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EVALUATION OF THE POINT LOAD TEST FOR ROCK

MISSOURI HIGHWAY AND TRANSPORTATION DEPARTMENT
FEDERAL HIGHWAY ADMINISTRATION

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STUDY 79-2

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ABSTRACT

NX cores from Missouri's major sedimentary rock types were tested by point load in both axial and diametral orientations and by unconfined compression for correlation purposes. A similar series of tests was performed on homogeneous mortar specimens. Data was analyzed to establish correlations between point load and unconfined test results and to determine the relative consistency of point load indices determined for both test orientations by various methods of computation. The overall correlations developed do not agree well with those reported in the literature. Methods of computing the point load index were also evaluated by the anisotropic index values computed from tests on mortar. Dividing force by the area of the failure surface was judged superior to the conventional method of computing point load index in which force is divided by the square of the distance between the platen points. Formulae were developed empirically for adjustment of axial indices to compensate for variations in length-diameter ratios. When computed by optimum means, the point load test was superior to the unconfined test in consistency of results obtained on NX rock cores but was somewhat less consistent in the comparative tests on mortar.

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LIST OF ABBREVIATIONS AND SYMBOLS

A	-	area
D	-	diameter
d	-	distance between loading points
I_a	-	anisotropic index
I_s	-	point load index
L	-	length of specimen
P	-	force
NX	-	a size of rock core with diameter of 2.04 ± 0.04 inches
Qu	-	unconfined compressive strength
r	-	correlation coefficient
S.D.	-	standard deviation
V	-	coefficient of variation

INTRODUCTION

This study was initiated to evaluate the reproducibility of the point load test and its relationship to unconfined compressive strength for various types of sedimentary rock found in Missouri.

The unconfined compression test, the conventional method of obtaining the strength of rock, is slow and expensive, requiring laboratory personnel and equipment to perform. The point load test has been gaining acceptance in the field of rock mechanics as a relatively quick and inexpensive field procedure. Advantages claimed are the ability to field test either cores or fragments of rock without preparation, at in-situ moisture content, with light weight and inexpensive equipment. Sealing, transporting, storing, soaking, sawing and polishing of core specimens are thereby avoided.

The theory of the point load test is to induce, through compressive loading between two pointed platens, internal tensile stresses sufficient to rupture the specimen. As tensile and compressive stresses are related in elastic materials according to Poisson's ratio, it should be possible to estimate the unconfined compressive strength from the point load required for failure.

In that the point load test is related to various other tensile tests for concrete and rock, the direct origins of the point load test are somewhat uncertain. In one of the earliest references available, a Russian, Protodyakonov (1), reported testing irregular rock specimens roughly shaped by hammering. Tensile strength was estimated by dividing the rupture load by the 2/3 power of the volume of the specimen as measured by the sand-displacement method. D'Andrea, Fischer and Fogelson (2), with the U.S. Bureau of Mines, concluded that, of various rock properties studied, only the point load tensile strength could be used alone to estimate compressive strength with fair accuracy. Hiramatsu and Oka (3) analyzed point load stresses theoretically and experimentally, including photo-elastic model tests which confirmed the minimal effects of surface geometry on tensile stress distributions.

Perhaps the most comprehensive and definitive treatments of the point load test as currently practiced are those of Broch and Franklin (4) and Bieniawski (5,6). Broch and Franklin proposed a standard test method and equipment requirements which are essentially those used in this study. Bieniawski has proposed a general relationship for unconfined compressive strength, Qu, and point load index, I_s, for NX cores, where $Qu = 24\ I_s$.

CONCLUSIONS

1. The unconfined compression test produced more consistent strength data than did the point load test in comparative testing of homogeneous mortar specimens. However, the unconfined produced somewhat less consistent results in comparative tests of cores of natural sedimentary rocks after the point load index was computed by means determined to produce optimum results.

2. The P/A (force divided by area) method appears to be the most rational method of calculating point load index since only this method produced an anisotropic index of approximately 1.0 with presumably homogeneous mortar specimens.

3. Restrictions proposed by others for the lengths of axial point load test specimens may be relaxed by calculating the index by a height adjustment factor which relates the test specimen to a common length based on a length to diameter ratio empirically determined to produce an anisotropic index of unity for mortar. On this basis the range of length to diameter ratios considered, 0.6 to 1.1, was determined to produce results that exceeded the consistency of diametral point load tests and approached that of the unconfined test.

4. Widely varying correlations of unconfined compressive strength to point load index were found by rock type with some rock types showing no correlation. The correlation for all data combined is markedly superior to that for any individual rock type. The overall correlations established for unconfined compression and point load testing do not agree well with that reported in the literature. This may be a consequence of the relatively limited range of strengths provided by the sedimentary rocks tested. Better agreement is probable if data from other types of rocks with higher average strengths were included.

IMPLEMENTATION

The point load test should not be used alone but as a supplement to the unconfined compression test. So used, it should permit a reduction in the number of unconfined tests required and should permit assessment of anisotropic properties where this may be relevant to the problem under investigation.

For use by designers, point load index values should be interpreted by the investigator in terms of estimated equivalent unconfined compressive strengths. This estimate should be based upon the apparent relationship revealed by the complementary test procedures for the particular formation and rock type being investigated, supplemented by the relationships developed and reported here and elsewhere.

The point load test procedures proposed by Broch and Franklin (4) should be followed except that the index should be calculated by the P/A method with axial test indices based upon length adjustment factors proposed in this report. The length to diameter ratios permissible for axial testing may encompass a range of 0.6 to at least 1.1.

SCOPE

A commercially available point load test apparatus was purchased and used in a testing program on NX cores from the major types of sedimentary rock found in Missouri. Point load testing was performed in both axial and diametral orientations with unconfined compression tests performed for correlation. A similar series of tests was performed on homogeneous specimens made from mortar. Point load test data was analyzed to establish correlations to unconfined compressive strength data and to determine the relative consistency of point load indices determined for axial and diametral test orientations by various methods of computation. Methods of computing the point load index were evaluated by the anisotropic index values computed from tests on mortar. Empirical formulae were developed for adjustment of the axial index to compensate for variations in the length-diameter ratio.

