CORRELATION BETWEEN THE POINT LOAD INDEX, $I_{s(50)}$, AND THE RESISTANCE TO UNCONFINED COMPRESSION IN LIMESTONE FROM THE COMUNIDAD VALENCIANA, SPAIN

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abstract

Unconfined compression strength is one of the most important engineering parameters in rock mechanics; it is used to characterize and study the behavior of solid rocks. A good correlation between the unconfined compression strength test (UCS) and the point load strength (PLS) can be very useful because it allows for faster and cheaper testing than others with the same security to calculate the structures and performances of solid rocks. A preliminary step to implement the correlation is to have a good method to determine resistance to point load. This determination is quite correct if you have a sufficient number of tests on specimens of the same nature, but usually it does not occur and it is necessary to apply a size correction factor, $F = (D_e / 50)^{\alpha}$*, with* $\alpha = 0.45$*. This paper is based on limestone from Comunidad Valenciana (Spain) because it represents a very high percentage of their rocks. The implementation has been conducted over 700 field and laboratory tests of which 255 are PLT test, 45 are UCS test, and the rest are other parameters like: porosity and specific weight, Slake Durability, and ultrasonic velocity, among others.*

keywords

resistance to point load, point load strength, Franklin test, size correction factor, unconfined compression

1 INTRODUCTION

The unconfined compression strength (UCS) is a fundamental parameter that is often costly, difficult, or even impossible to determine through normalized testing [1, 2, 3]; however, it may be determined with a simple geologist hammer or with the Schmidt hammer, among others [3]. But for more approximate values it may be obtained via simple in situ or laboratory tests, like the Brazilian traction test or the Point Load Test (PLT).

The different field and/or laboratory test methods to obtain more or less precisely, or at least estimate, the unconfined compression strength, σ_c , are summarized in Table 1.

Table 1. Methods for determining the unconfined compression strength, [4]

Application	Method	Is obtained by
Field Test	Organoleptic tests	Subjective estimate
	Schmidt hammer	Objective estimation
Laboratory test	Point load test	Correlation
	Brazilian Traction	
	Unconfined compression strength	Direct measurement

The Point Load Test (PLT) allows us to determine the Point Load Resistance Index or Point Load Strength, ($I_{s(50)}$). It is useful in classifying rocks by their resistance. Ever since its emergence, Franklin and other researchers have proposed a correlation with UCS [5], but this value should be adjusted according to the genesis of the different rock masses.

The PLT uses specimens formed by regular and irregular rock fragments or cylindrical specimens [6, 7] (generally from probes) that are not subjected to strict requirements to be admissible. This test may take place *in situ* or in the laboratory. This work presents the most appropriate values correlating PLT and UCS in limestone from a broad zone in eastern Spain, and proposing a value.

2 EXPERIMENTAL WORK

The samples tested in this research were collected from eight different locations in the Comunidad Valenciana, Spain, and come from healthy limestone masses that present a maximum equidistance of 140 km and a mean equidistance of 95 km. The largest excavations were conducted on Roadway CV-13, Torreblanca section (Castellón) and in the probes for the Project for the Cullera Tunnel (Valencia). The other investigations took place on the access road to the Loriguilla Reservoir (Valencia), Roadway N-332, Gandia section (Valencia), the Sierra Gorda quarry in Bellús (Valencia), the Guerola quarry in Onteniente (Valencia), a slope in Puebla de Arenoso (Castellón), and on the clearings to broaden Roadway A–7 in the Port of Albaida (Alicante).

With the samples from Roadway CV-13-Torreblanca, 44 Point Load (PL) tests were carried out in the laboratory, on controls carved into regular block shapes or cylinders, subjected to axial or diametric loads (diameters of 45 and 54 mm). To determine the UCS, we conducted ten tests (five with measurement of deformations through strain gauges) for diameters of 45 and 54 mm. The study zone exhibited limestone with karst evidence of the Upper Cretaceous, the rocky mass showed fractured sections and others that clearly revealed the structure of the mass.

For the Cullera Tunnel, 103 PL tests were conducted *in situ* with cylindrical controls, having diameters between 45 and 85 mm. For the UCS, 21 tests were performed (nine with strain gauges) using controls with diameters between 47 and 84 mm. Geologically, the zone crossed by the probes is fundamentally comprised of formations from the Upper Cretaceous: crystalline dolomite, limestone, polygamous breccias and marlstone.

On the access to the Loriguilla Reservoir, 10 PL tests were carried out in the laboratory, with irregular controls and carved with a regular block form and a cylinder (45-mm diameter). For the UCS, three tests were made with a measurement of the deformation through strain gauges (45-mm diameter). Lithologically, these were pisolitic and/or oolitic limestone from the Jurassic period.

With the samples from the roadworks being conducted to broaden the roadway surface on Roadway N-332 between Gandía and Xeresa, eight PL tests were conducted between the field and the laboratory, with controls in regular block form and cylindrical form using a diametric load (49-mm diameter). For the UCS, four tests were carried out, using controls with a 49-mm diameter. The zone is on a fringe of beige micritic limestone and Cretaceous age, massive but affected by some fracturing and karst processes.

