PREDICTION OF COMPRESSIVE STRENGTH FROM POINT-LOAD AND MOISTURE CONTENT INDICES OF HIGHLY ANISOTROPIC, COAL-BEARING STRATA OF THE ILLINOIS BASIN

BY

ROBERT ALAN BAUER

B. S., University of Illinois at Chicago Circle, 1976

THESIS

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The point-load index is used to estimate the unconfined compressive strength of rock samples. Previous researchers have developed relationships between the diametral point load index and the unconfined compressive strength for isotropic, high strength, igneous and metamorphic rocks and a few strong sedimentary rocks. Lately, testing of the strength of sedimentary rocks associated with coal deposits has become more prevalent. In many cases the point-load testing procedures and relationships between the point-load index and the unconfined compressive strength found for isotropic, high strength rocks are being used for the highly anisotropic, low strength, sedimentary rocks.

This study found that diametral point-load testing of highly anisotropic sedimentary rocks produces inconsistent results which cannot be used to accurately estimate the unconfined compressive strength. Axial point-load testing evaluated by the $T_{500}$ index, which incorporates the cross sectional area of the point-load sample, produces good correlation coefficients with the unconfined compressive
strength of five different lithologic rock types. The multiplying factors used with the $T_{500}$ index to estimate the unconfined compressive strength were 12.8, 14.3, 15.8, 22.9, and 26.7 for claystones, gray shales, limestones, black shales, and coals, respectively. The use of moisture content as an index to predict the compressive strength produced correlation coefficients ($r$) ranging from .67 to .90. Better correlations were sometimes produced when the $T_{500}$ and moisture content indices were combined into one equation for the prediction of the unconfined compressive strength.
ACKNOWLEDGMENTS

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CHAPTER I
INTRODUCTION

Researchers in the field of applied rock mechanics are continually seeking simple, low cost, convenient, reproducible tests to replace more expensive, time-consuming laboratory tests. These simple tests are known as index tests, and the property they measure is called an index property. The index property is used to estimate another property which is commonly required to mechanically characterize a sample.

A large number of samples are tested to fully portray any natural variability in a geologic material. Index tests allow the researcher to perform a large number of tests in a short period of time. Soil mechanics index tests are well defined and are used extensively to classify soil properties. Rock classification using index properties has not progressed to a similar level.

Rock strength is fundamental mechanical property. Rock strength is most commonly classified by the compressive strength test. This test requires precision-machined samples which limits testing to the laboratory. This test also requires long pieces of core, producing strength results from only the better quality sections of core and a bias which does not allow the researcher the opportunity to
fully assess the strength range of highly broken cores from weak strata.

A few index tests have been used to estimate the unconfined compressive strength from small pieces of broken core which are too small to use for unconfined compressive strength tests. Of these index tests, the point-load tests appear to have the widest application as shown by D'Andrea, Fischer, Fogelson (1965), McWilliams (1966), Carter and Sneddon (1977), and Read, Thornton, and Regan (1980). The point-load is not only an index test but is also a strength test for materials where the unconfined compressive strength test cannot be performed because of size constraints produced by small samples. In these cases, the point-load test replaces the unconfined compressive strength and eliminates the bias of strength results from only large pieces of intact core.

In the 1960s the point-load test was designed and first used to measure the tensile strength of igneous and high-grade metamorphic rock. Later it was used as an index to estimate the unconfined compressive strength. Nearly all the previous point-load testing and formulas to estimate strength properties were limited to isotropic, high strength, igneous and high-grade metamorphic rocks.

During the past decade rock mechanics testing of sedimentary rocks associated with coal-bearing strata has greatly increased. Some investigators have used the
point-load formulas and testing procedures designed for igneous and metamorphic rocks to estimate the strength indices of sedimentary rocks. Research and new formulas for the use of the point-load index for these highly anisotropic sedimentary rocks have not been published.

The increased testing of weak sedimentary rocks has led researchers to investigate other indices. Several authors, Bauer (1979, 1980), Jeremic (1981) and Sasaki, Kinoshita, and Ishijima (1981) have used moisture content indices to estimate the unconfined compressive strength of weak sedimentary rock.

The purpose of this study was to compare the results of axial and diametral point-load, indirect tensile, and unconfined compressive tests, as well as moisture content determinations on cores of limestone, shale, coal, and claystone from coal exploration programs conducted in the Illinois Basin of the Eastern Interior Basin of North America. The study examined the previously published point-load testing procedures and empirical relationships and compared them to the findings of this study.

This study also examined the variability of the empirical relationships between point-load indices and unconfined compressive strengths for: (1) the same rock stratigraphic unit from different areas of the State of Illinois, and (2) different lithologies associated with coal-bearing strata from specific localities. The use of
moisture content as a strength index and in conjunction with the point-load index was investigated. Formulas and procedures are presented to best estimate the unconfined compressive and tensile strength of the tested sedimentary rocks.
CHAPTER II
REVIEW OF RELATED STUDIES

Pells (1975) and Read, Thornton, and Regan (1980), have partially investigated the relationship between the point load index and the unconfined compressive strength of highly anisotropic sedimentary rocks from Australia. Both studies encountered results of high variability because the diametral point-load tests (Figure 1) were failing along bedding planes. They both concluded that diametral point load testing on highly anisotropic rocks was impracticable to derive unconfined compressive strength values. Read, Thornton and Regan (1980) proceeded to run the point-load test axially (Figure 1) and to use its value as a separate index and not to estimate the unconfined compressive strength.

The point-load test, as it is used today, was designed by Reichmuth (1963), who used it as a tensile strength index test. The formula included a constant which was derived for cores loaded diametrically. Later, Reichmuth (1968) expanded the formula and added shape and brittleness factors. Miller (1965) and D'Andrea, Fischer, and Fogelson (1965) used Reichmuth's method on various isotropic rock types and compared the point-load tensile strength to corresponding unconfined compressive strength. Test results from both
Fig. 1. Diametral and axial point-load tests.
studies produced high correlation coefficients of $r=.92$ and .95 respectively between predicted and measured unconfined compressive strength values.

