Investigating Relationships between Engineering Properties of Various Rock Types

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Abstract: In the literature on rock mechanics several authors have proposed different constants and empirical equations for the physical and mechanical properties of various rock types. This is because researchers usually base their results on the limited number of test samples and/or lithological units. The research presented here has been primarily performed to solve this variation problem. It makes sense to have a general expression for all types of rocks. To accomplish this, a considerable amount of data comprising 4,991 datasets was collected through an extensive literature review. The results were statistically analyzed to determine the range, mean, standard deviation for each investigated rock property. The physical and mechanical properties of the rocks were also plotted against each other in order to estimate one property from the other. The correlation coefficients and best-fit equations were determined by the least squares curve fit method. Usually good correlations were found between index tests, and physico-mechanical properties. At the end, various regression equations are proposed particularly applicable to the whole rock types.

Keywords: Intact rock, Physical and mechanical properties, Uniaxial compressive strength, Brazilian tensile strength, Point load strength, Young's modulus, Schmidt rebound hardness.

1. INTRODUCTION

It has been widely acknowledged that the knowledge of rock properties is one of the prime requirements for understanding the rock material. Rocks have been clearly defined according to their significant physical and mechanical properties. In addition, the physical and mechanical properties of intact rocks are very important in civil, mining, petroleum and geological engineering works that interact with rock such as underground structures, dams and foundations, tunneling and slopes. Mineralogy, grain size, density and porosity are assumed to be the intrinsic properties determining the rock strength. Mechanical properties such as hardness and strength, however, are not intrinsic material properties which mostly depend on the type of testing instrument and the test procedure adopted.

Index tests have traditionally been used to estimate one or more of the mechanical properties of rock. To evaluate rock strength and deformation usually direct tests are performed, but they are generally expensive and require considerable time, especially in the preparation of rock cores for testing. Instead, various indirect testing methods were developed and used to interpret the engineering properties of rocks. The indirect tests i.e., point load, Schmidt rebound hardness, Shore scleroscope hardness, p-wave velocity are relatively easy to perform, are not costly, require minimum sample preparation, and testing time is short. Researchers have been developed a number of correlations for the interpretation of rock properties obtained from indirect tests. For instance, some researchers have investigated the relationship between unconfined compressive strength (UCS) and point load strength (PLS) and considerably different constants have been found. This is because; they generally base their results on a limited number of observations and lithological units.

It is generally recognized that natural materials like rock tend to show a considerable variety of properties. However, not only does it vary widely with the property investigated but it also shows extensive deviation from test to test. Therefore, when defining a mechanical property it is extremely important to specify the test procedure adopted. For instance, various techniques of recording Schmidt rebound hardness have been consistently proposed in the literature [1]. It was seen that the rock property correlations reported in the technical literature often have a limited database and should be considered with caution. Also, there is no generally accepted empirical equation or approach in the literature to estimate different rock properties. Most of the researchers state that their model might be a useful tool for the rocks specific to their study, but their work was not conclusive and more data is required to improve the validity of the proposed models. Therefore, an attempt should be made to develop empirical equations essentially applicable to the whole rock range. Then, it will be possible to predict and estimate which property needs high quality core samples and

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sophisticated test equipment which property can be easily determined using simple and quick tests.

In this study, an extensive research on the literature was performed to assemble data on the physical and mechanical properties of intact rock from a wide array of different national and international published sources. This information will facilitate an assessment of physical and mechanical characterization of intact rock with a single expression. Instead having been based solely on the sample statistics that involve mostly a limited number of tests and rock samples; this work aims to find the population parameters by collecting all necessary information. The main objective of this study is, therefore, to derive simple empirical relationships between various physico-mechanical properties of intact rocks for a wide range of rock types at different origins. A second aim of the study is to acquire a single but still functional relationship for most of the rocks by presenting frequency distributions of several rock characteristics and well-known constants, which shows considerable variations from study to study in the literature.

2. THE DATABASE

The database comprised a wide variety of physical properties such as dry density (γ_d), specific gravity (G_s), water content (w), porosity (n) and wave velocity (p-wave) and mechanical properties such as uniaxial (σ_c) and triaxial compression tests, elastic modulus (E_t), Poisson's ratio (ν), Brazilian tests (T_0). Various index tests namely point load (Is), Schmidt hammer (R), Shore scleroscope (S_h), and slake durability (I_d) were also included. The data compiled embraces various types of rock from coal to granite, various compressive strength values from 0.4 MPa standing for "very weak rock" to 436 MPa falling in "extremely strong rock" and various testing conditions. Hence, the data is considered to be highly representative of all the variations that could be anticipated in rock engineering practices. In total, 4,991 rock types were sampled in the database and they are all employed for the statistical analysis. Names, types and classes of rocks are presented in Table **1** and the total sample numbers of each rock represented in the database are given in the brackets.

Table 1: Primary Rock Types in the Database Classified by Geologic Origin*

	Sedimentary						Metamorphic				Igneous					
	Clastic		Organic		Chemical		Foliated		Massive		Intrusive		Extrusive		Pyroclastic	
Coarse	Conglomerate	(27)			Limestone	(640)	Gneiss	(71)	Marble	(197)	Granite	(432)			Agglomerate	(8)
					Breccia	(12)	Argillite	(3)			Syenite	(49)				
					Calcarenite	(11)					Pegmatite	(3)				
											Tonalite	(4)				
											Monzonite	(10)				
											Dunite	(13)				
											Norite	(14)				
Medium	Sandstone	(711)			Travertine	(58)			Hornfels	(5)	Granodiorite	(57)	Andesite	(173)	Breccia	(6)
	Siltstone	(103)			Dolostone	(139)	Schist	(77)	Migmatite	(3)	Diorite	(38)	Dacite	(16)	Tuffite	(5)
	Greywacke	(30)			Caliche	(13)	Phyllite	(15)			Diabase	(23)	Trachyte	(2)		
											Dolerite	(35)				
Fine	Shale	(177)	Coal	(126)	Chalk	(101)	Slate	(14)	Quartzite	(43)	Gabbro	(29)	Basalt	(165)	Tuff	(288)
	Mudstone	(98)			Anhydrite	(10)	Granulite	(5)	Amphibole	(16)	Epidiorite	(5)	Rhyolite	(12)	Ignimbrite	(17)
	Marlstone	(144)			Gypsum	(46)	Serpentine	(24)			Pyroxenite	(5)			Pumice	(19)
	Claystone	(11)			Rock salt	(18)	Talc	(4)			Mylonite	(4)				
											Charnockite	(11)				
	Total	(1314)		(126)		(1062)		(229)		(254)		(750)		(367)		(342)

*Numbers in parenthesis refer to total number of rocks sampled in the database.

The following steps are carefully taken into consideration during the construction of the database.

- Each rock in the sample data is given a code representing the rock type and rock class.
- The correction for scale and shape effects was made using the suggestions of Hoek and Brown [2], Turk and Dearman [3] for UCS, Broch and Franklin [4] and ISRM [5] for PLS, if it was not already taken into account in the original study.
- A new conversion factor has been proposed and subsequently used in this study to convert the Schmidt hammer readings from N-type to L-type, since notable differences were reported between the two types of hammer values on the same rock specimens [1, 6, 7].
- The values for static and dynamic elasticity, effective and total (absolute) porosity, dry and saturated property, normal and parallel to foliation strength, bulk and grain density, fresh and weathered property, axial and diametric point load were taken into consideration in the database.

A summary of the physical and mechanical properties covered in the database is given in Table **2**, presenting the range, mean, standard deviation for each investigated rock property. It is understandable that the most represented rock property in the database is the uniaxial compressive strength with a total of 3,511 data points. Dry density and total porosity come after this property with a total representation of 3,068 and 2,901 values, respectively. The least represented tests in the database are the slake durability index with a total of 245 tests and Los Angles abrasion index with a total of 212 tests.

3. RESULTS AND DISCUSSION

In order to propose an indirect estimation by empirical equations, statistical methods are traditionally used. Bivariate correlation provides a means of summarizing the relationship between two different variables. Linear regression ($y=ax+b$) is the most

Table 2: Descriptive Statistics for the Investigated Properties of Rocks in the Database

	N	Min.	Max.	Mean	Std. Dev.
Dry density, gr/cm^3	3068	0.62	4.04	2.37	0.43
Specific gravity	2746	1.65	4.05	2.70	0.11
Effective porosity, %	1587	0.01	70.00	9.41	11.66
Total porosity, %	2901	0.01	74.92	13.14	13.65
UCS, MPa	3511	0.40	436.00	80.60	66.28
BTS, MPa	1864	0.05	45.10	7.32	5.80
PLS, MPa	1190	0.05	20.80	5.12	3.94
Shore scleroscope hardness	537	3.00	107.00	46.12	21.92
Schmidt rebound hardness	1002	9.50	72.00	41.43	12.31
P-wave velocity, km/s	1277	0.38	7.19	4.19	1.49
Cohesion, MPa	492	0.03	74.00	13.21	11.57
Internal friction angle, ^{o}	481	9.00	69.00	39.63	11.57
Young's modulus, GPa	2031	0.05	137.40	28.52	24.51
Poisson's ratio	1024	0.03	0.65	0.25	0.07
Slake durability index, %	245	29.75	99.80	90.17	14.22
Los Angles abrasion index, %	212	10.20	99.90	30.36	18.57
Fracture toughness, MPa.m$^{1/2}$	267	0.03	3.21	1.30	0.71
UCS/BTS ratio	1674	2.02	51.89	11.77	5.63
UCS/PLS ratio	1036	2.58	55.00	18.01	7.45
PLS/BTS ratio	659	0.50	7.50	1.79	0.85
Modulus ratio, E_t/σ_c	1769	12.08	2057.7	347.09	244.6

common statistical procedure for fitting a straight line to a set of experimental data and is based on the least-square curve estimation. In addition to linear regression analysis, power ($y=ax^b$), logarithmic ($y=a+\ln x$) and exponential ($y=ae^{bx}$) relationships between variables are also investigated [8]. The regression line equations could mostly be used to predict one property from the results of other empirical tests.

Many researchers have proposed different empirical equations concerning the rock properties. A supplementary list of the expressions proposed by different authors for the index, physical and mechanical properties of intact rocks is presented in Appendix Table **A1**. At a glance, one can easily discern that there are enormous variances between the proposed relationships for the various rock types by different authors. Although the number of tests for the indices is usually large enough for statistical inferences, the number of property tests is seldom sufficient to draw necessary conclusions for the whole rock spectrum. Usually, most of the researchers based their analysis on a limited number of rock types and testing range.

3.1. Significance of Porosity and Density Data

The presence of pores in the fabric of a rock material decreases its strength, and increases its deformability [9]. A small change of volume fraction of pores can produce considerable mechanical effects. Since sandstones and carbonate rocks, in particular, occur within a wide range of porosities, they exhibit a highly variable mechanical character; igneous rocks weakened by weathering processes also have typically high porosities. Most rocks have similar grain densities and therefore, have porosity and dry density values that are highly correlated. A low-density rock is usually highly porous. It is often sufficient, therefore, to quote values for porosity alone but a complete description requires values for both porosity and density [10].

It is clearly evident from Figure **1** that there is a highly significant linear relationship between the total porosity and dry density, as the former increases the later decreases. Here, it is necessary to clarify that since coal and evaporate rocks such as rock salt and gypsum, have considerably lower grain densities (1.7, 2.2 and 2.35, respectively) compared to rocks in general (2.7), they are instantaneously excluded in the following regression analysis between dry density and total porosity. The simple linear equation that relates dry density (γ_{dry}) dependent on total porosity (n_{tot}) is:

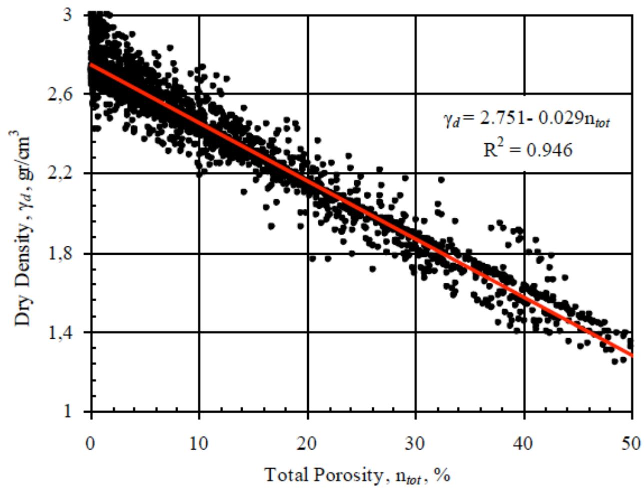

Figure 1: Relationship between total porosity and dry density.

$$\gamma_{dry} = 2.751 - 0.029 n_{tot} \qquad (r^2=0.946) \tag{1}$$

where γ_{dry} in gr/cm^3 and n_{tot} in %.