EQUIPMENT AND PROCEDURE

Equipment

A Soiltest Model RM-730 point load test apparatus was obtained for use in this study. This apparatus uses an Enerpac Model RC-102, 10 ton capacity hydraulic ram mounted inside a reaction frame. Pressure is provided by a hand operated Enerpac Model P-39 pump. Load is indicated by a 10,000 pound capacity Helicoid Type 430 gauge calibrated to read directly in pounds in 200 pound increments and equipped with a manually reset following pointer. Indicated gauge accuracy is ±½of 1% of scale range.

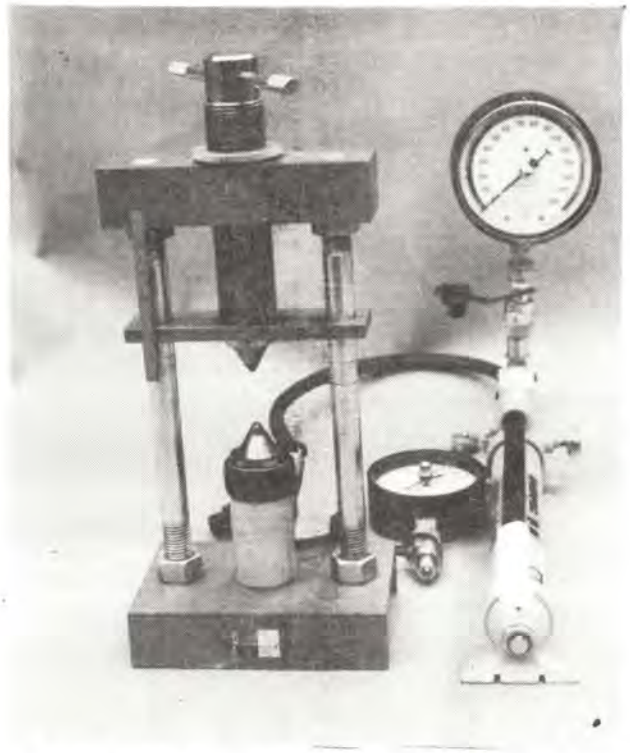

Figure 1. Soiltest Model RM-730 point load test apparatus.

Several changes were made to the equipment as received. The chrome plated points were not constructed of hardened steel nor ground to a radius of 5 mm as recommended by Broch and Franklin (4). The points were replaced with surface hardened points made from high carbon steel ground to the desired radius. The new points performed without deformation during all of the sedimentary rock testing. However, one of the few igneous rock (granite) cores tested was sufficiently hard to rupture the surface hardening.

Other changes included a snubber placed in the hydraulic circuit to minimize gauge backlash at failure. A lower load range gauge (0-1500 psi, ¼ of 1% of full range accuracy) was also ordered since it was found that most of the sedimentary rock specimens under test did not require more than a third of the capacity of the original gauge. Both gauges were fitted with quick connects. Unfortunately, the second gauge was not received until most of the testing program was completed. Relative insensitivity at low loads was therefore a drawback of the equipment as used.

Calibration of both gauges was accomplished using proving rings within the reaction frame with the points removed. A matter of concern during testing of very hard specimens, which was never satisfactorily resolved, was some apparent eccentricity as the points tended to diverge from a common axis along the curvature of cores while under heavy loading. It was suspected that this could influence the load gauge calibrations. It is believed that a more rigid, warp-free design for the reaction frame could help this problem.

Test Procedure

The procedures proposed by Broch and Franklin (4) for the point load test define specimen orientation with respect to the laminations or bedding planes encountered in cores of most sedimentary rock. By testing the rock core both normal and parallel to such planes of weakness it is possible to determine an anisotropic index, I_a, defined as the ratio of the strength indices obtained in the strongest and weakest directions. An anisotropic index of 1.0, the minimum factor possible, implies a perfectly homogeneous material without planes of weakness.

The most common method of point load testing is referred to as diametral testing where the load is applied parallel to the bedding planes and along the core diameter. For core specimens of most sedimentary rocks, the load is applied normal to the coring direction, resulting in a circular or slightly elliptical failure plane depending upon the exact orientation of the bedding planes. A length of core 1.4 times the diameter is recommended for this test orientation.

Axial testing is a second method in which the load is applied normal to the bedding plane orientation and at right angles to the direction of load application for diametral testing. This usually requires testing portions of broken core remaining from the initial diametral testing. The core length is recommended to be 1.1 ± 0.05 times the core diameter, a very tight limitation which can only sometimes be achieved without sawing and sample preparation.

An alternate procedure for testing of irregular lumps of rock also requires the load application to be referenced to the orientation of bedding planes. The accuracy of this method is reported by Protodyakonov (1) as being comparable to unconfined

compression testing of cores if a large number of such lump test results can be averaged. Test specimens are recommended to be egg shaped lumps, with maximum dimension about 1.4 times the shorter dimension and with the shorter dimension parallel to bedding laminations. This may be a difficult type of specimen to find or shape in the quantity required.

The strength value determined by point load testing is referred to as the point load index, I_s, and is commonly calculated as $I_s = P/d^2$ where P is the force required to rupture the specimen and d is the distance between the platen points. For NX cores tested diametrally, this distance was generally found to be 2.04 ± 0.04 inches.

COMPARATIVE TESTING OF PREPARED MORTAR SPECIMENS

To provide a comparison of the point load test to the unconfined compression test without the complications of anisotropy, samples were prepared of sand-cement mortar to provide homogeneous test specimens in sufficient number to establish statistical relationships. Mortars were prepared with Portland Cement contents of 10, 20 and 30 percent, using a well graded concrete sand. The specimens were formed in 2.5 inch diameter waxed tubes, moist cured for 28 days and sawed into 5 inch lengths for both unconfined compression and diametral point load testing. Axial testing was performed on remnants from the sawing and diametral testing. All test results are summarized in Table 1.