McWilliams (1966), Arscott and Hackett (1968), and Friedman and Logan (1970) used the axial point-load test to determine a preferred vertical weakness plane. Rock discs point-loaded axially fail along the weakest vertical plane. Discs cut and tested from oriented core can show if a weakness plane is in the same direction throughout the length of the core. All of the above authors found their tested material to contain a vertical preferred fracture plane. McWilliams (1966) and Friedman and Logan (1970) also performed "Brazilian" indirect tensile strength tests (Figure 2) on multiple discs at every 30° of azimuth. They showed that the weakest tensile plane defined by both the axial point-load test and the indirect tensile test matched. These findings were confirmed by Peng and Okubo (1978), Miller (1979), and Miller, Bauer, and Johnson (1980), on the investigations into preferred vertical weakness planes in Devonian aged black shales.

Broch and Franklin (1972) performed point-load tests on a limited number of high strength isotropic rocks and recommended replacement of Reichmuth's complex formula for tensile strength with a point-load strength index. They introduced a reference index of $I_{50}$ which is the point-load strength index of a 50mm diameter core. Broch and
Fig. 2. "Brazilian" indirect tensile strength test.
Franklin developed a correction chart so various sample sizes could be tested and changed to an $I_{50}$ value. They found that diametral point-load test samples needed length-to-diameter ratios of at least 1.4 to develop constant point-load values. They observed that the axial point-load values varied with changes in length-to-diameter ratios and never became constant. Therefore, the standardized length-to-diameter ratio for axial point-load tests was determined by finding the ratio which produced the same point-load index value as the constant value for the diametral point-load tests. Using this method the ratio of 1.1 was found for the axial point-load tests. They developed a correlation between the point-load index and the unconfined compressive strength. Their findings had a correlation coefficient of .88 and showed that 24 times $I_{50}$ equaled the unconfined compressive strength for the rock types tested. Broch and Franklin suggested a standardized procedure and point-load equipment dimensions. Their suggestions were adopted by the International Society of Rock Mechanics in 1973.

Bieniawski (1975) compared diametral and axial point load testing on samples of three rock types, namely norite, quartzite, and sandstone, using Broch and Franklin's procedures. He found the standard deviation for the diametral point-load index three times higher than for the axial point-load index for the sandstone samples.
Bieniawski did not address this problem of the large standard deviation for the diametral test results, but this was the first indication of problems of reproducibility of diametral point-load test results from anisotropic materials (sedimentary rocks). The standard deviation of the two tests were nearly identical for the norite tested, which has isotropic properties. The two tests being nearly equal, Bieniawski chose the diametral test as the most convenient to run since no sample preparation of the cores was required. Bieniawski produced factors of 24, 21, and 18 to be used with diametral point-loading of NX(54mm)-, BX(42mm)-, and EX(21.5mm)-diameter cores to estimate the unconfined compressive strength, for high-strength, isotropic rocks.

Peng (1976) used finite element analysis to investigate the radial and tangential stress distributions in samples, point-loaded in the axial direction. He found that stress distributions stabilized for specimens with length-to-diameter ratios less than 1.0. His work showed that thin discs of core could be used for axial point-load tests. This conflicts with Broch and Franklin's (1972) 1.1 length-to-diameter ratio of samples for axial point-load tests. Peng found that the complexities of the stress distributions developed in the sample made it difficult to develop a theory to determine the stress at which complete failure occurs. He advised that the test should not be used
to directly measure the tensile strength of a sample, but that the test was a reliable method to determine the preferred vertical orientation of fracture planes.

Brook (1977, 1979, 1980) developed a formula for point-load testing which incorporated the cross-sectional area of the sample and therefore eliminated the size and shape effects on the point-load index value. Brook's formula was procuded through the examination of the relationship between the amount of strain energy and the strained volume of the samples undergoing point-load testing. Mathematically he derived the relationship that the applied load was proportional to the strained cross-sectional area raised to the .75 power. The .75 factor expresses the size effect. Tsur-Lavie and Denekamp (1982) showed that .75 expresses the maximal size effect which is assumed to be related to a single fracture plane development in a sample. The size effect factor can be determined with a log-log plot of the load at failure versus the cross-sectional area. Theoretically, this produces a straight line with a slope of .75. Brook (1977) showed that the slopes varied from .71 to .87 with an average slope of .81 for various lithologies tested. Brook attributed the slope changes to different combinations of size and shape effects in different rocks. The slope value from the log-log relationship was used in the equation to produce the point-load index value normalized to a sample with a cross-sectional area of
500mm². Brook used the mathematically derived .75 slope in his formula since the actual slopes were near the .75 value. Brook's equation included the cross-sectional area of the samples so there was no size effect on the index values. The cross-sectional area used in the formula was the minimum area irrespective of the mode of failure which may produce a failure plane which is not perpendicular to the axis of the core. The samples can be any size as long as the ratio of length-to-diameter was less than 1.00 (Peng [1976]). Brook (1977, 1980) used the data from D'Andrea, Fischer, and Fogelson (1965) (49 tests), Broch and Franklin (1972) (15 tests), Bieniawski (1975) (9 tests) and his own test data (40 tests) to show that 12.5 times the $T_{500}$ value equals the unconfined compressive strength. Brook indicated that point-load samples run axially and diametrically produced similar testing results. These similar results proved that the materials used for these tests had isotropic properties. He also stated that the $T_{500}$ value was often very close to the indirect tensile strength value.