3.1.1. Porosity

Total or absolute porosity is a measure of the total void volume and can be obtained from the following formula,

$$n_{tot} = (1-\gamma_{dry}/G_s)100\% \tag{2}$$

where n_{tot} is total porosity (%), γ_{dry} is dry density (gr/cm^3) and G_s is specific gravity of rock.

The effective porosity, on the other hand, is a measure of the apparent void volume and is determined by the saturation and air porosimeter methods [11]. Unaltered rocks typically have porosity that is less than 20% which may increase due to weathering to values of 50% or even higher [12].

Porosity is the single most important physical property that influences rock strength. The pores in a rock which are prone to water saturation are principally known as effective porosity. With increasing effective porosity, penetration of water and consequently, the negative effects increase. The physical explanation of this is that high porosity contributes the networking (propagation) of stress-induced micro fractures [13]. It is established that the uniaxial compressive strength (σ_c) of porous rocks such as sandstones, granites, dolerites, basalts, dolomites, limestones, and chalks depend on porosity (n) (Appendix Table **A1**).

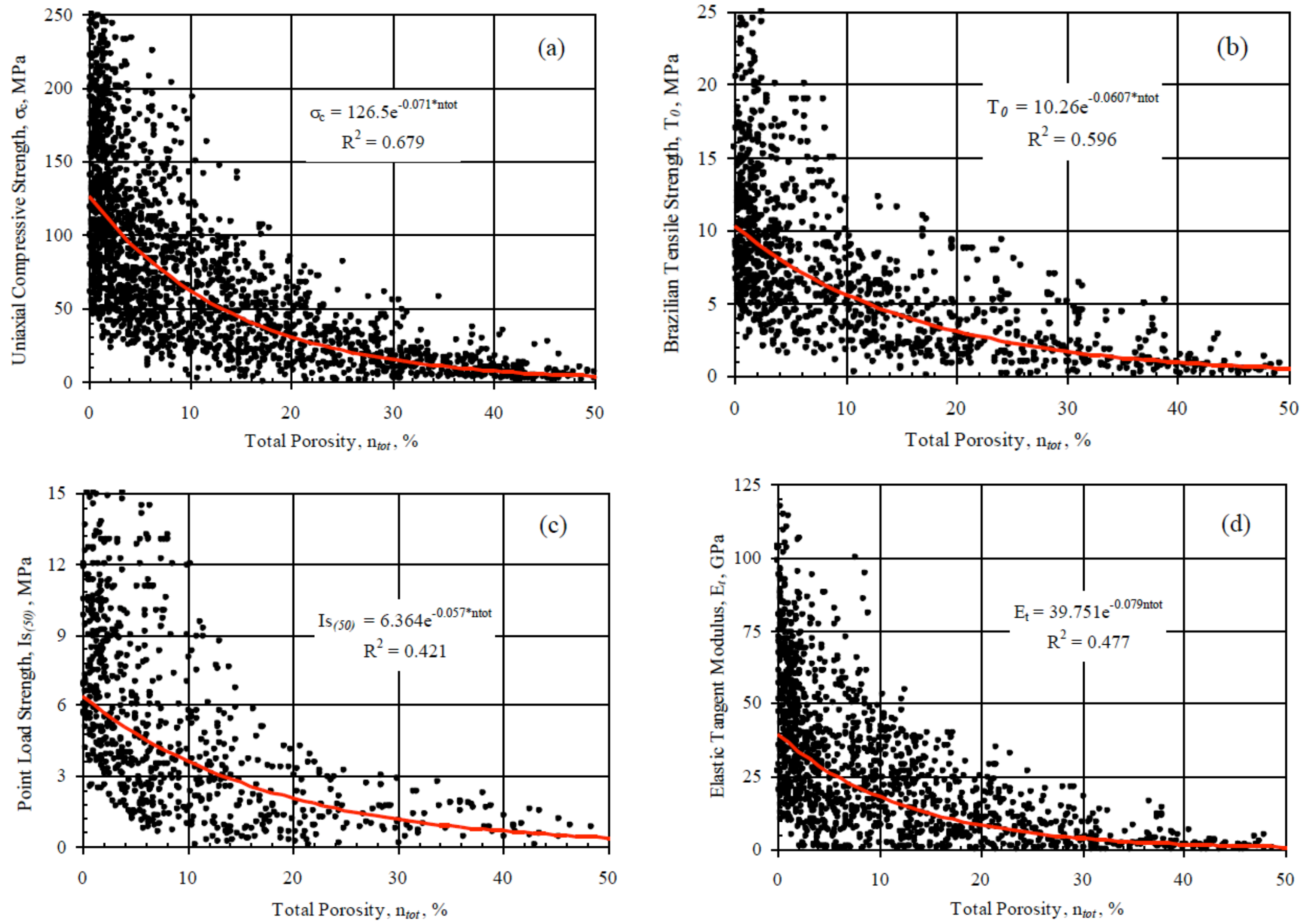

Figure 2: Total porosity vs. a. UCS, b. Brazilian tensile, c. Point load, and d. Young's modulus.

Figure **2** illustrates the relationship between total porosity (n_{tot}) and uniaxial compressive strength (σ_c) for all rocks. Clearly, the uniaxial compressive strength is inversely related to porosity. Large void space (high porosity) has a negative effect on rock strength. Similarly, there is a non-linear relationship of a hyperbolic nature between tensile and point load strengths, and Young's modulus and total porosity, as can be seen in the same figure. The plots indicate a sharp decrease in strength with an increase in the total porosity. The following formula was derived to relate uniaxial compressive strength to total porosity:

$$\sigma_c = 126.5\exp(-0.071 n_{tot}) \ (r^2=0.679) \quad (3)$$

where σ_c is the uniaxial compressive strength (MPa) and n_{tot} is the total porosity (%).

The unconfined compressive strength, Brazilian strength and the point load strength had highly significant relationships with total porosity. The relationships with effective porosity were not as good but were still significant (Table **3**). In other words, the compressive and tensile strengths considerably decrease as the porosity increases, this effect is the most pronounced for UCS (r^2=0.679) then for BTS (r^2=0.596) and then for Young's modulus (r^2=0.477) and PLS (r^2=0.421).

3.1.2. Density

One of the basic properties of a rock is that its density is influenced, primarily, by the specific gravities of the minerals it contains and the amount of unoccupied void space within it. The plots of four properties as a function of dry density are presented in Figure **3**. As the dry density increases, so does the unconfined compressive, Brazilian tensile, point load strengths, and Young's modulus increases exponentially. The relationship is significant in the case of the densities lower than 2.4 gr/cm^3 but less pronounced and scattered around the band of 2.4 and 2.9 gr/cm^3 owing to fact that most rocks typically have dry densities around this range as shown in Figure **4**. On the other hand it is noticed that there was a wide range in dry density values in the database. The lowest

Table 3: A Complete List of the Proposed Equations for all Rock Types in the Database

Equation	R	Equation	R
$\sigma_c = 10.04T_0$	0.77	$T_0 = 10.31\exp(-0.061n_{tot})$	0.77
$\sigma_c = 8.54T_0 + 18.21$	0.79	$T_0 = 11.83n_{tot}^{-0.506}$	0.67
$\sigma_c = 11.22T_0^{0.967}$	0.91	$T_0 = 0.049\exp(1.922\gamma_{dry})$	0.76
$\sigma_c = 15.15Is_{(50)}$	0.84	$T_0 = 0.173\gamma_{dry}^{3.903}$	0.75
$\sigma_c = 13.36Is_{(50)} + 14.05$	0.86	$T_0 = 0.524\exp(0.055R_L)$	0.75
$\sigma_c = 19.22Is_{(50)}^{0.879}$	0.90	$T_0 = 0.003R_L^{2.064}$	0.75
$\sigma_c = 0.259\exp(2.217\gamma_{dry})$	0.80	$T_0 = 6.78K_{IC}$	0.77
$\sigma_c = 1.07\gamma_{dry}^{4.543}$	0.79	$T_0 = 5.82K_{IC} + 1.64$	0.79
$\sigma_c = 1.819S_h$	0.78	$T_0 = 6.88K_{IC}^{0.948}$	0.87
$\sigma_c = 2.123S_h - 17.23$	0.79	$T_0 = 1.517Is_{(50)}$	0.82
$\sigma_c = 0.705S_h^{1.203}$	0.81	$T_0 = 1.361Is_{(50)} + 1.23$	0.83
$\sigma_c = 13.76\exp(0.032S_h)$	0.77	$T_0 = 2.038Is_{(50)}^{0.835}$	0.85
$\sigma_c = 102.6\exp(-0.085n_{eff})$	0.79	$T_0 = 2.403V_p - 2.15$	0.67
$\sigma_c = 98.37n_{eff}^{-0.551}$	0.74	$T_0 = 0.856\exp(0.459V_p)$	0.73
$\sigma_c = 126.8\exp(-0.071n_{tot})$	0.82	$T_0 = 0.647V_p^{1.629}$	0.77
$\sigma_c = 154.0n_{tot}^{-0.605}$	0.74	$Is_{(50)} = 6.286\exp(-0.056n_{tot})$	0.64
$\sigma_c = 3.50R_L - 67.68$	0.75	$Is_{(50)} = 8.428n_{tot}^{-0.507}$	0.63
$\sigma_c = 4.97\exp(0.058R_L)$	0.76	$Is_{(50)} = 0.039\exp(1.84\gamma_{dry})$	0.65
$\sigma_c = 0.022R_L^{2.123}$	0.76	$Is_{(50)} = 0.136\gamma_{dry}^{3.67}$	0.63
$\sigma_c = 29.57V_p - 32.45$	0.67	$Is_{(50)} = 0.222R_L - 3.94$	0.71
$\sigma_c = 7.215\exp(0.514V_p)$	0.76	$Is_{(50)} = 0.324\exp(0.057R_L)$	0.73
$\sigma_c = 5.912V_p^{1.741}$	0.80	$Is_{(50)} = 0.0015R_L^{2.123}$	0.74
$\sigma_c = 58.14K_{IC} + 35.34$	0.66	$Is_{(50)} = 1.567V_p - 1.145$	0.68
$\sigma_c = 88.37K_{IC}^{0.69}$	0.66	$Is_{(50)} = 0.613\exp(0.449V_p)$	0.79
$\sigma_c = 43.95\exp(0.586K_{IC})$	0.72	$Is_{(50)} = 0.751V_p^{1.261}$	0.77
$E_t = 0.309\sigma_c$	0.72	$R_L = 23.44\gamma_{dry} - 14.68$	0.60
$E_t = 0.280\sigma_c + 4.05$	0.68	$R_L = 8.33\exp(0.65\gamma_{dry})$	0.60
$E_t = 0.204\sigma_c^{1.072}$	0.78	$R_L = 12.54\gamma_{dry}^{1.322}$	0.59
$E_t = 3.326T_0$	0.79	$R_L = 0.424S_h + 21.63$	0.77
$E_t = 3.20T_0^{0.988}$	0.84	$R_L = 23.2\exp(0.011S_h)$	0.76
$E_t = 39.74\exp(-0.079n_{tot})$	0.62	$R_L = 7.88S_h^{0.435}$	0.79
$E_t = 47.41n_{tot}^{-0.616}$	0.79	$R_L = -0.721n_{tot} + 50.35$	0.62
$E_t = 0.032\exp(2.562\gamma_{dry})$	0.69	$R_L = 50.07\exp(-0.02n_{tot})$	0.63
$E_t = 0.146\gamma_{dry}^{5.382}$	0.62	$R_L = 52.8n_{tot}^{-0.154}$	0.59
$E_t = 0.68\exp(0.069*R_L)$	0.70	$V_p = 0.078R_L + 0.54$	0.63
$E_t = 0.0012R_L^{2.515}$	0.68	$V_p = 1.16\exp(0.025R_L)$	0.61
$E_t = 11.65V_p - 16.09$	0.62	$V_p = 0.109R_L^{0.934}$	0.62
$E_t = 1.578\exp(0.623V_p)$	0.62	$V_p = -0.101n_{tot} + 5.122$	0.75
$E_t = 1.314V_p^{2.073}$	0.75	$V_p = 5.176\exp(-0.032n_{tot})$	0.77
$\gamma_{dry} = 2.673 - 0.033n_{eff}$	0.77	$V_p = 5.459n_{tot}^{-0.245}$	0.71
$\gamma_{dry} = 2.715\exp(-0.017n_{eff})$	0.83	$V_p = 2.857\gamma_{dry} - 2.854$	0.76
$\gamma_{dry} = 2.751 - 0.029n_{tot}$	0.94	$V_p = 0.46\exp(0.861\gamma_{dry})$	0.76
	0.95	$V_p = 0.785\gamma_{dry}^{1.79}$	0.74
	0.97		0.82

σ_c: Uniaxial compressive strength (MPa), T_0: Brazilian tensile strength (MPa), E_t: Elastic tangent modulus (GPa), $Is_{(50)}$: Point load strength index (MPa), γ_{dry}: Dry density (gr/cm^3), n_{tot}: Total porosity (%), n_{eff}: Effective porosity (%), R_L: L-type Schmidt rebound hardness, S_h: Shore scleroscope hardness, V_p: P-wave velocity (km/s), K_{IC}: Fracture toughness (MPa.m$^{1/2}$).