TABLE 1

Summary of Tests on Mortar

	Unconfined Compression Tests, Qu.			Point Load Test Index, $I_s = P/d^2$						
				Diametral			Axial			
Cement Content	No.of Tests	Qu, (Ave)	V%	No.of Tests	I_s (Ave)	V%	No.of Tests	I_s (Ave)	V%	I_a
10%	16	588	17	50	47	45	25	126	26	2.68
20%	16	2,497	12	42	224	29	29	404	35	1.8
30%	16	3,412	14	42	323	12	34	502	26	1.55

NOTES: All test units in p.s.i.
V = Coefficient of variation
I_a = Anisotropic index
P/d^2 = Load divided by distance between points

For all test methods, coefficients of variation (V) were calculated. The coefficient of variation is the quotient of the standard deviation and average strength times 100 and indicates that percentage of the average strength, plus or minus, within which about 68 percent of the test results fall. In that these coefficients are a measure of the variability of the strength, by whatever means measured, they were used for comparison of relative repeatability between test methods, test orientations and methods of calculation of index values.

Sixteen unconfined compression tests at each cement content reveal a linear increase in strength versus cement content. Coefficients of variation calculated for the three series of tests (10, 20 and 30 percent cement content) were 17, 12 and 14 percent respectively. Comparable testing using point load procedures also indicate a linear increase of strength with cement content but with generally larger coefficients of variation. An

average of 2.8 diametral point load tests were performed for each unconfined compression test and an average of 1.6 axially oriented tests for each unconfined compression test. In the diametral orientation, coefficients of variation in point load index values of 45, 30 and 12 percent were calculated. Significantly, the coefficients for the diametral point load test and the unconfined compression test were about equal at the highest strength level, for that set with 30 percent cement content.

A correlation equation expressing the relationship of unconfined compression to diametral point load test data was calculated, using the least squares method, as Qu, psi = 10.5 I_s - 42. Adjusted for an origin through zero, Qu would approximate 11 I_s. It should be noted that Broch and Franklin (4) have reported a size effect relationship for correlations of index values to unconfined compressive strength for varying core diameters. Adjusting the 2.5 inch diameter mortar specimens by their procedure to the standard NX core diameter results in increases in I_s of from 8 to 12 percent. This modifies the correlation equation to Qu, psi = 8.7 I_s + 70 or, fitted through zero, approximately 10 I_s. This equation does not compare well to the general relationship reported by Bieniawski (5) for NX rock cores where Qu = 24 I_s.

Of those mortar specimens tested which met Broch and Franklin's (4) suggested L/D ratio requirement of 1.1 ± 0.05, only 3 specimens failed in the plane between the loading points. Failure generally occurred diagonally to the side of the specimen when the specimen length exceeded the minimum suggested length. When this occurred the results were discarded. Because complying with the recommended length-diameter ratio restrictions would also require laboratory sawing, thereby negating field use, it was decided to expand the range of length-diameter ratios tested to include all of the remnants from the diametral testing series. Accordingly, a total of 88 axial tests were performed on specimens having L/D ratios within the range of 0.6 to 1.1.

Correlation of these axial point load indices to unconfined compressive strengths gave the relationship Qu, psi = 7.1 I_s - 360 or, adjusted through zero, approximately 6 I_s. The coefficients of variation for the mortars with 10, 20 and 30 percent cement content were 26, 35 and 26 percent respectively. The average degree of anisotropic strength variation, I_a, between the diametral and axial tests was found to be 2.01. Considering that these specimens were prepared, homogeneous mortar specimens presumably without anisotropy, it was obvious that such axial test results were inconsistent with those of the diametral tests and should be adjusted.

Protodyakonov (1), in his irregular lump tests, attempted a rough approximation of the area of the surface of rupture in calculating tensile strengths. It was decided to similarly compute both diametral and axial index values, as shown on Table 2, as a function of the area, A, using the equation I_s = P/A. While the coefficients of variation for the diametral tests remained unchanged, those for the axial series were reduced to 16, 26

and 18 percent respectively for 10, 20 and 30 percent cement content and the average I_a value became a more logical 1.18. The equation developed for the diametral orientation is Qu, psi = 8.2 I_s - 60 or, struck through zero, approximately 8 I_s. The comparable equation for the axial orientation is Qu, psi = 8.3 I_s - 180 or, adjusted through zero, approximately 7 I_s.

TABLE 2

Summary of Axial Test Data on Mortar With and Without Length Adjustment in Calculating I_s

Cement Content	Point Load Index, $I_s = P/d^2$						Point Load Index, $I_s = P/A$					
	No Adjustment			Length Adjusted to 2.50"			No Adjustment			Length Adjusted to 2.16"		
	I_s	V%	I_a	I_s	V%	I_a	I_s	V%	I_a	I_s	V%	I_a
10%	126	26	2.68	24	54	1.98	88	16	1.4	61	20	1.03
20%	404	35	1.8	232	25	1.04	305	26	1.05	290	17	1.0
30%	502	26	1.55	337	19	1.04	436	18	1.09	423	13	1.06

NOTES: All test units in p.s.i.
A = area of the plane of failure

Further examination of the data for the axial tests indicated variations in the index value with varying sample lengths and suggested that the correlation between diametral and axial testing results could be improved if the axial specimens were adjusted to a common length. This confirms observations by others of variations in index values with varying lengths of sample. As previously noted, Broch and Franklin recommend a relatively constant length-diameter ratio of 1.1 ± 0.05.

