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An experimental investigation of the creep behavior of an underground coalmine roof with shale formation

Priyesh Verma
West Virginia University

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An experimental investigation of the creep behavior of an underground coalmine roof with shale formation

Priyesh Verma

Thesis submitted to the
Benjamin M. Statler College of Engineering and Mineral Resources
at West Virginia University
in partial fulfillment of the requirements for the degree of

Master of Science
in
Mining Engineering

Brijes Mishra, Ph.D., Chair
Yi Luo, Ph.D.
Keith A. Heasley, Ph.D.

Department of Mining Engineering

Morgantown, West Virginia
2013

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ABSTRACT

An experimental investigation of the creep behavior of an underground coalmine roof with shale formation

Priyesh Verma

Stability of the roof in an underground coalmine is imperative for safe and efficient mining. Mining of the coal seam disturbs the natural equilibrium of the in-situ stresses in the surrounding rock. Because of the opening, the surrounding stratum deforms to fill the opening and return to the pre-mining stress state. Such deformation, if allowed to continue, will lead to failure of the strata, which creates an unsafe work environment. Therefore, supports are used to reinforce the deforming strata and help prevent any further deformation in the rock.

However, various mines have reported failures in the supported entries (MSHA, 2013). These failures have been mostly called as "cutter-failure" (Peng, 2008) and are attributed to high horizontal stresses; however, surprisingly, another factor which is suspected to have an effect on such failure is the time-dependent deformation. Its effect on the roof fall activity is unknown and to the authors knowledge limited research has been reported on this subject. Additionally, organizations such as the ISRM (International Society of Rock Mechanics) and the ASTM (American Society for Testing and Materials) have not recommended any creep testing procedure. Therefore, the objective of this research was to investigate the time-dependent deformational behavior of immediate coal measures rock (shale) and then use phenomenological equations to fit the experimental data and produce input properties for numerical modeling.

Creep, or time-dependent, experiments were performed on shale and sandstone specimens under constant load and stress conditions. Both rock types showed development of creep strain; however, the shale specimens were more sensitive to the change in the stress conditions. Under uniaxial stress conditions, the specimens showed an increase in the creep strain as the stress increased. In triaxial conditions, the creep strain increased with an increase in the deviatoric stress conditions. It is believed that this research will provide some preliminary information on the time-dependent behavior of shale.

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Chapter 1

Introduction

1.1 Backdrop

The Appalachian states have the maximum number of underground coal mines which are present in multiple seams and the combined production of these mines comprises over 32% of the coal produced in the United States (EIA, 2013). Due to the presence of these large numbers of underground mines, strata control is one of the primary research areas.

Creation of an underground coal mine results in dramatic changes in the rock behavior. From measurements and field observations it is known that the earth is stressed and when an opening is created, it disturbs the natural stress equilibrium of the earth, producing induced stresses around the opening. The induced stresses strive to reach the equilibrium conditions by stressing the rock and inducing deformation. If no supports are provided, the opening might close immediately or over certain period of time. When supports are provided, the stress redistribution is restrained; however, the rock in the vicinity of the opening is expected to be under a constant state of stress. This constant stress condition acting on the rock for a certain time period also induces permanent deformation although in small amount, is known as creep or time-dependent deformation. Weak rocks such as shale have been shown to exhibit time-dependent deformation. Most of the underground coal mines in Appalachian coalfields have laminated shale as the immediate roof above coal seam and they are particularly known to have large number of roof fall incidents (MSHA, 2013).

Surprisingly, research on creep behavior of roof rocks has been limited to a few tests that were performed by the National Coal Board of the United Kingdom (Price, 1964). Most of the tests performed were uniaxial; and therefore, may not represent the true stress state that is usually observed in the immediate roof. In the United States, extensive creep tests were performed on rock salts and recently on Barnett shale (Sone and Zoback, 2010). However, similar test information on the roof rock of underground coal mines is not available. As the production life

of underground coal mines are often more than five to ten years, it is imperative that the time-dependent behavior of immediate roof is investigated for its effect and role on roof stability.

1.2 Objectives

The objective of this thesis is to study the time-dependent behavior of the immediate coal measures rock with shale formations. Organizations such as the ISRM and the ASTM do not have any recommended procedure to perform creep experiments, especially under triaxial conditions. So, this study is not limited to the investigation of creep of shale alone but also aims at the development a suitable testing procedure to perform creep experiments under both confined and unconfined conditions.

The following are the broad objectives of this thesis:

- a) Development of the uniaxial creep tests – single and multistage load conditions and validation of the test process by performing shake-down tests on Berea sandstone.
- b) Investigation of the time-dependent behavior of shale under single and multistage unconfined load conditions.
- c) Investigation of the time-dependent behavior of shale under multistage confined stress conditions.
- d) Finally, fitting of experimental results with a suitable creep law.

1.3 Thesis organization

This thesis is organized into five chapters. Chapter 2 will provide literatures that have advanced the knowledge on creep behavior for various kinds of rocks. Various testing methodologies and equipment used over the years in both unconfined and confined conditions, is also summarized. In chapter 3, a comprehensive testing procedure developed to perform creep experiments is explained. Chapter 4 will provide the results from the creep experiments followed by the discussion and finally, chapter 5 will summarize the conclusions from this study.

Chapter 2

Literature review

This chapter introduces the time-strain concept and provides summarized review of various research studies that have advanced the understanding of creep behavior in rock. Different testing methodologies and creep equipment which have evolved over the years in unconfined and confined conditions are also summarized. Finally, in-situ field measurements for different rock and salt types are also discussed.

2.1 The time – strain or creep curve

The time-strain, or creep, curves exhibited by different rock types subjected to constant load are basically similar in shape. The theoretical rock creep phenomena under constant load can be divided into three stages as shown in figure 2.1.

Figure 2.1 Theoretical time-strain behavior of rock under sustained load (Lama and Vutukuri, 1978).

It consists of the following stages:

1. Instantaneous elastic strain (AB)
2. Primary creep stage (BC)
3. Secondary creep stage (CD)
4. Tertiary creep stage (DE)

Immediately after the application of load, a rock specimen exhibits an instantaneous elastic strain (AB). Point B represents the start of constant load conditions. Primary creep (BC), which is the first stage, starts with the onset of the constant load conditions. It is also known as the transient creep stage. At this stage the creep rate decreases with time and it is sometimes called as delayed elastic deformation. Point C marks the end of the primary creep stage. At this point, if the load is suddenly brought to zero, there will be first an instantaneous elastic recovery (CF) followed by a 'time-elastic' recovery (FG). Thus, there is no permanent deformation in the rock specimen. The material remains elastic and this behavior is referred as time-dependent elasticity (Jaeger and Cook, 1979). However, if the load is not removed and is maintained constant beyond point C, the specimen starts to exhibit secondary creep, which is also known as steady state creep. Here the rate of strain is nearly constant. The duration of this creep phase is comparatively larger than the other creep phases. If the specimen is unloaded at any time, an asymptotic curve following a permanent deformation is commonly observed. If the load is maintained till point D, the tertiary creep stage starts. Here, the strain rate increases with an increase in time until the specimen eventually fails. This stage is sometimes called the accelerated creep stage. This phase is small and is fundamentally different from other two creep phases (Lama and Vutukuri, 1978).

The general equation for creep is

$$\varepsilon = \varepsilon_e + \varepsilon(t) + At + \varepsilon_t(t) \quad \text{Equation (2.1)}$$

Where:

- ε is the total strain in the rock specimen,
- ε_e represents the instantaneous elastic strain,
- $\varepsilon(t)$ represents primary or transient creep,
- At represents secondary or steady state creep,
- $\varepsilon_t(t)$ represents tertiary or accelerated creep,
- A is constant and depends on test conditions, t represents time.

The stages mentioned above have been observed in various rock types however; the mechanism behind these stages has not been fully understood (Lama and Vutukuri, 1978). Vast amounts of literature are available on creep of rock and rock salts, and it is impossible to cover and mention all the literatures. Therefore, the literature survey was restricted to the research work that was performed in coal mines, methods adopted during creep testing, and in-situ creep tests.

2.2 Review of literature

2.2.1 Unconfined creep testing of rock and coal

Norton (1929) was perhaps among the first few known researchers who investigated the creep resistance of steel at constant load and high temperature conditions. He was able to fit an exponential equation to the data between the observed flow rate and the applied stresses. This equation was commonly known as Norton's Power law and is frequently used to predict and model creep behavior. The highlight of his research on creep was the dependence of creep rate on stress and temperature. His research showed that a reduction in stress on the specimen at high temperature does not slow the creep rate. The creep rate was also affected by the temperature. Norton performed his experiments on metal using the Babcock and Wilcox creep testing machine (Figure 2.2) which was especially designed to provide a high range of temperatures for long period of time.

Figure 2.2 Babcock and Wilcox creep testing equipment (Norton, 1929).

Phillips (1932) was among the few known researchers to observe creep strains in a number of coal-measure rocks subjected to bending. He found that creep strain for an equal interval of time first increases and then decreases with each successive increment of load. He termed the load at which the time effect was maximum as the “*critical load*”. He also found that the time-dependent strain increases, when a beam of rock specimen previously loaded by bending in one direction was loaded in another direction for the same amount of time and load. However, the results obtained during his experiments were maybe because of non-homogeneous stresses in bending and hence, debatable.

Griggs (1939) performed some time dependent experiments to gain knowledge about the creep behavior of rocks. His work is considered as the starting point for the systematic investigation of creep behavior of rocks. He defined ‘*creep*’ as the slow deformation of solids under small loads acting over long period of time. He was able to measure creep in compression on different types of rocks for up to five hundred fifty days and observed measurable flow at stresses below the elastic limit. He proposed an empirical equation which had an “*elastic flow*” component and a “*pseudoviscous flow*” component.

$$S = A + B \log t + Ct$$

Equation (2.2)

Where:

S is the total deformation,

A is the elastic strain,

B log t is the primary creep (elastic Flow Component),

Ct is the secondary creep (pseudoviscous component),

B and C are the constants depending on applied stress,

T represents time.