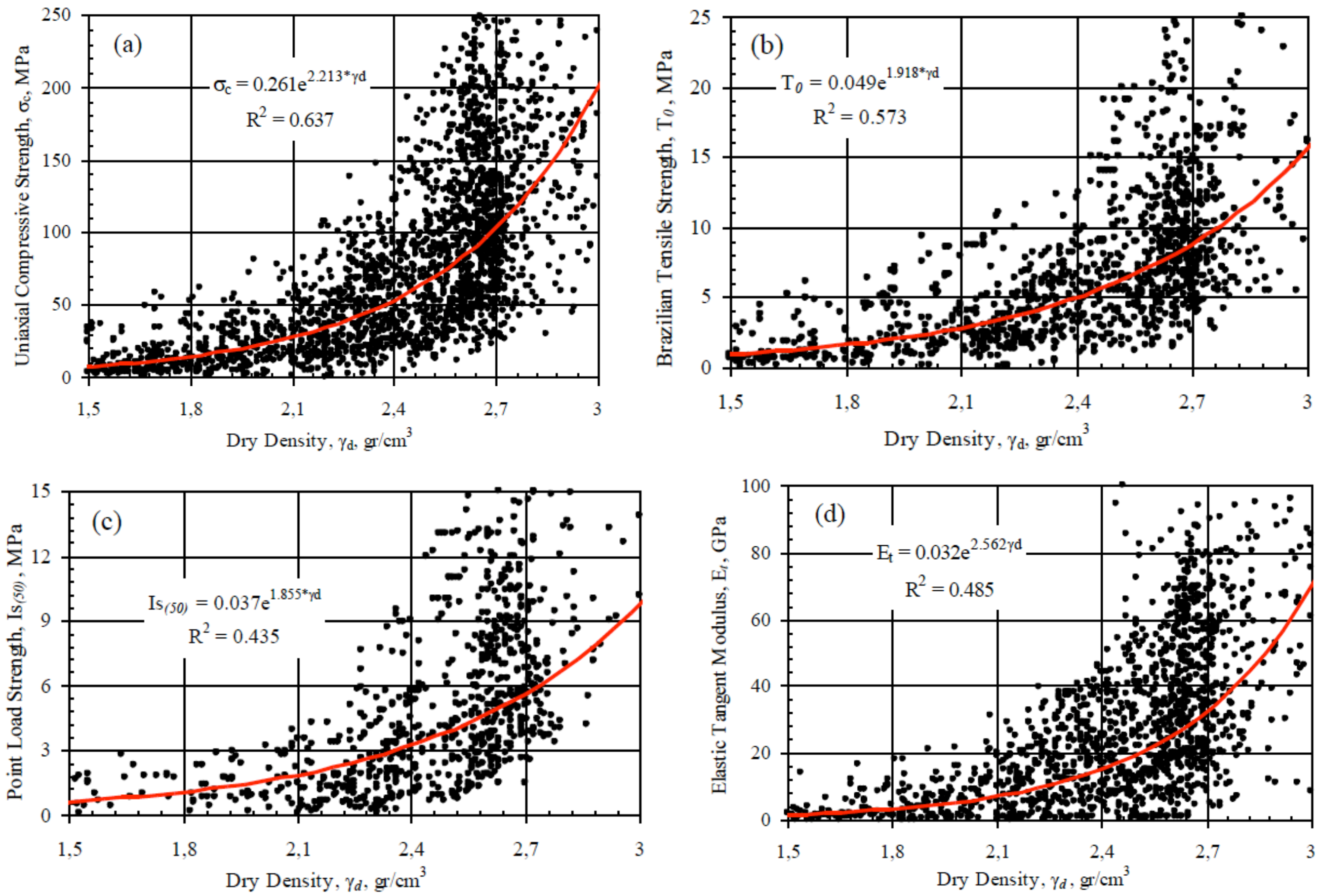

Figure 3: Dry density vs. a. UCS, b. Brazilian tensile, c. Point load, and d. Young's modulus.

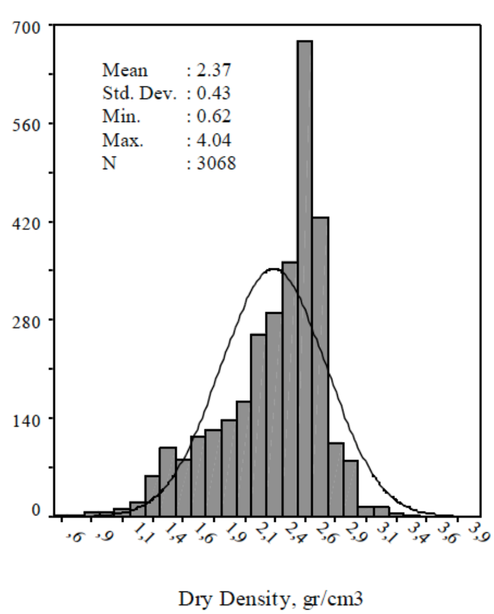

Figure 4: Histogram of dry density data.

value recorded from pumice is 0.62 gr/cm^3 and the highest value from quartzite at 4.04 gr/cm^3. The relationship between uniaxial compressive strength (σ_c) and dry density (γ_{dry}) is given as:

$$\sigma_c = 0.261\exp(2.213\gamma_{dry}) \quad (r^2=0.637) \tag{4}$$

where σ_c in MPa and γ_{dry} in gr/cm^3.

3.2. Uniaxial Compressive Strength

Uniaxial compressive strength (UCS) is one of the most important mechanical properties of rocks, which is mainly used for the design of engineering structures and characterization of intact rock materials. In rock engineering, the UCS value is typically determined by an unconfined compression test where a cylindrical core sample is loaded axially to failure, with no confinement (lateral support). Conceptually, the peak value of the axial stress is taken as the UCS of the sample. The test requires good quality test specimens of right circular cylinders having a height to diameter ratio 2.5-3.0 and a diameter of preferably not less than NX core size, approximately 54 mm [11].

The rocks in the database exhibit an average UCS of 80.60 MPa while they are extending from the lowest of 0.4 MPa for tuff to the highest of 436 MPa for basalt. According to ISRM [11] classification scheme, the strength of the rocks covered in the database corresponds to 3.3% “very weak” (1-5 MPa), 18.0% “weak” (5-25 MPa), 19.6% “medium strong” (25-50 MPa), 29.3% “strong” (50-100 MPa), 27.5% “very strong” (100-250 MPa), and 2.3% “extremely strong” (>250 MPa). Besides presenting the mean and range of collected data, it seems good to know how the data is distributed over the complete strength range. For this purpose, the histograms of uniaxial compressive, Brazilian tensile and point load strengths, and Young’s modulus for all rocks are given in Figure **5** with the associated descriptive statistics and a hypothetical normal distribution. A simple examination of the histograms reveals that the strength values mostly distributed in specific ranges and are also fairly skewed towards the higher values. This type of pattern is mostly characterized by a lognormal distribution [14] the only exception is the Young’s modulus, which shows a negative exponential character.

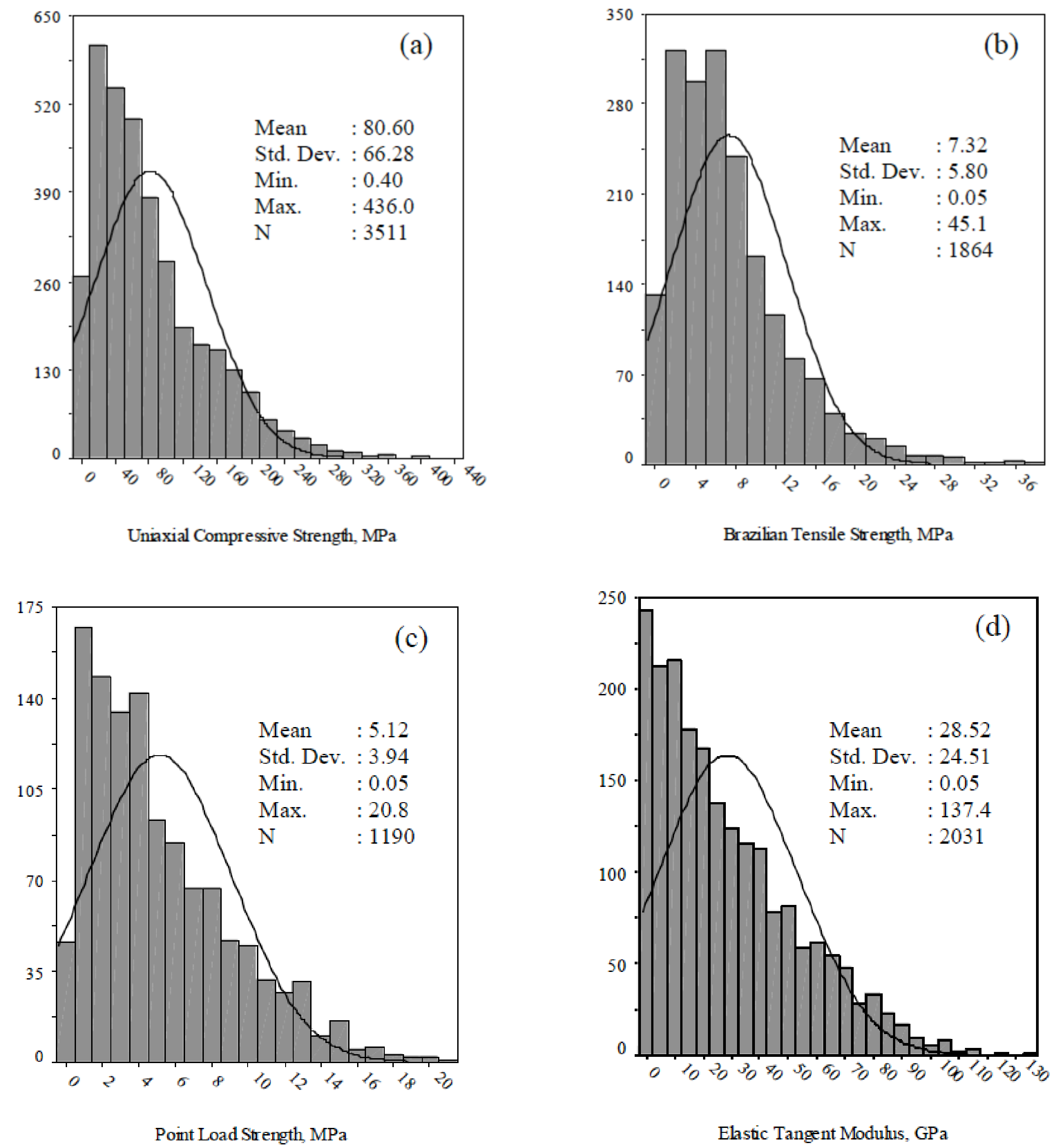

Figure 5: Histograms of a. UCS, b. BTS, c. PLS and d. Young’s modulus data.

3.3. Brazilian Tensile Strength

The tensile strength of rocks is one of the least investigated rock strength characteristics. In part, these are due to the use of compressive and shear stresses rather than tensile stresses in the design of rock structures. Rock is relatively weak in tension, and thus, the tensile strength (T_0) of an intact rock is considerably less than its compressive value (σ_c) [9]. The Brazilian tension test, also known as the splitting tensile test, is widely used to evaluate the tensile strength of rocks, as it is easy to prepare and test specimens. Compression-induced extensional fracturing generated in this test is also more representative of the *in situ* loading conditions and failure of rocks. In the Brazilian tension test, a circular disk placed between two platens is loaded in compression producing a nearly uniform tensile stress distribution normal to the loaded (vertical) diametric plane, leading to the failure of the disk by splitting [15]. The equation for Brazilian test is:

$$T_0 = 2P/\pi LD \qquad (5)$$

where P is the failure load, and L and D are the length and diameter of the disk.

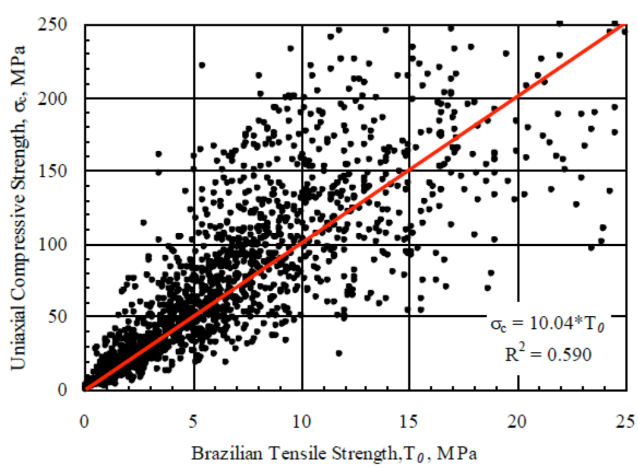

Figure 6: Relationship between Brazilin tensile and UCS.