Hassani, Scoble, and Whittaker (1980) developed a new point-load index versus size correlation chart and accompanying mathematical formulas. Hassani, Scoble, and Whittaker reported that their findings and those of Bieniawski (1975) and Carter and Sneddon (1977) concluded that more accurate and repeatable index values were derived by diametral point-load testing, as compared to the axial
test. In fact, neither Bieniawski (1975) nor Carter and Sneddon (1977) found more accurate and repeatable index values for diametral point-load testing. Bieniawski found values with less scatter for axial point-load testing of sandstone samples and nearly equal scatter for norite samples in comparison to diametral point-load testing. He advocated the use of the diametral test procedure based on similar scatter of results for the two tests and the convenience of the diametral point-load testing procedures. Carter and Sneddon found less scatters of axial point-load test results in mudstones and sandstone and only reported axial point-load test results in their article. Hassani, Scoble, and Whittaker (1980) did not show a comparison between data from their axial and diametral point-load tests. Their work was based on data from eight other authors along with their own test results, all based on diametral point-load testing. Their own tests were performed on sandstone, siltstone, and limestone. The authors' new formula standardized the point-load index \( I_{50} \) to an index value of a 50mm diameter core. They developed a relationship for the unconfined compressive strength equal to 29 times \( I_{50} \). Their coefficient of correlation between \( I_{50} \) values and unconfined compressive strength values was 0.94. They also found the tensile strength equal to 2.77 times the \( I_{50} \) value.
Thus, the literature review shows that the vast majority of previous point-load work was performed diametrally on isotropic material; any axial work performed was shown to be equal to the diametral values because of isotropic properties of the tested rock types. Emphasis was placed on diametral point-load work since it was the most convenient test to perform as it required no sample preparation prior to testing. It appears that Pells (1975), and Read, Thornton, and Regan (1980) were the only researchers to work with samples that had enough anisotropy to cause inconsistency in the diametral test results and curtailment of diametral testing.

A synopsis of point-load index formulas per author is shown in Table 1.
Table 1. Summary of point-load formulas per authors.

<table>
<thead>
<tr>
<th>AUTHOR(S)</th>
<th>FORMULAS</th>
<th>SAMPLE SIZE CONTRAINTS</th>
</tr>
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<tbody>
<tr>
<td>Reichmuth (1963)</td>
<td>$TS = 0.96L/D^2$</td>
<td>2</td>
</tr>
<tr>
<td>Broch and Franklin (1972)</td>
<td>$PLI = (I_d) = L/D^2$</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>Size correction chart to get $I_{50}$</td>
<td></td>
</tr>
<tr>
<td>Bieniawski (1975)</td>
<td>Broch and Franklin</td>
<td>1.4</td>
</tr>
<tr>
<td>Pells (1975)</td>
<td>Broch and Franklin</td>
<td>1.4</td>
</tr>
<tr>
<td>Brook (1980)</td>
<td>$PLI = T_{500} = M(L/A^{0.75})$</td>
<td>Any Ratio</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Any Ratio &lt; 1</td>
</tr>
<tr>
<td>Hassani, Scoble and Whitaker</td>
<td>Broch and Franklin</td>
<td>1.4</td>
</tr>
<tr>
<td>(1980)</td>
<td>New size correction</td>
<td></td>
</tr>
<tr>
<td></td>
<td>to get $I_{50}$</td>
<td></td>
</tr>
<tr>
<td>Read, Thornton, and Regan (1980)</td>
<td>Broch and Franklin</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.1</td>
</tr>
</tbody>
</table>

TS = Tensile Strength  
PLI = Point-Load Index  
D = Distance between loading points  
M = Proportionality constant  
L = Load at failure  
A = Cross-sectional area
CHAPTER III

GEOLOGIC SETTING AND CHARACTERISTICS OF SAMPLES

Samples for this study were derived from coal exploration projects conducted in Pennsylvanian sediments in the Illinois Basin Coal Field (Figure 3). The coal bearing strata in this basin underly the southern three-fourths of Illinois, the southwestern edge of Indiana and the northwestern portion of Kentucky. Samples were collected from southern, east-central, central, and north-central Illinois.

Tests on 26 different stratigraphic units were performed. Figure 4 shows a general stratigraphic column and the names of the 26 units selected for the test. The 26 units can be grouped in five different lithologies: underclay, coal, gray shale, black shale and limestone. The variations within each lithological group were small. Therefore, only a general description for each lithological group is given.

The underclays were light gray, massive, highly fractured with slickensided surfaces, and with carbonaceous root impressions in the upper part of the unit. The unit became calcareous downward and usually graded into a limestone or nodular limestone unit. The underclays were 3 to 6 feet thick. They are believed to represent a prodelta
Fig. 3. Illinois Basin Coal Field.
Fig. 4. Stratigraphic column showing the 26 units selected for tests for this study.
deposit upon which the coal swamps developed (Wanless, Baroffio and Trescott [1969]).

The coals were high-volatile B bituminous, normally bright-banded, with usually one well developed cleat set and another poorly developed one. The coals were 3 to 8 feet thick and were formed by accumulating plant matter in a swamp environment.

The gray shales tested were medium gray, poorly bedded, with siderite nodules or bands occasionally scattered throughout. The gray shale units usually became darker and less silty downward. The units were 5 to 80 feet thick. The gray shales are deposits from a terrestrial source, usually carried by and deposited along river systems and in fresh to brackish water lakes (Krausse et al [1979] and Treworgy and Jacobson [in press]).

The black shales tested were black, carbonaceous, fissile, hard, thinly bedded and occasionally phosphatic. The units were 1 to 4 feet thick. They were deposited in restricted anaerobic marine or lagoon conditions (Krausse et al [1979] and Palmer, Jacobson and Trask [1979]).

