1) to $\sigma'_v = 65$ MPa (sample S4). In order to determine the apparent pre-consolidation stress of the tested material, the results of an isotropic consolidation test are shown in Fig. 11 in terms of volumetric strain $\varepsilon_v$ vs. logarithm of effective vertical consolidation stress $\log(\sigma_v'_{\text{max}})$. Although the Barents Sea shale is a lithified material, a distinct change in the slope of the consolidation curves can be detected indicating an apparent pre-consolidation level. Using Casagrande (1936) procedure, an apparent pre-consolidation stress $\sigma_v'_{\text{max}}$ of about 40 MPa is obtained. This apparent pre-consolidation is much higher than the mechanical compaction from burial of Barents Sea shale to 1.37 km. Using this value yields an OCR of 5 and 3.3 for samples S1 and S2, respectively, while samples S3 and S4 are normally consolidated (OCR $= 1.0$) having been consolidated to stresses higher than 40 MPa. The effect of consolidation stress is clearly manifested on the stress–strain and stress path results and the mode of failure of the sample. OC samples show brittle behaviour with stiff stress–strain curves, dilative response and distinct peak stress followed by strain softening. OC samples developed well-defined shear failure surfaces. NC samples exhibit ductile behaviour with contractive response and a less defined peak and strain softening. NC samples do not show distinct failure surfaces after shearing. Similar to KWC and KBC, high OCR correlates

![Fig. 9. Normalized shear stress vs. axial strain plot for Barents Sea Shale at different initial consolidation stresses.](image)

![Fig. 10. Effective stress paths during undrained shearing of Barents Sea Shale.](image)

![Fig. 11. Determination of apparent pre-consolidation stress $\sigma'_v_{\text{max}}$ for Barents Sea shale from consolidation data.](image)
with brittleness and low OCR correlates with ductility. The main difference from KWC and KBC is that the OCR for Barents Sea shale also accounts for pre-consolidation from non-mechanical effects.

5. OCR as a measure of brittleness and shear strength

To examine further the use of OCR as an index of the degree of brittleness and magnitude of shear strength, a database of triaxial test results on 40 types of shales and mudrocks from the North Sea was assembled. Included in this database are results of CU triaxial tests with different consolidation stresses and unconfined compression tests (UCT). For some samples, compressional wave velocity (\(V_p\)) measurements were also performed. As noted above, OCR is widely used in soil mechanics as index of the undrained shear response of clay. However, the use of OCR to characterize the shear behaviour of shales and mudrocks has so far not been widely investigated. Ingram and Urai (1999) suggested using OCR as an index of brittleness, but raised the difficulty of determining or estimating the pre-consolidation stress for lithified materials. As noted above, undrained parameters are directly applicable to the field situation because of the very low permeabilities of mudrocks. It can be noted from Figs. 2, 6 and 9 that the undrained shear strength (corresponding to the peak shear stress) normalized with respect to the consolidation stress generally increases with increase of OCR. The higher normalized undrained shear strength, denoted as \(q_u/\sigma'_v\), with high OCR is attributed to the dilative response of OC materials. Due to dilation, the OC mudrock fails at an effective mean stress higher than the consolidation stress. In comparison, NC materials contract during shearing and fail at effective stresses lower than the consolidation stress.

An important observation that can be made from the experimental data on KWC, KBC and Barents Sea shale is the similarity in the normalized undrained peak shear strengths of NC materials. Such similarity in response points to the possibility of normalizing the undrained response of mudrocks, similar to the SHANSEP procedure used for clay. SHANSEP (stress history and normalized soil engineering properties) is a widely used procedure developed by Ladd and Foott (1977) to predict the undrained shear strength of clays. According to this procedure, the normalized undrained shear strength \(q_u/\sigma'_v\) of NC clays is unique, while the normalized undrained shear strength of OC clays can be represented by the relation

\[
\frac{q_u}{\sigma'_v} = a(OCR)^b
\]  

(9)

where \(b\) is an empirical constant, and \(a = (q_u/\sigma'_v)_{NC}\) is the normalized undrained shear strength of a NC material, i.e. the value of \(q_u/\sigma'_v\) for \(OCR = 1\).