The next step in the study is to relate the tensile and the uniaxial compressive strengths of the rocks. It is assumed that a fixed ratio exists between the tensile strength and compressive strength of the rocks. The plot of UCS as a function of the Brazilian tensile strength (BTS) is shown in Figure **6**. Since, physically, a zero BTS also implies a zero UCS, it is, therefore, required that the best-fit line passes through the origin in the linear regression analysis. The slope of the best fitting line passing through origin is found to be 10.04 and there is a strong correlation (R=0.77). In literature, many different ratios were recommended for UCS and BTS varying from 2 to 50 (Appendix Table **A1**). Based on all the data collected in this study it can be formally stated that rocks, in general, tend to have a tensile strength of 1/10 of their compressive strengths. The linear equation relating UCS (σ_c) to Brazilian tensile strength (T_0) is:

$$\sigma_c = 10.04T_0 \qquad (r^2=0.590) \qquad (6)$$

where both T_0 and σ_c is in MPa.

Furthermore, tensile strength to compressive strength ratio is one of the important fundamental properties of rock. Figure **7** shows the frequency distribution of this ratio for all rocks in the database. The ratio has a mean of 11.77 with a standard deviation of 5.63 and ranges between 2.02 and 51.9. The overall distribution of UCS/BTS ratio seems to be truly skewed towards the higher values. It can be seen graphically in Figure **8** that the ratio between UCS and BTS has an average value of 11.77, which slightly differs from the constant found by linear regression as 10.04 in Figure **6**. This discrepancy is basically caused by the techniques used in the calculations, the first one is simply the arithmetic mean of all ratios and the second one is the slope of the best fitting linear equation.

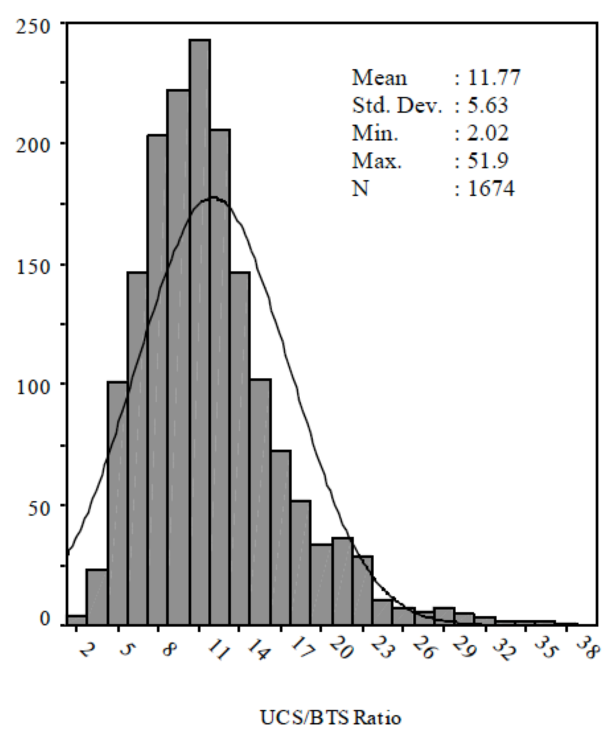

Figure 7: Histogram of UCS/BTS ratio data.

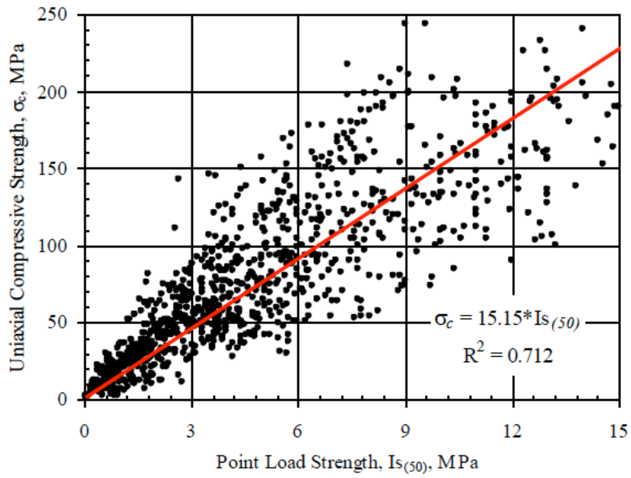

Figure 8: Relationship between UCS and point load strength.

3.4. Point Load Index

The point load test may be used as an index property in rock engineering applications where the true UCS is sought. This is because obtaining point load strength data is by far simpler than obtaining actual uniaxial compression test data, which requires sophisticated testing techniques and strict adherence to sample preparation standards. Therefore, it has become standard practice to rely on published correlations for predicting uniaxial strength from point load data. An index: strength ratio (UCS/Is) has been suggested by Broch and Franklin [4], ISRM [5], and Bieniawski [16] who propose conversion factors (*K*) ranging from 20 to 25 for intact rocks. However, there are many different constants mentioned in the literature ranging from 2-55 for different rock types (Appendix Table **A1**).

The test can be applied to rock samples with irregular or regular shapes. Three test methods; diametrical, axial, and block are available. For rocks possessing horizontal bedding or foliation, the diametric test is an unreliable indicator of the rock strength and axial testing perpendicular to the bedding is required to give a consistent rock strength index [17]. The equation for the diametrical test is:

$$I_s = P/D^2 \quad (7)$$

where P is the failure load and D is the diameter of the core sample.

It is obviously evident from Figure **8** that all data collected from uniaxial compressive strength and point load strength testing indicate a strong correlation between these parameters, although the proposed relationship differs considerably from the models found by Broch and Franklin [4], Bieniawski [16], and Cargill and Shakoor [18]. Again, since physically a zero $Is_{(50)}$ also implies a zero UCS, therefore, it is necessary for the best fit line pass through the origin in the linear regression analysis. The following equation allows the estimation of UCS as a function of the point load index for all rocks:

$$\sigma_c = 15.15 Is_{(50)} \quad (r^2=0.712) \quad (8)$$

where $Is_{(50)}$ is the point load index of the 50 mm diameter core.

In Figure **9**, it can be seen that the ratio between and has an average value of 18.01, which slightly differs from the constant found by linear regression as 15.15 in Equation 8. As noted before, this is due to the different techniques used in the calculations, the first value is a simple arithmetic mean of all ratios and the second value is the slope of linear line passing through the origin. The distribution of UCS/PLS ratio is slightly skewed towards the right. If the ratios greater than 30 are not taken into account, the histogram in Figure **9** can be said fairly symmetrical in shape.

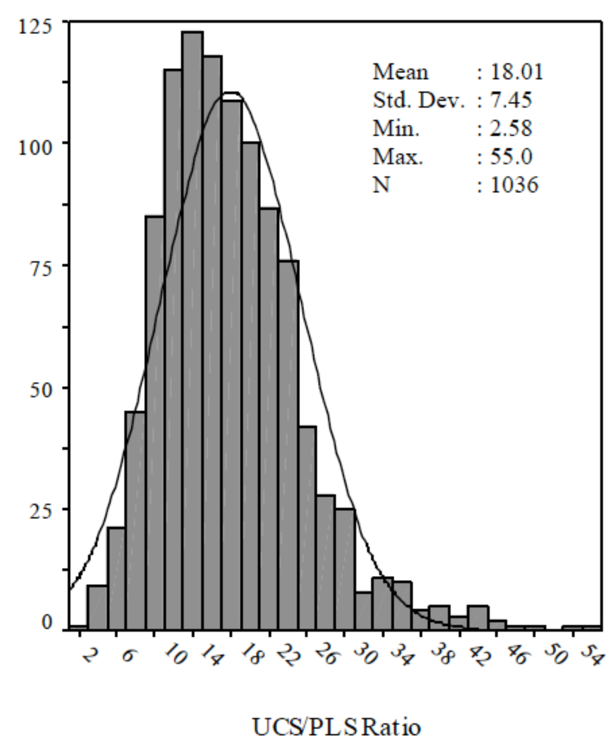

Figure 9: Histogram of UCS/PLS ratio data.

Figure **10** presents the relationship between Brazilian and point load tests, which are considered as two kinds of indirect tensile strength of the rocks. There is a common tendency in all XY-graphs correlating rock strengths up to now that the higher the strength values the more scattered the data points and it is also valid for this case. When the histograms of the strength properties of rocks are considered, they are typically skewed towards the right suggesting a lognormal distribution. If two log-normally distributed variables in scatter plots are compared the result will be a scattering pattern around the best-fit line at larger values. The linear equation relating Brazilian tensile strength (T_0) to point load strength ($Is_{(50)}$) is as follows:

$$T_0 = 1.517 Is_{(50)} \quad (r^2=0.670) \tag{9}$$

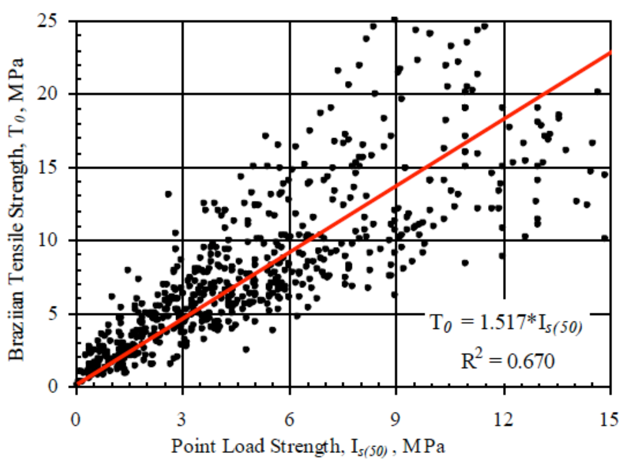

Figure 10: Relationship between point load and Brazilian tensile strengths.

The constant of the linear equation is somewhat interesting. The only difference between Brazilian tensile strength (Eq. 5) and point load index (Eq. 7) equations will the factor of $2/\pi$ in the former if it is provided that an equal dimension is used in the both experiments. The inverse of this factor is 1.57, which is very close to the constant of linear equation. Actually, the both tests are an indirect measurement of tensile strength of the rocks.

3.5. Schmidt Rebound Hardness

The Schmidt hammer is one of the widely used portable instruments for estimating rock strength indirectly. In the civil engineering and mining industries, it is used for non-destructive testing of the quality of concrete and rock, both in the laboratory and in the field. It measures the surface rebound hardness of the tested material. The plunger of the hammer is placed against the specimen and the specimen is depressed by pushing the hammer against the specimen. Energy is stored in a spring, which is automatically released at a prescribed energy level and impacts a mass against the plunger. The distance of rebound of the mass is measured on a scale and is taken as a measure of hardness [19]. Therefore, the harder the surface, the higher the rebound distances. The Schmidt hammer models are designed with different levels of impact energy, but the types L and N are more commonly adopted for the testing of rock and concrete with impact energy levels of 0.735 and 2.207 Nm, respectively [11].

The significant correlations have been found between the rebound values of two models in field applications [6] and it has been also stated by Buyuksagis and Goktan [1] and Aydin and Basu [7] that the correlations found between rebound values and UCS of rocks by using the N-type hammer are consistently higher than those of the L-type. In order to overcome the discrepancy between readings of the two models and harmonize the data for a single Schmidt rebound number, a new correction factor has been proposed specifically for this study which is presented in Figure **11**. According to the data collected from [1], [7], [20], the following linear relationship is typically acquired between N-type and L-type Schmidt hammer rebound numbers:

$$R_L = 0.84 R_N \quad (r^2=0.945) \tag{10}$$

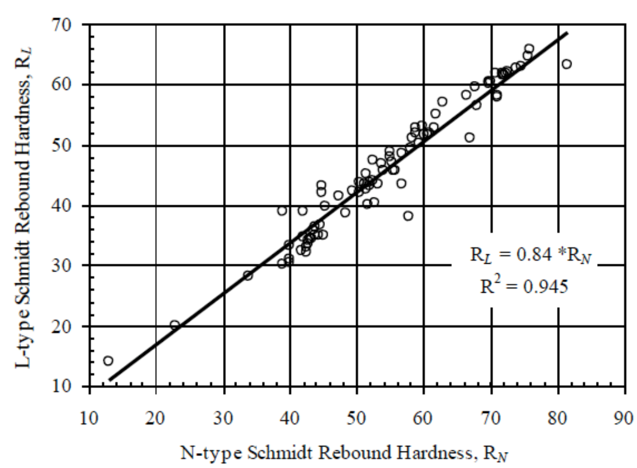

Figure 11: Relationship between N-type and L-type Schmidt rebound hardness.