The limestones were gray to buff in color, argillaceous, fossiliferous and occasionally contained shale partings. The units were 1 to 10 feet thick. They were deposited in open marine conditions (Givens [1968]).
CHAPTER IV

SAMPLING, PREPARATION, AND TESTING METHODS

The cores were supplied by companies performing coal exploration projects in the Illinois Basin. The diameters of the cores supplied were 47, 54, 63 and 75mm. All cores were wrapped in plastic tube-bags immediately after retrieval from the core barrel to protect them from moisture loss. In the laboratory, samples were taken from sections of core which looked uniform in appearance. All samples were wrapped in tape to hold them together and to protect them from the oil coolant subsequently used in the trim saw. Tape was removed prior to testing. Orientation lines were drawn the length of some cores prior to cutting the core into individual test samples, which allowed for the comparison of strength and fracture direction between different test samples. A sample set included enough pieces to obtain one unconfined compressive test and three to six tests of each point-load and indirect tensile tests and one moisture content determination. Some samples of shale were formed by separating them on bedding planes. Samples with major flaws, such as irregular circumferences, nodular inclusions, fractures, siderite bands in shale samples or shale layers in limestone samples, were not tested.
The natural moisture content of the cores was preserved by sealing the core in plastic tube-bags shortly after retrieval from the core barrel. Tests were usually performed on the cores within 4 to 5 months of drilling. Moisture content determinations were made on small samples shortly after drilling. These small samples were gathered from the sections of core where testing was expected to be performed. The moisture content of the small samples was compared with the moisture content of the sections of core tested at a later date. This procedure was performed to check the performance of the moisture preservation technique during the storage of core prior to testing. This method showed that no changes in the moisture content were found for the stored samples.

The unconfined compressive strength test was performed on right angle cylinders cut with an oil-bath saw. The ends were lapped to obtain a length-to-diameter ratio of 2 to 2.5 with a tolerance for non-parallelism between the end faces less than 0.063mm. Short samples with length-to-diameter ratios less than 2 but greater than 1 were occasionally tested when the proper length could not be obtained because of poor core conditions. Therefore, all compressive strength values were normalized to a length-to-diameter ratio of 1 by the formula presented in ASTM C 170-41T. Loading of the samples was performed under constant strain-rate conditions, the displacement rates varied from .06 to
.1mm/minute. Tests were completed in 6 to 15 minutes. No caps or lubricants were used on the sample ends. The uniaxial compressive strength was determined by dividing the ultimate axial force by the original area perpendicular to the axis of the core.

The point-load test was performed according to D'Andrea, Fischer, and Fogelson (1965) for axial loading and according to ISRM (1973) for diametral loading. The point load apparatus was designed with the standardized components presented in Broch and Franklin (1972) and ISRM (1973). The samples were loaded between two 60 degree cones with 5mm radius points. The cones were coaxially aligned and rigidly held in a specially designed jig. The load was increased until the sample failed along a fracture plane which intersected the two coaxial loading points. The exact solution of the imposed elastic stress has not been derived, but the field is axially symmetrical about the points up to the onset of the defined fracture. The length-to-diameter ratios of the point-load samples for axial loading were about .5 and always less than 1 (Peng [1976]), and for the diametral loading were 1.4 (Brock and Franklin [1972] and ISRM [1973]). Samples with ratios from 0.1 to 0.7 were used to investigate the size effect on axial point-load indices and to evaluate Brook's (1977) T<sub>500</sub> values. Hassani, Scoble, and Whittaker's (1980) I<sub>50</sub> equation and Brook's (1977) T<sub>500</sub> equations were tested for consistent values.
and correlation to compressive strength for both diametral and axial point-load testing.

Tolerances for parallelism between ends was not maintained. Most of the samples had their ends parallel from being cut parallel by the trim off saw.

The "Brazilian" indirect tensile test was performed on discs of core with length-to-diameter ratios of 0.5. The discs were compressed diametrically between high modulus platens (steel) (See Fig. 2). The compressive load produces a tensile stress perpendicular to the loading direction which is diametrically applied to opposite sides of the disc. The tensile stress is greatest at the center of the disc. This is where the tensile crack starts and propagates toward each platen. The maximum tensile stress developed at the time of failure is derived from the theory of elasticity. Various authors (Timoshenko [1934], Frocht [1948], Muskhelishvilli [1953], Solonlvikoff [1956], and Fairhurst [1964]) have studied the stress analysis of a circular element subjected to diametral line loads.

Tolerances for parallelism between ends of the discs was not a concern since the stresses and initiation of the fracture is at the center of the core. The ends of the core were only cut parallel to each other with a trim saw.

All strength testing was performed on a Tinius-Olsen Model 3000T-1 compression testing machine. The machine has a 300,000 pound capacity and is servo-controlled. The load
accuracy is better than 0.3 percent at all load ranges. A spherical head mount is incorporated in the frame to allow uniform loading and to compensate for minor misalignment.

Moisture content was calculated as a percent of the dry weight of the sample. The moisture sample was taken from the interior of the unconfined compressive strength sample immediately after testing, so slight drying of the exterior skin of the core did not influence the moisture measurements.
CHAPTER V
RESULTS

This study was based on 385 unconfined compressive strength tests, 1240 axial point-load tests, 90 diametral point-load tests, and 1240 indirect tensile strength tests. Approximately three point-load and three indirect tensile strength tests were run for each unconfined compressive strength test. Nearly 75 percent of the test samples were gray shales and therefore constituted the best statistical base of this study.

Diametral point-load testing was performed early in the investigation on 30 test sets of gray shale. Additional tests performed on the 30 sets included axial point-load, indirect tensile, and unconfined compressive strength tests. The diametral point-load indices calculated with Hassani, Scoble, and Whittaker's (1980) equation and Brook's (1977) equation indicated an average of standard deviation values of over 50 percent. The range of the standard deviation values was 23.5 to 81.3 percent. The relationship between Brook's (1977) $T_{500}$ index, and Hassani, Scoble, and Whittaker's (1980) $I_{50}$ index and corresponding unconfined compressive strengths had correlation coefficients of .02 and .05 respectively. The averaged multiplying factor used times $I_{50}$ to estimate unconfined compressive strength for
Table 2. Examples of indices of anisotropy for gray shale samples.

