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Overview of the Empirical Relations between Different Aggregate Degradation Values and Rock Strength Parameters

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Abstract

The aggregates are essential materials in civil engineering, they are used for railway and road constructions, for hydraulic engineering but they are also the base material of concrete. The crushed stones are exposed to several effects during their lifespan. Therefore, several tests were developed to evaluate their performance. One of the most important aspects is the resistance to degradation. However, degradation tests require special types of equipment and usually take longer than common strength tests which are more likely to be available for rock materials. Therefore, the empirical connections between strength and degradation values can be extremely useful in practice. The paper aimed to collect all available relationships and datasets from the literature that presents the relations between these different parameters – such as Aggregate Impact Value (AIV), Aggregate Crushing Value (ACV), Ten Percent Fines Value (TFV), Los Angeles Abrasion Value (LAAV), and micro-Deval Coefficient (MDE) – and rock strength parameters – such as Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$) – and to provide the best-fit formula for different rock types. The paper also highlights the difficulties and limitations of the compared relationships.

Keywords

aggregate degradation, rock strength, Uniaxial Compressive Strength, Point Load Strength Index

1 Introduction

Aggregates are essential materials in the construction industry, used in concrete, bitumen, road and railway infrastructures, etc. From extraction to installation, not to mention the entire lifespan, several effects have an influence on aggregates. During production, depending on the mining method, the rock environment might be exposed to blasting, then during loading, classification and production of smaller fractions, the rock material is exposed to significant impacting and breaking energy. During transportation, the grains may rearrange and are exerted by different forces, in addition, the grain distribution of the aggregate changes completely during the segregation. During installation, the grains receive additional impact and compression energy. The built-in crushed stone can be affected by both dynamic and static loads. The stresses between single particles as a result of the force effect between them can occur as tension (grain splitting), as shear generated by compression, as bending and as pure shear [1]. The different effects result in different stresses, like as impacting, crushing, abrasion, and disintegration [2]. In terms of the durability of the crushed stones, the resistance to degradation is one of the most important properties. Therefore, a number of tests have been developed to simulate these stresses and help to understand the mechanical weathering of aggregates.

The tests used in everyday practice, like as micro-Deval Abrasion Test, Los Angeles Abrasion Test, Impact Test and Crushing Test do not require special equipment. As a result, and because of the relatively quick and simple procedure, they have spread widely. Based on the standardized procedures of these tests and the various national standards and regulations, the various crushed stones are classified in terms of end use based on the parameters obtained as a result of these tests.

The current paper collects the available empirical relations between the mentioned aggregate degradation values and rock strength parameters from different literature. Beyond the relationships, different available datasets were also provided where authors did not establish a correlation.

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In the following, the test procedures are briefly presented which are necessary to determine the parameters that form the basis of the correlations collected in the article.

1.1 Micro-Deval Abrasion Test and Los Angeles Abrasion Test

Both micro-Deval and Los Angeles Abrasion Tests are kine-matic test methods in which case the samples are placed into a special drum rotated for the required revolutions. Both abrasion tests are included in the American standard [3–5] and in the European standard [6, 7]. These tests determine a coefficient, which is the percentage of the initial sample reduced to a size smaller than a required standard sieve size, in European Standards smaller than 1.6 mm due to rolling. While the micro-Deval Test determines a coefficient called micro-Deval Coefficient [6] or micro-Deval Abrasion Loss [3], the evaluation of Los Angeles Abrasion Test is also a coefficient called Los Angeles Coefficient [7] or Los Angeles Abrasion Loss [4, 5]. The main difference between the two test methods is that the Los Angeles Test method combines the actions of abrasion and impact. The drum is filled with steel spheres in addition to aggregates and trays are placed on the inside of the drum to pick up these steel balls which generate an impact-crushing effect after making a half-turn and falling free. The two tests require different apparatus. In case of Los Angeles Test method the size of the drum is larger and the aggregate (5000 or 10 000 g) and steel ball mixture shall be rotated for 500 or 1000 revolutions. During the micro-Deval Test method the test portion is 500 g aggregates and smaller charge or 10 000 g ballast aggregate without steel balls shall be rotated for 12 000–14 000 revolutions. It is important to note that Eurocode extends the test method for ballast aggregates.

1.2 Impact Test

The main aspect of the Impact Test method is to define the fragmentation caused by a falling hammer from a specified height. The result of the test is expressed as Aggregate Impact Value, which is the percentage of material passing through the 2.36 mm sieve and the initial mass of the test specimen. This test is prescribed by the BS 812-112 [8]. It is important to note that the Eurocode also determines an Impact Test as an alternative test method of Los Angeles Test to define the fragmentation of aggregates. The Eurocode Aggregate Impact Test method is fundamentally different from British standard recommendations. The test equipment size is larger by scales; therefore, the impact work performed by the hammer is also larger. While the EN Impact Test is extended to railway ballast aggregates the aggregate sizes bigger than 14.0 mm is not appropriate to be tested according to BS.

1.3 Crushing Test

The BS 812-110 gives a relative measure of the resistance of an aggregate to crushing under a gradually applied compressive load. The cylinder shall be filled with the prepared specimens, then the samples shall be loaded with a uniform rate to reach 400 kN in 10 minutes. The measure of fragmentation is called Aggregate Crushing Value (ACV). The BS 812-111 introduces a very similar aggregate testing method to the Crushing Test. The result of this method is the Ten Percent Fines Value (TFV) which is also a measure of the resistance of an aggregate to crushing under a gradually applied compressive load with the difference that the upper limit of the loading force is not fixed during this test. The procedure is repeated with various loads to determine the maximum load which generates the Ten Percent Fines Value (TFV). For the expression of fragmentation after loading, the sample shall be sieved over a 2.36 mm sieve and weighed and divided by the initial mass of the specimen after both tests. TFV is the loading force that generates 10% of degradation loss [9, 10]. The Hungarian Hummel test (MSZ 18287-3:1983) [11] and the Swiss Compression Test (EMPA) [12] operate on a similar principle. The latter method has been developed specifically for the railway faction (31.5/50 mm), but the Hummel is appropriate for fraction (16/40 mm) while the limitation of the BS test is 10/14 mm. The other difference is the definition of the degree of fragmentation. While the ACV is a simple loss percentage, in the other two test methods the change in particle distribution is measured.