There is a strong non-linear relationship between the SHV and the UCS, BTS, PLS and Young's modulus of rocks as shown in Figure **12**. The non-linear equation exhibited between UCS and Schmidt rebound hardness can be written as:

$$\sigma_c = 4.969 \exp(0.058 R_L) \quad (r^2=0.575) \tag{11}$$

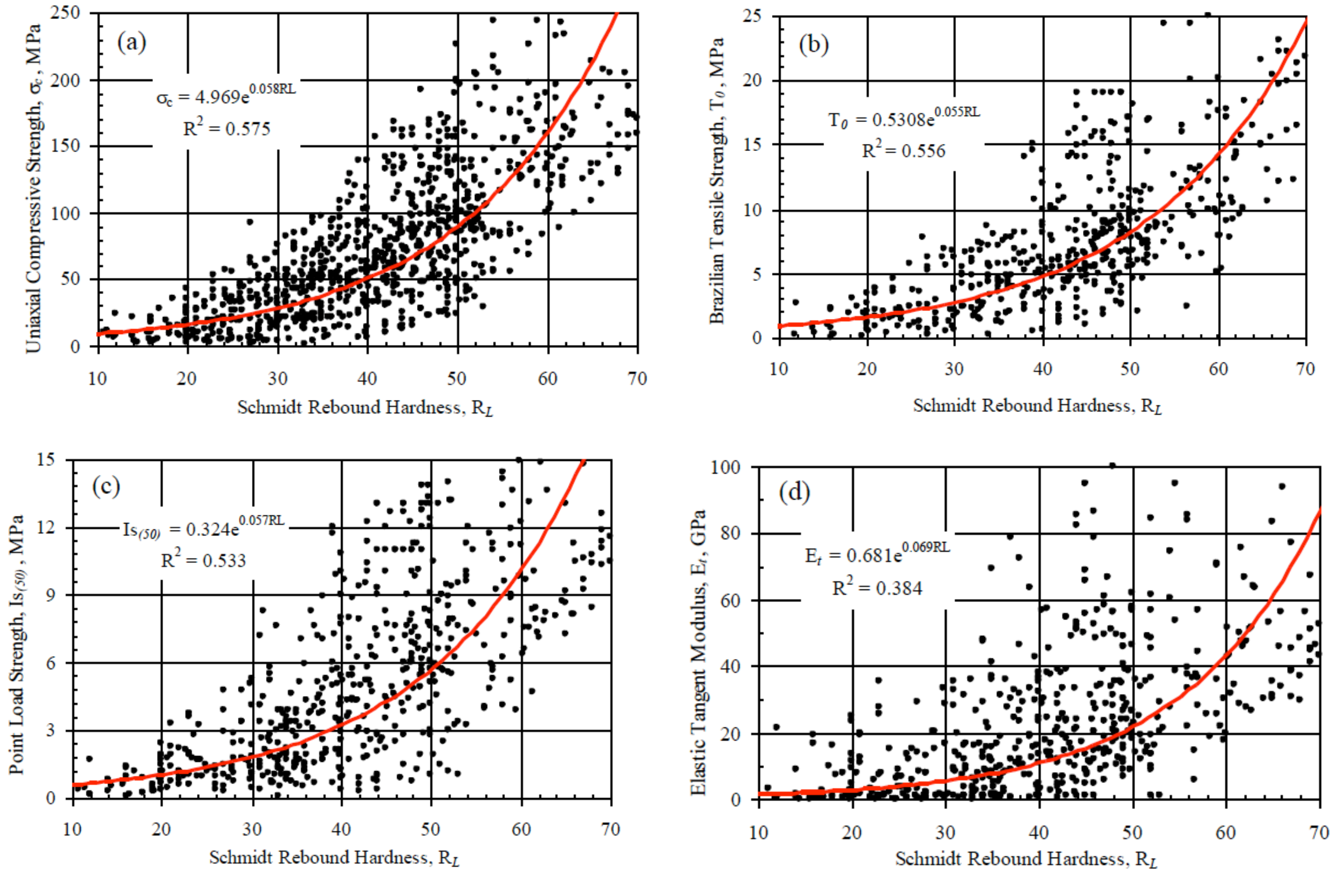

Figure 12: SHV vs. a. UCS, b. Brazilian tensile, c. Point load, and d. Young's modulus.

where σ_c is the uniaxial compressive strength in MPa and R_L the L-type Schmidt rebound value.

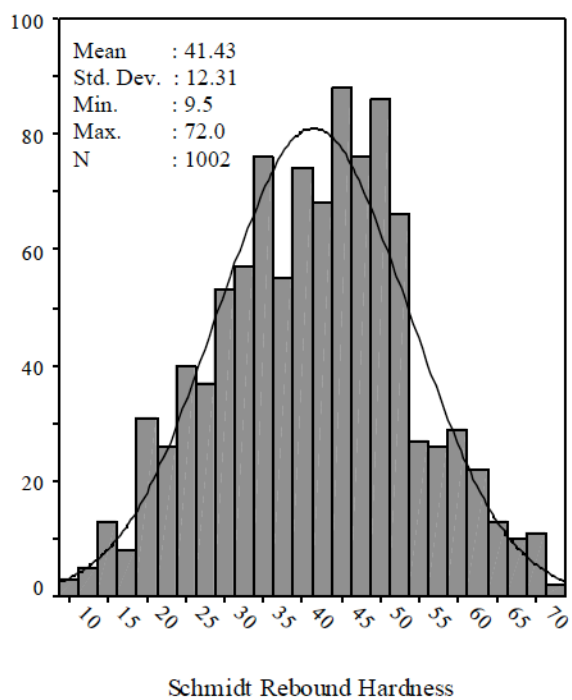

Figure 13: Histogram of Schmidt rebound hardness data.

The distribution (Figure **13**) of the Schmidt hammer values obtained from the collected data shows a noteworthy difference from the other property histograms. Most of the distributions evaluated in this study are usually skewed towards the larger values; the only exception is the distribution of dry density, which is skewed towards the smaller values. SHV histogram, however, shows a normal distribution. This may be attributed to the suggested methods for Schmidt rebound recording [11, 19]. In standard tests it is recommended to average certain readings whilst disregarding outliers. This may lead to this specific type of distribution. It is well established in statistics that the distribution of samples produced by averaging certain values can hypothetically produce bell-shaped (normal) distributions in accordance with the Central Limit Theorem whatever the distribution of the parent population from which the samples are drawn. Theoretically, if the parent population is normal, or the sample size is large (often n=10 or 20 will be large enough), then in either case the sampling distribution of average values has an approximately normal shape [21]. Also, there is a natural threshold at lower value (0) in SHV readings and it is limited on higher values. On the other hand, for the rest of the tests evaluated in this work, there is only a natural lower bound at 0; however, it is unlikely to extent an upper bound, which, theoretically, goes to infinity.

3.6. P-Wave Velocity

Ultrasonic techniques are non-destructive and easy to apply, both at site and laboratory conditions. The sound velocity of a rock mass is closely related to the intact rock properties. The P-wave velocity, as a natural characteristic of rocks and different materials, depends on their micro and macro structure, the existence of minor cracks, porosity and the characteristics of their mineralogical components, such as elastic parameters, density and micro-porosity [22]. In rock engineering, sound velocity (SV) techniques have increasingly been used to determine the dynamic properties of rocks [23]. The SV testing method determines the velocity of propagation of elastic waves in laboratory conditions. ISRM [11] describes three methods, the high and low frequency ultrasonic pulse techniques, and the resonant method. The velocities of longitudinal waves were determined using the pulse transmission method. The velocities of the P and S waves are calculated from the measured travel time and the distance between transmitter and receiver. In order to measure a SV index value, the Pundit testing machine is generally used. The Pundit has a pulse generator, transducers, and an electronic counter for time internal measurements.

There are statistically important correlations between P-wave velocity and both uniaxial compressive, Brazilian tensile, point load strengths and modulus of elasticity. The type of relationship obeys the law of either exponential or power as can be seen in Figure **14**. The relationship between p-wave velocity (V_p) and UCS (σ_c) can be formulated as:

$$\sigma_c = 5.912 V_p^{1.741} \qquad (r^2=0.645) \qquad (12)$$

where V_p in km/s and σ_c in MPa.

3.7. Effect of Water on Strength

It is well known that moisture content may influence the mechanical properties of rocks. Even igneous rocks are affected by the amount of water content. The work

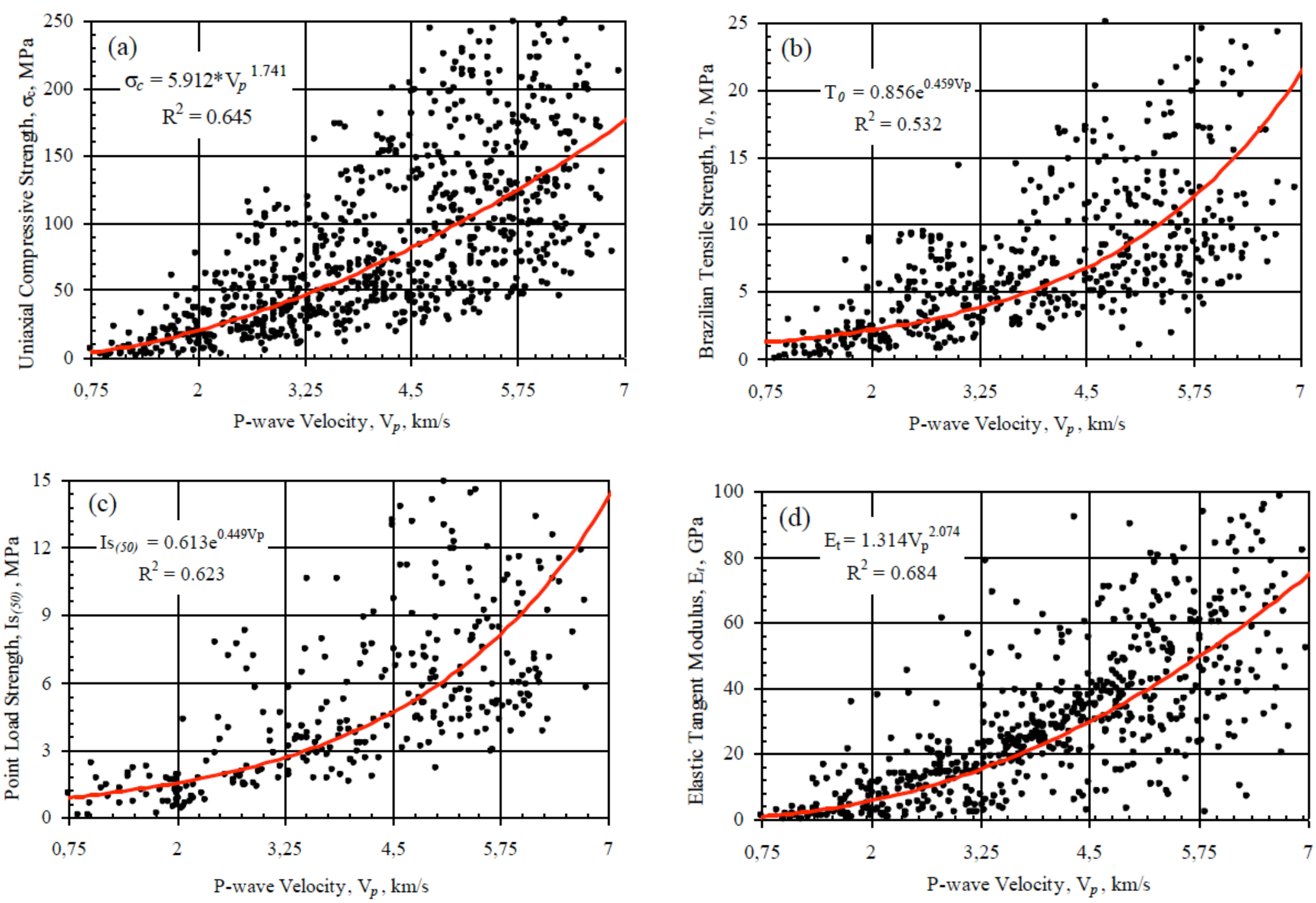

Figure 14: P-wave velocity vs. a. UCS, b. Brazilian tensile, c. Point load, and d. Young's modulus.

of many authors has conclusively shown that moisture has a significant effect on the strength of rocks [24-26] finding varying degrees of reduction in compressive strength, ranging from 6 to 85%, with increasing moisture content. Colback and Wiid [24] provided an example of the change in the UCS of quartzitic sandstone relative to the moisture content. The UCS of the rock decreases as the material becomes saturated. Dyke and Dobereiner [25] indicated a 25-35% loss in strength between UCS_{dry} and UCS_{wet} for British sandstones. The presence of liquids, most particularly water, substantially reduces the strength of the rocks. The lower strength was attributed to the decreasing of the surface free energy of the solid due to physical adsorption from the surrounding liquid [13].

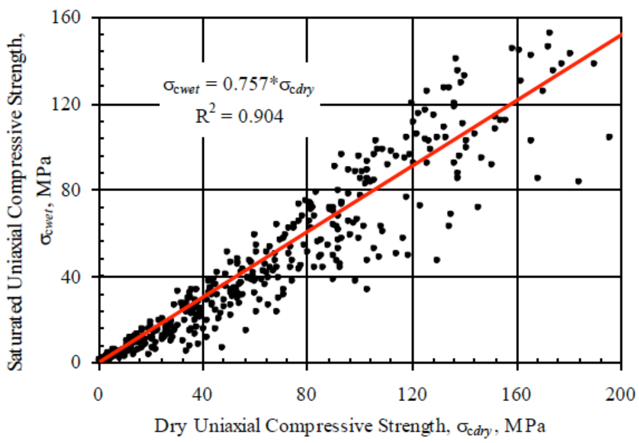

Figure 15: Relationship between dry and saturated samples of UCS.