He found that the rate of elastic flow varies inversely with time whereas the pseudo-viscous flow is constant under constant stress difference.

Pomeroy (1956) performed some creep tests in compression at room temperature on bituminous and anthracite coal. In the case of bituminous coal, he was able to fit his experimental data with Griggs equation (Equation 2.2). Figure 2.3 shows a typical graph showing the relationship between the deflection of a cantilever of bituminous coal and the duration of loading. However, he did not observe any creep behavior for anthracite coal. He inferred that this possibly was one of the reasons for coal outbursts in anthracite areas.

Figure 2.3 Deflection of bituminous coal with time (Pomeroy, 1956).

Price (1964) studied time-strain behavior of a number of coal measure rocks. He conducted creep experiments on Pennant sandstone, Calcareous sandstone, Wolstanton sandstone, Siltstone and nodular, muddy limestone. He used three types of loading devices, one for bending and two for compression. Figure 2.4 and 2.5 are schematics of test rigs used for compression. Figure 2.6 is a sketch of bending beam apparatus used. From the bending test results, he found that there exists a linear relationship between load and rate of 'secondary creep' for pennant and Wolstanton sandstone. He fitted Bingham rheological model to the experimental data. From the compression test, he found that when the rock specimen was subjected to incremental stress levels, the rate of secondary creep was inversely proportional to stress. He also conducted stress relaxation tests by loading, unloading and then reloading of the same specimen. He found that some specimen showed an overall expansion. He explained this time-dependent behavior in terms of the release of pockets of 'residual strain energy' from the specimen during the test.

Figure 2.4 Schematic representation of lever-system of large rig (Price, 1964).

Figure 2.5 Schematic representation of lever-system of small creep rig (Price, 1964).

Figure 2.6 Sketch of bending beam apparatus (Price, 1964).

Misra and Murrell (1965) performed creep experiments on different rock types up to a temperature of 750°C . For performing these creep experiments, they developed a test rig which can provide a constant load and temperature for longer durations. Figure 2.7 shows a schematic diagram of the compression creep testing machine used in the experiment. They found that the rate of creep increases with an increase in the temperature and stress for different rock specimens. They found that at a temperature below $0.2T_m$ (where T_m is the absolute temperature of melting), creep strain depends logarithmically on time and is proportional to stress and temperature. At a temperature of about $0.5 T_m$, creep strain was proportional to a fractional power of time and exponentially with stress.

Figure 2.7 Schematic diagram of compression creep testing machine (Misra and Murrell, 1965).

Singh (1975) observed creep behavior of Sicilian marble and other rock types. He loaded the specimens beyond their yield point and determined that the creep in the lateral direction was far greater than the creep in the axial direction. He successfully fitted a power curve to most of his creep data. He further determined that the steady state creep rate increases with an increase in the stress. Similar trends were observed when he used an incremental stress method.

Figure 2.8 Axial and lateral creep curves of Sicilian marble at a constant pressure (Singh, 1975).

Tandanand (1985) analyzed the moisture adsorption rate and strength degradation of Illinois shale. He applied continuum damage mechanics to determine the effect of moisture on strength of shale. He found out that the tangent modulus and compressive strength of shale are inversely proportional to moisture content, and when the moisture content in Illinois shale exceeds 8% of its dry weight, it loses its strength.

Park and Jung (1988) used the technique of holographic interferometry to study the creep behavior of rock and coal. They performed creep tests on shale, sandstone and coal samples collected from Alabama coal mines. They were able to plot all three stages of creep. They found that the creep behavior of rock and coal varies according to its physical condition.

Yang and Daemen (1997) conducted creep tests on tuff under compression at room temperature and at elevated temperature (204°C). From the experimental analysis and theoretical prediction, they concluded that creep strain increases with increasing temperature under constant loading conditions. They concluded that temperature also influences the creep behavior of rocks.

2.2.2 Confined creep testing of rock

Griggs (1936) performed a series of experiments at room temperature to examine closely the process of rock deformation under high confining stresses. The apparatus used to conduct his experiments established the standards for the new generation of triaxial test systems available in modern times. He performed experiments on cylindrical Solenhofen limestone specimens and found that with an increase in confining pressure, the rock failure changes from brittle to plastic. He further concluded that the rock would be more ductile when the differential pressure is

increased rapidly. Figure 2.9 shows the strain-time relationship when the differential stress was held constant. He found that the curves were exactly similar to what was observed in the “creep” of metal at high temperature.

Figure 2.9 Strain against time relationship at each point where the stress difference was held constant (Griggs, 1936).

Robertson (1960) studied the transient creep behavior of Solenhofen limestone specimens in compression at confining pressure up to 4000 bars. He observed a decrease in transient creep rate with an increase in confining pressure. From his creep experiments, he observed a shortening of the limestone specimen up to 48 percent from the original length. He also found that fracturing was one of the principal mechanisms of creep in limestone.

Rutter (1972) performed creep experiments on wet Solenhofen limestone at room temperature with 600 bars confining pressure under constant stress and constant force conditions. These two

conditions represent two scenarios (1) constant load on the specimen (2) constant stress in the specimen. In the 1st scenario the load on the specimen remains constant; however the stress changes due to the deformation. In the 2nd scenario the stress in the specimen is kept constant by adjusting the load with the change in the area of the specimen. The observed creep curve was found to be sensitive to stress changes due to a change in cross-sectional area of specimen. He also found that creep rate is extremely sensitive to small changes in applied differential stress.

Baud and Meredith (1997) conducted time dependent tests on water saturated samples of Darley Dale sandstone under triaxial conditions. Creep strain, acoustic emission and pore volume changes were recorded during each test. They found that creep rate and time-to-failure increases with an increase in the level of applied stress. They also found that acoustic emission and pore volumetry provide an overall damage measurements of sample. They concluded that the onset of tertiary creep starts when a certain damage level in the specimen is achieved.

Okubo et al. (2008) developed a transparent triaxial cell and performed creep tests on Tuff and shale specimens. Figure 2.10 shows the details of the transparent triaxial cell which was developed. With this cell, they observed changes in the specimen under confining pressure. Photographs were taken and processed to determine the amount of lateral deformation during the test from the primary to the tertiary creep stage. They found that the lateral strain rate was inversely proportional to the time remaining in the tertiary creep stage and hence, failure time could be estimated.

Figure 2.10 Details of transparent triaxial cell (Okubo et al., 2008).

Zhang et al. (2010) carried out temperature controlled triaxial compression and creep test on Ohya stone, a typical soft sedimentary rock, and proposed a thermo-mechanical constitutive model for soft rocks. From the experimental and theoretical calculated results, it was found that

creep failure time is largely dependent on temperature and creep failure will occur sooner at higher temperature.

2.2.3 Creep of rock in situ

Reynolds and Gloyna (1961) measured creep deformation at Grand Saline mine, Texas, and Hutchinson mine, Kansas, and derived an empirical equation expressing the creep rate as a function of time.

$$\dot{\epsilon} = Ae^{-kt} \quad \text{Equation (2.3)}$$

Where:

$\dot{\epsilon}$ is the creep rate,
 ϵ is the change in unit deformation,
 dt is the change in time,
 A and k are the constants and
 t represents time.

From their measurements, they found that the creep rate decreases as the tunnel age increases. They found that the vertical creep rate at the center of the opening was fourteen percent greater than at the wall. They concluded that the vertical creep rate at the center of the opening was approximately four times that of horizontal creep rate.

Barron and Toes (1963) performed creep measurements around the unlined portion of the shaft in the salt above the potash beds to obtain creep data which could be used to correlate theoretical ideas on understanding material behavior with field measurements. Measurements were made of displacements, relative to shaft axis, of points on the surface of the shaft and within the solid surrounding the shaft. Extensometers were used to measure the longitudinal deformation of boreholes around the shaft. From the measurements, they found that the radial displacement, U_r , for a point in the solid at a radius r from the shaft axis may be expressed in the form

$$U_r = -pB \left(\frac{a^2}{r} \right) \log_{10}(1 + bt) \quad \text{Equation (2.4)}$$

Where:

B and b are the constants,
a is shaft radius,
t represents time and
p is the pressure.

Experimental results agree with this $1/r$ dependency; however, results from surface points do not agree with this relation (Equation 2.4) which may be due to a change in material properties that has occurred between the surface and at four feet of depth. They further concluded that this type of measurement may be used to determine the shear creep function of the material in situ, which is important in correlating laboratory and field studies.

Bradshaw et. al (1964) developed an empirical creep equation (equation 2.5) by analyzing the laboratory pillar model tests results on Lyon mine salt specimen for 1000 hours or less duration at several different values of average pillar stress.

$$\dot{\epsilon} = B\sigma^m t^n \quad \text{Equation (2.5)}$$

Where:

$\dot{\epsilon}$ is the convergence rate (vertical convergence in micro inches per inch per day),
B is the constant which depends on unit of strain rate,
 σ is the average vertical stress (psi),
m is slope of $\dot{\epsilon}$ vs. σ on a log – log plot (positive),
t represents time in hours and
n represents the slope of $\dot{\epsilon}$ vs. t on a log – log plot (negative).

The developed empirical equation was able to predict vertical closure rates in salt mine openings up to 70 years old. From the results obtained from vertical and horizontal convergence measurements in the Hutchinson and Lyon mines, they found that vertical convergence rates have continued to decrease with time. They also determined that the horizontal closure rates and the transverse pillar expansion rates appear to be lower than the vertical closure rates for the first few years after an opening was created.

