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MECHANICAL PROPERTIES OF KENTUCKY OIL SHALES
AS RELATED TO MINE DESIGN APPLICATION

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Rock Mechanics Engineer
The Cleveland-Cliffs Iron Company

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Transportation Research Program
University of Kentucky

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Abstract: To develop oil shales as an alternate energy resource, it will be necessary to develop appropriate mining techniques. In this regard, the mechanical properties of the ore must be determined. Section I of this paper presents data from a laboratory study to determine these properties. Uniaxial compression tests, longitudinal frequency tests, indirect tensile tests, and triaxial tests were performed. Section II uses data from section I to design an example underground mine. Strength properties are considered in the design of roof spans and pillars. It is concluded that underground mining of Kentucky oil shale is feasible with a possible extraction rate of 70 percent or more through an aggressive rock mechanics program and good mining practices.

SECTION I-ROCK PROPERTIES

Introduction

Because of the abundance of oil shales in many south-central Kentucky counties, the development of these resources as an alternate source of energy appears favorable. However, to recover these materials through mining techniques, a thorough understanding of the mechanical properties of the material is necessary.

For this purpose, a number of holes were drilled in several south-central Kentucky counties and a series of cores containing the oil shales were obtained. These were sealed to prevent moisture loss and transported to the laboratory. A total of 34 specimens were tested in the laboratory and these are listed in Table 1. The mean moisture content of the 34 specimens was 0.96 percent with a standard deviation of 0.28 and a coefficient of variability of 29 percent.

The following tests were performed in the laboratory: (1) the uniaxial compression test, (2) the fundamental, longitudinal frequency test to determine the dynamic modulus, (3) the indirect tensile test, and (4) the triaxial compression test.

Uniaxial Compression Tests

Seventeen uniaxial compression tests were performed to determine the maximum compressive stress, elastic modulus and Poisson's ratio. The American Society for Testing and Materials standard test method No. D 3148 was used. Figure 1 shows the direction of loading in the uniaxial compression test. The specimens were loaded until failure at a rate that would produce failure in no less than five minutes but no more than 15 minutes.

Axial and radial strains were monitored by four SR-4 strain gages. The gages were mounted at the midpoint of the specimen, ninety degrees apart. The average of two gages (on opposite sides of the specimen) was used to calculate axial strain and, likewise, the average of the remaining two gages was used to calculate radial strain. Load on the specimen was monitored by a load transducer. A schematic diagram of the uniaxial compression apparatus is shown in Figure 2.

The axial strain at any point during the test is calculated as follows (1):

$$e_a = \frac{\Delta l}{l}$$

where

- $e_a$ = axial strain,
- $l$ = original undeformed length of specimen, and
- $\Delta l$ = change in length of specimen.
The radial strain is calculated from the equation (1);
\[ e_r = \frac{\Delta d}{d} \]
where:
- \( e_r \) = radial strain,
- \( d \) = original undeformed diameter of specimen, and
- \( \Delta d \) = change in diameter.

The compressive stress is calculated from the following equation (1):
\[ \sigma = \frac{P}{a} \]
where:
- \( \sigma \) = compressive stress,
- \( P \) = applied load, and
- \( a \) = original cross-sectional area of specimen.

Poisson's ratio is calculated as follows:
\[ v = \frac{e_t}{e_r} \]
where \( v \) = Poisson's ratio.

A typical example of the data as calculated in the above equations is shown in Figure 3.

Figure 3 also illustrates the method used to calculate the tangent elastic modulus from the uniaxial compression test. The slope of the axial stress-strain curve was determined at 50 percent of the maximum compressive stress according to the following equation:
\[ E_t = \frac{\Delta \sigma}{\Delta e} \]
where \( E_t \) = tangent elastic modulus from the uniaxial compression test. Table I lists a summary of the uniaxial compression test results. The maximum compressive stresses ranged from 1.17 x 10^4 to 1.94 x 10^4 pounds per square inch, with a mean value of 1.50 x 10^4 pounds per square inch and a standard deviation of 2.08 x 10^4 pounds per square inch. The coefficient of variability (standard deviation/mean x 100) was 14 percent. The tangent elastic modulus ranged from 1.02 x 10^4 to 3.36 x 10^4 pounds per square inch with a standard deviation of 5.97 x 10^4 pounds per square inch. The coefficient of variability for this parameter was 37 percent. The value of the tangent elastic modulus showed a considerable amount of variability. It is suggested that this is largely due to the nonuniformity of the specimens. It was noted that many of the specimens had small inclusions of pyrite crystals and it appeared that the specimens often failed around these crystals.