To investigate the effect of moisture content on the strength of all rocks, first, the influence of water content on the UCS (σ_c) is investigated. The measured strength under saturated conditions is plotted as a function of the strength under dry conditions in Figure **15**. It appears that the saturated strength is linearly related to the dry strength and also, the saturation of test samples can promote a considerable loss of compressive strength in the rocks. These reductions in strength vary between –5.56% and 85.9%, the average strength reduction being 34.14% for all rocks (Figure **16**). The minus sign in reduction refers to an increase in saturated strength. The effect of water content on some other mechanical properties of rocks can be briefly described in a similar way in the following equations and, for practical purposes, the complement of constants of linear equations between saturated and dry values can be loosely taken as the average percent reduction in strength. Saturated uniaxial compressive, Brazilian tensile, point load strengths and Young's modulus can be formulated in terms of their dry equivalents, as follows:

$$\sigma_{cwet} = 0.757\sigma_{cdry} \qquad (r^2=0.904) \qquad (13)$$

$$T_{0wet} = 0.789T_{0dry} \qquad (r^2=0.911) \qquad (14)$$

$$Is_{(50)wet} = 0.823Is_{(50)dry} \qquad (r^2=0.876) \qquad (15)$$

$$E_{twet} = 0.828E_{tdry} \ (r^2=0.888) \qquad (16)$$

It is obvious from Figure **16** that most of the rocks exhibit a marked decrease in their measured strengths when tested wet. There are different viewpoints as to what causes strength loss in saturated rocks. A noteworthy explanation is that the filling of pores inside the rock probably leads to the strength reduction due to pore water pressure.

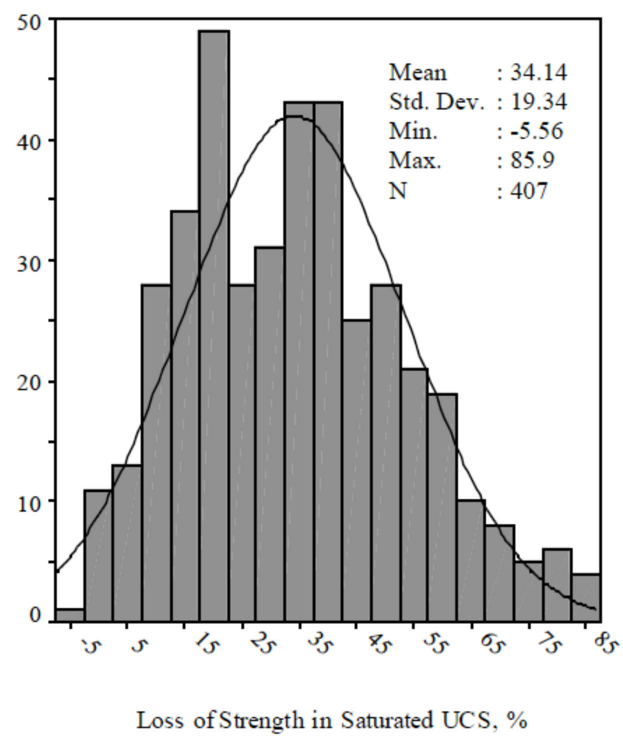

Figure 16: Histogram of % strength loss in saturated samples of UCS.

3.8. Modulus Ratio

The relationship between compressive strength and modulus of elasticity has been discussed by many researchers and they found a significant linear correlation between UCS and modulus of elasticity for different rock types (Appendix Table **A1**). The relationship between the elastic tangent modulus and UCS is graphically shown in Figure **17**. It can be easily seen that the value of the elastic modulus rises with increasing UCS. A linear relationship characterizes the correlation between the UCS and static modulus of elasticity of rocks in the database. The following formula relates the UCS to elastic Young's modulus:

$$E_t = 0.309\sigma_c \qquad (r^2=0.612) \qquad (17)$$

where E_t is the static modulus of elasticity in GPa and σ_c is in MPa.

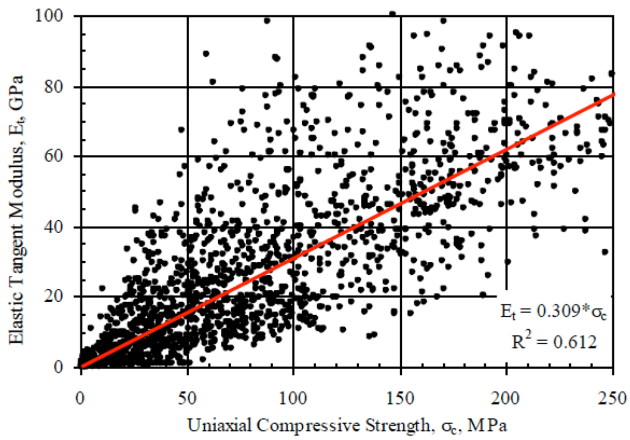

Figure 17: Relationship between UCS and Young's modulus.

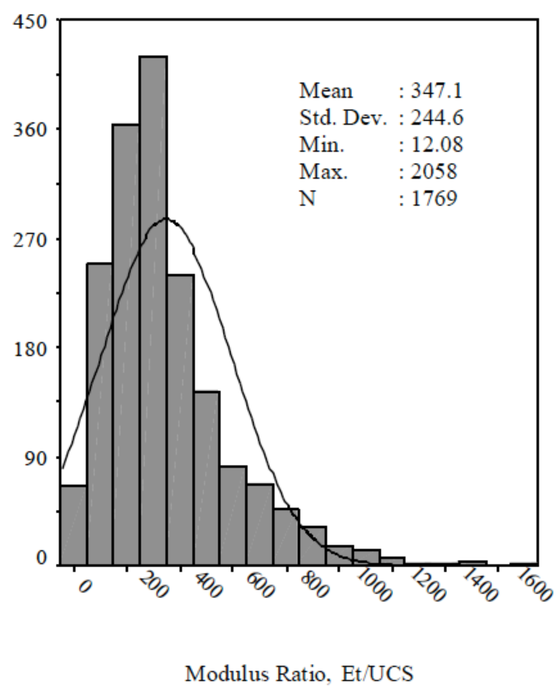

Figure 18: Histogram of modulus ratio data.

The constant of the equation, if multiplied by a factor of one thousand for unit conversion, is very close to the average modulus ratio (MR) of 347.1 given in Figure **18**. The modulus ratio is defined as the ratio of elastic tangent modulus to UCS [27] ($MR=E_t/\sigma_c$). Also, the same figure shows the modulus ratio of rocks in the database extending from a minimum of 12.08 for coal to a maximum of 2,058 for marlstone. The majority of the data (51.7%) have modulus ratios in the range of 200 to 500 (medium modulus ratio), 29.2% are less than 200 (low modulus ratio) and 19.1% have a high modulus ratio (>500) according to the engineering classification of intact rock proposed by Deere and Miller [27].

3.9. Shore Scleroscope

Originally designed for use on metals the Shore scleroscope is a non-destructive, hardness-measuring device. In this test, a diamond tipped indenter drops freely from a fixed height onto the surface of a specimen. The height of rebound indicates relative values of hardness (Shore hardness index, SHI), which may be correlated to the material strength. It is measured on a calibrated scale ranging from 0 to 140. The disadvantages of this test are that a large number of tests are required to give a good measure of the average hardness [28] and the measured hardness is sensitive to roughness of the specimen surface being tested [11]. Wuerker [29] showed plots of Shore hardness values against the UCS of more than 100 rock groups. Deere and Miller [27] published extensive studies on the relation between the Shore hardness and compressive strength of 28 different rocks, using the C-2 Shore scleroscope model.

A linear relationship is observed between Shore scleroscope hardness and both UCS and Schmidt rebound hardness as shown in Figure **19**. The equations that relate UCS and SHV in terms of SHI are given as:

$$\sigma_c = 2.11S_h - 16.23 \qquad (r^2=0.626) \qquad (18)$$

$$R_L = 0.424S_h + 21.73 \qquad (r^2=0.592) \qquad (19)$$

where σ_c is uniaxial compressive strength in MPa, R_L and S_h are the Schmidt rebound and Shore scleroscope values, respectively.

3.10. Fracture Toughness

Fracture toughness, K_C, is the resistance of a material to failure from fracture starting from a preexisting crack. In the case of testing on fracture toughness (mode I) two methods exist, the Chevron bend specimen and the short rod specimen,

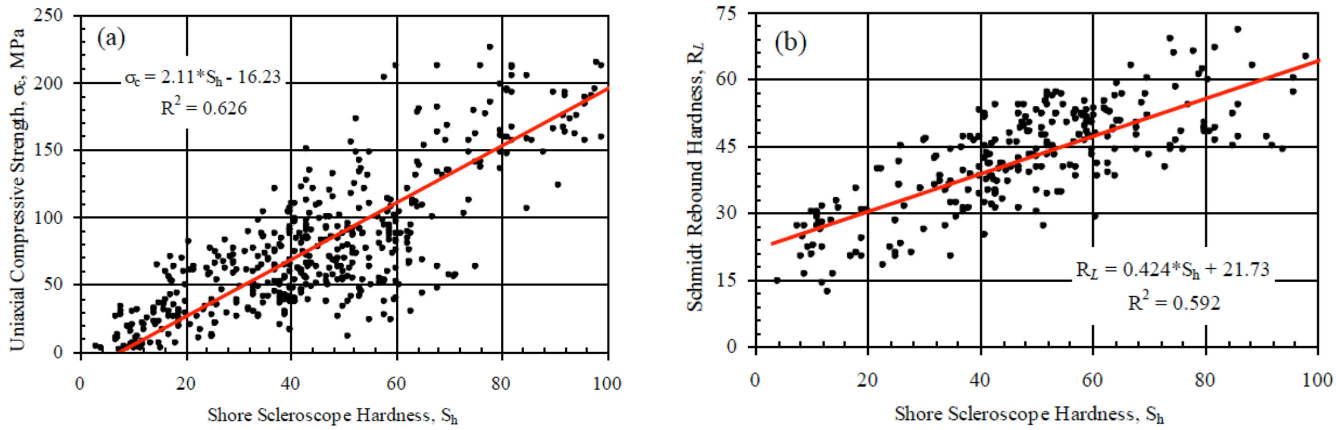

Figure 19: Shore scleroscope hardness vs. a. UCS, and b. Schmidt rebound hardness.

respectively. Both methods allow two levels of testing Level 1 requires only the recording of maximum load during bending and is supposed to be suitable for screening purposes. Level 2 testing needs load and displacement measurements requiring more sophisticated testing apparatus. Level 1 testing on fracture toughness K_{IC} may serve to obtain index values for intact rock with respect to its resistance to crack propagation [30]. In recent years there is a growing research into the measurement of the fracture toughness of intact rock. As such, fracture toughness values for rock do not exist in a large extent in the database for the comparison with the other tests. However, the practical usefulness of this test has been suggested by Gunsallus and Kulhawy [31].

A linear relationship is found between fracture toughness and Brazilian tensile strength as shown in Figure **20**. The following equation gives the estimation of Brazilian tensile strength (T_0) as a function of the fracture toughness (K_{IC}):

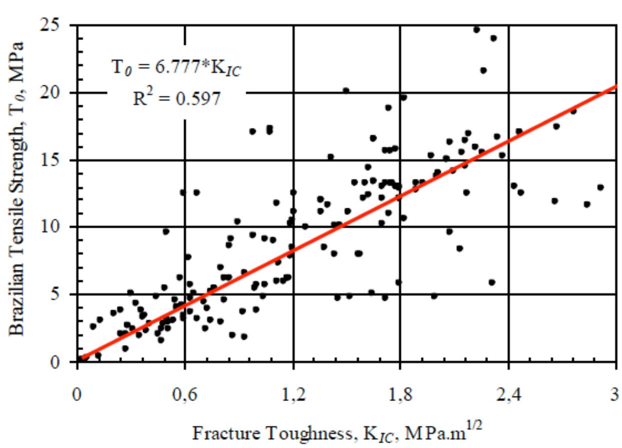

Figure 20: Relationship between fracture toughness and Brazilian tensile strength.

$$T_0 = 6.777K_{IC} \quad (r^2=0.597) \tag{20}$$

where K_{IC} is in MPa.m$^{1/2}$ and T_0 is in MPa.

A collection of the best-fitting equations for the properties of rocks in the database is given in detail in Table **3**. It includes all of the selected equations that are found to be significant whether they have been mentioned directly in the text. The correlation coefficient (R) measures the extent to which two variables are related to each other. A quick evaluation of Table **3** indicates that for all rock types in the database moderate to strong correlations (0.59 to 0.97) are found between different engineering properties of intact rocks.