<table>
<thead>
<tr>
<th>Diametral Point Load</th>
<th>Axial Point Load</th>
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<tbody>
<tr>
<td># of Samples</td>
<td>Avg. (MPa)</td>
</tr>
<tr>
<td>7</td>
<td>.182</td>
</tr>
<tr>
<td>3</td>
<td>.055</td>
</tr>
<tr>
<td>5</td>
<td>.156</td>
</tr>
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<td>2</td>
<td>.115</td>
</tr>
<tr>
<td>4</td>
<td>.492</td>
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<tr>
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<tr>
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<td>.560</td>
</tr>
<tr>
<td>2</td>
<td>.084</td>
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<tr>
<td>2</td>
<td>.622</td>
</tr>
<tr>
<td>2</td>
<td>.12</td>
</tr>
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</table>

Avg. 23.9
diametral testing of these shales was 140.8 with a standard deviation of 125 percent. Brook's (1977) $T_{500}$ index had an average multiplying factor of 82.1 with a standard deviation of 116 percent. These multiplying factors for diametral testing did not come close to the previous values of 16 by Reichmuth (1968), 24 by Broch and Franklin (1972), 12.5 by Brook (1977), 29 by Hassani, Scoble, and Whittaker (1980), and 20 to 22 by Read, Thornton, and Regan (1980).

Bedding plane anisotropy seemed to be the controlling factor for inconsistent diametral point-load test results. Therefore, the index of anisotropy was measured. An index of anisotropy, $I_a$, is defined as the ratio of point-load strength in the strongest direction divided by the strength in the weakest direction. Test results of this study determined the index of anisotropy, $I_a$, (using Broch and Franklin [1972] formulas) to be 2.0 to 71.9 (Table 2) for the gray shales tested. The weakest strength was calculated when the point-load test was applied diametrically and the samples separated along the bedding plane. These high values for the index of anisotropy showed that these shales had anisotropy up to 12 times greater than materials previously tested by a point-load apparatus. Previously published $I_a$ values in conjunction with point-load testing were 1.88 by Broch and Franklin (1972), and 1.05 to 5.7 by Greminger (1982).
Diametral point-load results from highly anisotropic materials cannot be used to estimate the unconfined compressive strength. This means that diametral testing does not provide a satisfactory correlation with the unconfined compressive strength of shale samples from cores drilled vertically in the Illinois Basin, because failure occurred along bedding planes and not across the fabric of the rocks.

Pells (1975), and Read, Thornton, and Regan (1980) had similar experiences of diametral point-load tests separating along bedding planes and concluded that a measurement of the stress needed to separate a sample along the bedding plane was not a measurement of the rock property meaningful to the unconfined compressive strength.

**Axial Point-Load Testing**

An alternative method of performing the point-load test is loading the samples axially, perpendicular to the bedding planes (Figure 1). The use of the point-load test in the field needs a formula that can incorporate dimension changes of the samples and produce a constant index value.

Reichmuth's (1963, 1968) (PLI), Hassani, Scoble, and Whittaker's (1980) ($I_{50}$), and Brook's (1977, 1980) ($T_{500}$) point-load indices were obtained for a set of 120 axial point-load tests on samples with varying length-to-diameter ratios. The point-load indices were
compared to the unconfined compressive strength and produced correlation coefficients of .61, .61, and .64, respectively. The modified Reichmuth's formula gave various index values for changes in sample length. Thin samples gave high index values and thick samples gave low index values. The Hassani, Scoble, and Whittaker's $I_{50}$ formula produced indices for individual samples in a sample set with 3.3 times more standard deviation than Brook's $T_{500}$ formula.

These point-load formulas used to evaluate the 120 point-load samples with various length-to-diameter ratios showed similarly low correlations with the unconfined compressive strength. The $T_{500}$ formula showed only a slightly higher correlation coefficient but had a much lower standard deviation among individual point-load samples of a sample set, making it the best formula of the three.

These three point-load formulas were also compared for all the gray shale samples (Table 3). Overall, the $T_{500}$ formula produced consistently slightly better correlations with the unconfined compressive strength as compared to the other two indices. Therefore, the rest of the study concentrated on the $T_{500}$ index.

The high standard deviation among individual point-load samples produced by Reichmuth's (1963, 1968) and Hassani, Scoble, and Whittaker's (1980) formulas did not have a large impact on the correlation to the unconfined compressive strength, since 3 to 6 individual point-load test results
were averaged to produce one point-load index value. The formulas producing high standard deviations among individual point-load samples can cause problems when only a few point-load samples are available for testing.

**Axial T\(_{500}\) Index**

This study has shown that the point-load testing of highly anisotropic materials produced the best correlation with the unconfined compressive strength when the T\(_{500}\) formula (Brook [1977, 1980]) was used on the test performed in the axial direction. Thus, axial point-load tests evaluated by using Brook's (1977, 1980) formula were performed on a wide range of sample sizes from individual rock units. The log-log plot of load at failure versus the cross-sectional area had slopes from .70 to 1.1 with an average value of .84 for gray shales (Figure 5).

The changes in slope and load-at-failure intercept among the five data sets on Figure 5 were probably caused by slight material differences. The samples for each data set were taken from a section of core 1 to 1.5 feet long. Each data set had nearly linear relationships, with very little scatter, indicating similar material properties over the 1- to 1.5-foot length of core sampled. The range of slopes was larger than Brook (1977), but the average slope value was close to the one Brook found. No improvement in the correlation of the T\(_{500}\) index with the unconfined
Fig. 5. Log-log plot of load at failure versus cross-sectional area of axial point-load samples. Slope value of line is shown. All samples are gray shale.

Data shown has been divided into two graphs because of data overlap.
Table 3. Correlation coefficients between estimated and measured unconfined compressive strength based on three various point-load indices.