**Dynamic Modulus Tests**

The dynamic Young's Modulus was determined for all test specimens. The procedure used is outlined in standard test method No. C 215 of the American Society for Testing and Materials. Standard test method No. D 2845 could also have been used, however, the limitations on dimensions were more severe than in C 215.

A schematic diagram of the testing apparatus is shown in Figure 4. The oscillator was used to induce a sinusoidally varying longitudinal wave in the specimen. The specimen response was determined by the accelerometer at the opposite end of the specimen cylinder. The outputs from the oscillator and the accelerometer were input to a dual beam oscilloscope. When the oscilloscope is made to display the forcing
Figure 3. Example of Data Illustrating Stress as a Function of Strain and Poisson's Ratio and Method of Obtaining the Tangent Elastic Modulus.

The function versus the accelerometer output, an ellipse of some shape will result. The oscillator is tuned through a range of frequencies and the frequency at which the major and minor axes of the ellipse align vertically and horizontally, is the fundamental longitudinal frequency.

The dynamic Young's Modulus is calculated from the following equation (2):

$$E_d = Dw (n)^2$$

where:

- $E_d$ = dynamic Young's modulus,
- $n$ = fundamental longitudinal frequency,
- $W$ = weight of specimen,
- $D = 0.01318 (L/d)^2$,
- $L$ = specimen length, and
- $d$ = specimen diameter.

Table 1 lists the dynamic Young's Modulus for each of the specimens. The values varied widely from $1.87 \times 10^5$ to $5.55 \times 10^6$ pounds per square inch. The mean value for the 34 specimens was $3.72 \times 10^5$ pounds per square inch with a standard deviation of $9.57 \times 10^4$ pounds per square inch. Again, the coefficient of variability was 26 percent.

Indirect Tensile Test

Fourteen indirect tensile tests were performed according to ASTM standard test No. C 496. The specimens were loaded with the same compression machine as used in the uniaxial compression tests. In this test, only the load is monitored. The loading configuration on the specimen is illustrated in Figure 5. Although the specimen is tested in compression, the specimen fails in tension along, or close to, the surface shown as a dashed line in Figure 5. For this reason the test is described as an indirect tensile test.

The load is applied continuously, at a constant rate within the range of 100 to 200
THEORETICAL FAILURE SURFACE (TENSION)

Figure 5. Loading Direction in the Indirect Tensile Test.

Table 1. Summary of All Laboratory Tests

<table>
<thead>
<tr>
<th>SAMPLE HOLE</th>
<th>DEPTH (feet)</th>
<th>MAXIMUM HORIZ. STRAIN (%)</th>
<th>MAXIMUM VERT. STRAIN (%)</th>
<th>MAXIMUM POISSON'S RATIO</th>
<th>MAXIMUM COMPR. STRESS (PSI)</th>
<th>MAXIMUM TENSILE STRESS (PSI)</th>
<th>MAXIMUM TENSILE MODULUS (MILLIONS PSI)</th>
<th>DYNAMIC MODULUS (MILLIONS PSI)</th>
<th>TANGENT MODULUS (MILLIONS PSI)</th>
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TRIAXIAL TESTS

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</tbody>
</table>

INDIRECT TENSILE TESTS

pounds per square inch per minute of tensile stress until the specimen fails. The tensile stress is calculated as follows (1):

\[ T = \frac{2P}{ld} \]

where
- \( T \) = tensile stress,
- \( P \) = maximum applied load indicated by the testing machine,
- \( l \) = length of specimen, and
- \( d \) = diameter of specimen.

Table 1 lists the results of these tests. The values ranged from 2.17 \times 10^3 to 2.95 \times 10^3 pounds per square inch. The mean value was 2.50 \times 10^3 pounds per square inch and a standard deviation of 250 pounds per square inch. This is a coefficient of variability of 10 percent.
Triaxial Test

Three triaxial tests were performed. In the triaxial test, the specimen is sealed in a rubber membrane and is then placed in a fluid-filled chamber. By applying air or hydraulic pressure to the chamber, the specimen can be placed under an isotropic confining stress. The specimen is then loaded vertically to failure. Figure 6 illustrates the loading pattern on a triaxial specimen and Figure 7 is a schematic diagram of the triaxial apparatus. The procedures used in this study are described in ASTM standard No. D 2664.