Adjusting axial index values to a common 2.49 inch length, based on a 1.0 L/D ratio determined to produce an average anisotropic index of 1.0 for the P/d^2 method of computation, modified the results to those shown in Table 2. A similar adjustment was made to a common 2.16 inch length based on a 0.86 L/D ratio similarly determined for the P/A method of calculation. Length adjusted index equations were developed empirically from the test data with D used as diameter and d as distance between the test points. The equation for the $I_s = P/d^2$ method of calculation is:

$$(1)\quad I_s \text{ (Adjusted), psi} = P/d^2 - \frac{(P/d^2 + 155)\left(0.99 - \frac{d}{D}\right)}{\left(1.37 - \frac{d}{D}\right)}$$

The equation for the P/A method of calculation is:

$$(2)\quad I_s \text{ (Adjusted), psi} = P/A - \frac{(P/A + 110)\left(0.86 - \frac{d}{D}\right)}{\left(1.82 - \frac{d}{D}\right)}$$

Using the second equation, the coefficients of variation for I_s were reduced substantially, to 20, 17 and 13 percent for the three respective cement contents. However, index values were not significantly improved by the first equation which resulted in respective coefficients of variation of 54, 25 and 19 percent. A comparison of index values and coefficients of variation for both methods of calculation using length correction are shown on Table 2.

The equations for correlation of unconfined compressive strength to length adjusted index values are, Qu, psi = 8.2 I_s + 420 (or about Qu = 10 I_s when struck through zero) for the P/d^2 method of calculation and Qu, psi = 8.0 I_s + 80 (or about Qu = 8 I_s when struck through zero) for the P/A method.

Correlation coefficients (r), shown in Table 3, relate average unconfined compressive strengths for each cement content to corresponding average point load indices as determined for the two test orientations and four methods of index calculation. These coefficients are derived by the least squares method as used to determine first degree equations of correlation and reference the fit of the data points to the equation with a value of 1.0 being a perfect fit and a value of zero indicating no correlation.

TABLE 3

Correlation of Qu to I_s for Mortar Using Various Methods of Calculating I_s

Test Orientation and Method of Calculating Point Load Index, I_s		Correlation Coefficient (r) for Qu vs. I_s	Coefficient of Variation V%
Diametral	P/d^2	0.942	20.5
Diametral	P/A	0.952	18.7
Axial	P/d^2	0.796	33.5
Axial	P/A	0.905	21.9
Axial, Length Adjusted	P/d^2	0.933	23.9
Axial, Length Adjusted	P/A	0.955	16.3

The correlation coefficients determined ranged from a low of 0.80 to a high of 0.96. The lowest (poorest) value was for the P/d^2 method of calculating axial index values without length adjustment of the random lengths tested. All other values exceeded 0.90. A value of 0.95 was found for the diametral orientation using the P/A method of calculation and 0.96 for axial tests with P/A index calculation after the random lengths were adjusted to a common value.

Averages of the coefficients of variation, termed standard errors, were calculated as the square root of the quotient of the averaged squared deviations and the number of coefficients, divided by the average of the dependent variable times 100, and are also shown on Table 2. This shows the unconfined compression test to be superior to all forms of the point load test with a standard error of only 15 percent.

Comparable data generated for the point load tests ranged from a low of 16.3 to a high of 33.5 percent. The P/A method of calculation was shown to be superior to the P/d^2 method in both test orientations. The highest value found, 33.5 percent, was for the axial test, on random lengths from 1.2 to 2.7 inches, without length adjustment and using the P/d^2 method of calculation. However, an improved value of 16.3 percent, the lowest or best for the point load test, was determined using length adjustment and the P/A method of calculation. This approaches the value determined for unconfined testing.

NX ROCK CORE TESTS

Test results from NX (2.04 ± 0.04 inches diameter) rock cores are summarized on Table 4. This table was compiled from a total of 145 unconfined compression tests, 360 diametrally oriented point load tests and 437 axially oriented point load tests. The tested cores were visually classified into general rock types as claystone, sandy shale, limey shale, sandstone, limestone, dolomitic limestone, argillaceous limestone, and granite. All cores were selected from routine production from bridge foundation investigations conducted throughout the state of Missouri. The various rock types were not available in equal proportions. The number of tests on each core specimen varied according to suitability of the available samples. Only two igneous rock samples were available and, while the results are reported, no statistical comparisons were made.

The samples selected for axial testing varied in length to diameter (L/D) ratios from 0.6 to 1.2. Average specimen lengths for each of the rock categories are summarized on Table 6. Test results were discarded if the plane of failure did not occur between the loading points.

Tables 4, 5, 6 list, for each rock type, total number of tests, number of core specimens tested, average unconfined compressive strength, average index values, coefficients of variation and anisotropic indices for axial and diametral orientations of point load testing and for the various methods of calculation of the point load index.

TABLE 4

Summary of Test Data on NX Cores Using the P/d^2 Method of Calculating I_s

	Unconfined Compressive Strength, Qu.				Point Load Test, $I_s = P/d^2$								
					Diametral				Axial				
Rock Type	No.of Tests	No.of Spec.	Ave Qu	V%	No.of Tests	No.of Spec.	Ave I_s	V%	No.of Tests	No.of Spec.	Ave I_s	V%	I_a
Claystone	17	14	392	209	53	13	9	158	80	14	93	77	10.7
Sandy Shale	18	10	509	77	40	10	15	133	58	10	46.7	80	3.3
Shale (Limey)	18	7	869	82	41	7	38	163	59	7	77	58	1.3
Sandstone	24	24	3,559	82	101	24	214	112	100	24	283	56	1.3
Dolomitic Limestone	23	23	5,916	60	70	23	317	47	71	24	447	37	1.5
Argillaceous Limestone	5	5	7,179	23	8	5	467	42	11	5	736	86	1.6
Limestone	38	38	9,151	48	43	38	531	38	55	36	823	51	1.6
Granite	2	2	20,548	(N/A)	4	1	1,265	(N/A)	3	2	1,674	(N/A)	1.3
All Data Above (Standard Error, %)	145	123		(62)	360	121		(67)	437	122		(81)	
All Limestone Above (Standard Error, %)	66	66		(46)	121	66		(41)	137	65		(64)	

NOTES: All test units in p.s.i.
% Standard Error equals the square root of the quotient of the squared deviation and the number of coefficients divided by the average of the dependent variable times 100.