<table>
<thead>
<tr>
<th>Number of Samples</th>
<th>Lithology per Project Sites</th>
<th>$I_{50}$</th>
<th>PLI $^2$</th>
<th>$T_{500}$ $^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>39</td>
<td>Energy Shale</td>
<td>.76</td>
<td>.69</td>
<td>.76</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>.95</td>
<td>.88</td>
</tr>
<tr>
<td>40</td>
<td>Energy Shale</td>
<td>.61</td>
<td>.64</td>
<td>.63</td>
</tr>
<tr>
<td>25</td>
<td>Francis Creek</td>
<td>.87</td>
<td>.86</td>
<td>.88</td>
</tr>
<tr>
<td>21</td>
<td>Farmington Shale</td>
<td>.85</td>
<td>.84</td>
<td>.86</td>
</tr>
<tr>
<td>100</td>
<td>Canton Shale</td>
<td>.62</td>
<td>.74</td>
<td>.86</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>.97</td>
<td>.98</td>
</tr>
<tr>
<td>23</td>
<td>Lawson Shale</td>
<td>.55</td>
<td>.53</td>
<td>.57</td>
</tr>
</tbody>
</table>

$^1$Hassani, Scoble and Whittaker (1980)

$^2$Reichmuth (1963, 1968)

$^3$Brook (1977, 1980)
compressive strength was gained with the use of the individual slope values as compared to the mathematically derived .75 value. The .75 slope value was therefore used in the $T_{500}$ equation for this study.

Each lithology tested produced a different average multiplier to be used times the $T_{500}$ index to estimate the unconfined compressive strength. The averaged multiplier value for the tested claystones, gray shales, limestones, black shales, and coals were 12.8, 14.3, 15.8, 22.9, and 26.7 respectively. The multipliers were based on limited numbers of samples, except for the gray shale samples. The gray shale data were based on 263 sample sets. The other test data were based on an average of 30 sample sets per lithology. All the multipliers were larger than the 12.5 value presented by Brook (1977, 1980).

Figure 6 shows the relationship between the $T_{500}$ index and the unconfined compressive strength of 263 sample sets of gray shale. The correlation coefficient for the data was .80.

A comparison of the $T_{500}$ index and the unconfined compressive strength of the same stratigraphic unit (Energy shale) across the state showed a good correlation of .88. This unit represented 90 sample sets from all the gray shale test samples. The $T_{500}$ index and unconfined compressive strength relationship for this single unit is shown in Figure 7. The averaged multiplying factor used with the
Fig. 6. Axial point-load index value of $T_{500}$ versus unconfined compressive strength for all gray shale samples.
Fig. 7. Axial point-load index value of $T_{500}$ versus unconfined compressive strength for Energy Shale samples.
The $T_{500}$ index to estimate the unconfined compressive strength was 15.9 for the Energy shale samples. The range of the averaged multiplying factors for the Energy shale at each project site across the state was 14.4 to 17.4.

The best correlation was found between the axial point-load index of Brook (1977, 1980) and the unconfined compressive strength. This correlation was not as good as previous published correlations of .92 (Miller [1965]), .94 (D'Andrea, Fischer, and Fogelson [1965]), .88 (Broch and Franklin [1972]), .95 to .85 (Hassani, Scoble and Whittaker [1980]) and .97 (Read, Thornton and Regan [1980]). However, none of these previous studies worked with highly anisotropic materials.

The shales had grain size and material variations irregularly spaced generally parallel to bedding planes throughout the cores. The unconfined compressive strength index was an averaged strength of multiple thin layers of material with slight physical differences. The point-load test samples had physical variations similar to the unconfined samples, but since they were much smaller in length, they may not have had exactly the same compositional differences as the longer unconfined compressive strength test samples. Testing of multiple axial point-load samples (Figure 5) showed only small strength variations as indicated by the minor amount of scatter along the slope lines. This variation was further reduced by averaging the
point-load index values from multiple (3 to 6) samples. This problem of small compositional variations cannot be entirely overcome when testing finely bedded materials, such as shale.

The change in the multiplying factor between lithologic rock types and among the same stratigraphic unit at different locations showed that the multiplying factor for each lithologic unit needs to be characterized for individual locations. If this can not be performed because of a lack of samples, then the multipliers presented in this study would be the better approximations in contrast to previously published multipliers.

\[ T_{500} \text{-Moisture Content Composite Index} \]

Another simple index used to estimate unconfined compressive strength was the moisture content. Figure 8 shows the semi-log plot of logarithm of moisture content versus the unconfined compressive strength for all the Energy Shale samples. The correlation coefficient for all the Energy Shale samples was .90. This relationship was better than the use of the \( T_{500} \) index to estimate the unconfined compressive strength. The relationship of logarithm of moisture content versus unconfined compressive strength for all the gray shales showed a correlation coefficient of .76. This was slightly lower than the correlation coefficient of .80 found for the relationship of
ENERGY SHALE SAMPLES

\[ Q_u = -66.3 \times \log W\% + 71.33 \]

\[ r = .90 \]

Fig. 8. Semi-log plot of moisture content versus unconfined compressive strength for Energy Shale samples.
the $T_{500}$ index and the unconfined compressive strength for all the gray shale samples.

The use of these indices for estimation of the unconfined compressive strength of the other lithologies showed the moisture content index generally had the better correlation. The comparison of correlations for the moisture content index versus the $T_{500}$ index for the other lithologies were: claystone: .67 to .44, limestone: .59 to .46, and black shales: .78 to .49. The correlations for these lithologies were based on a limited number of samples and should be considered preliminary results.

The estimation of the unconfined compressive strength by using only the moisture content index is not advised. For several test sites the bedrock strength was reduced through micro-fracturing caused by local movements or tectonic activities. These small movements late in the history of the bedrock materials did not change the moisture content. The moisture content did not change since micro-fracturing probably did not produce significant new void spaces as compared to the existing void space already in the rock. This produced a rock with lower strength compared to its moisture content index. A mechanical test such as the point-load index or unconfined compressive strength should be used to first characterize the materials so that various indices, such as moisture content, can be used with confidence.
Multiple indices may be combined into one formula to produce better estimates of the unconfined compressive strength. In this study the two indices of point-load and moisture content were combined into one formula. For example, the predicted unconfined compressive strength of the Energy shale samples using this method had a correlation coefficient of .92 (Figure 9) when compared to the measured unconfined compressive strength. This correlation was slightly higher than the .88 and .90 correlation coefficients produced by the use of the $T_{500}$ index and moisture content index respectively. The combination of the two indices did not always produce a better correlation coefficient as compared to using one of the indices separately (Table 4). This problem may have been a function of the number of sample sets tested, since testing twenty or more sample sets yielded better correlations with the combined indices. The averaged correlation coefficient between the measured and predicted unconfined compressive strength using the equations with combined indices for all the lithologies was .896. D'Andrea, Fischer, and Fogelson (1965) combined eight indices to produce a correlation coefficient of .986 for a comparison between predicted and measured unconfined compressive strength.
Fig. 9. Unconfined compressive strength predicted by $T_{500}$ and moisture content indices, versus the measured unconfined compressive strength.
Table 4. Correlation coefficients between estimated and measured unconfined compressive strength based on various indices.