Generally, triaxial test data are used to define the shear strength parameters commonly known as the internal friction angle, $\phi$, and cohesion, $c$. To determine these parameters, the confining stress and the maximum applied vertical stress are plotted in a "Mohr's stress diagram" as illustrated in Figure 8. Because of the heterogeneous nature of rock, three triaxial tests are normally performed and the results plotted as shown in Figure 9. A "best-fit" straight line is then fitted tangent to the three Mohr's circles. The zero intercept of this line is defined as the cohesion, and the angle of this line with the horizontal is the angle of internal friction. These parameters are often used in the design of soil and rock structures.

Figure 6. Loading Scheme in the Triaxial Compression Test.

Figure 7. Schematic Diagram of Triaxial Compression Apparatus.
The results of the three triaxial tests in this study are shown in Figure 9. Confining pressures of 2000, 3000, and 4000 pounds per square inch were used. The test with 3000 pounds per square inch confining pressure (sample 19) had an equipment malfunction during the tests which made the failure stress abnormally low. However, if the average stress values of the uniaxial compression tests (zero confining pressure) is plotted with the triaxial tests, the shear strength parameters can be determined. From Figure 9 the internal friction angle, \( \phi \), was determined to be 26.5 degrees and the cohesion is 4800 pounds per square inch.

Some of the results of this section will now be used in an example mine design application.

\[ \phi = 26.5^\circ \]
\[ C = 4800 \text{ psi} \]

Figure 9. Results of Triaxial Compression Tests.
Each mine must be evaluated independently due to the extreme variability in the physical properties of natural materials from one location to the next. Often, many critical design criteria must be determined from the examination of drill core specimens until access to the ore body is gained.

At present, knowledge of Kentucky oil shale physical properties is limited to the results from laboratory testing as presented in Section I. From these results we can predict with a limited degree of accuracy how the oil shale, and surrounding rock, will behave in a mining situation. Once underground workings are established, the true in-situ behavior of a mine in Eastern Oil Shales can be determined and the mine design fine tuned accordingly.

Selection of a Mining Method

Before any design work can begin, a mining method must, of course, be selected. There are three factors that tend to limit the mining methods that may be employed for a given deposit. These are as follows:

1. Spatial characteristics of the deposit.
2. The physical properties of the deposit and surrounding strata.
3. Environmental requirements.

Because of the abundant reserves of Kentucky oil shale located in areas with favorable stripping ratios, the first generation of commercial mines will employ surface mining techniques. In several locations, highwall to ore ratios will become impractical as mining moves towards the foothill regions, at which time, adits may be driven into the highwall along the mining zone to establish an underground mine. Although the data in Section I could be applied to surface mining, it is the second generation of mines (underground) to which this report will address itself.

TYPICAL LITHOLOGY—CASEY CO., KENTUCKY

Figure 10
The spatial characteristics of oil-bearing shales in Kentucky has been well established through a core drilling program conducted by the Institute for Mining and Minerals Research (IMMR). They can best be classified as flat laying, thick bedded deposits, occurring as multiple seams in some locations, but only as a single seam (the Cleveland Member of the Ohio Shale Formation) in the area for this report. This type of layout is most effectively mined by longwall or room and pillar mining techniques.

There are three areas, hanging wall, ore zone, and footwall, that must be considered when analyzing the physical characteristics of a potential underground mine (see Figure 10).

- **Hanging Wall:** The Borden Formation consists of shales and siltstones, and comprises the majority of the overburden. As a result of the extreme weathering which occurred to core specimens from the Borden Formation no physical property testing was done in this zone. While this does not provide us with any information for empirical design it does indicate the necessity to protect the roof rock from heave or pillar bottom punching.

- **Ore Zone:** Cleveland Member of the Ohio Shale Formation. See Appendix B for the derivation of the properties listed below.

- **Foot Wall:** Huron Member of the Ohio Shale Formation. See Appendix C for the derivation of the properties listed below.

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<tr>
<th>Property</th>
<th>Cleveland Member</th>
<th>Huron Member</th>
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<tr>
<td>Compressive Strength</td>
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<td>5,624 psi</td>
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<td>Young's Modulus</td>
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<tr>
<td>Flexural Strength</td>
<td>2,343 psi</td>
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</table>

* Sorption values are essential when designing the roof rock, would tend to punch into the back creating extremely adverse ground conditions (see Figure 12).