Headley (1967) measured the convergence rate at five salt mines in England, Canada and the United States. From analyzing his field measurements, he found that the power curve relationship (equation 2.6) between convergence rate and calculated pillar stress would provide the best fit for his data.

$$\dot{\epsilon} = A\sigma^n \quad . \quad \text{Equation (2.6)}$$

Where:

$\dot{\epsilon}$ is the convergence rate,
 σ is the stress and
A and n are the constants.

He also suggested that failure based on a limiting vertical deformation would be more realistic approach to evaluate pillar stability in salt.

Overall, the literature review presented in the previous sections discusses the articles that pioneered laboratory creep testing and in situ creep analysis in rocks. A wide variety of creep testing methods and machines have been used for investigating the creep behavior of several kinds of rocks. The variability of testing methods and machines, and the absence of a standardized test procedure posed a unique question - which method is best suitable for creep testing of shale that constitutes the immediate roof of underground coalmines? In addition to the above mentioned problem, the tests mentioned in previous literatures were performed at different stress conditions and at different temperatures. The fact that none of these tests represented roof-type conditions also poses doubts on using coefficient values derived from these tests. Therefore, to find answers to the problems mentioned above and to gain knowledge on creep of rock, it was imperative that laboratory tests be performed on the shale specimens.

Chapter 3

Design of experiments

This chapter discusses the developments of comprehensive testing procedure for determining the time-dependent properties of coal measures rocks. Compression testing machines used under unconfined and confined conditions are explained in detail. Strain measuring devices and the calibration used to measure creep strains in prepared samples are also briefly summarized.

3.1 Introduction

When an entry is driven into the coal seam, the pre-existing *in-situ* stress is disturbed and induced stress surrounds the entry (Hoek and Brown, 2002). The induced stress deforms the rock and if the deformation is allowed to continue then the entry may experience roof fall. However, in practice, and by regulation (MSHA, 2012), the entries have to be provided with artificial supports. Even with installation of the supports, the immediate roof exposed in the entry is under the influence of the induced and the in-situ stresses (Lu and Hebbelwhite, 1998). The long term effects of these stresses on the rock and the roof fall is not comprehensively known and therefore needs to be investigated. In addition, the effect of stress change on the roof rock due to the advancement of the entry and the opening of adjacent entry is also not available. Therefore, this thesis aims to address this gap by performing and analyzing detailed laboratory creep tests on shale rocks. The laboratory tests were delegated into two broad categories:

1. The unconfined test, which will include single and multistage tests.
2. The confined test, which will include three confined stress conditions with multistage test procedure.

In the unconfined test scheme, a number of rock specimens were tested under different constant load conditions. For shakedown tests, a total of ten Berea sandstone and for immediate roof rocks, eleven shale specimens were tested. The specimens, in single and multistage stages, were subjected to constant loads which were 65, 70, 75, 80 and 85% of their peak strength and

for a period ranging from 2 to 8 days. The experiments were performed in a servo-controlled stiff compression testing machine.

In the confined test category, the rock specimens were subjected to a constant triaxial state of stress for a time period of 24 hours. For maintaining the constant triaxial state of stress, the stress difference between the axial and the confining stress was maintained constant. During the test, axial, radial, volumetric strain and shear strain were recorded. The confined creep tests were performed on three shale samples at different confining stresses. All the confined experiments were performed using GCTS Triaxial testing system.

All experiments were performed under controlled temperature and humidity conditions. Also, the strain measuring setup was sensitive enough to capture small strain developed in the specimen during the experiments.

3.2 Preparation of specimen

Most of the rocks tested under the above test scheme was supplied in the form of shale cores by the Imperial mine, located in Upshur County, West Virginia. Shale cores from various depths were supplied, but a depth range of 376.8' - 386.8' was chosen for the test scheme because it represented the immediate roof of the coal seam. Also, initially specimen from Berea sandstone block was also prepared to perform some initial shakedown creep tests.

All specimens were prepared in the rock mechanics laboratory in accordance with American Society of Testing Materials (ASTM) and/or the International Society of Rock Mechanics (ISRM) test standards. Each of the specimens was assigned a unique identification number for tracking within the WVU laboratory.

Three steps were used to develop the test samples. The methods include coring, cutting and grinding. Initially cores were drilled from the intact rock blocks using the diamond core drill with water as a coolant. Cored cylinders were cut into an approximate length (maintaining standard length/diameter ratio ≥ 2.0 and ≤ 2.5) by a rotary saw, utilizing a diamond-impregnated blade with water-cooling. After air-drying for two to seven days, the ends of these cylinders were ground flat and were made parallel to within $\pm 0.001''$ with the help of an automatic grinder. The final length-to-diameter ratio was between 2.0 – 2.3 for all the prepared specimen. No attempt

was made to prepare the sides of the specimens obtained from the mine. The sides of the specimens of Berea sandstone were smooth and straight as they were cored in laboratory under ideal conditions.

Dimensions of each of the prepared specimen were measured using a digital scale. The diameter was measured at the top, in the middle and at the bottom of the specimen and an average value was recorded for each specimen.

Figure 3.1(a, b, c) illustrates the steps involved in the specimen preparation process of Berea Sandstone. All shale cores were first wrapped with a scotch tape before cutting and grinding process. This was necessary because shale is extremely weak and water sensitive, and can break easily along the bedding or parallel layering when it comes in contact with water, which is used during cutting and grinding operation. The steps involved in for preparation process for the shale cores are shown in figure 3.2 (a, b).

Figure 3.1 Specimen preparation stage: (a) original block, (b) drilled holes, (c) prepared specimens for Berea Sandstone.

Figure 3.2 Specimen preparation stage: (a) Rock cores (b) Finished specimen after cutting and grinding for Imperial Shale.

3.3 Unconfined creep tests

The following section contains a description of the compression testing machine used, strain measuring arrangement, specimen testing procedure and analysis of data.

3.3.1 Compression testing machine

The unconfined creep tests were performed in a servo-hydraulic, Material Test System (MTS 440) in the Rock Mechanics Laboratory of the department of Mining Engineering, WVU, shown in the figure 3.3.

Figure 3.3 MTS servo controlled compression testing machine and its components - (1) Machine Load (2) Glass Shield (3) Hydraulic Actuator (4) Manual Control System (5) Strain Gauge Control Panel (6) Computer (7) MTS Data Acquisition System (8) Upper steel platen (9) Specimen (10) Lower steel platen.

The MTS machine load frame was designed to counter the forces applied to the test specimens during compression and fatigue testing. The load frame consists of four vertical columns that join a movable crosshead and a fixed platen. The crosshead is vertically adjustable to accommodate specimens of various lengths. The crosshead, once in position, locks into place to prevent slippage or backlash (MTS reference manual, 1990).

Figure 3.4 Load frame dimensional drawing (MTS reference manual, 1990).

The hydraulic actuator system installed in the MTS 440 is a double-acting, double-ended heavy duty actuator that operates under precision servo valve control in a closed-loop system. The actuator is used to generate a precise force on the specimen or to accurately control the piston rod displacement. The actuator piston rod movement is accomplished by supplying high-pressure fluid to one side of the actuator piston and opening the other side to a return line. The differential pressure across the piston forces the piston rod to move.

The amount of the hydraulic fluid, the speed and the direction of piston rod movement is controlled by a servo valve (figure 3.5). If hydraulic pressure is applied to port A and port B is opened to the return line, the piston rod extends from the actuator. If the hydraulic pressure is applied to the port B and port A is opened to the return line, the piston rod retracts. The force applied to a specimen attached between the actuator piston rod end and a reaction mass (load frame crosshead) is the product of the applied differential hydraulic pressure and the effective piston area. The internally mounted linear variable differential transformer (LVDT) provides a displacement indication of the actuator piston rod. The LVDT is an electromechanical device that provides an output voltage proportional to the displacement of a linearly moveable core

extension. The core extension is axially oriented in the LVDT coil. The LVDT coil is connected to the LVDT mounted in the LVDT assembly (Figure 3.5) (MTS reference manual, 1990).

Figure 3.5 Cutaway view of hydraulic actuator system (MTS reference manual, 1990).

The machine's LVDT on the vertical stroke of the hydraulic ram is used to measure the axial deformation of the specimen. Because of the actuator, the LVDT measures the total deformation of the specimen as well as the deformation of platens and loading crossbar frame (Figure 3.6). Following ASTM Standards-D7012, a suitable calibration for machine deformation was applied to the test data to get the true specimen axial strain values. The calibration was performed in the laboratory using a cylindrical steel specimen with documented elastic properties.

MTS 440 was housed in a climate control laboratory where temperature and humidity was kept constant. This was done to eliminate any effect of temperature and moisture on the specimen and the machine.

Figure 3.6 Load distribution of the specimen and MTS machine during creep test (Nie, 2011).

3.3.2 Testing procedure

A) Determination of Uniaxial compressive strength

Uniaxial compressive strength (UCS) tests were performed on sandstone and shale specimens. All the UCS tests were performed using the ASTM D7012 test procedures. The strength tests were performed on a servo-hydraulic, MTS test system (MTS 440) shown in the figure 3.4. The uniaxial compressive strength (σ_u) of the test specimen was calculated as follows:

$$\sigma_u = P/A \quad \text{Equation (3.1)}$$

Where:

- σ_u is the uniaxial compressive strength (UCS),
- P is the failure load and
- A is the cross-sectional area of the specimen.

B) Unconfined creep test

Prior to perform any creep tests, specimens were stored in the laboratory under a controlled condition of temperature ($70 \pm 2^\circ\text{F}$) and humidity ($55 \pm 2\%$) for a minimum period of seven days. Two methods were used to perform creep tests in compression.