CONCLUSIONS

An extensive review of rock mechanics literature reveals that many studies are performed to investigate the relationships between rock strength, deformation and rock hardness. Many simple and complex empirical models are increasingly proposed from these studies. In the current research a comprehensive database was accomplished to provide the basis for the detailed analysis of rocks at different origin and it is aimed to gain insight on existing studies looking for the relationships between rock physical and mechanical properties.

Based on the results of the regression analysis and frequency histograms of rock properties evaluated in the present study the following conclusions can be derived:

- It was seen in the frequency histograms that most of the rock properties and ratios tend to skew in the direction of higher values except for dry density and Schmidt rebound hardness.

- There is a common tendency in all XY-plots relating rock strength properties that the higher the values the more scattered the data points. This is typically due to correlating two log-normally distributed variables, which generally produce this type of pattern [32].

- The constant of 10.04 found in linear equation relating UCS to Brazilian tensile strength statistically validates the customary statement "generally rocks have tension strength of one tenth of their compression strength" [9].

- The problem in establishing a single factor relating $Is_{(50)}$ and UCS values is highlighted. The constant 15.15 found from the linear equation and the average ratio of 18.01 between the UCS and the point load strength for all rocks are both comparatively less than the 24 quoted by Broch and Franklin [4].

- The results show that moisture substantially reduces the strengths of rocks. The wetting of the rock samples causes an average unconfined compressive strength loss of 34%.

- The current work clearly confirms that although the limited test data for a specific rock type is more likely to produce good correlations, it is still possible to obtain some significant relationships for different kinds of rocks.

- The relevant relationships are considered to be the best suited for the prediction of engineering properties of all rock types since the data included in the analyses cover a wide range of property and rock lithology.

- A final remark can be pointed out here that the data are scattered in the most of the plots. A possible reason for the scattering is that the test values in the plots were collected from a wide range of scientific sources covering different rock units and testing procedures. On the other hand, if some data could be re-plotted for the similar rock classes or rock groups, this procedure may be increase the reliability of relevant correlations. The relationships between the properties of the same rock type and similar rock units should be investigated in the future.

APPENDIX

Table A1: A Supplementary List of the Empirical Equations Proposed by Various Authors

Reference	Equation	R	Rock unit
Wuerker [29]	$\sigma_c = 2.76S_h$		More than 100 rock groups
	$\sigma_c = 5\ldots22T_0$		
Fairhurst [15]	$\sigma_c = 11.5T_0$		
Hobbs [33]	$T_0 = 0.25\sigma_c + 4.6$	0.88	Massive and laminated rocks
	$\sigma_c = 2.84T_0 - 3.34$		
D'Andrea *et al.* [34]	$\sigma_c = 15.3Is_{(25)} + 16.3$	0.95	49 lithologic units
Deere & Miller [27]	$\sigma_c = 9.97\exp(0.02R_L{*}\gamma_{dry})$	0.94	28 different lithology
	$\sigma_c = 28.75\exp(0.009S_h{*}\gamma_{dry})$	0.92	
	$\sigma_c = 31.19\gamma_{dry} - 36.27$	0.60	
	$\sigma_c = 3.54S_h - 42.85$	0.90	
	$\sigma_c = 8.59R_L - 240.62$	0.88	
	$\sigma_c = 20.7Is_{(54)} + 29.6$	0.92	
	$E_t = 0.19R_L{*}\gamma_{dry}^2 - 7.87$	0.88	
	$E_t = 0.042\gamma_{dry}{*}S_h + 12.62$	0.80	
	$E_t = 0.094\gamma_{dry}{*}R_L - 20.28$	0.85	
	$E_t = 0.74S_h + 11.52$	0.75	
	$E_t = 1.786R_L - 29.59$	0.73	
Smorodinov *et al.* [35]	$\sigma_c = 0.0864\exp(0.291\gamma_{dry})$		Carbonate rocks
	$\sigma_c = 254\exp(-0.091n_{eff})$		
Broch & Franklin [4]	$\sigma_c = 23.7Is_{(50)}$	0.88	15 different rocks
Szlavin [36]	$\sigma_c = 20NCB + 12.4$	0.88	
	$\sigma_c = 2.1S_h - 35.2$	0.84	

	$\sigma_c = 3.6T_0 + 15.2$	0.76	
	$T_0 = 0.37S_h – 3.9$	0.81	
	$T_0 = 0.16\sigma_c + 4.4$	0.76	
	$T_0 = 3.1NCB + 5.8$	0.73	
Bieniawski [16]	$\sigma_c = 24Is_{(54)}$		Sandstone, quartzite, norite
Dearman & Irfan [37]	$\sigma_c = 0.00016R_L^{3.47}$	0.86	Granite
	$E_t = 1.89R_L - 60.55$	0.93	
Hassani *et al*. [38]	$\sigma_c = 29Is_{(50)}$	0.94	Limestone, siltstone , sandstone
	$\sigma_c = 10.5T_0 + 1.2$	0.85	
Kidybinski [39]	$\sigma_c = 0.477\exp(0.045R_N * \gamma_{dry})$	0.82	Coal, shale, mudstone, sandstone
Singh *et al*. [40]	$\sigma_c = 2.00R_L$	0.72	Sandst., siltst., mudst., seatearth
	$T_0 = 0.23R_L – 0.81$	0.72	
Sheorey *et al*. [41]	$\sigma_c = 0.40R_N - 3.60$	0.94	Coal
Gunsallus & Kulhawy [31]	$\sigma_c = 16.5Is_{(50)} + 51.0$	0.69	Sandstone, limestone, dolostone
	$\sigma_c = 12.4T_0 – 9.0$	0.76	
	$K_{IC} = 0.0044\sigma_c + 1.04$	0.72	
	$K_{IC} = 0.0736T_0 + 0.76$	0.73	
	$K_{IC} = 0.0995Is_{(50)} + 1.11$	0.67	
Huang & Wang [42]	$K_{IC} = 0.65V_p – 1.68$	0.90	
Haramy & DeMarco [43]	$\sigma_c = 0.994R_L - 0.383$	0.70	Coal
	$\sigma_c = 0.287R_L^{1.33}$	0.85	
Ghose & Chakraborti [44]	$\sigma_c = 0.88R_L - 12.11$	0.87	Coal
	$T_0 = 0.06R_L – 0.92$	0.81	
Van Heerden [23]	$E_t = 0.075E_d^{1.560}$	0.98	10 different rocks
Singh & Eksi [45]	$\sigma_c = 23.31Is_{(50)}$	0.95	Gypsum, marlstone
	$\sigma_c = -1.14 + 27.2NCB$	0.94	
	$\sigma_c = 2.5S_h$	0.94	
	$Is_{(50)} = 1.1NCB$	0.94	
	$S_h = 10.2NCB$	0.95	
Vallejo *et al*. [46]	$\sigma_c = 12.5Is_{(50)}$	0.62	Shale
	$\sigma_c = 17.4Is_{(50)}$	0.38	Sandstone
O'Rourke [47]	$\sigma_c = 4.85R_L - 76.18$	0.77	5 different rocks
	$\sigma_c = 21.8Is_{(50)} + 43.2$	0.77	
Ojo & Brook [13]	$\sigma_c = 3.54(S_h – 12)$		Sandstone, mudstone
Xu *et al*. [48]	$\sigma_c = 2.98\exp(0.06R_L)$	0.95	Mica-schist
	$E_t = 1.77\exp(0.07R_L)$	0.96	
	$\sigma_c = 2.99\exp(0.06R_L)$	0.91	Prasinite
	$E_t = 2.71\exp(0.04R_L)$	0.91	
	$\sigma_c = 2.98\exp(0.063R_L)$	0.94	Serpentinite
	$E_t = 2.57\exp(0.03R_L)$	0.88	
	$\sigma_c = 3.78\exp(0.05R_L)$	0.93	Gabro
	$E_t = 1.75\exp(0.05R_L)$	0.95	
	$\sigma_c = 1.26\exp(0.52R_L * \gamma_{dry})$	0.92	Mudstone
	$E_t = 0.07\exp(0.31R_L * \gamma_{dry})$	0.89	
Cargill & Shakoor [18]	$\sigma_c = 3.32\exp(0.043R_L * \gamma_{dry})$	0.93	Sandstone
	$\sigma_c = 18.17\exp(0.018R_L * \gamma_{dry})$	0.98	Carbonates
	$\sigma_c = 23.0Is_{(50)} + 13$	0.94	13 lithologic units
Sachpazis [49]	$\sigma_c = 4.29R_L - 67.52$	0.96	33 different carbonates
	$E_t = 1.94R_L - 33.93$	0.88	
Ghosh & Srivastava [50]	$\sigma_c = 16.0Is_{(50)}$	0.75	Granitic rocks
Christaras [51]	$\gamma_{dry} = 0.75 + 0.30V_p$	0.92	Marly limostone

	$\sigma_c = 6.202\exp(0.48V_p)$	0.97	
Grasso *et al*. [52]	$\sigma_c = 9.30Is_{(50)} + 20.04$	0.71	Mudstone
	$T_0 = 1.53Is_{(50)} – 0.21$	0.89	
	$\sigma_c = 25.67Is_{(50)}^{0.57}$	0.73	
	$T_0 = 1.01\exp(0.47Is_{(50)})$	0.93	
	$\sigma_c = 9.68\exp(0.045R_L)$	0.75	
	$E_t = 1.28\exp(0.033R_L)$	0.72	
	$\sigma_c = 2.83\exp(1.14V_p)$	0.64	
	$E_t = 0.29\exp(1.08V_p)$	0.80	
Whittaker *et al*. [53]	$T_0 = 9.35K_{IC} – 2.53$	0.77	
	$K_{IC} = 0.708 + 0.006\sigma_c$		
	$K_{IC} = 0.27 + 0.107T_0$		
	$K_{IC} = 0.336 + 0.026E_t$		
Vernik *et al*. [54]	$\sigma_c = 254(1- 0.027n_{tot})^2$	0.96	Arenite
Singh & Singh [55]	$\sigma_c = 23.37Is_{(50)}$	0.98	Quartzite
Arioglu & Tokgoz [56]	$T_0 = 0.081\sigma_c^{0.983}$	0.85	20 different rocks
Ulusay *et al*. [57]	$\sigma_c = 19.5Is_{(50)} + 12.7$		
Kahraman [58]	$\sigma_c = 0.00045(R_N*\gamma_{dry})^{2.46}$	0.96	10 different lithology
Gokceoglu [59]	$\sigma_c = 0.0001R_L^{3.27}$	0.84	Marl
Chau & Wong [60]	$\sigma_c = 12.5Is_{(50)}$	0.73	Granite, tuff
Zhixi *et al*. [61]	$K_{IC} = -0.332 + 0.361V_p$	0.96	Sandstone
	$K_{IC} = 0.054V_p + 0.388$	0.75	Shale
Karpuz & Pasamehmetoglu	$V_p = 6.05n_{eff}^{-0.47}$	0.95	Andesite
[62]	$V_p = 6.03 – 0.194n_{eff}$	0.90	
	$n_{eff} = 23.2\exp(-0.04R_L)$	0.96	
	$n_{eff} = 25.2 – 0.28R_L$	0.88	
	$Is_{(50)} = 871n_{eff} - 1.48$	0.80	
	$Is_{(50)} = 98.8n_{eff} - 1.18$	0.91	
	$V_p = 4.33 + 1.22\ln(\sigma_c)$	0.91	
	$V_p = 0.48 + 0.069R_L$	0.95	
	$Is_{(50)} = 0.0465V_p^{2.18}$	0.94	
	$R_L = 9.51Is_{(50)}^{0.47}$	0.97	
	$R_L = 17.26\ln(\sigma_c) - 67.25$	0.94	
Brown & Reddish [63]	$K_{IC} = 3.21\gamma_{dry} - 6.95$	0.95	17 different rocks
	$K_{IC} = 3.35\gamma_{dry} - 6.87$	0.84	
Holmgeirsdottir & Thomas	$\sigma_c = 4.65S_h – 40.46$		15 different rocks
[64]	$\sigma_c = 3.0S_h – 22.8$		
Zhang *et al*. [65]	$T_0 = 8.88K_{IC}^{0.62}$	0.97	
Tugrul & Zarif [66]	$\sigma_c = 162.9\gamma_{dry} – 362$	0.93	Sandstone
	$\sigma_c = 140.16\exp(-0.19n_{eff})$	0.97	
Tugrul & Zarif [67]	$\sigma_c = 8.36R_L - 416$	0.87	Granitic rocks
	$\gamma_{dry} = 2.644 - 0.025n_{tot}$	0.86	
	$V_p = 6.52 - 0.36n_{tot}$	0.81	
	$\sigma_c = 577.2\gamma_{dry} – 1347$	0.82	
	$\sigma_c = 35.54V_p – 55$	0.80	
	$\sigma_c = 201 - 78.22n_{eff}$	0.81	
	$\sigma_c = 183 - 16.55n_{tot}$	0.83	
	$\sigma_c = 15.25Is_{(50)}$	0.98	
	$T_0 = 0.15\sigma_c – 0.73$	0.92	
	$E_t = 0.35\sigma_c – 12$	0.94	
Koncagul & Santi [68]	$\sigma_c = 0.895S_h + 41.98$	0.57	Shale