Pillar punching could be a problem in room and pillar mining as well. The accepted solution to this would be designing the pillars to yield as load increases rather than remain stiff and punch into the back. A yielding pillar can create other problems in a headgate or tailgate entry with rib spalling or by failing in a brittle fashion rather than yielding. Pillar design will be described further later in this report.

It must be noted that the properties of the Borden Formation used in the above discussion were derived from core samples taken in Lewis and Fleming counties, 100 miles or more from the location of core samples used for testing in Section I of this report. It is uncertain how much the physical properties of a formation could change over this distance. Cleveland Member core samples from Lewis and Fleming counties displayed a compressive strength up to 90 percent less than the values used in this report. This does highlight the need for site-specific testing before attempting a final design.

The behavior of the back is essential to a longwall operation and allows for fewer design options than a room and pillar system. With the uncertainty of the Borden Formation's behavior as a mine back it would not be prudent to attempt first generation
underground mining with a longwall system. Also, at this time off the shelf longwall equipment is not available for cutting ore with a compressive strength of 12,000 plus psi. Therefore, a room and pillar mining system will be examined as the mining technique to be employed.

The environmental requirements will not be discussed in this report. The surface mining which will undoubtedly precede any underground mining would address this issue. It would be safe to assume either underground mining technique could meet whatever requirements necessary.

**Roof Span**

In designing a room and pillar layout one of the first considerations is the safe maximum roof span. As discussed previously, the extreme slaking observed in Borden Formation core samples indicates a need to protect the roof rock from weathering. To accomplish this, a two-foot-thick zone (see Figure 13). Using eight-foot resin-grouted roof bolts, a composite beam composed of two feet of Cleveland Member shale, being the thinnest part of the beam, will encounter the highest stress. From the indirect tensile test the modulus of rupture (R₀) is calculated to be:

\[ R₀ = 5,624 \text{ psi, for Cleveland shale.} \]

Because of the uncertainty resulting from limited test data and experience, a safety factor of eight will be used. The maximum allowable stress will be:

\[ σ_{allow} = \frac{R₀}{F.S.} \]

\[ σ_{allow} = \frac{5,624}{8} = 703 \text{ psi} \]

Knowing the allowable stress, a composite beam formula can be rearranged to calculate the maximum roof span.
UNCAVED (YIELDING) BACK IN LONGWALL AREA SHEDS OVERBURDEN LOAD TO CHAIN PILLARS

LOCAL ROOF FAILURES

STIFF PILLARS, HAVING A HIGHER COMPRESSIVE STRENGTH THAN THE HANGING WALL, WILL PUNCH INTO THE ROOF, I.E. ROOF FAILS BEFORE PILLARS WILL YIELD.

LONGWALL ENTRY STABILITY

\[ \sigma_{\text{max}} = \frac{L^2}{t_1} \left( \frac{y_1 t_1}{y_2 t_2} \right) \]
\[ L^2 = \frac{\sigma_{\text{max}} t_1}{\left( \frac{y_1 t_1}{y_2 t_2} \right)} \]

where
\[ \sigma_{\text{max}} = \text{maximum allowable stress} = 703 \text{ psi} \]
\[ L = \text{length of the beam} \]
\[ t_1 = \text{thickness of Cleveland unit} = 2 \text{ ft} \]
\[ t_2 = \text{thickness of Borden unit} = 5 \text{ ft} \]
\[ y_1 = \text{unit weight of Cleveland shale} = 145 \text{ lb/ft}^3 \]
\[ y_2 = \text{unit weight of Borden shale} = 155 \text{ lb/ft}^3 \]

solving for \( L \)
\[ L = 36.4 \text{ feet} \]

The maximum stress in the Borden shale unit of the beam would be:
\[ \sigma_{\text{max}} = \frac{(36.4)^2}{5} \left( \frac{(145)(2)+(155)(5)}{(2+5)} \right) \]
\[ \sigma_{\text{max}} = 280 \text{ psi} \]

The safety factor for the Borden shale can be calculated from its flexural strength.

\[ F.S. = \frac{R_y}{\sigma_{\text{max}}} \]
\[ F.S. = 1.671/280 = 5.9 \]

This is lower than the safety factor for the Cleveland shale but still acceptable. The above design equation reflects the only flexural strength of a composite beam clamped at each end and does not take into consideration any frictional resistance to bending between bedding planes.
The span calculated above will be used as the maximum allowable span, which occurs across the diagonal of an intersection. The maximum room span will be (see Figure 14):

\[ L_{room} = L_{max} (\sin 45°) \]

\[ L_{room} = 26.4 \text{ (sin 45°)} = 25.74 \text{ ft} \]

say 25 feet.