At the Sierra Gorda quarry in Bellús, 65 samples were extracted from the same block for PLT (regular blocks) and four for UCS (with measurement via strain gauges). The zone is located in sandy marls with limestone intercalations, most frequently towards the top part, which contain worm, hedgehog, and Inoceramus tracks (bioturbations).

Seeking a better correlation with the limestone subjected to the study, the research was broadened to include the following sites:

Guerola quarry in Onteniente, where eight PL tests were carried out along with a UCS test. The lithostratigraphic series presented great continuity of the sedimentation since the Neocomian to the Pliocene, given that practically all the floors are represented in which, currently, the Cretaceous and the Tertiary periods are divided.

Slope in Puebla de Arenosa with nine PL tests and one UCS test. The geology in the zone has an alternating series, sometimes metric, massive and marl limestone or marlstone less highlighted. This unit loses carbonate laterally with the appearance of alternations that have a lower content of limestone which have been denominated alternations of marls, marl limestone and limestone.

Clearings in the broadening of A–7 in the Port of Albaida, with eight PL tests and one UCS test. The study zone has a geology composed of a 100-m thickness, comprising microcrystalline limestone clay processes quite similar to the prior, but displaced in a fine, wavy, and sometimes leafy stratification, yellowish marl interbanks are very frequent, particularly towards the upper part of the package.

The numbering system shown in Table 2 will be used for the whole text.

Table 2. Location of the study sites in the Comunidad Valenciana

SITE	PROJECT / WORK
1	Excavations on roadway CV–13
2	Specimens of the Cullera tunnel project
3	Access slope to the Loriguilla reservoir
4	Slope on N–332 in Gandía – Xeresa
5	Sierra Gorda quarry in Bellús
6	Clearing in Puebla de Arenoso
7	Causeway on A–7 (Port of Albaida)
8	Guerola quarry in Onteniente.

3 TESTS PERFORMED

For a better follow up of the results obtained, the following briefly reviews the tests performed in this research.

3.1 INDEX AND ALTERABILITY PROPERTIES

The samples subjected to PLT and UCS were measured and weighed in terms of their volume, density, specific weight, humidity, and the specific relative weight of the particles in order to know with a good approximation the total porosity of the samples. Likewise, their alterability was analysed via static and dynamic immersion tests, through the Dusseault test [8], and the Slake-Durability test [9], respectively, which determines their alterability against water immersion cycles with pounding. The values obtained were used for an upcoming publication.

3.2 POINT LOAD TEST (PLT)

The Point Load Test is also known as the Franklin test. The first reference is by Reichmuth, D. (1968) [10], who proposed obtaining the resistance to a point load and a determination of the unconfined compression strength via correlation, regardless of whether they are irregular, regular, or cylindrical rock controls [11]. This test has not been designed for soft rocks or with anisotropy, although its application according to the norm permits an “anisotropy index” (I_a) in terms of the function of the results obtained with the application of the load in a parallel and perpendicular manner to the anisotropy planes.

The PLT has two clear advantages: one is the portability of the press, which permits its use in the laboratory and *in situ* (outcrops, probing, or excavations); the other advantage, and the main one, is the requirement level on the samples to be tested, only requiring some geometric minimums (Table 3), with an indispensible condition that the rupture of the specimen is produced by a fracture plane (or several) that contain the two points of the load application (Figure 1).