<table>
<thead>
<tr>
<th>Number of Samples</th>
<th>Lithology per Project Sites</th>
<th>( T_{500} )</th>
<th>( W_Z )</th>
<th>( T_{500} + W_Z )</th>
</tr>
</thead>
<tbody>
<tr>
<td>39</td>
<td>Energy Shale</td>
<td>.765</td>
<td>-.771</td>
<td>.856</td>
</tr>
<tr>
<td>10</td>
<td>Energy Shale</td>
<td>.894</td>
<td>-.837</td>
<td>.872</td>
</tr>
<tr>
<td>40</td>
<td>Energy Shale</td>
<td>.632</td>
<td>-.686</td>
<td>.686</td>
</tr>
<tr>
<td></td>
<td>All Energy Shale Samples</td>
<td>.876</td>
<td>-.900</td>
<td>.921</td>
</tr>
<tr>
<td>25</td>
<td>Francis Creek</td>
<td>.882</td>
<td>-.866</td>
<td>.921</td>
</tr>
<tr>
<td>21</td>
<td>Farmington Shale</td>
<td>.863</td>
<td>-.573</td>
<td>.853</td>
</tr>
<tr>
<td>100</td>
<td>Canton Shale</td>
<td>.750</td>
<td>-.543</td>
<td>.762</td>
</tr>
<tr>
<td>4</td>
<td>Canton Shale</td>
<td>.981</td>
<td>-.996</td>
<td>.993</td>
</tr>
<tr>
<td>23</td>
<td>Lawson Shale</td>
<td>.573</td>
<td>-.493</td>
<td>.615</td>
</tr>
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</table>

| 13                | St. David                   | .488           | -.639   | .615             |
| 8                 | Brereton                    | .330           | -.658   | .420             |
| 8                 | Bankston Fork               | .766           | -.713   | .894             |
|                   | All Limestone Samples       | .461           | -.587   | .662             |

| 13                | All Samples                 | .093           | -.775   | .130             |

| 19                | All Samples                 | .436           | -.679   | .557             |

| 15                | #5 Coal                     | -.177          |         |                  |
| 11                | #5 Coal                     | .867           |         |                  |
| 5                 | #6 Coal                     | .017           |         |                  |
| 14                | Other seams combined        | -.149          |         |                  |
Vertical Preferred Weakness Planes

The point-load test can be used to determine other characteristics of rock cores. Several parallel lines were drawn the length of sections of core to indicate relative orientation of core samples trimmed into individual test samples. This allowed comparisons to be made between different directions of the vertical planes of weakness when long sections of core were available for testing. Many of the composites of tested core sections of this study showed that preferred weakness planes (Figure 10) did exist. If the cores had no preferred weakness planes, a composite of the fracture planes would have shown random directions. McWilliams (1966), Arscott and Hackett (1968), Friedman and Logan (1970), Peng and Okubo (1978), Miller (1979), Miller, Bauer, and Johnson (1980), and Lajtai (1980) used axial point-load testing with orientated core to determine preferred vertical weakness planes. McWilliams (1966) and this study used arbitrarily orientated core.

The directions of the preferred fracture planes from axial point-load testing were also compared to the failure plane directions of the unconfined compressive strength samples. Core samples collected from areas devoid of multiple tectonic events (a majority of the samples for this study) showed good correlation between the directions of failure planes between the two tests (Figure 11). Roughly 75 percent of the tested sample sets showed the fracture
Fig. 10. Typical composite of axial point-load fracture directions of 16 limestone samples from 5 foot thick unit showing preferred direction of failure.
Fig. 11. Histogram of frequency versus angle between the strike of the unconfined compressive strength and the strike of the axial point-load fracture planes.
planes of the two tests within 20 degrees of each other. These observations indicated that preferred weakness planes controlled the failure plane direction of the compressive strength tests and the point-load tests. Friedman and Logan (1970) showed that the strike of the failure planes between axial point-load tests and triaxial compressive strength tests of sandstones had the same direction.

In this study, a preferred vertical weakness plane in the cores of gray shale defined where and in which direction the axial cracks developed, and therefore controlled the strike of the shear plane development in the unconfined compressive strength samples.

The prediction of the direction of shear plane development in the unconfined compressive strength samples by the axial point-load test did not work for samples from areas where multiple tectonic movements had taken place. Figure 12 shows the frequency versus the degrees between the strike of the fracture plane directions of the axial point-load test and the unconfined compressive strength tests from samples taken from an area with movements in multiple directions. The samples were taken from an area that was near the intersection of two fault systems and a dome, the Wabash and Cottage Grove Fault Systems and Hicks Dome. All the movements associated with these structures had probably produced weakness planes with multiple directions, none more dominant than the others.
Fig. 12. Histogram of frequency versus angle between the strike of the unconfined compressive strength and the strike of the axial point-load fracture planes for an area subjected to multiple directional movements.
Bock (1979) stated that when compressing brittle material, axial cracks occurred before the ultimate strength was obtained. These axial cracks can define where the shear plane developed in the compressed brittle material (Figure 13). Both the axial cracks and the shear plane(s) had the same strike.