TABLE 5

Summary of Test Data on NX Cores Using the P/A Method of Calculating I_s

Rock Type	Unconfined Compressive Strength, Qu. No.of Tests	No.of Spec.	Ave Qu	V%	Point Load Test, I_s = P/A Diametral No.of Tests	No.of Spec.	Ave I_s	V%	Axial No.of Tests	No.of Spec.	Ave I_s	V%	I_a
Claystone	17	14	392	209	53	13	11	163	80	14	55	78	5.0
Sandy Shale	18	10	509	77	40	10	19	129	58	10	50.2	120	2.6
Shale (Limey)	18	7	869	82	41	7	49	159	59	7	78	56	1.6
Sandstone	24	24	3,559	82	101	24	264	112	100	24	246	57	1.1
Dolomitic Limestone	23	23	5,916	60	70	23	399	47	71	24	388	34	1.0
Argillaceous Limestone	5	5	7,179	23	8	5	607	42	11	5	561	70	1.1
Limestone	38	38	9,151	48	43	38	667	42	55	36	723	46	1.1
Granite	2	2	20,548	(N/A)	4	1	1,565	(N/A)	3	2	1,532	(N/A)	1.0
All Data Above (Standard Error, %)	145	123		(62)	360	121		(69)	437	122		(70)	
All Limestone Above (Standard Error, %)	66	66		(46)	121	66		(41)	137	65		(42)	

NOTE: All test units is p.s.i.

TABLE 6

Summary of Axial Test Data Using Length Adjustment in Calculating I_s

Rock Type	No.of Tests	No. of Samples	Tested Lengths, In. Ave	S.D.	Point Load Index, I_s, Length Adjusted: $I_s = P/d^2$ Ave I_s	V%	I_a	I_s = P/A Ave I_s	V%	I_a
Claystone	80	14	1.79	0.28	63	193	7.0	60	76	5.5
Sandy Shale	58	10	1.79	0.08	12	151	-1.3	54	91	2.8
Shale (Limey)	59	7	2.1	0.19	127	58	3.3	116	57	2.4
Sandstone	100	24	1.85	0.18	211	65	1.0	265	57	1.0
Dolomitic Limestone	71	24	1.85	0.24	397	53	1.3	430	39	1.1
Argillaceous Limestone	11	5	1.76	0.45	538	43	-1.2	601	50	1.0
Limestone	55	36	1.81	0.29	641	39	1.0	748	29	1.1
All Data Above (Standard Error, %)	437	122				(59)			(52)	
All Limestone Above (Standard Error, %)	137	65				(45)			(38)	

NOTES: All test units in p.s.i.
S.D. = standard deviation

In addition to the two point load test orientations, the following four methods of calculation of the point load index were evaluated and are summarized in the tables:

(1) Using the distance between the loading points, d, where $I_s = P/d^2$, for both axial and diametral test orientations.

(2) Using the area of the failure surface, A, where $I_s = P/A$, for both axial and diametral test orientations.

(3) For the axial direction of testing, adjusting the length of the specimen to a common L/D ratio of 1.0 as determined for the mortar specimens for the $I_s = P/d^2$ method of calculation.

(4) For the axial direction of testing, adjusting the length to a common L/D ratio of 0.86 as determined for the mortar specimens for the $I_s = P/A$ method of calculation.

Disregarding the 2 tests on granite, average coefficients of variation or standard errors were calculated for each of the 7 rock types for unconfined compression, axial and diametral point load tests and for four methods of computing the index. Since most sedimentary rocks are non-homogeneous with anisotropic properties to some degree, the standard errors for the tests on rock were predictably higher than for mortar by all test methods with a range of 52 to 81 percent for point load versus 62 percent for unconfined compression tests. The relatively higher value for unconfined tests on rock, as compared to those on mortar, probably reflects the greater effect of non-homogeneity upon the longer specimens used for the unconfined tests. However, experience with the mortar tests was confirmed in that the length adjusted, P/A method of calculating axial point load was again the lowest or best of the point load test methods with a standard error of 52 percent. The axial, P/d^2 method of calculation without adjustment of the random lengths tested was again the highest at 81 percent.

Substantially lower values of standard error were found by considering only the various types of limestone, presumably among the harder and more homogeneous of the sedimentary rocks. For the limestone family, the range was 38 to 64 percent for point load and 46 percent for unconfined compressive tests. Again the lowest, or best, value was for length adjusted P/A method of calculating axial point load index.

Correlation coefficients (r) are shown in Table 7 for comparison of average unconfined compressive strengths to corresponding average point load indices as determined for the two test orientations and four methods of index calculation. Widely varying degrees of correlation were found by rock type. For some rock types, no correlation at all is indicated. Overall correlation coefficients were also calculated for all data combined, disregarding rock type. An r value of 0.68 is indicated for diametrally oriented tests using both the P/A and the P/d^2 methods of calculation. The highest or best values

were found for axially oriented tests, 0.79 for the P/d^2 method and 0.78 for the length adjusted P/A method of calculation. Correlation equations for the latter and for the P/A calculated diametral tests are listed in Table 8. Not included in this table are simplified equations for these cases for all data, where, struck through zero, Qu approximates 13 I_s and 12 I_s respectively or an average of 12.5 I_s.