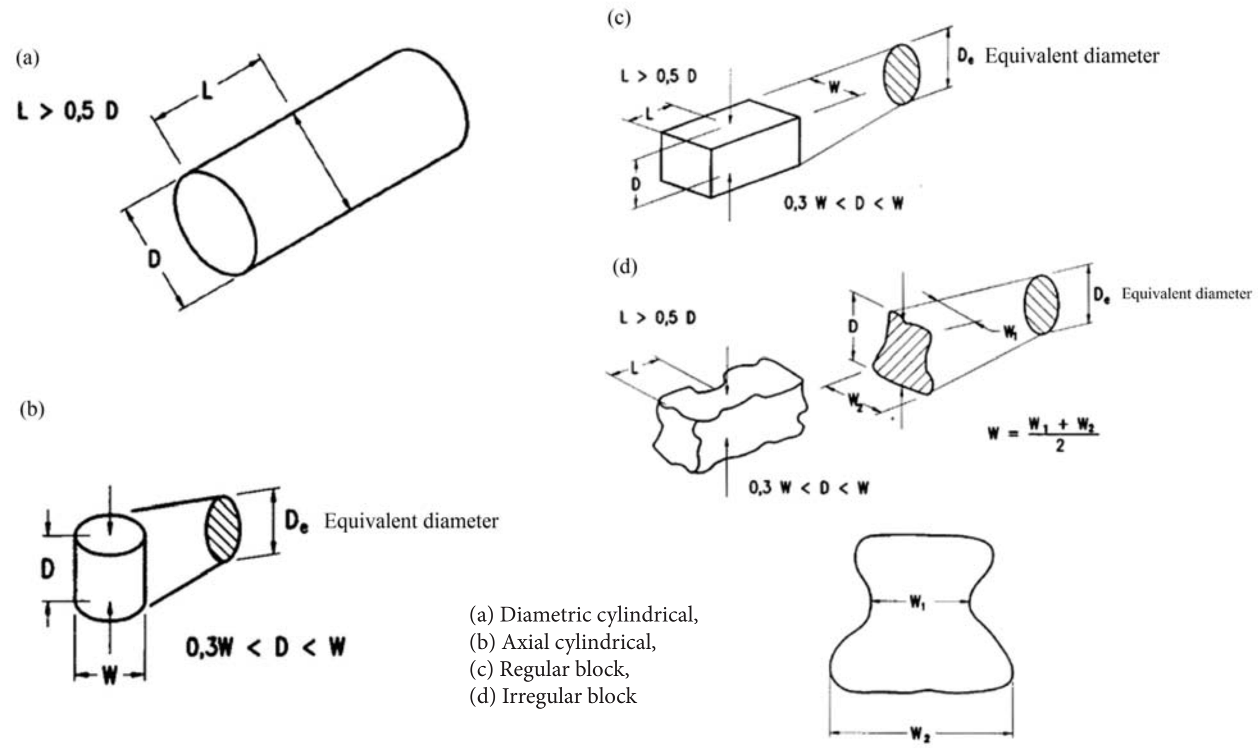

Figure 1. Geometric requirements of the specimens for the Point Load Test (UNE 22950-5:1996).

Table 3. Geometric requirements of the specimens for the Point Load Test (Source: author's work).

Type of test	Distance between load points (D)	Length of the specimen (L)	Width of the specimen		
			Less (W_1)	Higher (W_2)	Medium (W)
Diametric cylindrical	D	L > 0.5 D	-	-	-
Axial cylindrical	0.3 W < D < W	-	-	-	W
Regular block	0.3 W < D < W	L > 0.5 D	-	-	W
Irregular block	0.3 W < D < W	L > 0.5 D	W_1	W_2	½ (W_1+W_2)

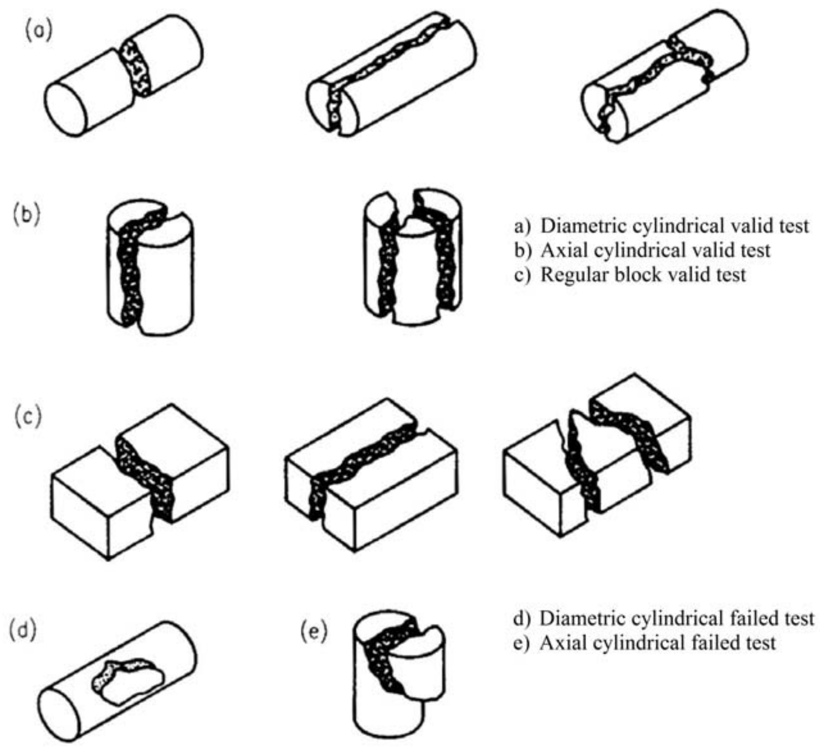

Figure 2. Rupture forms for valid and null tests [7].

The PLT is normalized for cylindrical specimens that are 50 mm in diameter, obtaining the Point Load resistance index, $I_{s(50)}$. When the diameter of the sample subjected to a diametric load (or the equivalent diameter, D_e, for other types of specimens and loads) differs from 50 mm, we should correct by size.

This correction by size is carried out by testing the samples with diverse sizes, whose breaking loads are represented against D_e^2, in double logarithmic scale, obtaining for D_e^2 = 2.500 mm^2 the value of P_{50}, by interpolation or extrapolation. The points represented in this graphic should be aligned along a straight line, ignoring (although not suppressing) those deviating substantially from the line.

When few controls are available to conduct a curve for the correction by size, we can use the following correction factor F, proposed in the norm:

$$F = \left(\frac{D_e}{50}\right)^{0.45} \qquad (1)$$

where D_e is the equivalent diameter.

In this case, the Point Load resistance index is obtained as:

$$I_{s(50)} = \frac{P_{50}}{50^2} \qquad (2)$$

where $I_{s(50)}$ is the Point Load index normalized to a cylindrical specimen of 50 mm in diameter, subjected to a diametric test.

P_{50} is the tensile strength to the point load for a diametric test with a cylindrical specimen of 50 mm in diameter or rather with the correction by size:

$$I_{s(50)} = F \cdot I_s = F \cdot \frac{P}{D_e^{\,2}} \qquad (3)$$

The resistance to the point load is also an identifying parameter for the geomechanic characteristics of the rock, similar to how they are classified according to their unconfined compression strength. (Table 4).