In addition to the previous descriptions, it has to be mentioned that several large-scale tests were developed, since the original scale of the railway ballast is larger and due to the larger fraction size, the size effect of the conventional tests might significantly affect the evaluability of the results. That is why extensive literature recommends to carry out a large-scale test instead of the tests detailed above in the case of large particle size [13–15].

In order to understand the mechanical behavior of crushed stones in railway ballast layer, many other large-scale tests [15] have been developed to better represent on-site conditions and effects, which is a major challenge [16]. Different scale of ballasted track models [17–20] and large-scale versions of box tests [21–27], direct shear tests [28–30], oedometer tests [13], shaker tests [31] and triaxial tests [14, 15, 20, 32–51] have been carried out with the aim of understanding or even modeling the mechanical process of the ballast layer of the railway track structure.

In the last two decades, there has been increased research on the discrete element modeling (DEM) of aggregates. In many cases, models were calibrated using an experimental test (box test [52], direct shear test [53–55], triaxial test [56–58], micro-Deval test [59, 60], oedometer test [52], Hummel test [61, 62]).

There is a significant difference in the dimensions of the test equipment and the time required for execution between standard and large-scale tests. The large-scale tests have been developed with the aim of helping to learn about the mechanical properties of crushed stones, however, they are difficult to transfer to everyday laboratory practice and qualification. If a significant drawback of the conventional tests is that the preparation, completion and evaluation may require up to 3-4 days, the large-scale tests are even more circumstantially. In addition, the space requirements of the test equipment are problematic, and their operation also requires much more energy. All in all, the already expensive devices with high maintenance costs and a low number of test repetitions are economically unfavorable. As a result, apart from a few equipments put into operation for research purposes, these tests will not spread in laboratories operated for profit.

Several research studies have concluded that the standard degradation parameters are insufficient to characterize the durability of aggregates. The current paper does not deal with all the factors influencing the results of standard degradation tests, but many articles evaluate which physical, geological, etc., features have significant effects on them. [63–68].

Turk and Dearman [69] concluded that the degradation properties of aggregates depend on the fraction size range which size effect is rock strength related. If an appropriate quantity of aggregates or standard rock samples are not available to test with the help of a well-defined correlation between aggregate degradation and rock strength properties, the missing mechanical properties can be predicted [2, 70, 71]. The application of a well-defined correlation can help efficient and economic rock mining. Since the suitability of the given rock unit as aggregate material can be predicted from some preliminary rock strength tests. Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$) are the most often used parameters to determine the strength of the rock.

1.4 Uniaxial Compressive Test

Both the ISRM and ASTM suggest methods for determining the Uniaxial Compressive Strength (UCS). The cylinder specimens with specified size should be loaded continuously in a quasi-static way, at a constant stress rate until the failure occurs [72, 73]. The size and shape of the specimens depends on the standards. The UCS is expressed as the quotient of the recorded compressive force for failure and the original loaded area.

1.5 Point Load Test

The suggested method for determining Point Load Strength was published by ISRM and ASTM also [74, 75]. The Point Load Strength Test is an index test with the aim of helping the qualification of the rock materials. In rock mechanics, it is also used to predict other strength parameters when the circumstances do not allow carrying out other tests like the Uniaxial Compressive Strength Test. In addition to regular cylindrical specimens, the ISRM recommendation also allows the examination of irregular specimens, so-called irregular lumps. Similar to the UCS test method, gradually increased load is applied to break the sample. In addition to the specimen dimensions, the failure load is measured. The Uncorrected Point Load Strength (I_S) is defined as:

$$I_S = \frac{P}{D_e^2} , \tag{1}$$

where D_e is the equivalent core diameter and is calculated according to the given formula of the ISRM prescription.

Since I_S varies as a function of the diameter, a uniform Point Load Strength has been introduced to characterize the rock sample. The named size-corrected Point Load Strength Index ($I_{S(50)}$) is defined as the value of IS which would have been measured on a specimen with a diameter of 50 mm. The ISRM also recommends methods to calculate this value.

2 Relationships between Uniaxial Compressive Strength (UCS) and aggregate degradation properties

Cargill and Shakoor [76] tested sandstones, limestones, marble, dolomite, and gneiss. The Uniaxial Compressive Strength (UCS) tests were carried out according to the ASTM D2938 and they defined the Los Angeles Abrasion Values (LAAV) by using ASTM C131 ("grading A", which is equal to 9.5/37.5 mm). The purpose of the paper was to find the correlation between Uniaxial Compressive Strength and Los Angeles Abrasion Values. The possibility of using the established relationship was seen in those cases when the rock is greatly fragmented to carry out the Uniaxial Compressive Strength Test. The authors divided the Los Angeles Abrasion Values by the dry density (see the established relationship in Table 1) by which they found reduced scatter in the data points. An inverse relation has been shown between UCS and LAAV parameters. The researchers established the fact that under 100 MPa of UCS the decrease of LAAV is more intensive than in the case of higher Uniaxial Compressive Strength. The reason for the scatter in the measurements was assumed as an effect of the grain size variation. They found the Los Angeles Abrasion Test applicable to estimate the UCS but noted the need for specified equations for different types of rocks.