**Piller Design**

The conventional method of relating compressive strength (C) with a safety factor, to overburden pressure, will be used for pillar design. Several more sophisticated techniques taking in-situ angles of internal friction, confining pressures and width to height ratios into account, have been proposed in recent years. These more advanced methods should be considered once site-specific information is available for final design.

Knowing the maximum room span and the depth of overburden a series of calculations can be made to determine the safety factor of various pillar dimensions (see Figure 15) as shown in Table 2.

\[ F.S. = \frac{C_o}{\sigma} \]

where

- \( C_o \) = compressive strength (12,141 psi)
- \( \sigma \) = pillar load
- \( \gamma \) = unit weight of overburden
- \( (155 \text{ lb/ft}^2) \)

\[ \sigma_1 = 145 \text{ lb/ft}^3 \]

\[ \sigma_2 = 155 \text{ lb/ft}^3 \]
### Table 2. Factors of Safety for Pillar Design

<table>
<thead>
<tr>
<th>L (Ft)</th>
<th>X (Ft)</th>
<th>Y (Ft)</th>
<th>P (psi)</th>
<th>( \sigma ) (psi)</th>
<th>F.S.</th>
<th>R_e</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>20</td>
<td>25</td>
<td>0.22</td>
<td>2,200</td>
<td>5.52</td>
<td>78%</td>
</tr>
<tr>
<td>25</td>
<td>25</td>
<td>25</td>
<td>0.25</td>
<td>1,337</td>
<td>6.27</td>
<td>75%</td>
</tr>
<tr>
<td>25</td>
<td>30</td>
<td>25</td>
<td>0.27</td>
<td>1,776</td>
<td>6.84</td>
<td>73%</td>
</tr>
<tr>
<td>25</td>
<td>30</td>
<td>30</td>
<td>0.30</td>
<td>1,628</td>
<td>7.46</td>
<td>70%</td>
</tr>
</tbody>
</table>

\[ L_{ROOM} = L_{MAX} \sin 45^\circ \]

**ROOM WIDTH, PLAN VIEW ROOM INTERSECTION**

Figure 14
and $c = \frac{d}{R_p}$

where $d$ = depth of overburden (450 ft)
$R_p$ = pillar ratio

and $R = \left( \frac{x}{x+L} \right) \left( \frac{y}{y+L} \right)$

where $x$ = pillar width
$y$ = pillar length
$L$ = room span (25 ft.)

and $e = \frac{1-R}{100}$

where $e$ = extraction ratio

Experience in western oil shale mines has shown the use of large diameter blast holes to be most economical. Blast damage up to 30 times the hole diameter could occur if no attempt is made to use controlled blasting techniques such as decoupling. A conservative mine design must take into account, especially in main entries where life of mine access is required.

If, for example, a 3.0 inch diameter blast hole were used, a zone of blast damage around the perimeter of the pillar up to 7.5 feet in depth could exist. Presumably a bedding plane will be found to form the immediate roof line and prevent the propagation of blast damage into the roof beam. The compressive strength of the rock in the blast damage area would be reduced 50 percent or more. The roof beam depends upon a solid foundation on each end, the effective room width would have to be reduced to provide the necessary support and, therefore, the extraction ratio would also be reduced (see Figures 16 and 17).

Assuming 50 percent strength in the blast damage zone would require the addition of 3.75 feet to each rib, or 7.5 feet to the cross sectional dimensions to maintain the same safety factors, see Table 3.

Of course once again actual field experience will finally dictate what extent blast damage, or its control, will have on pillar design.

\[
PILLAR\ RATIO\ (R_p) = \frac{(X)(Y)}{(X+L)(Y+L)}
\]

PILLAR RATIO CALCULATION

Figure 15
TRUE EXTRACTION RATIO ($R_e'$) = \left\{1 - \frac{(X')(Y')}{(X+L')(Y+L')}\right\} \times 100$

EXTRACTION RATIO CALCULATION

Figure 16

MAX. BEAM LENGTH ($L$)

BORDEN FOR.