1. Single-stage unconfined creep test

Specimens are loaded to a certain pre-determined percentage of their peak load value and maintained constant for a given period of time. The LVDT installed within the machine was used to measure the creep strains in the specimen.

2. Multi-stage unconfined creep test

Specimens are loaded stepwise, in an incremental way, starting with a relatively low percentage of peak load value and then increasing the load following a set of predefined time intervals. With this method, more information can be obtained about the creep behavior of a particular rock type with a single specimen.

A series of six single-stage unconfined creep tests were conducted on Berea sandstone specimens. A specimen was loaded up to 65% of its peak load value, thereafter maintaining the load constant for 12 hours. Additionally, other tests were also performed on sandstone specimens at 70, 75 and 80% of their peak load value for the same period of 12 hours. Before starting the creep tests, the scotch tape was removed from the shale specimens. Single-stage creep tests were performed by loading four different shale specimens at 60, 70, 75 and 85% of their peak load value, thereafter maintaining the constant load for a period of 48 hours. For multistage unconfined creep tests, three shale specimen were loaded in an incremental way at a load level of 65, 70, 75, 80, 85, and 90% of its peak load value, where increment of load was done every 24 to 48 hours.

3.3.3 Analysis of data

The unprocessed data from the MTS machine gives time (sec), axial load (pound) and axial displacement (inches). Axial stress was calculated from the axial load and the cross sectional area of the specimen. A suitable calibration for machine deformation was applied on the test data to obtain the true axial strain value. The observed strain during constant loading is assumed to be here as creep strain. The total strain developed in any rock specimen during the creep tests can be written as a sum of instantaneous elastic strain and creep strain (see Equation 3.2)

$$\varepsilon = \varepsilon^{(i)} + \varepsilon^{(c)} \quad \text{Equation 3.2}$$

Where:

ϵ is the total strain developed in the rock specimen,

$\epsilon^{(i)}$ is the instantaneous elastic strain and

$\epsilon^{(c)}$ is the creep strain.

For single-stage creep test, creep strain was calculated by subtracting the instantaneous elastic strain ($\epsilon^{(i)}$) from the total observed strain (ϵ).

In the case of multistage creep testing, increment of strain at the beginning of the second stage was calculated as $\epsilon_2^{(i)} - \epsilon_1^{(i)}$ (see figure 3.7). Here, $\epsilon_2^{(i)}$ is the instantaneous elastic strain in the second stage and $\epsilon_1^{(i)}$ is the instantaneous elastic strain due to first stage. This increment in elastic strain (caused due to increment of load after each stage) was calculated for subsequent stages and subtracted from the overall observed axial strain to obtain creep strains for each stage.

Figure 3.7 Creep curve during multi-stage creep test (Hult, 1966).

3.4 Confined creep test

This section contains a description of the triaxial testing machine, the strain measuring arrangements, the specimen testing procedure and the analysis of data.

3.4.1 Triaxial testing machine

Confined creep tests were performed on a servo-hydraulic, GCTS triaxial test system (GCTS RTX – 1500) (see figure 3.8) in the rock mechanics laboratory of the mining engineering department at WVU.

Figure 3.8 GCTS triaxial test system with various components. (1) Hydraulic pump (2) Cell pressure intensifier (3) Pore pressure intensifier (4) Hydraulic supply hoses (5) Triaxial cell (6) Test frame (7) Computer (8) Temperature indicator (9) Digital controller.

The GCTS triaxial rock testing system (RTX-1500) has the following characteristics (GCTS technical reference manual, 2010):

- Direct closed-loop digital servo control of axial stress, average principle stress, axial strain, radial strain, and several other calculated triaxial variables.
- Standard 1,500 kN Load capacity and 1,750 kN/mm stiffness.
- GCTS High Pressure Triaxial cell with internal instrumentation to measure local axial & radial strains.
- 140 MPa servo-controlled pressure intensifier system for cell and pore pressure.
- Available options: axial & circumferential deformation measurement system, platens with ultrasonic transducers, and high temperature control subsystem.
- Ideal for performing unconfined compression, triaxial, bending, indirect tension, fracture, creep, post failure behavior, & other compression tests.
- Optional automatic hydraulic lift for triaxial cell and roller assembly for fast & easy specimen setup.
- The RTX-1500 meets the specifications of the International Society for Rock Mechanics (ISRM) for triaxial testing of rock samples.

The GCTS triaxial rock testing system (RTX-1500) has the following axial and circumferential measurement device for high pressure triaxial cell (GCTS technical reference manual, 2013):

- Set of upper and lower LVDT holder rings for two deformation sensors positioned at 180 degrees to measure average axial strain. Accommodates specimens with a diameter from 35 mm to 54 mm (see figure 3.9 (a)).
- Circumferential roller assembly and LVDT holder to measure the circumferential deformation in the specimen. Accommodates specimens with initial diameter from 35 to 54 mm (1.3 to 2.12 inch) (see figure 3.9 (b)).
- LVDT's with ± 2.5 mm (0.09 inch) range (see figure 3.9 (a)).

(a) LVDT holder ring with attached LVDT's

(b) Circumferential roller assembly with LVDT

Figure 3.9 Axial and circumferential measurement devices

3.4.2 Testing procedure

The multistage confined creep test was designed to observe the creep of rock specimen under constant deviatoric conditions. The triaxial test cell is a complex testing machine and therefore a detailed test procedure was developed and implemented. The GCTS software was configured for the desired creep test by incorporating the multistage parameters.

A multistage, confined creep test was initiated by first applying the required confining pressure to the specimen. When the confining pressure reached the preselected value, the axial load was increased monotonously to the targeted stress difference value. The confining pressure and stress difference are then maintained at their specified level for the entire duration of the test.

During the test, axial force, confining pressure, deviatoric stress, axial stress, axial displacement and radial displacements were recorded. When the test reached the pre-assigned time duration the test was concluded and the stress difference was reduced to zero by continuously decreasing the axial load at a slow and steady rate while maintaining the confining pressure constant. The reduction in the stress procedure was executed with extreme caution to avoid any shock loading on the specimen. Once the stress difference reaches zero, the 1st stage is then transitioned into the second stage, which followed the same procedure as described for the 1st stage. Increments in stress difference was achieved with each successive stage. It is also mentioned here that the confining pressure was held constant at all stages and only the axial load was changed to achieve the desired stress difference.

Multistage confined creep tests were conducted on three Imperial shale specimens with each stage consisting of eight hours at relatively low confining pressure of 250, 500 and 750 psi.

3.4.3 Data acquisition variables

The data acquisition system was set to monitor various test variables as shown in Table 3.1. Different triaxial parameters were recorded during the multistage confined creep test.

Table 3.1 Test output and their definitions.

#	Identifier	Name	Formula
1.	E_a	Axial Strain	$\Delta L/L$
2.	E_c	Circumferential Strain	$\Delta D/D$
3.	E_v	Volumetric Strain	$\epsilon_a + 2\epsilon_c$
4.	γ	Shear Strain	$\epsilon_a - \epsilon_c$
5.	A_c	Corrected Area	$\frac{A_0 \times (1 - \epsilon_v)}{(1 - \epsilon_a)}$
6.	σ_d	Deviator Stress	$\frac{P - \sigma_c \times A_{\text{piston}}}{A_c}$
7.	σ_a	Axial Stress	$\sigma_d + \sigma_c$
8.	σ_c	Confining Stress	Direct Input

Remark	ΔL is the axial deformation of the specimen L is the actual length of the specimen ΔD is the measured circumferential deformation D is the diameter of the specimen A_o is the original area of the specimen P is the axial load A_{piston} is the area of the piston
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Note, creep strain in the multistage confined creep test was calculated in the same way as is calculated for multistage unconfined creep test (see figure 3.7).

Chapter 4

Results and Discussion

This chapter provides the results obtained from the unconfined and confined creep tests performed on shale and sandstone. Steady state creep law values were determined for each specimen tested. The test results and discussion are presented together.

4.1 Uniaxial compression strength test results

Griggs (1939) defined “Creep” as the slow deformation of solids under small loads acting over long periods of time. The rock is stressed below its failure strength for producing deformation that will be dependent not only on the stress but also on the duration of the test. To determine the ultimate strength of the rock types, eight compression tests were performed on the Berea sandstone and Imperial shale specimens.

Table 4.1 provides our laboratory determined strength values for the shale and the sandstone specimens. The shale specimens were found to be of higher values when they were compared against the published values (Peng, 2008). The specimens were thoroughly examined and core logs were cross checked to find any anomalies in the specimen. Presence of sandstone intrusion may be a possible reason for such high value.

The axial stress-strain curves for the rocks tested in the present investigation exhibited a non-linear behavior in the beginning, a linear behavior in the middle and then again a non-linear behavior near the end of the test (figure 4.1). The initial non-linear behavior was attributed to the fact that rocks have inherent cracks and micro pores which closes as the stress increases. Once all the cracks close, the rock becomes stiffer and starts to behave elastically and hence a linear behavior is observed in the middle of the stress strain plot. Fracture initiation takes place by the development of cracks within the specimen with further increase in the stress. Crack or fracture developed within the specimen gradually coalesces as stress increases and hence, non-linear behavior was observed towards the end of the curve.

Table 4.1 Summary of UCS test results on different rock specimen.

Name	CRSA2	CRSA10	CRSA11	CRSA12	IMP1	IMP5	IMP6	IMP7
Rock type	Berea Sandstone	Berea Sandstone	Berea Sandstone	Berea Sandstone	Imperial Shale	Imperial Shale	Imperial Shale	Imperial Shale
Length (inch)	4.419	4.426	4.457	4.431	4.352	4.077	4.000	4.088
Diameter (inch)	1.990	1.990	1.990	1.990	1.983	1.985	1.977	1.980
L/D ratio	2.220	2.224	2.239	2.226	2.194	2.053	2.023	2.064
Young's modulus ($\times 10^6$)	2.04	2.32	2.24	2.38	1.92	1.83	1.57	1.83
UCS (psi)	7,627	11,080	9,350	7,526	12,042	12,378	11,793	13,687
Average UCS (psi)	8,895				12,475

Figure 4.1 Stress versus axial strain curves for rock specimen tested under load control.