The habitual ranges of the Point Load Index for diverse types of rocks are shown in Figure 3; this scale is one of the most used in rock mechanics.

3.3 Test of unconfined compression strength (UCS)

The unconfined compression strength was determined directly through a "Test of the unconfined compression resistance" (UCR).

Table 4. Classification of rocks in terms of their resistance to a point load [12].

$I_{s(50)}$ (MPa)	Resistance to point load	
	Author: Garnica et al, 1997	Author: Carol, 2008
< 0. 03	Extremely low	Very low
0.03–0.1	Very low	
0.1–0.3	Low	Low
0.3–1.0	Moderate	Medium
1.0–3.0	High	High
3.0–10.0	Very high	Very high
> 10.0	Extremely high	Extremely high

The test consists of applying an axial force F_c to a cylindrical specimen of area A, bringing it to rupture with a press. The resistance is given by the equation:

$$\sigma_c = \frac{F_c}{A} \qquad (4)$$

The requirements according to the UNE 1990 norm [2] for the specimens to be tested are:

1. Cylindrical shape with dimensions:
   - Height/diameter ratio from 2.5 to 3.0.
   - Diameter, over 10 times the rock's maximum grain size.
   - Diameter, not less than 50 mm.

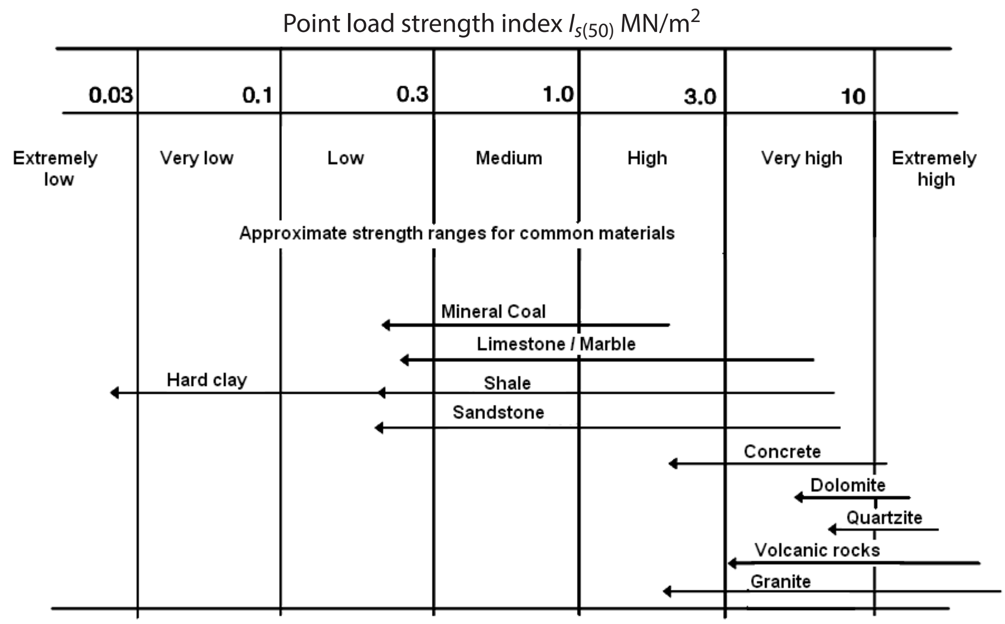

Figure 3. Variation of the Point Load Indexes for different types of rocks [4].

2. The specimens will be elaborated through perforation, turned cuts and polishing or any other appropriate method, so that:
   - The lateral surface of the specimen is smooth and free of irregularities.
   - The bases must be flat and form a straight angle with the axis of the specimen.
   - Avoid using coating materials as equalizers to achieve the parallelism required for the bases of the test specimen. If it was necessary to deviate from this norm due to the characteristics of the material to be tested, this must be indicated in the test protocol.

The tolerances of their elaboration will be those indicated in Table 5.

Table 5. Tolerance of the elaboration of rock specimens for the unified compression test [2].

Tolerance for	Deformability of the rock		
	Little	Medium	High
Deviation of the generatrix with respect to the axial direction	±0. 3 mm	±0.4 mm	±0.5 mm
Flatness of the base	±0.02 mm	±0.5 mm	±0.1 mm
Deviation with respect to right angle, the angle of the axis of the cylinder with the base	10′	20′	30′

Hence, the requirements regarding the sample to be tested are not easy to fulfil and are sometimes impossible, as described in the following.

Regarding the elaboration of the specimens:

3.3.1 Tolerance

Regarding the tolerance on the surface finishes (primarily on the lateral surfaces), it is habitual for the perforation systems and the nature of the rocks leads to us not being able to meet the requirements (Table 5).

3.3.2 Equipment

Special equipment is required for drilling and cutting, and above all for polishing; costly equipment, as well as the consumables are used (diamond dust, etc.), which are practically not available in any laboratory.

3.3.3 Facing

The facing issue is of special importance. The ISRM recommendations did not permit any system that was not mechanical; the current Spanish norm, [2], preserves that imposition, but indicates that: “If it were necessary to deviate from this norm due to the characteristics of the material to be tested, it must be indicated in the test protocol”, leading to the generalization of the already-extended custom of facing the specimens with sulphur.