COMPOSITE ROOF BEAM

CLEVELAND MEMBER

EFFECTIVE ROOM WIDTH ($L'$)

BLAST DAMAGE ZONE

PILLAR RIB

HURON MEM.

COMPOSITE ROOF BEAM

Figure 17
Conclusions

The lack of site-specific information on the Borden Formation, which comprises the hanging wall and most of the overburden, limits the confidence with which a mine design can be applied. Questions regarding possible pillar punching and blast damage cannot be properly answered with the available laboratory data, and most likely will not be fully understood until in-situ observations are possible. What benefits could be gained from a yielding pillar design can only be ascertained in an active mine.

Even with the conservative design approach taken, the mechanical properties of Kentucky oil shale indicate successful underground mining is feasible. Overall extraction of 70 percent or more will be possible through an aggressive rock mechanics program and good mining practices. A pilot or experimental mine would provide the clues to many of the questions raised in this report.

REFERENCES


APPENDIX A

PHYSICAL PARAMETERS OF BORDEN FORMATION SHALES

Basic data from IMMR, Resource Assessment of Oil-Bearing Shales in Lewis and Fleming Counties, Kentucky, Section A4.5.

1. Unconfined compressive strength mean compressive strength minus the standard deviation.

<table>
<thead>
<tr>
<th>Test Results</th>
<th>11,618 psi</th>
<th>15,300 psi</th>
<th>9,000 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>12,039</td>
<td>-</td>
<td>8,883</td>
</tr>
<tr>
<td>s</td>
<td>-3,156</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

C_o 8,883 psi
2. Young's Modulus (E), mean value minus the standard deviation, using $E_t$ at 50 percent failure load.

Test Results

- $1.475 \times 10^6$ psi
- $2.630 \times 10^6$ psi
- $0.986 \times 10^6$ psi

$x = 1.697 \times 10^6$

$s = -0.844 \times 10^6$

$E = 0.899 \times 10^6$ psi

3. Indirect Tensile Strength ($T$) mean value minus the standard deviation.

Test Results

- $1.475 \times 10^6$ psi
- $2.630 \times 10^6$ psi
- $0.986 \times 10^6$ psi

$x = 1.697 \times 10^6$

$s = -0.844 \times 10^6$

$E = 0.899 \times 10^6$ psi

4. Flexural Strength ($R_0$)

- $R_0 = 2.4 T$
- $R_0 = 2.4 (2,343) = 5,624$ psi

APPENDIX C

PHYSICAL PARAMETERS OF HURON MEMBER SHALE

1. Unconfined Compressive Strength ($C$) mean compressive strength minus the standard deviation.

Test Results

- $12,881$ psi
- $14,957$ psi
- $16,047$ psi

$x = 14,937$

$s = 14,921$

$C = 12,872$ psi

2. Young's Modulus (E), mean value minus the standard deviation.

Test Results

- $1.157 \times 10^6$ psi
- $1.624 \times 10^6$ psi
- $1.435 \times 10^6$ psi

$x = 1.405 \times 10^6$

$s = -0.219 \times 10^6$

$E = 1.186 \times 10^6$

3. Indirect Tensile Strength ($T$) mean value minus the standard deviation.

Test Results

- $2,707$ psi
- $2,342$

$x = 2,514$ psi

$s = -171$

$T = 2,343$

4. Flexural Strength ($R_0$)

- $R_0 = 2.4 T$
- $R_0 = 2.4 (2,166) = 5,199$ psi

APPENDIX B

PHYSICAL PARAMETERS OF CLEVELAND MEMBER SHALE

1. Unconfined Compressive Strength ($C$) mean compressive strength minus the standard deviation.

Test Results

- $16,325$ psi
- $16,843$

- $11,721$ psi

$x = 14,926$ psi

$s = -2,734$

$C = 12,141$ psi

2. Young's Modulus (E), mean value minus the standard deviation.

Test Results

- $1.492 \times 10^6$ psi
- $1.932 \times 10^6$ psi

$x = 1.492 \times 10^6$

$s = -0.447 \times 10^6$

$E = 1.045 \times 10^6$ psi

3. Indirect Tensile Strength ($T$) mean value minus the standard deviation.

Test Results

- $2,707$ psi
- $2,342$

$x = 2,514$ psi

$s = -171$

$T = 2,343$

4. Flexural Strength ($R_0$)

- $R_0 = 2.4 T$
- $R_0 = 2.4 (2,166) = 5,199$ psi