Individual stress-strain curves and photographs of the rock specimens tested under this section are included in Appendix A. The post-test pictures showed various modes of fracture for each individual rock specimen. The tests provided the UCS values to start with the unconfined creep tests which are provided in the sections that follow.

4.2 Single stage unconfined creep test results

The UCS values obtained from the compression tests were then used for selecting the load levels for the subsequent creep tests. The specimens were loaded to a predefined percentage of their peak load value and maintained constant for a period of 12 to 48 hours. The load level was varied from 60 to 85% of the failure load with test performed at every 5% load interval. This variation in load was adopted in the absence of any consistent procedure to perform these creep tests. The LVDT installed within the MTS machine was used to measure the strain developed within the specimen. The machine was recording data points at every 15 minutes after the load was maintained constant.

In tables 4.2 and 4.3, the steady-state creep rates for the Berea sandstone and Imperial shale are presented. Summary of the test results are shown in table 4.2 and 4.3. A total of six Berea sandstone specimens were tested, however two specimen failed within few minutes due to variability in the strength of the prepared specimen. Similarly, a total of four Imperial shale specimens were tested of which two specimens failed prematurely.

Table 4.2 Creep rates from single stage unconfined creep test results on Berea sandstone specimens.

Name	Length (inch)	Diameter (inch)	% Peak Load value	Time (hours)	Stress (psi)	Steady-state creep rate per hour
CRSA7	4.421	1.990	65% (17,790 pound)	12	4,670	4.33×10^{-05}
CRSA5	4.412	1.990	70% (18,976 pound)	12	5,050	5.24×10^{-05}
CRSA1	4.441	1.990	75% (20,525 pound)	12	5,559	5.59×10^{-05}
CRSA3	4.445	1.990	80% (21,817 pound)	12	5,977	7.69×10^{-05}
CRSA6	4.441	1.990	75% (20,525 pound)	N.A.	5,559	N.A.
CRSA4	4.408	1.990	82% (22,520 pound)	N.A.	6,185	N.A.

Table 4.3 Creep rates from single stage unconfined creep test results on Imperial specimens.

Name	Length (inch)	Diameter (inch)	% Peak Load value	Time (hours)	Stress (psi)	Steady-state creep rate per hour
IMP3	4.336	1.983	60% (23,110 pound)	45	6,430	1.07×10^{-05}
IMP8	4.355	1.980	70% (26,874 pound)	47	7,669	1.47×10^{-05}
IMP4	4.354	1.981	75% (28,789 pound)	N.A.	N.A.	N.A.
IMP2	4.328	1.988	85% (31,610 pound)	N.A.	N.A.	N.A.

The creep rates in table 4.2 showed an increase in the rate as the load increased from 65 to 80% of peak load. The specimens were kept under constant load condition and therefore the stress shown in figure 4.2 and 4.3 were calculated using the original cross-sectional area of the specimen. In the loading stage, the specimens underwent deformation, which was observed in figures 4.2 and 4.3. Specimens of Berea sandstone showed an instantaneous elastic strain up to 0.15%, however, the shale specimens showed an increment in the elastic strain up to 0.35%. The increase in the deformation of the specimens was due to the closure of the cracks, platen adjustment of the specimen and closure of pores. The specimens were maintained under constant load for 12 to 48 hours which showed the development of both primary and secondary creep strain which is shown in figures 4.4 and 4.5.

Figure 4.2 Stress against axial strain curve observed during creep test for Berea Sandstone specimens.

Figure 4.3 Stress against axial strain curve observed during creep test for Imperial Shale specimens.

For the Berea sandstone specimens (figure 4.4), the increase in the constant load levels had a profound effect on the amount of the strain. The specimen CRSA7 was loaded up to 65% and CRSA3 up to 80%, showed significant difference in the creep strain. The reason behind such difference is unknown, however in rock salts it is mostly attributed to the rate of dislocation climb of the crystals of the salt (Lama and Vutukuri, 1978). The shale specimens were tested for 48 hours and they showed a larger difference in the creep strain for loads applied at 60 to 70% of the failure load. Also in figures 4.4 and 4.5, specimens CRSA1, CRSA3 and IMP3 show deflection at the 11th, 7th and 23rd hour, which was probably due to possibly small structural failure in the specimen. When the instantaneous elastic strain was removed and only creep strain was plotted against time to better show the curves (figure 4.6), the various stages of creep was observed as was previously discussed in figure 2.1.

Figure 4.4 Axial strain against time curve observed during constant creep test for Berea Sandstone specimen.

Figure 4.5 Axial strain against time curve observed during creep test for Imperial Shale specimens.

From the creep theories discussed earlier, the graph was divided into three stages; (1) primary, (2) secondary and (3) tertiary. The stages were determined by analyzing the change in the strain rate with time. When creep strain for the four sandstone specimens was plotted (figure 4.7), it was evident that as the load level increases there was a significant increase in the creep strain.

Figure 4.6 Axial creep curve of Berea Sandstone specimen (CRSA7) at a constant stress of 4670 psi.

Figure 4.7 Effect of stress level on creep test of Berea sandstone specimen.

In figure 4.8, the creep rate against time is provided for the four sandstone specimens. The strain rate for the two specimens at low load showed a fairly steady state, while at the higher loads, the specimens showed a higher creep rate in the early loading phase which was decreasing to attain the steady state if the test would have continued beyond 12 hours.

Figure 4.8 Creep rate against time curve at different stress level for Berea sandstone specimens.

Figure 4.9 Effect of stress level on creep test of Imperial shale specimen.

For the shale, the two specimens attained the steady state from 15th hour of the initiation of the test. The steady state creep rate for shale and sandstone specimen (see Table 4.2 and 4.3) was determined by taking the slope of creep strain - time curve when the specimen showed the development of a steady state. Individual creep curves of the rock specimens tested under this section are included in Appendix B.

Figure 4.10 Creep rate against time curve at different stress level for Imperial shale specimens.

Once the steady state creep rate was determined, the laboratory results were fitted against the phenomenological model. The simplest and the most widely used phenomenological equation is the steady state or Norton's creep law (Eq. 4.1) (Drescher 2002):

$$\dot{\epsilon}_{ss} = A\sigma^n \quad \text{Equation (4.1)}$$

Where:

$\dot{\epsilon}_{ss}$ is the steady state creep rate,

A and n are the empirical constants,

σ is the applied axial stress.

The values of A and n are the material constants and are different for different rock types. They were obtained by taking log of the equation 4.1 which will transform it into a linear equation as shown below.

$$\log \dot{\epsilon}_{ss} = \log A + n \log \sigma \quad \text{Equation (4.2)}$$

The log values of the experimentally measured creep rate were plotted against the log of the stress values. The coefficient 'A' and the exponent 'n' was determined by adding a linear trend line to the values plotted in log creep rate against log axial stress curve (Figure 4.11). Table 4.4 shows the values of steady state creep law for Berea sandstone. Value of 'n' and log 'A' can be

calculated directly from the equation of the fitted line. For Berea sandstone, the calculated values of coefficient 'A' is equal to $1.48 \times 10^{-13} \text{psi}^{-1} \text{hr}^{-1}$ and 'n' is equal to 2.3.

Table 4.4 Steady state creep rate and log values for creep rate and stress values for Berea Sandstone

Specimen Name	% Peak load	Stress (psi)	Steady state creep rate per hour	Log (stress)	Log (Creep rate)
CRSA3	80%	5,977	7.96×10^{-05}	3.7449	- 4.2254
CRSA1	75%	5,559	5.95×10^{-05}	3.7764	- 4.0990
CRSA5	70%	5,050	5.25×10^{-05}	3.7032	- 4.2800
CRSA7	65%	4,670	4.33×10^{-05}	3.6693	- 4.3633

Figure 4.11 Fitting of linear trend line in log (creep rate) against log (stress) curve for Berea Sandstone.

Using the same method described above, the coefficients were obtained for the shale specimens. Table 4.5 shows the values for the steady state creep law for the Imperial shale with values of coefficient 'A' equal to $1.40 \times 10^{-12} \text{psi}^{-1} \text{hr}^{-1}$ and 'n' equal 1.8 (figure 4.12).

Table 4.5 Values for the steady state creep law for Imperial Shale

Specimen Name	% Peak load	Stress (psi)	Steady state creep rate per hour	Log (stress)	Log (Creep rate)
IMP3	60%	6,430	1.07×10^{-05}	3.8082	- 4.9710
IMP8	70%	7,669	1.47×10^{-05}	3.8847	- 4.8326

Figure 4.12 Fitting of linear trend line in log (creep rate) against log (stress) curve for Imperial Shale.

In the absence of any specific guideline for creep testing from ASTM and ISRM, the initial unconfined creep experiments were designed to formulate the test parameters and to refine the test procedure and analysis of the experimental data. These initial test results laid the foundation for the subsequent multistage unconfined creep tests. The multistage tests were designed to use a single specimen to produce multiple results. In addition, these tests are a better representative of the stress regime acting on the rock mass because it depicts the change in the stress that occurs in the sequential development of the entry. Lu and Hebbelwhite (1998) showed that the load on the bolt changes when the entries are driven. They also found that the load on the bolt increases when adjacent entries are driven. They concluded that load acting on the bolts follow multiple patterns, which is due to the heterogeneity of the strata. To account for this change in stress conditions, multistage uniaxial and triaxial creep tests will provide realistic behavior of the rock when subjected to long-term stress conditions. The sections that follow provide results on multistage unconfined creep tests on imperial shale specimens.