Regarding specimen dimensions:

3.3.4 Length

Discontinuities of the rocky core may hinder the obtaining of rock cylinders with sufficient length (L>125 to 150 mm, between two planes perpendicular to the axis, for 50-mm diameter).

3.3.5 Grain size

In rocks formed by thick grains (> 5 mm) the diameter of the specimen must be over 50 mm; in cases of some rocks with centimetre grains or clasts, the proportions of the specimens are practically impossible to obtain (and in the case of it being possible, it may not be broken with just a conventional press).

3.3.6 Correction of the size and shape of the specimen

Also, even if all the recommendations of the norms are fulfilled, no general and unique formulation is available to permit extrapolating the results of the tests of the real in situ conditions of the rocky core. The variation of the compression strength of the matrix depends on the size (diameter) and form (height/width) of the specimen.

It has been observed that the resistance diminishes as the specimen size increases, given that, among other things, when the size increases it is more likely for a structural defect to appear and develop the rupture of the rock.

Figure 4 shows the results of the unconfined compression tests performed on samples from different lithologies [13]. The values from these tests were divided by the resistance of a rock specimen that awas 50 mm in diameter, for each lithology presented. In this way, the data presented are dimensionless and, furthermore, differences due to humidity content, load rate, etc., are eliminated.

The unconfined compression strength, σ_c , for a sample of diameter d, is related to that corresponding to a 50-mm diameter (σ_{c50}) with:

$$\frac{\sigma_c}{\sigma_{c50}} = \left(\frac{0.05}{d}\right)^{0,18} \qquad (5)$$

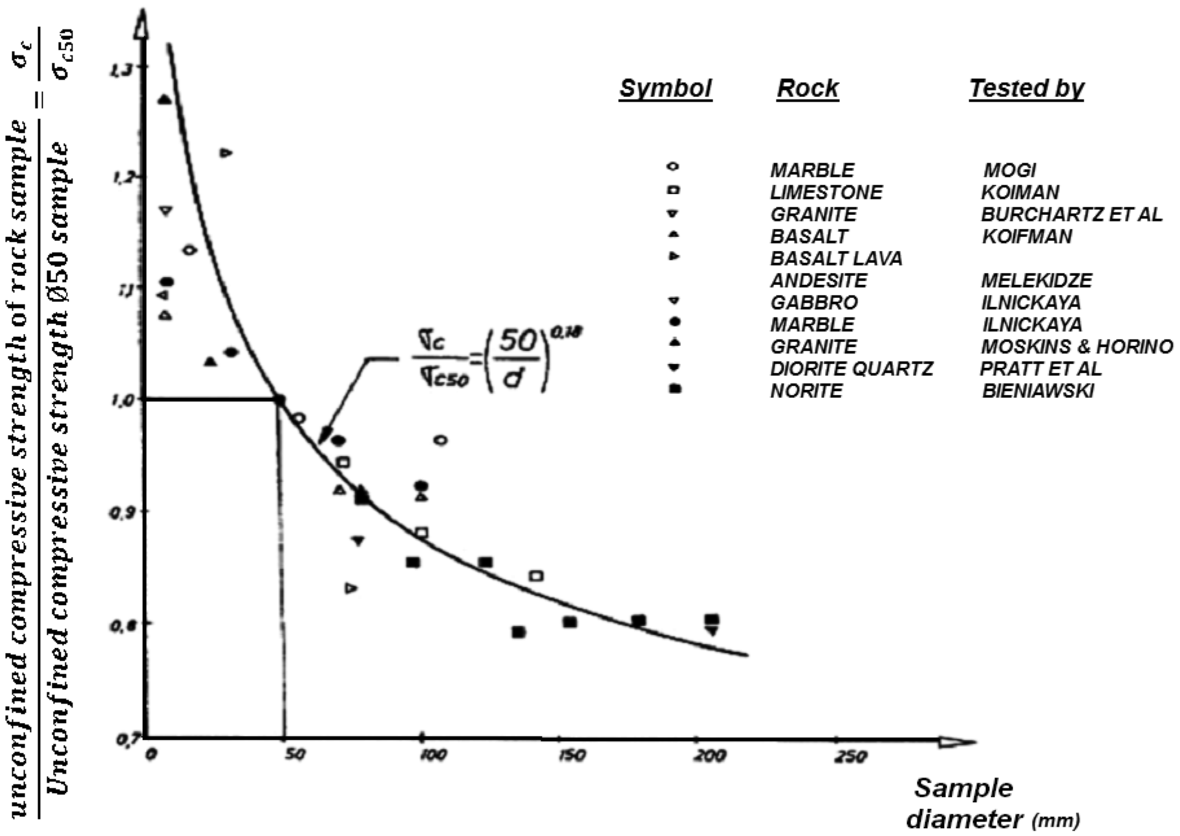

Figure 4. Variation of UCS with specimen diameter [13], in [14].

Likewise, the resistance increases with a diminished height/width ratio and it is greater when the confinement of the control increases. The critical size is defined as a specimen size for which an increase in its dimensions does not result in an appreciable decrease of its resistance. However, this test is the most widely accepted value to know the resistance to compression of a rocky core.