One of the authors, Shakoor and his colleague, Brown conducted a study in 1996 [77]. They performed Uniaxial Compressive Strength Tests and Los Angeles Abrasion Tests on 15 carbonate rocks. The aggregate materials were

Table 1 Comparison of the correlations between Los Angeles Abrasion Value (LAAV) and rock strength parameters – Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$) – developed in previous studies

Equations	R^2	Rock types	Standards	References
$UCS = 1450 \times (LAAV/\rho_d)^{-0.91}$	0.85	Sandstones, Limestones, Marble, Dolomite, and Gneiss	ASTM D 2938 (1984) ASTM C131	Cargill and Shakoor [76]
$LAAV = \exp(3.85 - 0.004 \times UCS)$	0.76	All rock types	ASTM C131 (1989) Deere and Miller (1966)	Al-Harthi – 1 [71]
$LAAV = 88.01 - 12.35 \times \ln(UCS)$	0.78	All rock types	ASTM C131 (1989) Deere and Miller (1966)	Al-Harthi – 2 [71]
$LAAV = 143.78 - 24.12 \times \ln(UCS)$	0.63	All rock types	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV = 150.81 - 26.23 \times \ln(UCS)$	0.50	Igneous	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV = 511.42 \times UCS^{-0.62}$	0.81	Metamorphic	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV = 536.89 \times UCS^{-0.60}$	0.50	Sedimentary	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV = 168.41 - 29.19 \times \ln(UCS)$	0.96	Porosity n=0.18-0.38%	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV = 132.73 - 21.89 \times \ln(UCS)$	0.68	Porosity $n < 1\%$	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV = 634.04 \times UCS^{-0.68}$	0.79	Porosity n=1-5%	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV = 4236.1 \times UCS^{-1.05}$	0.75	Porosity $n > 5\%$	ASTM C131-66 ISRM (1985)	Kahraman and Fener [78]
$LAAV/V_p = 22.67 - 3.91 \times \ln(UCS)$	0.80	Limestones, Travertines, Marbles, Andesite	ASTM C131-01 (2006) TSE 699 (1987)	Ugur et al. [79]
$LAAV = 497.64 \times UCS^{-0.67}$	0.71	Sedimentary	ASTM C131-66 (2006) ISRM (1981)	Ozcelik [80]
$LAAV = 382.26 \times UCS^{-0.65}$	0.85	Metamorphic	ASTM C131-66 (2006) ISRM (1981)	Ozcelik [80]
$LAAV = 132.41 \times UCS^{-0.45}$	0.83	Igneous	ASTM C131-66 (2006) ISRM (1981)	Ozcelik [80]
$LAAV = 115.74 - 20.52 \times \ln(UCS)$	0.86	Trachyte, Mafic, Ultramafic	ASTM C131 (1989) ASTM C170-90 (1999)	Rigopoulos et al. [81]
$LAAV = 33.02 - 0.13 \times UCS$	0.91	Limestones, Dolomite, Sand and Gravel, Tephra–phonolite, Trachybasalt	TS EN 1097-2 (2010) TS EN 1926 (2007)	Tuncay et al. [82]
$LAAV = 162.7 \times \exp(-0.013 \times UCS)$	0.33	All rock types	ASTM C131 (1989) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 99.86 \times \exp(-0.01 \times UCS)$	0.46	Charnokite	ASTM C131 (1989) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]

Continuation of Table 1

Equations	R^2	Rock types	Standards	References
$LAAV = 208.79 \times \exp(-0.013 \times UCS)$	0.78	Gneiss	ASTM C131 (1989) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 110.22 - 0.56 \times UCS$	0.94	Granite	ASTM C131 (1989) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 75.49 \times \exp(-0.009 \times UCS)$	0.80	Quartzite	ASTM C131 (1989) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 42.30 - 0.15 \times UCS$	0.57	Limestone, Dolomite	ASTM C131/C131M-14 (2006) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 1 [2]
$LAAV = 125.53 \times UCS^{-0.35}$	0.68	Limestone, Dolomite	ASTM C131/C131M-14 (2006) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 2 [2]
$LAAV = 56.2 - 2.8 \times I_{S(50)}$	0.51	Granite	ASTM C131-81 (1981) ISRM (1985)	Irfan [88]
$LAAV = \exp(3.85 - 0.087 \times I_{S(50)})$	0.77	All rock types	ASTM C131 (1989) ISRM (1985)	Al-Harthi – 1 [71]
$LAAV = 50.35 - 12.93 \times \ln(I_{S(50)})$	0.79	All rock types	ASTM C131 (1989) ISRM (1985)	Al-Harthi – 2 [71]
$LAAV = 127.96 \times I_{S(50)}^{-0.80}$	0.72	All rock types	ASTM C131-66 ISRM (1985)	Kahraman and Gunaydin [89]
$LAAV = 104.92 - 40.14 \times \ln(I_{S(50)})$	0.81	Porosity n < 1%	ASTM C131-66 ISRM (1985)	Kahraman and Gunaydin [89]
$LAAV = 104.36 \times I_{S(50)}^{-0.68}$	0.77	Porosity n > 1%	ASTM C131-66 ISRM (1985)	Kahraman and Gunaydin [89]
$LAAV/V_p = 12.45 - 4.82 \times \ln(I_{S(50)})$	0.67	Limestones, Travertines, Marbles, Andesite	ASTM C131-01 (2006) TSE 699 (1987)	Ugur et al. [79]
$LAAV = 52.36 \times I_{S(50)}^{-0.48}$	0.68	Sedimentary	ASTM C131-66 (2006) ISRM (1985)	Ozcelik [80]
$LAAV = 57.73 \times I_{S(50)}^{-0.61}$	0.87	Metamorphic	ASTM C131-66 (2006) ISRM (1985)	Ozcelik [80]
$LAAV = 52.61 \times I_{S(50)}^{-0.56}$	0.90	Igneous	ASTM C131-66 (2006) ISRM (1985)	Ozcelik [80]
$LAAV = 45.72 - 2.78 \times I_{S(50)}$	0.45	All rock types	ASTM C131 (1989) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 39.97 - 1.58 \times I_{S(50)}$ [1]	0.69	Charnokite	ASTM C131 (1989) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 39.14 - 1.48 \times I_{S(50)}$	0.83	Gneiss	ASTM C131 (1989) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 51.77 - 3.38 \times I_{S(50)}$	0.80	Granite	ASTM C131 (1989) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 22.863 - 0.5497 \times I_{S(50)}$	0.69	Quartzite	ASTM C131 (1989) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
$LAAV = 41.33 - 11.46 \times \ln(I_{S(50)})$ [2]	0.81	Basalt, Granite, Gabbro, Diorite, Sandstone, Limestone, Dolomite, Quartzite, Schist, Phyllite and Amphibolite	IS 2386 part IV (1963) IS 8764 (1998)	Ahmad et al. [91]
$LAAV = 80.35 \times I_{S(50)}^{-0.88}$	0.93	Igneous, Sedimentary	EN 1097-2:2010 ASTM D 5731-08	Fotev and Angelova [92]
$LAAV = 48.10 - 4.38 \times I_{S(50)}$	0.63	Limestone, Dolomite	ASTM C131/C131M-14 (2006) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 1 [2]
$LAAV = 56.95 \times I_{S(50)}^{-0.51}$	0.71	Limestone, Dolomite	ASTM C131/C131M-14 (2006) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 2 [2]