4.3 Multistage unconfined creep test results

The immediate roof of the majority of the underground coal mines in the United States is composed of shale (Peng, 2008). Shale is fined-grained, highly-compacted and contains various amounts of clay. It exhibit fissility or splitting along closely spaced, near parallel surfaces, or laminations (Peng, 2008). Shale formations are inherently weak in strength, and often in the presence of moisture there is further degradation in the strength. Therefore, core recovery is not

only difficult but under certain conditions is impossible. Preparation of the specimen from broken rock blocks is equally difficult because of their low strength. Also, the effect of stress change on the roof rock due to the advancement of the entry and the opening of adjacent entry is also not available. Therefore, multistage tests are conducted to utilize the same specimen for multiple conditions.

In this phase of the experiment, multistage tests were performed on three shale specimens - IMP3, 8 and 11, and under four constant load stages (figure 4.13) which are 65, 70, 75 and 80% of the ultimate failure load value. The load was applied at the rate of 100 psi/sec until it reached the designated load and was then kept constant for 24-48 hours.

Figure 4.13 Stress against axial strain curves observed for IMP8.

On the completion of the 1st stage the load was programmed to increase at the rate of 100 psi/sec to the load value of the second stage. The procedure was repeated at interval of 48 hours and stopped when the test time reached 192 hour (4 stages) or the 8th day of the test (figure 4.14). For specimen IMP8, the axial strain against time is shown in figure 4.14. It is observed that in the 1st stage of strain against time curve the specimen showed a non-linear behavior. This was also observed in the creep strain against time plot, which excluded the presence of the instantaneous strain (figure 4.15). This non-linear behavior was due to the presence of cracks and voids inside the rock specimen which closes during the first stage when the load is maintained constant for 48 hours. Also, it was observed that when the specimen was loaded in the second, third and the fourth stage similar behavior was not observed.

Figure 4.14 Multistage axial strain against time curves observed for IMP8.

Figure 4.15 Creep Strain against time curve for IMP8.

On observing the strain creep rate against time (figure 4.16), the strain rate for stages 2, 3 and 4 were similar, however, for the 1st stage, the strain rate was low and the specimen was still under primary or transient creep stage. For the other three stages, the strain rate was high when the test began and then slowly attained a steady state within 15 hours of the test. The specimen did not show any sign of failure after the test was completed. The results of the experiments are presented in Table 4.6. The steady-state creep rate was calculated by finding the slope of creep strain against time curve when the specimen attained the steady state. Individual creep curves for each of the specimen tested under this section is shown in Appendix C.

Figure 4.16 Creep rate against time curve for IMP8.

Table 4.6 Summary of multistage unconfined creep test results on Imperial shale specimens

Name	Length (inch)	Diameter (inch)	% Peak Load	Time/Stage (hours)	Average Stress (psi)	Steady-state creep rate per hour
IMP3	4.421	1.990	65% (25,036pound)	24	7,060	N.A.
			70% (26,963pound)	24	7,684	4.3303×10^{-06}
			75% (28,888pound)	24	8,307	5.6939×10^{-06}
			80% (30,814pound)	24	8,930	6.3514×10^{-06}
IMP8	4.342	1.981	65% (24,980pound)	48	8,105	N.A.
			70% (26,901pound)	48	8,728	4.7253×10^{-06}
			75% (28,822pound)	48	9,351	6.2242×10^{-06}
			80% (30,744pound)	48	9,974	6.7767×10^{-06}
IMP11	4.339	1.988	85% (32,905pound)	48	10,601	5.6691×10^{-06}
			90% (34,840pound)	48	11,224	6.3534×10^{-06}

The steady state creep values were then used for generating the coefficient of ‘A’ and ‘n’ as discussed in the single stage tests. Table 4.7 shows the values of the parameters that are needed for determining the coefficient of the steady state creep law. The first phase was not included for both IMP3 and IMP8 rock specimen for calculating the coefficient because the specimen appeared to be in its primary or transient creep stage.

Table 4.7 Values for the steady state creep law for specimen IMP3, IMP8 and IMP11

Phase	% Peak load	Stress (psi)	Steady state creep rate per hour	Log (stress)	Log (Creep rate)
Phase 2	70%	7,684	4.33×10^{-06}	3.8855	- 5.3634
Phase 3	75%	8,307	5.69×10^{-06}	3.9194	- 5.2445
Phase 4	80%	8,930	6.35×10^{-06}	3.9508	- 5.1971
Specimen IMP8					
Phase 2	70%	8,724	4.73×10^{-06}	3.9409	- 5.3251
Phase 3	75%	9,351	6.22×10^{-06}	3.9708	- 5.2062
Phase 4	80%	9,974	6.67×10^{-06}	3.9988	- 5.1758
Specimen IMP11					
Phase 1	85%	10,601	5.67×10^{-06}	4.0253	- 5.2464
Phase 2	90%	11,224	6.35×10^{-06}	4.0501	- 5.1970

For specimen IMP8 (figure 4.17) the points are fitted with a straight line and the coefficient values are the equation coefficient. Similarly, two other specimens IMP3 and IMP11 coefficient values 'A' and 'n' are determined and are provided in table 4.8. The coefficient values shown in table 4.8 are then compared with the values obtained in the single stage tests and it was found that the 'n' values are near to the single stage values while the 'A' values are different for different stress conditions and creep rates and therefore, for modeling purposes it is imperative that the immediate roof rocks selected for creep tests should be performed at the stress levels that are near to the in-situ stress conditions.

Figure 4.17 Fitting of linear trend line in log (creep rate) against log (stress) curve for IMP8.

Table 4.8 ‘A’ and ‘n’ values for the steady state creep law

SPECIMEN	A	n
IMP3	$4.94 \times 10^{-16} \text{ psi}^{-1}\text{hr}^{-1}$	2.56
IMP8	$3.01 \times 10^{-16} \text{ psi}^{-1}\text{hr}^{-1}$	2.59
IMP11	$5.30 \times 10^{-14} \text{ psi}^{-1}\text{hr}^{-1}$	1.99

The tests performed in this section were under uniaxial stress conditions, which takes only the vertical stresses into the consideration. However, the immediate roof is under both horizontal and vertical stresses. Therefore, the unconfined tests performed in the previous section does not represent the real stress conditions of the immediate roof rock. For understanding the true creep behavior of the immediate roof, the rock needs to be tested under triaxial conditions. The sections that follow provide the result and discussion for the multistage triaxial tests that were performed on shale specimens at different stress conditions.

4.4 Multistage confined creep test results

In triaxial tests the rock is subjected to two stresses – axial and confining. The triaxial stress state, when compared with in-situ stress (σ_x , σ_y and σ_z) represents the three principal stresses: vertical (σ_x) and two horizontal stresses (σ_y and σ_z) (Hoek and Brown, 2002). In the triaxial stress test, the two horizontal stresses are essentially assumed to be equal. For this section, multistage tests were adopted instead of single stage tests. The reason for the selection of multistage tests was the limited availability of test specimens and also to minimize the variability of the test results performed on different test specimens.

Multistage confined creep tests were performed on Imperial shale specimen at relatively low confining pressures of 250, 500 and 750 psi. A total of one specimen was tested at each different confining pressure out of which one specimen failed in the second stage at 750 psi confining pressure. The testing procedure discussed earlier (see section 3.4.2) was performed on the three shale specimens; IMP13, IMP14 and IMP15. It is known that under confining stress, the strength of the rock generally increases; and therefore using the peak strength values obtained from the uniaxial tests cannot be used to select the stress levels to perform confined creep experiments.

Therefore, three triaxial strength tests were performed on shale specimens at 250, 500 and 750 psi. Table 4.9 provides our laboratory determined triaxial strength values for shale specimens under different confining pressure and its values at 70, 75 and 80% strength.

Table 4.9 Triaxial strength of the specimen with the creep reduced values.

#	Specimen IMP9	70%	75%	80%
Failure strength at 250 psi confining pressure	9,581 psi	6,706 psi	7,185 psi	7,664 psi
	Specimen IMP 10			
Failure Strength at 500 psi confining pressure	9,500 psi	6,650 psi	7,125 psi	7,600 psi
	Specimen IMP12			
Failure strength at 750 psi confining pressure	11,981 psi	8,386 psi	8,985 psi	9,584 psi

The constant stress condition applied to the specimen differs from the constant load applied under uniaxial stress conditions. With the use of the lateral strain transducer, the change in the area was continuously measured and the axial stress was adjusted to maintain a constant stress in the specimen. The procedure described earlier is represented in the figure 4.18. Creep test was initiated by applying the required confining pressure. Once the confining pressure reached the preselected value, the first stage of the confined creep experiment was initiated. The load was increased from O to A, the deviatoric stress was maintained at a constant value from A to B. On completion of the 1st stage, the specimen was brought back to hydrostatic condition by slowly unloading the specimen at the rate of 100 psi/sec, till the targeted stress difference was brought to zero. The unloading of the specimen to the hydrostatic condition ensures that shock loading of the specimen was eliminated. Once the stress difference was brought to zero, the next stage was initiated, where the load was increased slowly to achieve a higher deviatoric stress value than the previous stage and the same procedure was repeated. The confining pressure is maintained constant throughout the experiment.

Figure 4.18 Deviatoric stress path for multistage creep test under triaxial stress conditions.