To determine the applicability of the SHANSEP procedure to mudrocks, the normalized undrained shear strength of 40 types of mudrocks and shales are plotted against OCR in Fig. 12. The results show a linear relationship between the logarithms of \(q_u/\sigma'_v\) and OCR, which agrees with the power function given in Eq. (9). The reasonably good correlation between \(q_u/\sigma'_v\) and OCR for 40 types of materials is promising. Thus, it seems that OCR provides a good index of the increase in shear strength and brittleness of mudrocks. The average values of \(a\) and \(b\) for the different materials are equal to 0.39 and 0.89, respectively. The normalized undrained shear strength \(a = (q_u/\sigma'_v)_{NC}\) of 0.39 appears to be a reasonable average value for mudrocks under NC conditions. In comparison, Gutierrez et al. (1996a,b) found values of \(a = 0.29\) and \(b = 0.93\) for several onshore shales. For several clays, Ladd and Foott (1977) obtained values of \(a = 0.20\) and \(b = 0.77\).

The log–log normalized undrained shear strength vs. OCR relation gives a better linear fit for each individual mudrocks type. Examples are shown in Fig. 13 for KBC with \(a = 0.47\) and \(b = 0.66\), and Fig. 14 for Barents Sea shale with \(a = 0.38\) and \(b = 0.59\). It is re-emphasized that the data shown in Fig. 12 and the constants \(a = 0.39\) and \(b = 0.89\) are for mudrocks with different degrees of diagenesis and cementation. Although the normalized undrained shear strength of shale samples with
Fig. 14. Normalized undrained shear strength as function of OCR for Barents Sea Shale.

different degrees of diagenesis are similar for the same OCR, more lithified samples are actually stronger (i.e. have higher undrained shear strength \( q_u \)) than younger uncemented samples because of higher values of apparent pre-consolidation stress. The average relationship given in Fig. 12 should be used only to get a rough estimate of undrained shear strength in the absence of data based on actual field materials.

It is also noted that the data shown in Fig. 12 include test results from both isotropically and anisotropically consolidated samples, and thus the normalized relationship (Eq. (9)) and the definition of OCR based on effective vertical stresses (Eq. (7)) appear to be valid irregardless of the stress condition at consolidation. A specific example of the similarity of test results of isotropically and anisotropically materials is given in Figs. 6 and 13 which show comparatively similar normalized undrained shear strengths for the isotropically consolidated samples M4 and M5 and the anisotropically consolidated sample M6.

For almost all of the mudrocks included in the database, it was not possible to separate the mechanical pre-consolidation stress from the increase in pre-consolidation stress due to other effects such as cementation. The effects of the cementation on the pre-consolidation stress of clays has been shown conclusively by Bjerrum and Wu (1967), who found that cemented clays exhibited apparent maximum past stress significantly higher than maximum past overburden stress. One of the few studies on the effects of chemical diagenesis on the compaction behaviour of shales was reported by McGown and Ladd (1982). They observed an increase in apparent pre-consolidation stress due to diagenesis by precipitation of calcium carbonate (CaCO\(_3\)) in the Nebraska Pierre Shale. They found out that a relatively small amount of CaCO\(_3\), above a threshold of about 2% and below 50% by weight, appears to cement the structure of Nebraska Pierre Shale and increase its apparent pre-consolidation stress.

The normalized behaviour given in Eq. (9) can be used without the need to determine the separate contributions of stress history and chemical diagenesis on the apparent pre-consolidation stress. Two approaches can be used to determine \( \sigma'_{\text{max}} \) experimentally: (1) Casagrande (1936) procedure where the apparent pre-consolidation corresponds to the sharpest bend in consolidation plot as shown in Fig. 11, and (2) Addis (1987) procedure where the apparent pre-consolidation stress corresponds to the point where the \( K_0 \)-value changes in a uniaxial strain compaction test. The overconsolidation ratio can also be estimated by carefully evaluating the effective stress paths and by determining the stress level at which the response changes from brittle-to-ductile behaviour. In the next section, empirical procedures will be developed for an approximate determination of \( \sigma'_{\text{max}} \) in the field.