In tests for UCS in this research, loads were applied from 11.4 to 161.2 MPa, at a rate of 0.5 MPa/s, for six different types of diameters (83, 71, 63, 54, 48, 45 mm) for a total of 45 tests. The highest value found for the unconfined compression strength in the limestone tested was 161.2 MPa for a 45-mm diameter; the lowest was 11.4 MPa for a 71-mm diameter.

4 RESULTS

A total of 255 point load resistance tests were performed, in situ and in the laboratory, on all types of specimens, brought to rupture through an axial and diametric load. The tests were executed with equivalent diameters from 24 to 82 mm and $I_{s(50)}$ values were obtained from 1.21 to 6.40 MPa. The results were studied for four resistance sub-groups. For this, a statistical analysis was performed of the measurements by lithology groups (Table 5).

5 STATISTICAL TREATMENT

As mentioned in Section 2, there are eight points in the Comunidad Valenciana (Table 2), and this numbering system will continue to be used.

In the first place, the comparison of the samples tested by works (site) was conducted with the $I_{s(50)}$ variable, shown in the box-and-whisker chart in Figure 5.

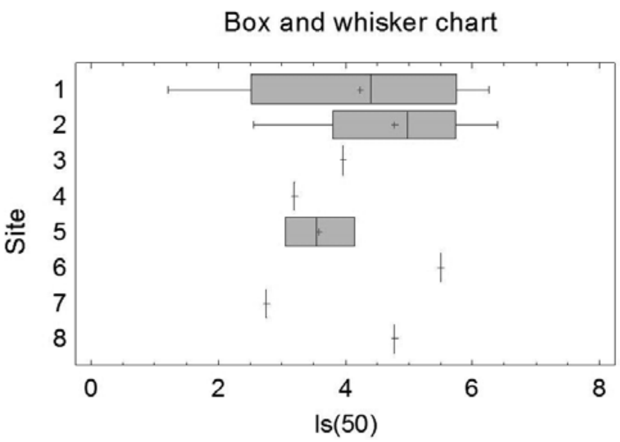

Figure 5. $I_{s(50)}$ box-and-whisker chart (all tests).

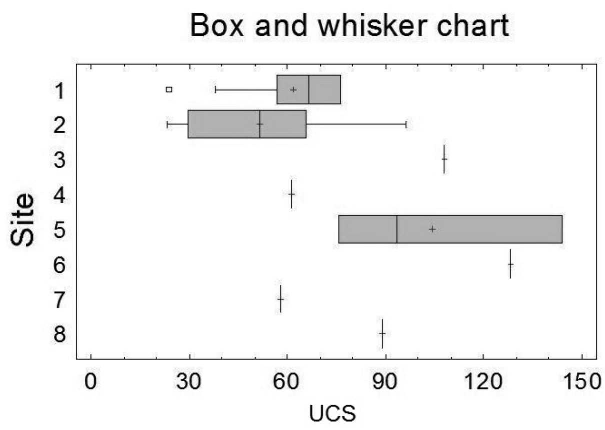

Figure 6. σ_c box-and-whisker chart (all tests).

With this procedure we compared the values obtained via a statistical test; F-test in the analysis of variance chart (Table 6) tests for any significant difference among the measurements.

Table 6. Analysis of variance for $I_{s(50)}$.

Variance analysis					
Source	Sums of Squares	DF	Mean Square	Coefficient-F	P-Value
Among groups	9.55454	7	1.36493	0.67	0.6925
Within the groups	54.7059	27	2.02615		
Total	64.2605	34			

The $I_{s(50)}$ analysis of variance chart breaks down the variance of data into two components: a component among groups and a component within each group. The F-ratio, which in this case is equal to 0.67, is the quotient of the estimation among the groups and the estimation within the groups. Given that the p-value of the F-test is above 0.05, there is no statistically significant difference among the measurements from the 8 points in the Comunidad Valenciana with a 95% confidence index. In the multiple range contrast analysis for $I_{s(50)}$, no statistically significant differences were noted between any pair of measurements at the 95% confidence level. It was also noted that all the groups are homogeneous.

Then the measurements were compared with the σc variable and Figure 6 (box-and-whisker chart) shows the mean, median, and the first and third quartiles.

The analysis of variance chart F-test (Table 7) tests for any significant difference among the measurements.

The coefficient F in Table 7, which in this case is equal to 4.33, is the quotient of the estimation among the groups and the estimation within the groups. Given that the p-value for the F-test is below 0.05, indicating statistically significant differences among the measurements from the 8 points in the Comunidad Valenciana at a 95% confidence level.

Table 7. Analysis of variance for σ_c.

Variance analysis					
Source	Sums of Squares	DF	Mean Square	Coefficient-F	P-Value
Among groups	13465.5	7	1923.7	4.33	0.0025
Within the groups	11981.9	27	443.7		
Total	25447.8	34			

Table 8. Multiple range contrast for σ_c.

Method: 95 LSD percentage					
Site	Frequency	Mean	Homogeneous groups		
1	15	61.8	X		
2	12	51.4	X		
3	1	108.0		X	X
4	1	61.2	X	X	
5	3	104.3		X	X
6	1	128.3			X
7	1	57.9	X	X	
8	1	88.9	X	X	X

In conclusion, to make a correlation between the resistance to ultimate compression and $I_{s(50)}$, it is necessary to perform a multiple range test for the UCS and group them thus (Table 8).