In the triaxial creep test of specimen IMP15, the specimen was tested at three constant deviatoric stress values (σ_d): 6413, 6897 and 7379 psi (figure 4.19) and at a constant confining pressure of 250 psi. The test was conducted for a total of 24 hours with each stage being 8 hours. Parameters such as load and deformation were monitored continuously. On completion of the test, the axial stress against strain was plotted as shown in figure 4.20. The permanent deformation due to creep was observed as the test progressed from stage 1 to 3.

Figure 4.19 Deviatoric Stress against time relationship for IMP15.

The permanent deformation in the 1st stage (see figure 4.20) was mainly due to closure of pores and voids. This behavior was also observed in multistage uniaxial creep tests. The

nonlinear response of the rock to the applied stress was observed when the specimen was unloaded and reloaded for the subsequent stages (figure 4.20).

Figure 4.20 Axial Stress against Axial Strain relationship for IMP15.

The axial creep strain for each stage was calculated (see section 3.3.3) and plotted against time as shown in figure 4.21. The first stage showed a non-linear behavior with 0.03% straining while in second and third stage the specimen showed less than 0.02 % increase in the strain.

Figure 4.21 Axial creep strain against time relationship for IMP15.

The strain rate for each stage was successively higher which showed that the creep deformation for rock was dependent on stress. The change in the stress accelerated the development and propagation of crack which resulted in further deformation of the specimen.

Figure 4.22 Creep rate against time relationship for IMP15.

Further, the volumetric strain for the three stages was found to mimic the deformation of the specimen. Higher volumetric strain was observed in the third stress stage which meant that the specimen dilated more in the third stage. The result indicates the onset of shear fracture which may have led to the failure of the specimen.

Figure 4.23 Volumetric strain against time relationship for IMP15.

For the specimen IMP13, the confining pressure was raised from 250 to 500 psi and the deviatoric stress was held constant at 5,547, 5,994 and 6,422 psi. The axial creep strain against time is plotted in figure 4.24. In multistage creep experiments, it was found that the specimen showed higher deformation in the first stage than in the subsequent stages. Additionally, it was

observed that the specimen showed erratic volumetric deformation (figure 4.23) in the first stage with possibly some failure before getting stabilized.

Figure 4.24 Axial creep strain against time for specimen IMP13.

Figure 4.25 Volumetric Strain against time for specimen IMP13.

Stabilization occurred due to confining pressure restricting the development of the fracture. In the third stage, which has the higher deviatoric stress values, the specimen showed accelerating volumetric deformation with time (figure 4.25). Such behavior is a possible indication of the failure of the specimen, if the test would have continued for more hours. The strain rate shown in

figure 4.26 was similar to the earlier results. The rates attained steady state approximately four hours from the initiation of the test.

Figure 4.26 Creep strain rate against time for IMP 13.

Lastly, the specimens were tested at a confining pressure of 750 psi and deviatoric stress values of 7,574 and 8,174 psi. The specimen showed dramatic behavior in the second stage as it failed after one and half hours from the initiation of the second stage (figure 4.27). The axial creep strain development in the first stage shown in figure 4.27 was lower than the values reported for the test performed at 250 and 500 psi. With increase in the confining stress there was less development in the creep strain; however when the specimen enters the second stage of the test, the creep strain accelerates after one and half hour from the initiation of the test. The strain then accelerated nonlinearly till four and half hours from the initiation of the second stage when the test was stopped. Figure 4.28 shows the failed specimen with the shear fracture as well as fracture extending throughout the specimen.

Figure 4.27 Creep strain rate against time for IMP14.

Figure 4.28 Failed Specimen IMP14 showing the shear fracture outlined in white sketch.

As previously discussed the volumetric strain is an indicator of the development of shear fracture; the volumetric strain behavior in the second stage shown in the figure 4.29 validates the above statement. The strain developed within the specimen with time resulted in failure of the specimen when the deviatoric stress was increased from first stage (7,574 psi) to second stage (8,174 psi). The volumetric strain in the second stage increases abruptly within one hour of the initiation of the second stage and after reaching a peak of 0.07%, the strain descends sharply indicating the failure of the specimen.

Figure 4.29 Volumetric Strain against time for specimen IMP14.

The creep rate is plotted in figure 4.30 and it was observed that the strain rate in the 1st stage showed high rates as compared to the strain rates measured at 250 and 500 psi confining pressure. In the second stage the specimen never attained steady state; instead the strain rate increased after one hour from the initiation of the test.

Figure 4.30 Creep Strain rate against time for specimen IMP14.

In earlier paragraphs, the creep behavior of the specimens were analyzed under multistage triaxial conditions. The summarized values for the steady-state creep rates for the three specimens are presented in table 4.10. It was observed that the secondary creep rate decreases as the confining pressure increased from 250 to 750 psi.

Table 4.10 Summary of multistage confined creep test results on Imperial shale specimens.

Name	Length (inch)	Diameter (inch)	Confining Pressure (psi)	Time/Stage (hours)	Stress Difference (psi)	Axial steady-state creep rate per hour
IMP15	4.273	1.987	250	8	6,413	1.30×10^{-05}
				8	6,897	1.35×10^{-05}
				8	7,379	1.59×10^{-05}
IMP13	4.411	1.982	500	8	5,547	8.42×10^{-06}
				8	5,994	9.80×10^{-06}
				8	6,422	1.01×10^{-06}
IMP14	4.401	1.989	750	8	7,574	6.51×10^{-06}
				1.3	8,174	2.91×10^{-04}

The data obtained from the tests were then analyzed to determine the coefficient of the steady state creep law. For specimen IMP 15 and IMP13, the log of stress and creep rate were tabulated (table 4.11) and then plotted as shown in figure 4.31 and 4.32. The points were then fitted with a linear line and the equation coefficients were determined.

Table 4.11 Values for the steady state creep law for specimen IMP15 and 13

Phase	Confining Pressure (psi)	Deviatoric Stress (psi)	Steady state creep rate per hour	Log (stress)	Log (Creep rate)
Phase 1	250	6,413	1.30×10^{-05}	3.8070	-4.8860
Phase 2	250	6,897	1.35×10^{-05}	3.8386	-4.8696
Phase 3	250	7,379	1.59×10^{-05}	3.8679	-4.7986
Specimen IMP13					
Phase 1	500	5,547	8.42×10^{-06}	3.7440	-5.0746
Phase 2	500	5,994	9.80×10^{-06}	3.7777	-5.0087
Phase 3	500	6,422	1.01×10^{-06}	3.8076	-4.9956

Figure 4.31 Fitting of linear trend line in log (creep rate) against log (stress) curve for IMP15.

Figure 4.32 Fitting of linear trend line in log (creep rate) against log (stress) curve for IMP13.

From the linear fit of the data, the coefficient of steady state creep law was determined and the values for 'A' and 'n' are provided in table 4.12. When the values are compared with the results obtained from uniaxial tests it was found that the 'n' values are in the range from 1.25 to 2.59 while the values from A varied from 1.69×10^{-10} to 4.94×10^{-16} $\text{psi}^{-1}\text{hr}^{-1}$. Variations in the value of the coefficients are directly related to the state of stress, however, the exact mechanism of crack development and propagation is still an area lacking full understanding.

Table 4.12 'A' and 'n' values for the steady state creep law.

SPECIMEN	A	n
IMP15	$4.85 \times 10^{-11} \text{ psi}^{-1}\text{hr}^{-1}$	1.42
IMP13	$1.69 \times 10^{-10} \text{ psi}^{-1}\text{hr}^{-1}$	1.25

Chapter 5

Conclusions

The main objective of this research was to understand the time-dependent deformation of immediate coal mine roof rock, in particularly roof with shale formations. Development of an entry in a coal seam disturbs the natural stress equilibrium present in the earth crust. In addition to the weight of rock itself, the rock is subjected to in-situ and induced stresses, which induces deformation in the rock surrounding the entry. The roof is critically stressed and continues to deform after secondary supports are provided. One of the major indicators of continuous deformation is the roof fall in supported sections of the mine. It is suspected that the presence of stress for a certain period, induces time-dependent deformations in the rock, which may induce the failure of the rock in the supported sections.

Surprisingly, there is absence of any research performed in this area especially in the United States. Therefore, this thesis was planned to address this gap by performing laboratory experiments. Further while the experimental creep analysis was being planned, it was found that there was an absence of any standards on consistent test procedures. Therefore, to develop proper test procedure for creep tests, the investigation was initiated with shakedown creep tests on Berea sandstone specimens. Based on the initial results, a test procedure was developed and subsequently seventeen shale specimens were tested under both single and multistage unconfined and multistage confined conditions to understand this behavior under different stress and load conditions. The contributions of this thesis are discussed in the following:

- Shale specimen showed time-dependent stain in both single and multistage unconfined and confined creep tests.
- For the single stage unconfined creep tests, the shale specimens showed significant increase in the creep strains and creep rates with an increase in load levels. Similar results were obtained for the multistage unconfined creep tests. For multistage confined creep tests, similar behavior for shale specimen was again observed when the stress was maintained constant for longer period of time. Both creep strain and creep rate showed a

significant increase in their values with increase in stress conditions. It was concluded that both creep strain and creep rate are stress dependent.

- For both multistage unconfined and confined creep tests, the first phase showed a non-linear behavior which is mainly attributed to the fact that rock specimens have inherent cracks and voids which closes as the load/stress is maintained constant for longer period of time.
- The multistage procedure developed to conduct creep tests provided a more realistic representation of the stress regime acting on the rock in the field. Additionally, the same specimen can be utilized for analysis of multiple conditions.
- Steady state or Norton's creep law provided a good fit to experimental data with R^2 value greater than 0.9 for most of the specimen tested. The coefficient A and n are the physical constant which are determined based on creep test data for each specimen. The values of coefficient 'n' ranges from 1.25 to 2.59 while 'A' ranges from 1.69×10^{-10} to 4.94×10^{-16} $\text{psi}^{-1}\text{hr}^{-1}$. This variation in the values is directly related to the applied stress on the specimen. Therefore, it is imperative to perform creep tests using in-situ stress conditions.
- The exact mechanism of creep, micro-crack development and failure in shale is still unknown and an area of future research.