	$\sigma_c = 0.658I_{d2} + 9.081$	0.63	
	$S_h = 0.37I_{d2} - 5.23$	0.56	
Starzec [69]	$\gamma_{dry} = 0.2V_p + 1.73$	0.74	Crystalline rocks
	$E_t = 0.48E_d - 3.26$	0.91	
Bearman [30]	$K_{IC} = 0.209I_{s(50)}$	0.95	12 different rocks
Katz *et al*. [70]	$\sigma_c = 2.208\exp(0.067R_N)$	0.96	Limestone, sandstone
	$E_t = 0.00013R_N^{3.09}$	0.99	Syenite, granite
	$\gamma_{dry} = 1.308\ln(R_N) - 2.874$	0.96	
Altindag [71]	$K_{IC} = -0.221 + 0.003\sigma_c$	0.96	Marble, limest., sandst., andesite
	$K_{IC} = -0.957 + 0.281T_0$	0.90	
	$K_{IC} = -0.916 + 0.163E_t$	0.81	
	$K_{IC} = 0.632 + 0.325Is_{(50)}$	0.70	
	$K_{IC} = -0.820 + 4.731\log(NCB)$	0.75	
Gupta & Rao [12]	$E_t = 0.286\sigma_c^{0.98}$	0.93	Igneous rocks
	$E_t = 0.080\sigma_c^{1.91}$	0.77	Sedimentary rocks
	$E_t = 0.150\sigma_c^{1.11}$	0.91	All rocks
Tugrul & Zarif [72]	$\gamma_{dry} = 2.70 - 0.033n_{tot}$	0.88	Limestone
	$n_{tot} = 0.62V_p + 5.37$	0.78	
	$\sigma_c = 538.9\gamma_{dry} - 1309$	0.89	
	$\sigma_c = 16.73V_p + 21.25$	0.94	
	$\sigma_c = 144 - 17.29n_{tot}$	0.77	
	$\sigma_c = 14.38Is_{(50)} + 42$	0.92	
	$T_0 = 0.56\sigma_c - 15$	0.90	
	$E_t = 0.512\sigma_c - 20.41$	0.90	
Sulukcu & Ulusay [73]	$\sigma_c = 15.31Is_{(50)}$	0.83	23 different rocks
	$T_0 = 2.30Is_{(50)}$	0.80	
Kahraman [74]	$\sigma_c = 6.97\exp(0.014R_N * \gamma_{dry})$	0.78	Carbonates
	$\sigma_c = 9.95V_p^{1.21}$	0.83	
	$\sigma_c = 23.6Is_{(50)} - 2.69$	0.93	Coal measure rocks
	$\sigma_c = 8.41Is_{(50)} + 9.51$	0.85	Other rocks
Chaterrejee & Mukhopadhyay	$\sigma_c = 55.57\gamma_{dry} - 100.75$	0.94	Sandstone, siltstone, limest., shale
[75]	$\sigma_c = 10.33T_0^{0.89}$	0.97	
	$\sigma_c = 64.23\exp(-0.085n_{eff})$	0.96	
	$E_t = 0.73\sigma_c + 0.17$	0.96	
Yilmaz & Sendir [76]	$\sigma_c = 2.27\exp(0.059R_L)$	0.98	Gypsum
	$E_t = 3.15\exp(0.054R_L)$	0.91	
Zhang [77]	$T_0 = 6.88K_{IC}$	0.97	
Lashkaripour [78]	$\sigma_c = 21.43Is_{(50)}$	0.93	Mudrock
	$\sigma_c = 210.12n_{tot}^{-0.821}$	0.82	
	$E_t = 0.103\sigma_c^{1.086}$	0.90	
	$E_t = 37.9n_{tot}^{-0.863}$	0.83	
Vasarhelyi [79]	$E_t = 0.178\sigma_c$	0.86	Sandstone
	$\sigma_{csat} = 0.759\sigma_{cdry}$	0.95	
Quane & Russel [80]	$\sigma_c = 24.4Is_{(50)}$		Strong rocks
	$\sigma_c = 3.86Is_{(50)}^2 + 5.65Is_{(50)}$		Weak rocks
Alber & Brardt [81]	$K_{IC} = 0.0654\exp(0.681V_p)$	0.94	
	$K_{IC} = 0.015\exp(1.74\gamma_{dry})$	0.87	
Hudyama *et al*. [82]	$\sigma_c = -49.36\ln(n_{tot}) + 189.35$	0.79	Tuff
Tugrul [83]	$\sigma_c = 195\exp(-0.21n_{tot})$	0.89	Sandst., basalt, limest., granodior.
	$\sigma_c = 125\exp(-0.20n_{eff})$	0.89	

	$\gamma_{dry} = 2.713 - 0.033n_{tot}$	0.97	
	$\gamma_{dry} = 2.684 - 0.151\log(n_{eff})$	0.92	
	$n_{tot} = 4.36\log(n_{eff}) + 1.17$	0.91	
Yasar & Erdogan [22]	$\gamma_{dry} = 1.1623\ln(S_h) - 2.093$	0.89	Limest., sandst., marble, basalt
	$\gamma_{dry} = 0.9377\ln(R_L) - 1.03$	0.92	
	$R_N = 56.883\ln(S_h) - 181.38$	0.91	
	$n_{eff} = -0.2R_L + 11.21$	0.89	
	$n_{eff} = -9.06\ln(S_h) + 38.042$	0.64	
	$\sigma_c = 1x10^{-8}S_h^{5.555}$	0.91	
	$\sigma_c = 4x10^{-6}R_L^{4.292}$	0.89	
Jeng *et al*. [84]	$\sigma_c = 133.7\exp(-0.107n_{tot})$	0.89	Sandstone
Tsiambaos & Sabatakakis [85]	$\sigma_c = 7.3Is_{(50)}^{1.71}$	0.91	Sedimentary rocks
	$\sigma_c = 23.0Is_{(50)}$	0.87	
Yasar & Erdogan [86]	$V_p = 0.032\sigma_c + 2.02$	0.89	Carbonate rocks
	$V_p = 0.094E_t + 1.75$	0.93	
	$V_p = 4.32\gamma_{dry} - 7.51$	0.90	
Palchik & Hatzor [87]	$Is_{(50)} = 7.74\exp(-0.039n_{tot})$	0.92	Chalk
	$\sigma_c = 273.2\exp(-0.076n_{tot})$	0.81	
	$\sigma_c = 8\ldots18Is_{(50)}$		
Dincer *et al*. [88]	$\sigma_c = 2.75R_L - 36.83$	0.97	Andesite, basalt, tuff
	$E_t = 0.47R_L - 6.25$	0.92	
	$E_t = 0.17\sigma_c + 0.28$	0.92	
Basarir *et al*. [89]	$\sigma_c = 10.96Is_{(50)}$	0.79	Dacite
	$\sigma_c = 4.72R_N^{0.69}$	0.81	
	$\sigma_c = 0.68\gamma_{dry}*V_p^{2.69}$	0.81	
Aydin & Basu [7]	$\sigma_c = 1.45\exp(0.07R_L)$	0.92	Granite
	$E_t = 1.04\exp(0.06R_L)$	0.91	
	$\sigma_c = 0.92\exp(0.07R_N)$	0.94	
	$E_t = 0.72\exp(0.05R_N)$	0.92	
	$n_{tot} = -0.43R_L + 30.4$	0.89	
	$n_{eff} = -0.32R_L + 21.15$	0.90	
	$\gamma_{dry} = 0.01R_L + 2.00$	0.92	
Kahraman *et al*. [90]	$\sigma_c = 10.91Is_{(50)} + 27.41$	0.78	38 different rocks
Vasarhelyi [91]	$\gamma_{dry} = -0.0268n_{tot} + 2.71$	0.99	Limestones
	$T_0 = 0.129\sigma_c$	0.86	
	$\sigma_c = 0.056\exp(2.75\gamma_{dry})$	0.80	
Fener *et al*. [92]	$\sigma_c = 9.81Is_{(50)} + 39.32$	0.85	11 different rocks
	$\sigma_c = 4.24\exp(0.059R_N)$	0.81	
Sousa *et al*. [93]	$\sigma_c = 124.28n_{eff}^{-0.56}$	0.81	Granite
	$V_p = 4.083n_{eff}^{-0.42}$	0.89	
	$\sigma_c = 4.0V_p^{1.247}$	0.85	
Kahraman & Alber [94]	$\sigma_c = 17.91Is_{(50)} + 7.93$	0.89	Fault breccia
Aydin & Basu [95]	$T_0 = 0.00004\exp(4.6\gamma_{dry})$	0.83	Igneous rocks
	$T_0 = 8.3\exp(-0.14n_{eff})$	0.83	
	$T_0 = 10.74\exp(-0.111n_{tot})$	0.86	
Kolay & Kayabali [96]	$\gamma_{dry} = 0.239Is_{(50)} + 1.535$	0.69	Coarse grained rocks
	$I_{d2} = 29.0\exp(0.412\gamma_{dry})$	0.75	
	$I_{d2} = 10.48Is_{(50)} + 44.5$	0.80	
Palchik [97]	$\sigma_c = 7164n_{tot}^{-2.05}$	0.99	Sandy shale
	$c = 0.55\exp(0.088\sigma_c)$	0.97	
	$c = 43.9n_{tot}^{-1.1}$	0.92	

	$\phi = 93.53 - 1.24n_{tot}$	0.99	
Buyuksagis & Goktan [1]	$\sigma_c = 2.482\exp(0.073R_L)$	0.94	Granite, marble, limest. travertine
Shalabi *et al*. [98]	$\sigma_c = 3.201R_L - 45.6$	0.76	Dolostone, limestone
	$\sigma_c = 3.326S_h - 79.76$	0.80	
	$E_t = 0.971S_h - 26.907$	0.92	Shale
	$\sigma_c = 1.581S_h - 62.2$	0.85	
	$\sigma_c = 73\gamma_{dry} - 110.32$	0.62	
	$E_t = 0.531\sigma_c + 9.57$	0.84	
Sharma & Singh [99]	$\sigma_c = 0.0642V_p - 117.99$	0.90	Seven rock types
	$I_{d2} = 0.0069V_p + 78.577$	0.78	
	$ISI = 0.0118V_p + 58.105$	0.81	
Yagiz [100]	$\sigma_c = 0.0028R_L^{2.584}$	0.92	Travertine, limestone, schist
	$E_t = 1.233R_L - 17.8$	0.85	
	$V_p = 0.537R_L^{0.562}$	0.77	
	$\gamma_{dry} = 6.434R_L^{0.348}$	0.78	
	$n_{eff} = 344.3\exp(-0.115R_L)$	0.71	
Mishra & Basu [101]	$\sigma_c = 5BPI$	0.93	Granite, schist, sandstone
	$\sigma_c = 14.63Is_{(50)}$	0.94	
	$\sigma_c = 2.38\exp(0.065R_L)$	0.93	
Fereidooni [102]	$I_{d2} = 12.89\gamma_{dry} + 62.61$	0.95	Hornfels
	$R_L = 48.48\, n_{eff}^{-0.23}$	0.96	
	$T_0 = 3.5\times10^{-6}R_L^{3.80}$	0.96	
	$T_0 = 2.28Is_{(50)} - 4.66$	0.97	
	$\sigma_c = 24.36Is_{(50)} - 2.14$	0.99	
	$\sigma_c = 0.02R_L^{2.28}$	0.96	
	$\sigma_c = 10.03T_0 + 55.19$	0.96	
Hebib *et al*. [103]	$\sigma_c = 0.0322\exp(3.017\gamma_{dry})$	0.93	Sedimentary rocks
	$\sigma_c = -31.14\ln(n_{eff}) + 108.47$	0.92	
	$\sigma_c = 2.8555\exp(0.063R_L)$	0.87	

σ_c: Uniaxial compressive strength (MPa), T_0: Brazilian tensile strength (MPa), E_t: Elastic tangent modulus (GPa), E_d: Dynamic elasticity modulus (GPa), γ_{dry}: Dry density (gr/cm^3), $Is_{(50)}$: Point load strength index (MPa), n_{tot}: Total porosity (%), n_{eff}: Effective porosity (%), R_L: L-type Schmidt rebound hardness, R_N: N-type Schmidt rebound hardness, S_h: Shore scleroscope hardness, I_{d2}: Slake durability index (%), V_p: P-wave velocity (km/s), c: Cohesion (MPa), ϕ: Internal friction angle (^{o}), K_{IC}: Fracture toughness (MPa.m$^{1/2}$), NCB: Cone indenter number, ISI: Impact strength index (%), BPI: Block punch index (MPa).

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