[1] presumably incorrectly published or fitted on the measured values
[2] originally presumably incorrectly published: $LAAV=41.33+11.46\times\ln(I_{S(50)})$

laboratory crushed materials from the same rock blocks from the cores that were produced for the compressive test. They carried out multiple linear regression considering properties like as dry density, and absorption. In this paper, only the measured data were plotted in Fig. 1(a). For the Uniaxial Compressive Strength determination, they used the ASTM D4543, while the Los Angeles Abrasion Tests were carried out on "grading A" (9.5/37.5 mm) aggregates prescribed in the ASTM. The aim of their study was to develop an equation with which the UCS is predictable as a function of LAAV, dry density, and absorption. They also performed a bivariate regression analysis to diagnosticate the possibility of UCS directly from LAAV. They found that the added properties to the LAAV as dry density, and absorption give a more reliable equation to predict the UCS.

The aim of the study of Al-Harthi [71] was to determine the strength characteristics (AIV, ACV, LAAV) of crushed aggregates from a field index (Schmidt hammer and Point Load). All three main rock groups according to origin (igneous, sedimentary and metamorphic) were tested. Both the Schmidt hammer and the Point Load Tests were carried out on aggregates. Instead of direct Uniaxial Compressive Strength Tests, the UCS was determined from Schmidt rebound hammer readings. Both exponential and logarithmic relationships were defined between