Table 8 applies a multiple comparison procedure to determine the measurements that are significantly different from the others. It may also be noted that three homogeneous groups were identified, according to the alignment of the X sign in the column, meaning that no significant Fisher (LSD) differences exist among the homogeneous groups.

6 UCS REGRESSION ANALYSIS AGAINST PLT

Initially, a linear regression was performed, defining the ratio between the independent and dependent variables. The correlations were made for the three types of homogeneous groups (Table 8), obtaining the four main homogeneous subgroups according to the multiple contrast for σ_c , which are shown in the following:

Table 9. Correlation for subgroup 1.

Series	Homogeneous Groups
□	1 y 2
△	4, 7 y 8
◇	3, 5 y 6

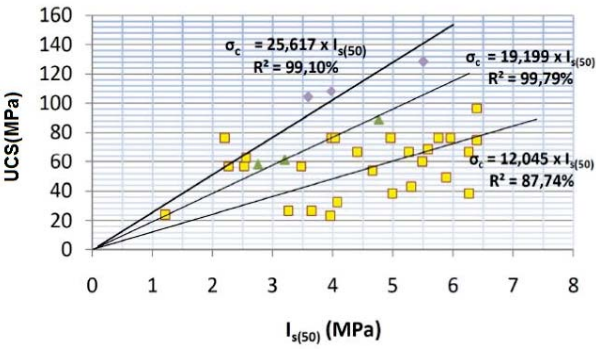

Figure 7. σ_c regression and the $I_{s(50)}$ (Subgroup 1).

Table 10. Correlation for subgroup 2.

Series	Homogeneous Groups
□	1 y 2
△	3, 4 y 7
◇	5, 6 y 8

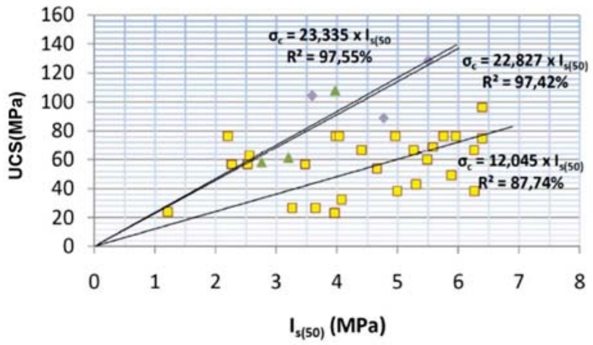

Figure 8. σ_c regression and the $I_{s(50)}$ (Subgroup 2).

Table 11. Correlation for subgroup 3.

Series	Homogeneous Groups
□	1, 2 y 7
△	4, y 8
◇	3, 5 y 6

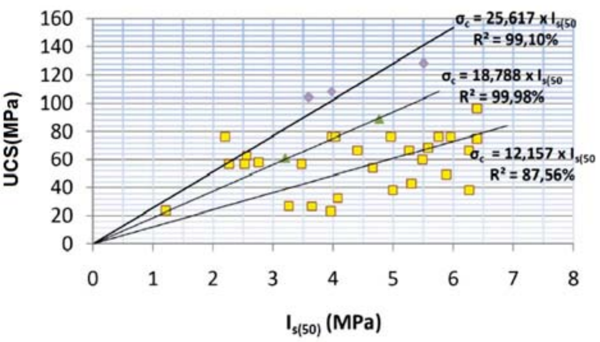

Figure 9. σ_c regression and the $I_{s(50)}$ (Subgroup 3).

Table 12. Correlation for subgroup 4.

Series	Homogeneous Groups
□	1 y 2
△	3, 5 y 7
◇	4, 6 y 8

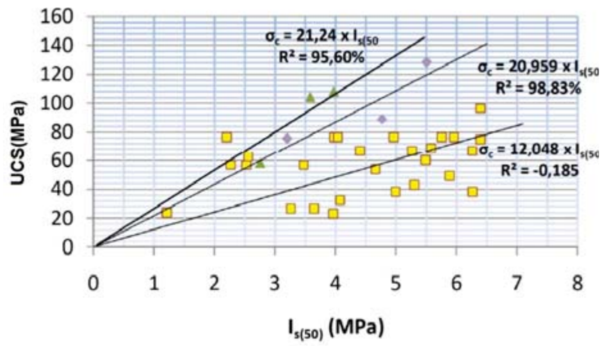

Figure 10. σ_c regression and the $I_{s(50)}$ (Subgroup 4).

In general, the summary of the correlations for the four subgroups is shown in Table 13.

Table 13. Summary of correlations for the 4 subgroups.

Subgroup	R^2	UCS
1	99.79%	19.199 x $I_{s(50)}$
2	97.55%	23.335 x $I_{s(50)}$
3	99.98%	18.788 x $I_{s(50)}$
4	98.83%	20.959 x $I_{s(50)}$

7 CONCLUSIONS AND DISCUSSION

The indirect methods used to estimate the stress parameters in the rocky core are numerous and varied, but if the norms are followed and all the procedures are performed, the values may be coherent and close to reality.

One of the most common methods is the linear correlation between UCS and $I_{s(50)}$, the latter is obtained in the PLT (Equation 6).

$$\sigma_c = \beta \times I_{s(50)} \qquad (6)$$

It was proven that there is no unique value of β for all rocks (Table 14), as initially proposed by Broch and Franklin [11]; the results herein show that for the same type of rock (limestone) the obtained values of β are between 12 and 26.