To provide a more direct measure of brittleness, OCR is related to the brittleness index BRI proposed by Ingram and Urai (1999), which is based on the ratio of the unconfined compressive strengths of mudrocks at OC and NC conditions. Assuming that the same ratio can be obtained from the normalized undrained shear strengths at OC and NC conditions, the BRI can also be written as:

\[
\text{BRI} = \frac{\sigma_{\text{OC}}}{\sigma_{\text{NC}}} = \frac{\left(\frac{q_u}{\sigma'_{\text{OC}}\text{max}}\right)_\text{OC}}{\left(\frac{q_u}{\sigma'_{\text{NC}}\text{max}}\right)_\text{NC}} = \text{OCR}^b
\]

(10)

Ingram and Urai (1999) specified that for BRI \( > 2 \), the degree of embrittlement increases with increasing BRI. This value corresponds to OCR \( = 2.5 \) for an average \( b=0.89 \) from the mudrock database. Thus, OCR \( = 2.5 \) will be used as threshold value above which leakage due to shear fracturing of seals is deemed to be likely.

6. Field application

In order to facilitate the use of OCR to predict brittleness, shear failure and fracturing in the field, procedures will be developed to predict the apparent pre-consolidation stress from simple index tests and field measured data. One parameter that can be easily obtained in the field is the unconfined compressive strength \( \sigma_c \). This parameter can be indirectly determined, for instance, from point load tests or indentation tests on drill cuttings (e.g. Szwedzicki and Donald, 1996). Fig. 15 presents a plot of \( \alpha'_{\text{max}} \) vs. \( \sigma_c \) (both in MPa), which shows good correlation between the two parameters of the following form:

\[
\alpha'_{\text{max}} = 8.6(\sigma_c)^{0.55}
\]

(11)

Although there is limited data, typical \( \sigma_c \) values tend to be in the range of 5–30 MPa for mudrocks (Urai, 1995). It should be reasonable to expect that a correlation exists between \( \alpha'_{\text{max}} \) vs. \( \sigma_c \) since both can be considered as material parameters which depend only on the degree of mechanical compaction and cementation of a material. The validity of Eq. (11) is supported by data on North Sea chalks (Havmøller and Foged, 1996), which shows a comparable correlation between \( \alpha'_{\text{max}} \) vs. \( \sigma_c \) (Fig. 15).

A more convenient procedure is to relate \( \alpha'_{\text{max}} \) with data that can be directly measured in the field such as the
compressional wave velocity $V_p$. One such correlation, shown in Fig. 16 in terms of $s_{\text{cmax}}$ (in MPa) and compressional wave velocity $V_p$ (in m/s), is obtained as:

$$s_{\text{cmax}} = 1 \times 10^{-7} (V_p)^{2.5}$$

(12)

Again, data are limited but are within the typical range of $V_p$ of 2–5 km/s for mudrocks (Urai, 1995). To support the reliability of Eq. (12), an additional relationship is formulated via the following correlation developed by Urai (1995) between $s_c$ (in MPa) and $V_p$ (in m/s):

$$\log s_c = -6.36 + 2.45 \log(0.86V_p - 1172)$$

(13)

Substituting Eq. (13) in Eq. (11) provides the second $s_{\text{cmax}}$ vs. $V_p$ curve shown in Fig. 16, which is in close agreement with Eq. (12). The similarity of the two curves shown in Fig. 16 appears to support the correlation between $s_c$ and $V_p$.

In order to determine OCR, the current state of stress in the field must also be known. The present effective vertical stress $\sigma'_v$ is obtained from total vertical stress $\sigma_v$ and the pore pressure using the effective stress principle. The total vertical stress $\sigma_v$ is based on the integration of the total density (solids plus fluid density of the formation established from well logs) over the depth $z$ of the formation. To estimate the pore pressure $p_p$, Eaton (1975) method is used

$$p_p = \rho_f - \rho_v \left(\frac{R_o}{R_N}\right)^x$$

(14)

where $\rho_f$ is the overburden gradient, $\rho_v$ is the normal hydrostatic gradient for sea water, $R_o$ is the observed resistivity, $R_N$ is the normal compaction trend resistivity, and $x$ is an empirically derived exponent. Similar procedures for estimating pore pressures from electric logs are given in Hottmann and Johnson (1965), and Foster and Whalen (1966). Later these techniques have been extended to include the use of seismic and petrophysical data (Eberhart-Phillips et al., 1989).