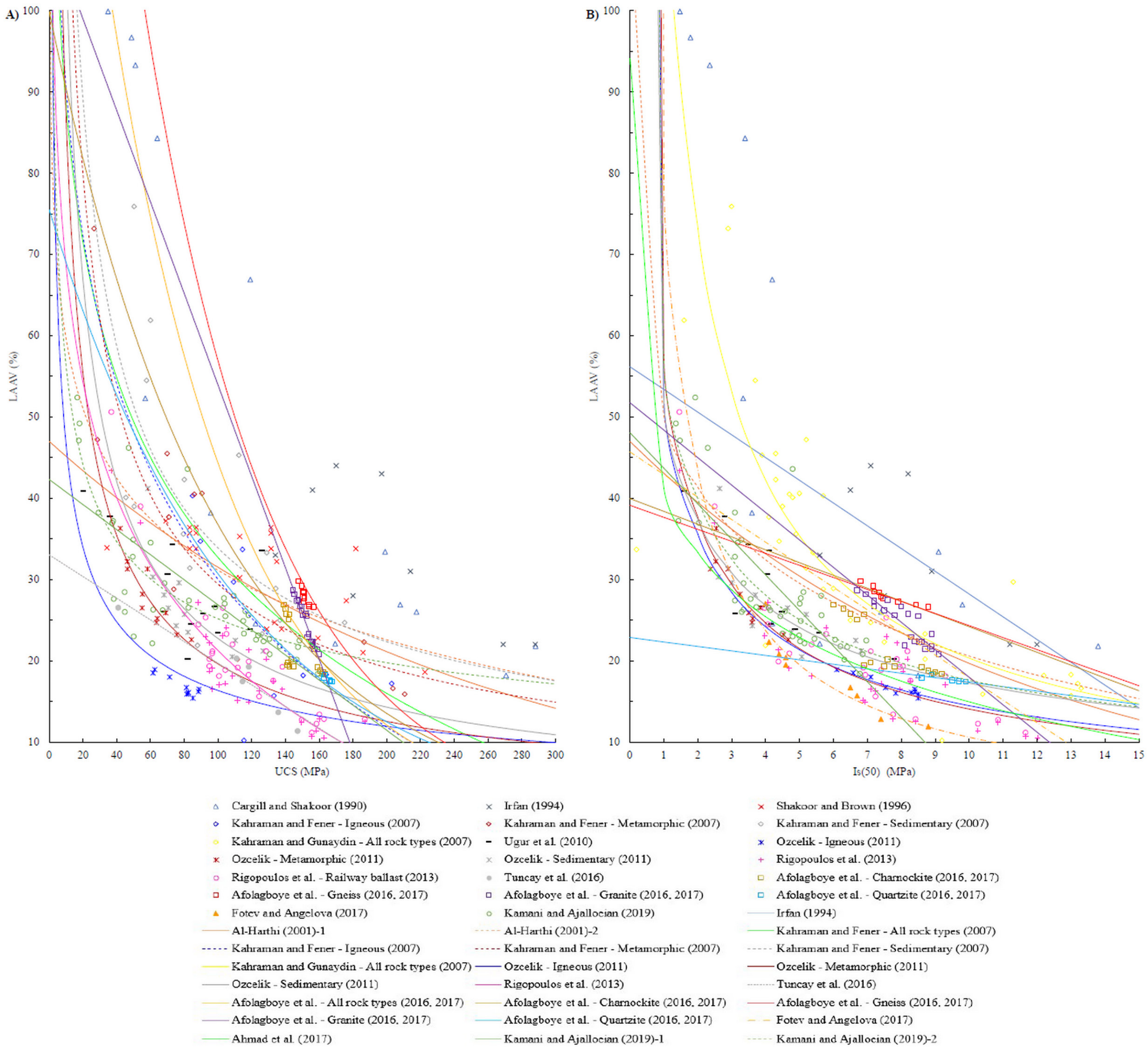

Fig. 1 Aggregate degradation test results and established relationships between aggregate degradation parameters and rock strength parameters based on previous studies. (a) Relationships between Los Angeles Abrasion Value (LAAV) and Uniaxial Compressive Strength (UCS); (b) Relationships between Los Angeles Abrasion Value (LAAV) and Point Load Strength Index ($I_{S(50)}$)

the parameters. As a result, statistically significant relationships were found between UCS and ACV, AIV, and LAAV parameters.

Kahraman and Fener [78] investigated the opportunity of predicting the LAAV from UCS. As Al-Harthi [71], these researchers tested a wide variety of igneous, sedimentary and metamorphic rock types. The study was completed by examining the effect of porosity. The correlation coefficients of the regression analyses were found to increase by considering this property also. "Grading D" (4.75/9.5 mm) was used for the Los Angeles Abrasion Test (ASTM C131-66).During the regression analysis linear, logarithmic and exponential curve fittings were carried out (see Table 1).

In the study of Ugur et al. [79] limestones, travertines, crystalline marbles and andesite were tested to evaluate the correlations between the LAAV and mechanical properties covering Uniaxial Compressive Strength, Point Load Strength Index (which are discussed later). The Los Angeles Abrasion and the Uniaxial Compression Tests were performed according to a Turkish national standard. The LAAV was measured on 10/20 mm fraction size of aggregates rotated both 100 and 500 revolutions. Since the P-wave velocity (V_p) was also recorded, the LAAV results were divided by that, to create a significant relationship. The advantage of this procedure is that since the P-wave velocity is highly related to the porosity, density, mineral composition, size and frequency of fractures in the rock structure, these geological features can be taking account also. An inverse relationship was found between LAAV and UCS and a logarithmic regression curve was fitted to the results (see Table 1).

The purpose of the study of Ozcelik [80] was to give an estimator correlation of LAAV from some physical and mechanical properties. The ASTM C131-66 method was used to determine the LAAV on "grading D" (4.75/9.5 mm) aggregates. The UCS was performed following the ISRM (1981). 32 different rock groups were tested including igneous, sedimentary and metamorphic origins. Ozcelik found that is impossible to create a relationship that covers all the rock types, therefore it is required to carry out the regression analysis for all types separately (see Table 1).

Rigopoulos et al. [81] investigated the interrelation between LAAV and UCS on different ophiolitic rocks (dunites, harzburgites, lherzolites, troctolites, diorites and trachytes) from Greece. The Los Angeles tests were performed according to ASTM C131, using the "A" gradation (9.5/37.5 mm) and extended to railway ballast aggregates following the Eurocode recommendations. The established relation was applied on the 9.5/37.5 mm fraction size, so the data and correlations given below are also valid for this size range. As a result of the regression analysis, it is identifiable that the LAAV is negatively correlated with UCS (see Fig. 1(a)).

The study of Tuncay et al. [82] focused on the other recently most widely used abrasion resistance aggregate test beyond the Los Angeles Abrasion Test, the micro-Deval Abrasion Test also. Mineralogical, petrographic, mechanical, physical and chemical properties of rock samples with different origins (limestones, dolomite, sand and gravel, tephra–phonolite, trachybasalt) were examined. In terms of rock strength, the Uniaxial Compression Test, while in terms of aggregate degradation resistance, the Los Angeles and micro-Deval Abrasion Tests were used. The compression and abrasion tests were carried out according to the Turkish version of EN standards. A linear relationship was established between LAAV and UCS.

Afolagboye et al. [83] evaluated the durability of Southwest Nigerian aggregates. The resistance to degradation was determined by the use of Los Angeles Abrasion Value (LAAV), Aggregate Crushing Value (ACV), Aggregate Impact Value (AIV) and Ten Percent Fines Value (TFV). In the case of Los Angeles Tests, the ASTM was followed, while the other degradation values were defined according to the British Standard, BS 812 prescriptions. Uniaxial Compressive Strength Tests expended this research and later with Point Load Strength Tests [84] with both. The study of Afolagboye et al. [84] aimed to identify the possibility of using these rock strength parameters to predict the mechanical weathering resistance of aggregates. The rock strength parameters were measured according to the ISRM. The tables (Table 1, Table 2, Table 3, Table 4) represent the established correlations between UCS and the degradation values for combining different rock types and individual rock classes (for charnockite, gneiss, granite, quarzite) also. For the combined rock lithologies inverse linear relationship was established between UCS, ACV (see Table 2) and AIV (see Table 4), while inversely direct one between UCS and TFV (see Table 3). Between UCS and LAAV exponential correlation was applied (see Table 1). Generally, for the separated rock groups non-linear relationships were obtained between UCS and the degradation parameters. In conclusion, it was noted that the rock class should be considered to create correlations between the rock strength parameters of the parent rock and degradation values of the crushed stones.