Table 14 summarizes the most important results of the β factor for all the conducted tests.

Table 14. Summary of correlations.

β	R^2 (%)	Places of the C.V.
13.612	84.48	8 points of C.V
12.045	87.74	CV-13 and Cullera
12.157	87.56	CV-13, Cullera and Puerto Albaida
19.199	99.79	N-332, Puerto Albaida and Onteniente
22.827	97.42	Loriguilla, N-332 and Puerto Albaida
18.788	99.98	N-332 and Onteniente
20.959	98.83	Loriguilla, Bellús and Puerto Albaida
25.617	99.10	Loriguilla, Bellús and Puebla de Arenoso
23.335	97.55	Bellús, Puebla de Arenoso and Onteniente
21.240	95.60	N-332, Puebla de Arenoso and Onteniente

Table 14 shows the set of subgroups statistically separate. The subgroups that have a better correlation (shaded in grey) have R^2 values greater than 87%.

Other researchers have found similar values of β for various types of rocks (Table 15).

Table 15. Estimated values of the coefficient to apply in the point load strength [17].

Rock types	Strength	$C_0/I_{s(50)}$
Igneous, compact	Medium-High	20-25
Foliated metamorphic	Medium-High	16-22
Foliated metamorphic	Low	12-16
Well-cemented calcareous	Medium-High	18-24
Well-cemented sedimentary	Low	10-15
Poorly cemented sedimentary	Low	6-10

However, when β is obtained for a concrete lithology (subgroups 1, 2, 3, 4) of Table 14, a correlation coefficient above 87% is presented. This suggests the existence of an adequate β for a specific lithology, but when seeking to widely use the determination of σ_c with $I_{s(50)}$, it is convenient to carry out a prior "calibration" with a UCS test to permit establishing the most appropriate β.

Here we found that when having to use a value of β for "limestone-type" rocks from the Comunidad Valenciana the "mean" value is β=13.6.

Finally, for future research, we suggest investigating the correlation of σc with the $I_{s(50)}$, controlling the humidity content of each sample to be tested, given that a high impact has been detected in the results.

REFERENCES

[1] ASTM International (2009). D 7012-04 Standard test method for compressive strength and elastic modulus of intact rock core specimens under varying states of stress and temperatures, West Conshohocken, USA.

[2] UNE (1990). Parte 1, ensayos para la determinación de la resistencia: Resistencia a la compresión uniaxial 22-950-90, AENOR. Madrid, España.

[3] Çobanoğlu, İ., Çelik, S.B. (2008). Estimation of uniaxial compressive strength from point load strength, Schmidt hardness and P-wave velocity. Bull Eng. Geol. Environ. 67, 491-498. DOI 10.1007/s10064-008-0158x.

[4] Serón, J.B. (1997). Propiedades básicas de las Rocas. Aplicación a las clasificaciones geomecánicas. I Curso sobre tecnología de métodos modernos de sostenimiento de Túneles, Valencia. España, 17.

[5] Romana, M. (1999). Correlation between uniaxial compressive and Point Load (Franklin test)

strength for different rock classes. In: 9th ISRM Congress, Paris, France, 1, 673-676l.
[6] ASTM International (2009). D 5731-05 Standard test method for determination of the point load index of Rock, West Conshohocken. U.S.A.
[7] UNE (1995). Parte 5, ensayos para la determinación de la resistencia: resistencia a Carga Puntual, AENOR, 22-950-95,Madrid, España.
[8] Normas del Laboratorio de Transportes NLT. Estabilidad de áridos y fragmentos de roca frente a la acción de desmoronamiento en agua 255/99, Madrid, España.
[9] International Society for Rock Mechanics ISRM. Test al desmoronamiento (Slake durability) 21997, Lisboa, Portugal.
[10] Reichmuth, D.R. (1968). Point-load testing of brittle materials to determine tensile strength and relative brittleness. Proceedings of the ninth symposium on Rock Mechanics. University of Colorado, 134-159.
[11] Broch, E., Franklin, J.A. (1972). The point load test. International Journal Rock Mechanics, Mining and Science 9, 669-697.
[12] Garrido, M.E., Hidalgo, C., Preciado, J.I. (2010). Prácticas de laboratorio geotecnia y cimientos I. Departamento de Ingeniería del Terreno. Universidad Politécnica de Valencia, Ref.:2010.418, Editorial UPV.
[13] Hoek and Brown (1980). Underground excavations in Rock. Institution of Mining and Metallurgy, London.
[14] Instituto Tecnológico Geominero de España (ITGE) (1991). Mecánica de rocas aplicada a la minería metálica subterránea. Ministerio de Industria, Comercio y Turismo. Secretaria general de la Energía y Recursos Minerales, 160.
[15] Preciado, J.I. (2008). Ensayos de laboratorio, Geotecnia y Cimientos II [Diapositiva]. Valencia, España: Universidad Politécnica de Valencia.
[16] Serón, J.B., Preciado, J. (2007). Instrumentación geotécnica [Apuntes de clase]. Ref. 2007.113, Valencia, Universidad Politécnica de Valencia.
[17] Romana, M. (1994). Test de compresión puntual de Franklin. Revista de Ingeniería Civil, 116-120.