The effective horizontal stress $\sigma'_h$ is determined from in situ stress measurements or estimated from empirical stress vs. depth relationships (e.g. Breckels and van Eekelen, 1982). Once the effective stresses are known, the current shear stress $\tau'$ can be calculated from Eq. (5). OCR is then calculated from Eq. (7), and the undrained shear strength $q_r$ is estimated from Eq. (9) using $a = 0.39$ and $b = 0.89$ in case actual rock mechanical data are not available. Shear failure of the seal is likely to have occurred if the current shear stress exceeds the undrained shear strength, i.e. $q > q_r$. Finally, leakage is likely to have occurred if OCR > 2.5.

It should be pointed out again that simple procedure presented above is only valid if the effective vertical stress is the maximum effective principal stress as encountered in horizontally layered rocks without tectonic stresses. Application to more complicated geologic situation would require analysis of the stress situation in the field. The procedure is also valid only for the current stress situation in the field. It may be possible for leakage to have occurred in the past under a different stress field, and this will require knowledge of paleostress magnitudes and orientations.

7. Shear fractures as conduits of fluid flow

As shown by experimental data, shear failure of mudrocks at brittle conditions results in the formation of dilative fracture planes. To investigate how shear induced fractures affect fluid flow in mudrocks under compressional effective stresses, a special experiment was performed. This experiment involved a triaxial specimen of KBC with height equal to the diameter which was loaded in triaxial compression until a shear failure surface was formed. A short sample was used to force a shear
induced fracture, once created, to remain in contact with the filters on both top and bottom loading plates. Thus, fluid can flow through the fracture from top to bottom of the sample. During the shear loading, a constant fluid pressure difference of 8 MPa test was applied from top to bottom of the sample.

The results of the test are shown in Fig. 17 in terms of the shear stress vs. axial strain and the flow rate measured concurrently during testing. Because of the low confinement, the specimen exhibited a distinct peak shear stress followed by strain softening. Just before the maximum shear stress was reached, a marked increase in flow rate was observed which coincided with the formation of a fracture plane. This result illustrates that a shear fracture created under effective compressive stresses in a mudrock can provide sufficient permeability for leakage to occur even when a compressive effective normal stress acts on the failure plane and the basin is under a compressional stress regime.

8. Conclusions

Triaxial tests have been performed to study the shear behaviour, brittle-to-ductile transition and the hydraulic conductivity of shear induced fractures in shales and mudrocks. Based on a study of the undrained shear behaviour of 40 rock samples from the North Sea and adjacent area, it was shown that the overconsolidation ratio OCR can be used as a measure of the undrained shear strength and brittleness of mudrocks and shales, and for determining the likelihood of leakage through hydrocarbon seals. Overconsolidation in mudrocks and shales is caused by a combination of mechanical compaction and other non-mechanical factors leading to an apparent pre-consolidation. Mudrocks which reach effective confining stresses above the apparent pre-consolidation stress are normally consolidated (NC), while those at effective confining stresses below apparent pre-consolidation are overconsolidated (OC). NC mudrocks or those with low OCR show ductile behaviour. OC mudrocks with high OCR show brittle behaviour characterized by stiff response with well-defined peak shear stress and distinct strain softening. Shear failure of brittle mudrocks results in the formation of distinct shear fractures. A special triaxial test showed that shear fractures can increase the permeability of mudrocks by providing a path for channelized flow, and results indicate that shear fractures in brittle materials can continue to dilate during post-peak shearing.

The normalized undrained shear strength \( q_u/\sigma_u^{0} \) was also found to correlate well with OCR following the SHANSEP normalization procedure used for clays. An average undrained shear strength criterion of \( (q_u/\sigma_u^{0}) = 0.39(OCR)^{0.89} \) is proposed based on data from different North Sea mudrocks and shales. Correlations of the apparent pre-consolidation stress with unconfined compressive strength \( \sigma_c \) and compressional wave velocity \( V_p \) are also proposed. In the absence of field specific data, the undrained shear strength criterion and the pre-consolidation stress correlations, together with in situ stress and pore pressure data, can be used for preliminary evaluations of cap rock integrity in horizontally layered rocks. Evidently, more experimental and field data are needed to improve the reliability of the proposed procedure.

References