TABLE 7

Correlation Coefficients for Qu Vs. I_s for Various Methods of Calculating I_s

	Correlation Coefficients, r					
	Point Load Index, $I_s = P/d^2$			Point Load Index, $I_s = P/A$		
Rock Type	Diametral	Axial	Axial (Length Adjusted)	Diametral	Axial	Axial (Length Adjusted)
Claystone	0.08	0.30	0.04	0.09	0.45	0.37
Sandy Shale	0.70	0.72	0.73	0.58	0.67	0.65
Shale, Limey	-0.18	0.80	0.31	-0.18	0.71	0.38
Sandstone	-0.01	0.26	0.24	-0.01	0.26	0.50
Limestone, Shaley	0.17	0.39	0.59	0.17	0.52	0.68
Limestone, Dolomitic	0.37	0.29	0.37	0.37	0.36	0.39
Limestone	0.57	0.70	0.69	0.57	0.66	0.70
Average of Rock Types	0.24	0.49	0.42	0.23	0.52	0.51
All Data	0.68	0.73	0.79	0.68	0.77	0.78

TABLE 8

Correlation Equations for Qu and I_s Calculated by the P/A Method

	Diametral				Axial I_s, Length Adjusted			
Rock Type	Equation	I_s Mean	V%	r	Equation	I_s Mean	V%	r
Claystone	$Qu = 51.5\ I_s - 5,330$	112	167	0.09	$Qu = 53.5\ I_s - 2,820$	60	76	0.33
Sandy Shale	$Qu = 25.6\ I_s + 131$	17	78	0.58	$Qu = 8.4\ I_s + 117$	54	91	0.69
Shale, Limey	N/A	49	176	0	$Qu = 29.3\ I_s - 2,540$	116	57	0.38
Sandstone	N/A	264	115	0	$Qu = 71.9\ I_s - 15,500$	265	57	0.27
Limestone, Shaley	$Qu = 38.5\ I_s - 16,200$	609	48	0.17	$Qu = 6.5\ I_s + 3,400$	601	50	0.70
Dolomitic Limestone	$Qu = 52.6\ I_s - 14,900$	399	44	0.37	$Qu = 103\ I_s - 38,200$	430	39	0.20
Limestone	$Qu = 28.6\ I_s - 10,700$	685	31	0.57	$Qu = 19.5\ I_s - 5,490$	748	29	0.71
All Data	$Qu = 20.0\ I_s - 2,650$	391	62	0.68	$Qu = 17.5\ I_s - 1,970$	400	54	0.77

NOTE: All equation units in p.s.i.

Correlations developed by the least squares method for all orientations and methods of calculating point load index are also exhibited as plots of 1st degree equations on figures 2 through 7. The plots extend through the range of one standard deviation,

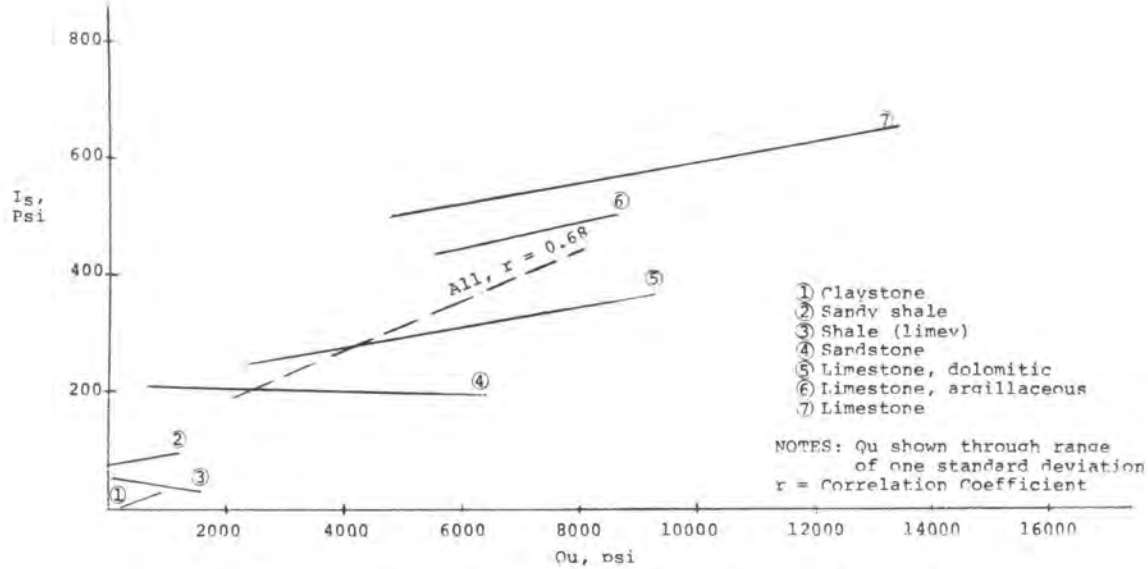

Figure 2. Correlations of Qu to I_s for diametral tests using the $I_s = P/d^2$ method of calculation.