Table 2 Comparison of the correlations between Aggregate Crushing Value (ACV) and rock strength parameters – Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$) – developed in previous studies.

Equations	R^2	Rock types	Standards	References
ACV = exp(3.71 – 0.005 × UCS)	0.86	Igneous, Metamorphic, Sedimentary	BS 812: part 110 (1990) Deere and Miller (1966)	Al-Harthi – 1 [71]
ACV = 78.82 – 11.73 × ln(UCS)	0.89	Igneous, Metamorphic, Sedimentary	BS 812: part 110 (1990) Deere and Miller (1966)	Al-Harthi – 2 [71]
ACV = 68.37 – 0.29 × UCS	0.64	All rock types	BS 812: part 110 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 52.61 × exp(–0.005 × UCS)	0.84	Charnockite	BS 812: part 110 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 2507.3 × exp(–0.03 × UCS)	0.52	Gneiss	BS 812: part 110 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 74.17 – 0.33 × UCS	0.83	Granite	BS 812: part 110 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 52.05 × exp(–0.006 × UCS)	0.61	Quartzite	BS 812: part 110 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 40.45 – 0.16 × UCS	0.78	Limestone, Dolomite	BS 812: part 110 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 1 [2]
ACV = 43.11 × exp(–0.006 × UCS)	0.81	Limestone, Dolomite	BS 812: part 110 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 2 [2]
ACVv36.5 – 1.64 × $I_{S(50)}$	0.54	Granite	BS 812: part 110 (1975) ISRM (1985)	Irfan [88]
ACV = exp(3.71 – 0.11 × $I_{S(50)}$)	0.90	Igneous, Metamorphic, Sedimentary	BS 812: part 110 (1990) ISRM (1985)	Al-Harthi – 1 [71]
ACV = 43.08 – 12.32 × ln($I_{S(50)}$)	0.91	Igneous, Metamorphic, Sedimentary	BS 812: part 110 (1990) ISRM (1985)	Al-Harthi – 2 [71]
ACV = 43.91 – 2.44 × $I_{S(50)}$	0.67	All rock types	BS 812: part 110 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 34.76 – 1.21 × $I_{S(50)}$	0.96	Charnockite	BS 812: part 110 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 54.22 – 3.76 × $I_{S(50)}$	0.73	Gneiss	BS 812: part 110 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 39.87 – 1.93 × $I_{S(50)}$	0.68	Granite	BS 812: part 110 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 22.33 – 0.47 × $I_{S(50)}$	0.70	Quartzite	BS 812: part 110 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
ACV = 45.60 – 4.36 × $I_{S(50)}$	0.79	Limestone, Dolomite	BS 812: part 110 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 1 [2]
ACV = 0.53 × $I_{S(50)}{}^2$ – 8.76 × $I_{S(50)}$ + 53.50	0.82	Limestone, Dolomite	BS 812: part 110 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 2 [2]
$I_{S(50)}$ = 15.31 × exp(-0.054 × ACV)	0.70	Basalt, Granite, Gabbro, Diorite, Sandstone, Limestone, Dolomite, Quartzite, Schist, Phyllite and Amphibolite.	IS 8764 (1998) IS 2386 part IV (1963)	Ahmad et al. [91]

The aim of the research of Capik and Yilmaz [85] was to find prediction models to estimate the micro-Deval Coefficient (MDE) from rock properties as Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$). The study showed inverse relationships between MDE and the strength parameters of the rock. Generally, it can be conducted that the MDE decreases with the increase of the rock strength. During both the UCS micro-Deval Tests, authors followed the recommendations of ASTM. A wide variety of rocks with different origins were tested (sedimentary, igneous). During the regression curve fitting, linear, logarithmic and exponential functions were used. The best-fitting is indicated in this paper (Table and Figure on page 13).

Table 3 Comparison of the correlations between Ten Percent Fines Value (TFV) and rock strength parameters – Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$) – developed in previous studies

Equations	R^2	Rock types	Standards	References
TFV = 0.69 × UCS + 6.11	0.24	All rock types	BS 812: part 111 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 122.85 – 0.13 × UCS	0.06	Charnokite	BS 812: part 111 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 133.95 – 0.20 × UCS	0.05	Gneiss	BS 812: part 111 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 1.07 × UCS – 47.36	0.30	Granite	BS 812: part 111 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 35.62 × exp(0.008 × UCS)	0.93	Quartzite	BS 812: part 111 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 6.49 × $I_{S(50)}$ + 59.44	0.32	All rock types	BS 812: part 111 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 0.36 × $I_{S(50)}$ + 100.64	0.005	Charnokite	BS 812: part 111 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 114.73 – 1.45 × $I_{S(50)}$	0.17	Gneiss	BS 812: part 111 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 7.17 × $I_{S(50)}$ + 58.26	0.32	Granite	BS 812: part 111 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
TFV = 3.84 × $I_{S(50)}$ + 94.08	0.92	Quartzite	BS 812: part 111 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]