References

1. Barron, K. and Toes, N.A., 1963, "Deformation around a mine shaft in salt," *Proceedings of the Rock Mechanics symposium*, Ottawa, pp. 115-134.
2. Baud, P. and Meredith, P.G., 1997, "Damage accumulation during triaxial creep of darley dale sandstone from pore volumetry and acoustic emission," *International Journal of Rock Mechanics and Mining Science*, Vol. 34, No. 3-4, paper no. 024.
3. Bradshaw, R.L., Boegly, W.J. and Empson, F.M., 1964, "Correlation of convergence measurements in salt mines with laboratory creep-test data," *Proceedings of the sixth symposium on Rock Mechanics*, Rolla, pp. 501-514.
4. Drescher, K., 2002, "An investigation into the mechanism of time-dependent deformation of hard rocks," Masters' Thesis, University of Pretoria, South Africa, pp. 1-264.
5. EIA (DOE/EIA-0035(2013/08) - Monthly Energy Review, August 2013.
6. GCTS technical reference manual, 2010.
7. Griggs, D., 1936, "Deformation of rock under high confining pressure – 1: Experiments at room temperature," *The Journal of Geology*, Vol. 44, No. 5, pp. 541-577.
8. Griggs, D., 1939, "Creep of rocks," *The Journal of Geology*, Vol. 47, No. 3, pp. 225-251.
9. Headley, D.G.F., 1967, "An appraisal of convergence measurements in salt mines," *Proceedings 4th Canadian Rock Mechanics Symposium*, Ottawa, pp. 117-135.
10. Hoek, E. and Brown, E.T., 2002, "Underground excavations in rocks," The Institute of mining and metallurgy, pp. 1-525.
11. Hult, J., 1966, "Creep in engineering structures," Blaisdell Publishing Company, pp. 1-113.
12. Jaeger, J. C. & Cook, N.G.W., 1979, "Fundamentals of rock mechanics – Third edition," Chapman and Hall publication, London, pp. 1-583.
13. Lama, R.D. & Vutukuri, V.S., 1978, "Handbook on mechanical properties of rocks (Volume III), Trans Tech Publications, Clausthal, Germany, pp. 1-405.
14. Lu, T. and Hebblewhite, B., 1998, "Field monitoring of rock bolting performance in weak roof strata," *17th International Conference on Ground Control in Mining*, Morgantown, WV, USA, pp. 243-248.

15. Misra, A.K. and Murrell, S.A.F., 1965, "An experimental study of the effect of temperature and stress on the creep of rocks," *Geophysical Journal of the Royal Astronomical Society*, Vol. 9, No. 5, pp. 509-535.
16. MSHA 2013 – "Preliminary accident reports, Fatalgrams and Fatal Investigation Reports Coal Mine".
17. MTS Reference manual, 1990.
18. Murrell, S.A.F. and Misra, A.K., 1962, "Time-dependent strain or 'creep' in rocks and similar non-metallic materials," *Bulletin - Institution of Mining and Metallurgy*, Vol. 71, No. 7, pp. 353-378.
19. Nie, D., 2011," Experimental analysis of post-failure behavior of coal and rock under laboratory compression tests," Masters' thesis, West Virginia University, Morgantown, pp. 1-110.
20. Norton, F.H., 1929, "The creep of steel at high temperature," McGraw-Hill book company Inc., New York, pp.1-90.
21. Okubo, S., Fukui, K. and Hashiba, K., 2008, "Development of a transparent triaxial cell and observation of rock deformation in compression and creep tests." *International Journal of Rock Mechanics & Mining Sciences*, Vol. 45, pp. 351-361.
22. Park, D.W. and Jung, J.O., 1988, "Application of holographic interferometry to the study of time-dependent behavior of rock and coal," *Rock Mechanics and Rock Engineering*, Vol. 21, No. 4, pp. 259-270.
23. Phillips, D.W., 1932, "Further investigation of the physical properties of coal-measure rocks and experimental work on development of fractures," *Transactions Institution of Mining Engineering*, Vol. 82, pp. 432-450.
24. Pomeroy, C.D., 1956, "Creep of coal at room temperature," *Nature*, Vol. 178, No. 4527, pp. 279-280.
25. Price, N. J., 1964, "A study of the time-strain behavior of coal-measure rocks," *International Journal of Rock Mechanics and Mining Sciences*, Vol. 1, No. 2, pp. 277-303.
26. Reynolds, T.D. & Gloyne, E.F., 1961, "Creep measurements in salt mines," *Proceedings of the fourth symposium on Rock Mechanics*, March 30, 31 – April 1, pp. 11-17.

27. Robertson, E.C., 1960, "Creep of solenhofen limestone under moderate hydrostatic pressure," *Memoir – Geological Society of America*, Vol. 79, pp. 227-244.
28. Rutter, E.H., 1972, "On the creep testing of rocks at constant stress and constant force," *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, Vol. 9, No. 2, pp. 191-195.
29. Singh, D.P., 1975, "A study of creep of rocks," *International Journal of Rock Mechanics and Mining Science & Geomechanics Abstracts*, Vol. 12, No. 9, pp. 271-275.
30. Sone, H. and Zoback, M.D., 2010, "Strength, creep and frictional properties of shale gas reservoir rocks," *44th US Rock Mechanics Symposium and 5th U.S.- Canada Rock Mechanics Symposium*, Salt Lake City, Utah, June 27-30.
31. Tandanand, S., 1985, "Moisture adsorption rate and strength degradation of Illinois shales," *26th US Symposium on Rock Mechanics*, Rapid City, South Dakota, June 26-28, pp. 591 – 600.
32. Yang, C., and Daemen, J. J. K., 1997, "Temperature effects on creep of tuff and its time-dependent damage analysis," *International Journal of Rock Mechanics and Mining Science*, Vol. 34, No. 3-4, Paper No. 345.
33. Yu, C.W., 1998, "Creep characteristics of soft rock and modelling of creep in tunnel," Ph.D. thesis, University of Bradford, U.K, pp. 1-190.
34. Zhang, S., Nakano, H., Xiong, Y., Nishimura, T. and Zhang, F., 2010, "Temperature-controlled triaxial compression/creep test device for thermodynamic properties of soft sedimentary rock and corresponding theoretical prediction," *Journal of Rock Mechanics and Geotechnical Engineering*, Vol. 2, No. 3, pp. 255-261.

Appendix A

Stress-Strain curves observed during uniaxial compressive strength tests

Figure A1 Stress versus stain curve for CRSA2.

Figure A2 Intact Specimen.

Figure A3 Shear failure.

Figure A4 Stress versus strain curve for CRSA10.

Figure A5 Intact Specimen.

Figure A6 Shear failure.

Figure A7 Stress versus strain curve for CRSA11.

Figure A8 Intact Specimen.

Figure A9 Failed Specimen.

Figure A10 Stress versus strain curve for CRSA12.

Figure A11 Intact specimen.

Figure A12 Observed single cone failure.

Figure A13 Stress against strain curve for IMP1.

Figure A14 Intact Specimen.

Figure A15 Axial splitting type failure.

Figure A16 Stress versus strain curve for IMP5.

Figure A17 Intact Specimen.

Figure A18 Shear type failure.

Figure A19 Stress versus strain curve for IMP6.

Figure A20 Intact specimen.

Figure A21 Single cone failure.

Figure A22 Stress versus strain curve for IMP7.

Figure A23 Intact Specimen.

Figure A24 Axial splitting type failure.

Appendix B

Individual creep curves observed during single stage unconfined creep tests

A) Berea Sandstone

Figure B1 Stress versus strain curve for CRSA7.

Figure B2 Strain versus time curve for CRSA7.

Figure B3 Creep Strain versus time curve for CRSA7.

Figure B4 Creep rate versus time curve for CRSA7.

Figure B5 Stress versus strain curve for CRSA5.

Figure B6 Strain versus time curve for CRSA5.

Figure B7 Creep strain versus time curve for CSRA5.

Figure B8 Creep rate versus time curve for CRSA5.

Figure B9 Stress versus strain curve for CRSA1.

Figure B10 Strain versus time curve for CRSA1.

Figure B11 Creep strain versus time curve for CRSA1.

Figure B12 Creep rate versus time curve for CRSA1.

Figure B13 Stress versus strain curve for CRSA3.

Figure B14 Strain versus time curve for CRSA3.

Figure B15 Creep strain versus time curve for CRSA3.

Figure B16 Creep rate versus time curve for CRSA3.

B) Imperial Shale

Figure B17 Stress versus strain curve for IMP3.

Figure B18 Strain versus time curve for IMP3.

Figure B19 Creep strain versus time curve for IMP3.

Figure B20 Creep rate versus time curve for IMP3.

Figure B21 Stress versus strain curve for IMP8.

Figure B22 Strain versus time curve for IMP8.

Figure B23 Creep strain versus time curve for IMP8.

Figure B24 Creep rate versus time curve for IMP8.

Appendix C

Individual creep curves observed during multistage unconfined creep tests

Figure C1 Stress versus strain curve for IMP3.

Figure C2 Strain versus time curve for IMP3.

Figure C3 Creep strain versus time curve for IMP3.

Figure C4 Creep rate versus time curve for IMP3.

Figure C5 Strain versus time curve for IMP11.

Figure C6 Creep Strain versus time curve for IMP11.

Figure C7 Creep rate versus time curve for IMP11.

Figure C8 log (Creep rate) versus log (Stress) curve for IMP3. Figure C9 log (Creep rate) versus log (Stress) curve for IMP11.