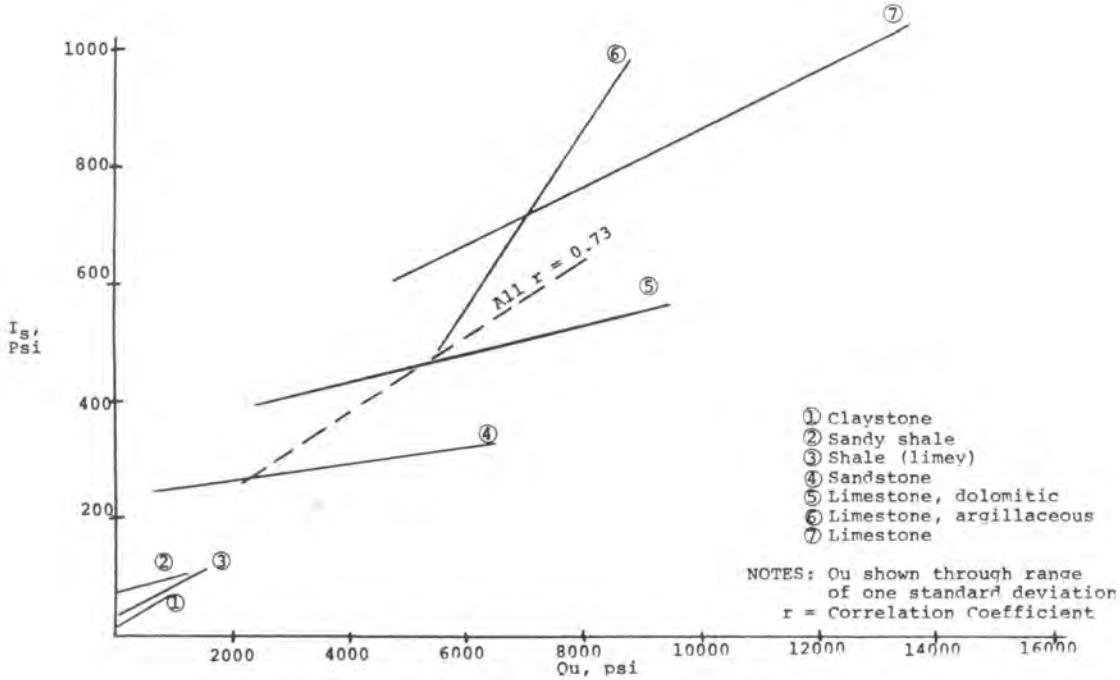

Figure 3. Correlations of Qu to I_s for axial tests using the $I_s = P/d^2$ method of calculation.

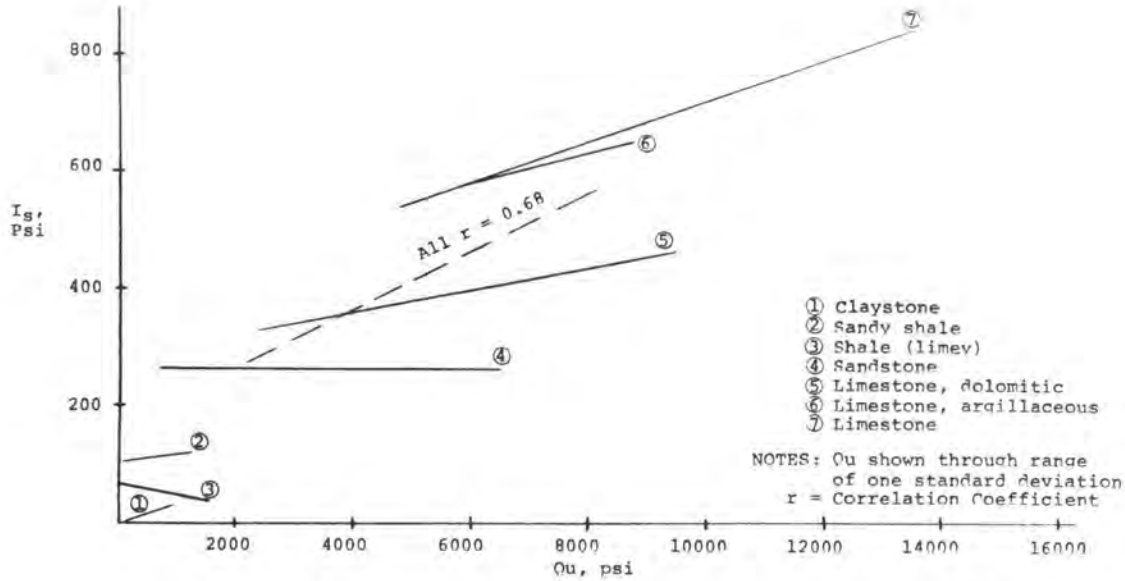

Figure 4. Correlations of Qu to I_s for diametral tests using the $I_s = P/A$ method of calculation.

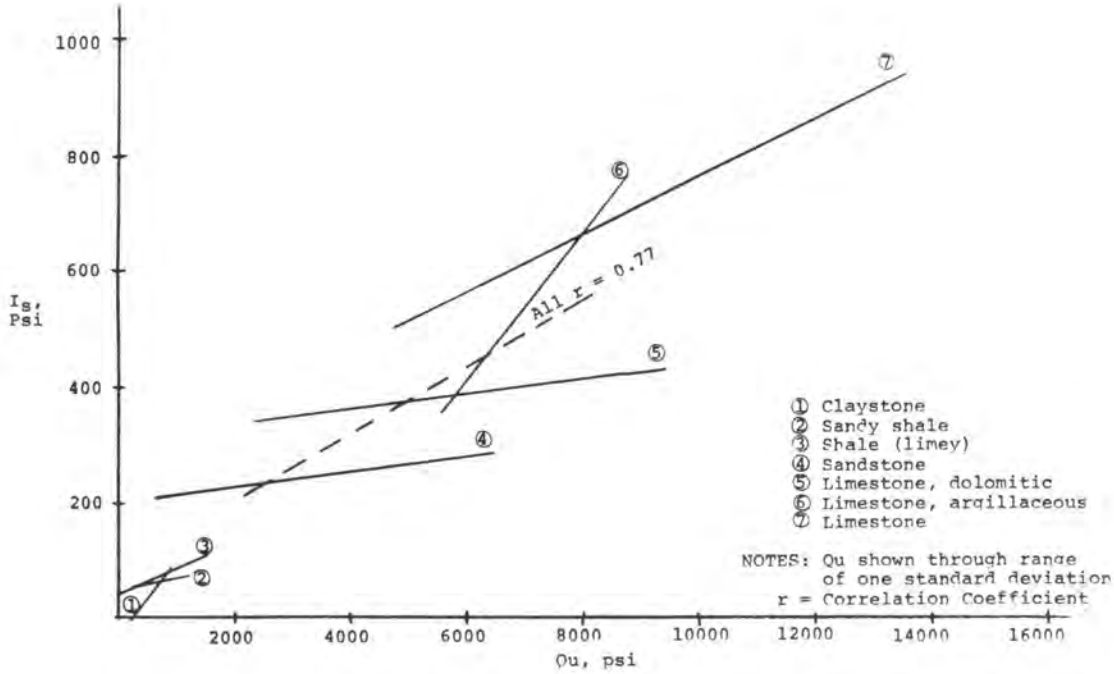

Figure 5. Correlations of Qu to I_s for axial tests using the $I_s = P/A$ method of calculation.