Kamani and Ajalloeian [2] performed several types of aggregate degradation tests to evaluate the resistance of aggregates to different mechanical weathering processes (LAAV, AIV, ACV). Uniaxial Compression and Point Load Tests were used to investigate the interrelation between rock strength and degradation parameters. As in the case of the most previously discussed studies, the aim of the research was to create correlations for an estimation that can be used to predict the degradation resistance of crushed stones from rock strength parameters. Carbonate (limestone, dolomite, travertine, marly limestone) samples were tested. Both the UCS and $I_{S(50)}$ were determined according to the ISRM. The LAAV was identified according to ASTM C131/C131M-14 on "grading B" (9.5/19 mm), while at the determination of ACV and AIV the BS 812 was used on the fraction size range of 9.52/12.7 mm. Both linear and non-linear regressions were based on the measured UCS and ACV, AIV and LAAV values (see e.g.: Fig. 1(a), Fig. 2(a), Fig. 4(a)). The best fits are summarized in the tables (Table 1, Table 2, Table 4). Generally, the non-linear interrelations led to higher correlation coefficients and an inverse relationship can be detected between UCS and the degradation parameters. In the case of ACV the exponential, in the case of AIV the logarithmic models proved to be the best. It was found that UCS has a large effect on ACV, which is explained by the fact that the loading process is similar during the two test methods. The lowest correlation coefficient was given for LAAV.

In the study of Czinder and Török [86] Hungarian andesite lithotypes were tested. The abrasion resistance of the aggregates was determined by micro-Deval Abrasion Tests on 10/14 mm fraction size followed the EN 1097-1:2012 method. The UCS tests were carried out according to the American Standard. A wide variety of regression analyses was performed in terms of the type of the curve fitting (linear, exponential and logarithmic). The exponential curve fitting resulted in the highest correlation, therefore this equation can be seen in Table 5. The established relationship was slightly modified in the Ph.D. dissertation of Czinder [87].

3 Relationships between Point Load Strength Index ($I_{S(50)}$) and aggregate degradation properties

Irfan [88] studied granites from Hong Kong by classifying them according to their grain size. The subgroups were fine-grained granite (grain size of less than 2 mm), fine- to medium-grained granite (about 2 mm), medium-grained granite (2 to 6 mm) and coarse-grained granite (over 6 mm). To define the rock strength Point Load Strength Tests were carried out on irregular lumps. Several aggregate abrasion tests were used like as Aggregate Crushing Value (ACV),

Table 4 Comparison of the correlations between Aggregate Impact Value (AIV) and rock strength parameters – Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$) – developed in previous studies

Equations	R^2	Rock types	Standards	References
AIV = exp(3.72 – 0.005 × UCS)	0.84	Igneous, Metamorphic and Sedimentary	BS 812: part 112 (1990) Deere and Miller (1966)	Al-Harthi - 1 [71]
AIV = 78.47 – 11.87 × ln(UCS)	0.87	Igneous, Metamorphic and Sedimentary	BS 812: part 112 (1990) Deere and Miller (1966)	Al-Harthi – 2 [71]
AIV = 49.78 – 0.22 × UCS	0.64	All rock types	BS 812: part 112 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 47.48×exp(–0.007 × UCS)	0.70	Charnokite	BS 812: part 112 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 127.99×exp(–0.013 × UCS)	0.65	Gneiss	BS 812: part 112 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 127.32×exp(–0.013 × UCS)	0.79	Granite	BS 812: part 112 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 69.13×exp(–0.011 × UCS)	0.93	Quartzite	BS 812: part 112 (1990) ISRM (1979)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 36.73 – 0.15 × vUCS	0.65	Limestone, Dolomite	BS 812: part 112 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 1 [2]
AIV = 69.29 – 10.52 × ln(UCS)	0.73	Limestone, Dolomite	BS 812: part 112 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 2 [2]
AIV = exp(3.71 – 0.12 × $I_{S(50)}$)	0.85	Igneous, Metamorphic and Sedimentary	BS 812: part 112 (1990) ISRM (1985)	Al-Harthi – 1 [71]
AIV = 42.20 – 12.41 × ln($I_{S(50)}$)	0.86	Igneous, Metamorphic and Sedimentary	BS 812: part 112 (1990) ISRM (1985)	Al-Harthi – 2 [71]
AIV = 30.84 – 1.80 × $I_{S(50)}$	0.64	All rock types	BS 812: part 112 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 26.08 – 1.21 × $I_{S(50)}$	0.89	Charnokite	BS 812: part 112 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 25.52 – 1.03 × $I_{S(50)}$	0.76	Gneiss	BS 812: part 112 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 27.89 – 1.35 × $I_{S(50)}$	0.67	Granite	BS 812: part 112 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 15.71 – 0.48 × $I_{S(50)}$	0.96	Quartzite	BS 812: part 112 (1990) ISRM (1985)	Afolagboye et al. [83] Afolagboye et al. [84]
AIV = 42.84 – 4.31 × $I_{S(50)}$	0.77	Limestone, Dolomite	BS 812: part 112 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 1 [2]
AIV = 0.76 × $I_{S(50)}^2$ – 10.67 × $I_{S(50)}$ + 54.25	0.83	Limestone, Dolomite	BS 812: part 112 (1990) Ulusay and Hudson (2007)	Kamani and Ajalloeian – 2 [2]
$I_{S(50)}$ = 25.01 – 6.51 × ln(AIV)	0.77	Basalt, Granite, Gabbro, Diorite, Sandstone, Limestone, Dolomite, Quartzite, Schist, Phyllite and Amphibolite	IS 8764 (1998) IS 2386 part IV (1963)	Ahmad et al. [91]

Aggregate Impact Value (AIV), Ten Percent Fines Value (TFV) determined in BS 812 and Los Angeles Abrasion Value (LAAV) according to ASTM 131-81 (on the nominal size of 20 mm). He found that the mechanical properties of the granite aggregates are highly affected by the rock strength which is porosity, density, grain size, microfracture and mineralogical composition related (see Fig. 1(b), Fig. 2(b), Fig. 3(b), Fig. 4(b)). He found that the greater the grain size, the higher the value of ACV and AIV. During the study the Uniaxial Compressive Strength was also defined from the Schmidt hammer measurements. The aggregate degradation parameters are presented as a function of this defined UCS in the previous part.