**Point-Load Testing and "Brazilian" Indirect Tensile Tests**

The orientation of the preferred weakness planes had an impact on the indirect tensile strength testing of samples. The weakest tensile strength was measured when the preferred fracture plane intersected the two platens which applied the load (Figure 14). This produced tension forces across the weakness planes and the sample failed at a low tensile value. When the load was applied perpendicular to the preferred weakness plan, the tension forces were parallel to the weakness plane (Figure 15) and the largest tensile strength values were produced. This study found a ratio of strongest to weakest indirect tensile strength values ranging from 1.02 to 2.08 for gray shales and limestones showing good preferred weakness planes. The average value was 1.20. This means that if the axial point-load test was not used to define the preferred weakness plane then arbitrary directional testing of the indirect tensile strength may have produced tensile strengths up to 2.08 times stronger than its weakest value.
Fig. 13. Diagram of the direction of shear plane development in an unconfined compressive strength sample controlled by axial cracks (from Bock, 1979).
Fig. 14. Indirect tensile test with load applied along preferred fracture planes, producing the weakest measured tensile strength.

Fig. 15. Indirect tensile test with load applied perpendicular to preferred fracture planes, producing the strongest measured tensile strength.
The indirect tensile strength may be estimated with the use of the point-load test. This study found the average multiplying factor to be 1.34 for gray shales, 1.1 for coal, 1.02 for limestone, 1.3 for claystone and 1.7 for black shale. These values were averages which, in the case of the black shales, limestone, coal and claystones, were based on a limited number (20 to 30) of samples for each lithology. The multiplying factors were averages of indirect tensile tests performed in all directions relative to the preferred weakness plane. All had poor to fair correlation coefficients ranging from .20 to .84 for the relationship of $T_{500}$ point-load index to the indirect tensile strength.

The relationship between $T_{500}$ point-load index and the weakest indirect tensile strength produced by loading the sample along the preferred weakness plane was investigated for 40 samples of gray shales. The results showed that the averaged ratios of indirect tensile strength to $T_{500}$ was 1.02, or approximately 1. Therefore, the $T_{500}$ point-load index approximately equals the indirect tensile strength in the weakest direction. This corresponded with Brook's (1980) findings. These findings for the indirect tensile strength - $T_{500}$ point-load index were approximations and only preliminary data based on limited tests.

Preferred vertical weakness planes in a core form the plane where the weakest tensile strength is found for
"Brazilian" indirect tensile testing. The existence of this weakness plane in highly anisotropic rocks can only be easily defined by axial point-load testing. Diametral point-load testing of highly anisotropic rocks generally fails along the bedding planes and will not define the vertical weakness plane.

Previous researchers (Friedman and Logan [1970], Peng and Okubo [1979], Lajtai [1980], Miller, Bauer and Johnson [1980]) showed the averaged indirect tensile strengths in the strong direction (loads perpendicular to weakness plane) 1.13 to 1.64 times stronger than the averaged values in the weak direction (loads applied along weakness plane). The axial point-load test should be used to define the preferred weakness plane before indirect tensile tests or beam tests are performed. Then the samples may be tested in their weakest configuration to insure the greatest margins of safety. McWilliams (1966) and Lajtai (1980) used axial point-load tests to develop multiplying factors of 2.22 and 3.76 respectively to be used times the point-load index to estimate the indirect tensile strength. Brook (1980) found that the $T_{500}$ point-load index value approximated the indirect tensile strength value.
CHAPTER VI

CONCLUSIONS

1. Diametral point-load testing cannot be used to estimate the unconfined compressive strength of highly anisotropic materials. Bedding plane separation produced by diametral point-load test did not characterize the strength of the rock material in a manner that was meaningful to estimating the unconfined compressive strength. Since Illinois Pennsylvanian sediments are nearly flat lying, vertical cores from these sediments should not be used with the diametral point-load tests to accurately estimate the unconfined compressive strength.

2. The best estimation of the unconfined compressive strength with point-load testing for highly anisotropic sedimentary rocks was produced with axial point-load testing evaluated by the $T_{500}$ index. The $T_{500}$ formula incorporates the cross-sectional area of the sample in the calculation. This allows field point-load testing devices to be used on pieces of core of sedimentary rocks which can be easily separated along bedding planes to produce field test samples of varying sizes.

3. The other point-load test indices evaluated in this study showed nearly as good a correlation with the
unconfined compressive strength as the $T_{500}$ index but the standard deviation of individual point-load test values for the other indices was over three times as much as those for the $T_{500}$ index. Good correlation of the other indices with the unconfined compressive strength was produced in this study by averaging 3 to 6 axial point-load values which reduced the effect of the high standard deviation. When only a few point-load samples are available, these indices may produce improper results.

4. Multiplying factors to be used with the $T_{500}$ index to estimate the unconfined compressive strength were developed for each lithology. Good estimates of the unconfined compressive strength were also produced using the moisture content as an index. The estimate of the unconfined compressive strength by the combination of the axial point-load index and moisture content produced the best correlations with the lithologies having a large number of tested samples.

5. The $T_{500}$ index value approximates the indirect tensile strength value when the indirect tensile strength test is performed by loading the specimen along the plane of preferred vertical weakness - a plane that often is found in flat-lying sedimentary rocks. "Brazilian" indirect tensile strength values depend on the direction of the preferred weakness plane and the direction of tensile stresses produced during testing. Without performing axial
point-load testing to define the preferred weakness plane, indirect tensile and beam strength test values may be as much as twice as strong as the values from the weakest tensile strength direction.
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VITA

Robert Alan Bauer was born in Chicago, Illinois, on January 19, 1952. He attended the University of Illinois at Chicago Circle from 1970 to 1976, receiving a Bachelor of Science Degree in Geological Science in 1976. He attended the University of Illinois at Urbana-Champaign from 1978 to 1980.

He briefly worked at the McHenry County Planning Commission before accepting a staff position in 1976 at the Illinois State Geological Survey. He has been involved with the fields of Engineering-Geology, Subsidence, Coal Mining and Rock Mechanics.

He holds membership in a number of professional societies, including the Society of Mining Engineers, the International Society for Rock Mechanics, and Association of Engineering Geologists.

He has authored or co-authored the following papers and presentations:


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