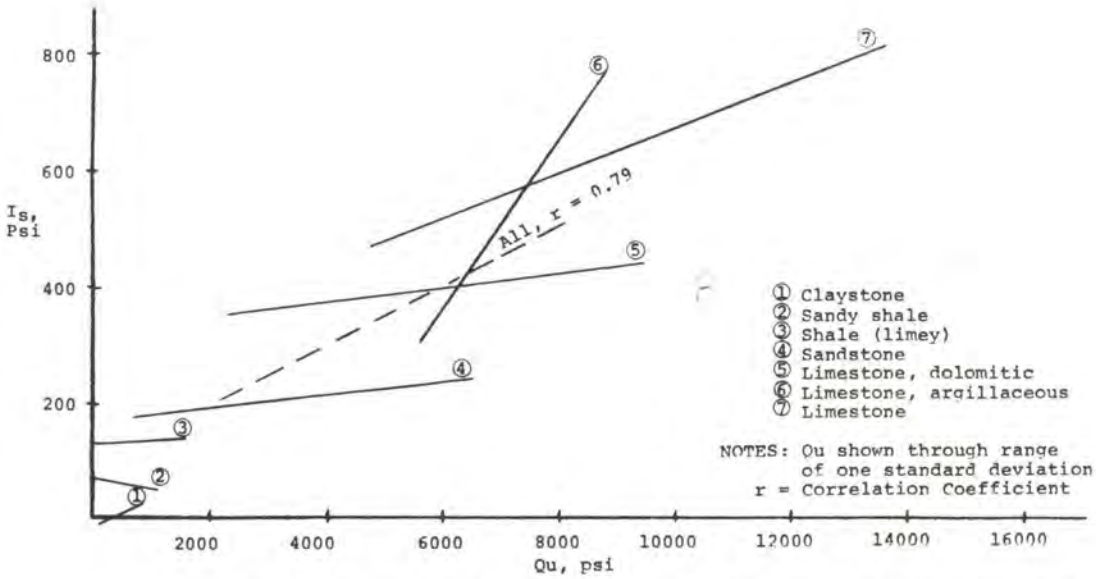

Figure 6. Correlations of Qu to I_s for axial tests using the length adjusted, $I_s = P/d^2$ method of calculation.

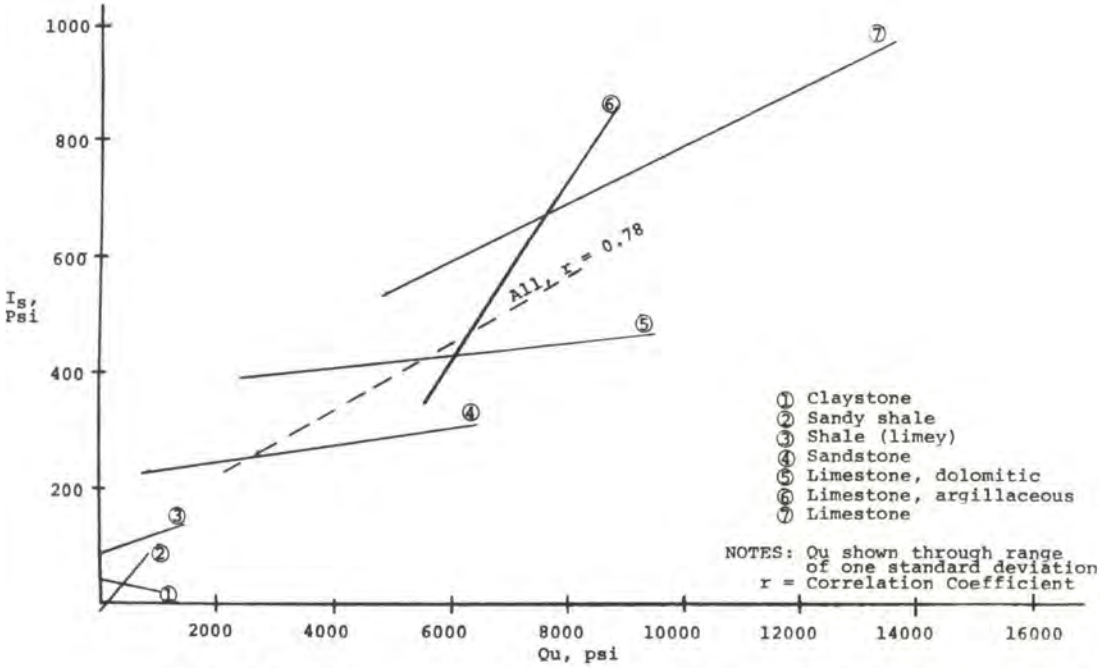

Figure 7. Correlations of Qu to I_s for axial tests using the length adjusted, $I_s = P/A$ method of calculation.

plus or minus the averaged unconfined compressive strength for the rock type noted. The axial test correlations encompass a range of specimen length to diameter ratios of 0.6 to 1.2. Also plotted on each figure are the overall correlation by that method for all tests combined. These plots show graphically that correlations for individual rock type are at significant variance from the overall correlation, regardless of the method of calculation, and that the overall correlation is superior to any individual correlation. However, the overall correlations developed do not compare well to the general relationship proposed by Bieniawski for NX cores where Qu approximates 24 I_s. It should be noted however that Bieniawski's relationship encompassed a range of unconfined strengths greater by a factor of more than three. It is likely that expanding the range of rock strengths tested beyond that provided by Missouri's sedimentary formations would improve the degree of agreement with Bieniawski's general relationship. This is supported by the tendency found for improved correlation coefficients and coefficients of variation with increasing strength with both the mortar and rock tested.

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