As previously discussed, Al-Harthi [71] established relationships between the Point Load Strength Index ($I_{S(50)}$) and aggregate degradation values as ACV, AIV, and LAAV (see Table 1, Table 2, Table 4). The regression analysis showed a relatively high degree of correlation for

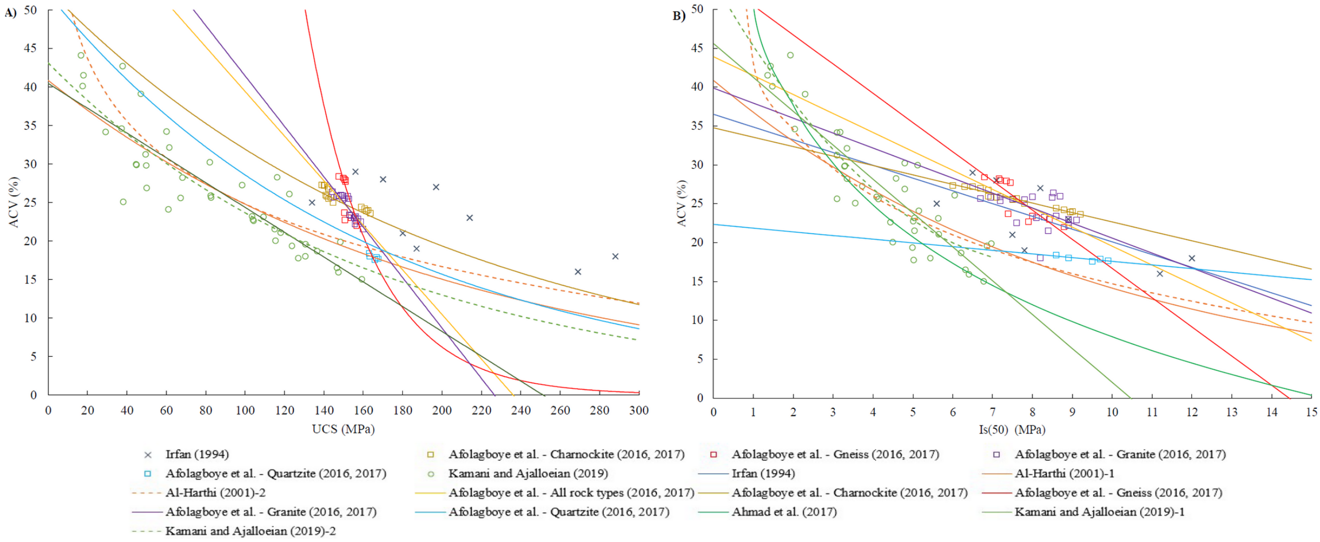

Fig. 2 Aggregate degradation test results and established relationships between aggregate degradation parameters and rock strength parameters based on previous studies. A) Relationships between Aggregate Crushing Value (ACV) and Uniaxial Compressive Strength (UCS). B) Relationships between Aggregate Crushing Value (ACV) and Point Load Strength Index ($I_{S(50)}$)

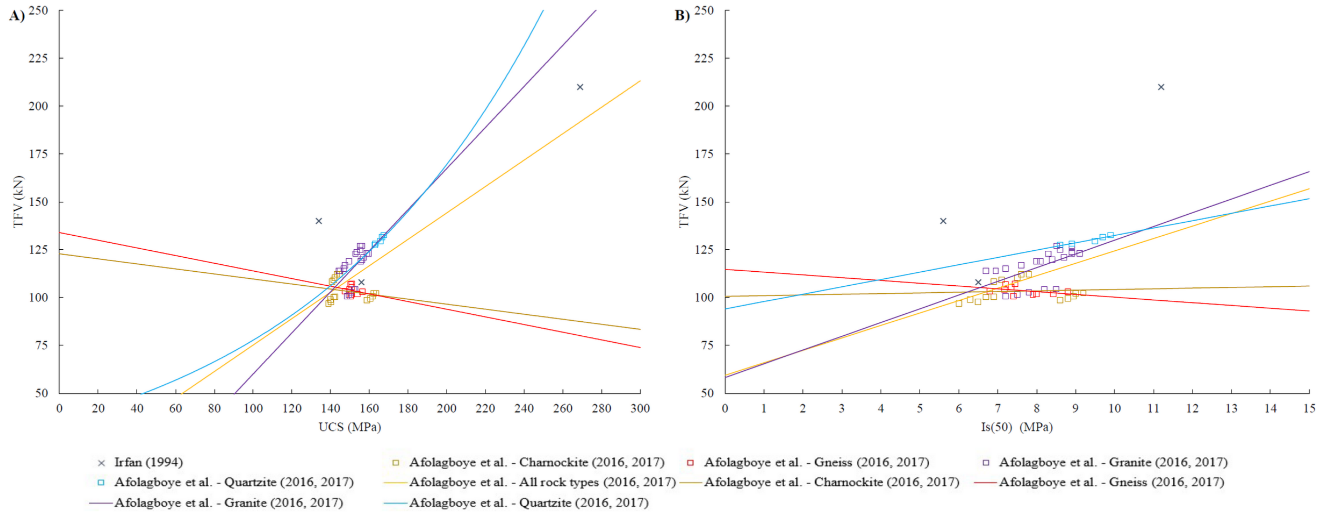

Fig. 3 Aggregate degradation test results and established relationships between aggregate degradation parameters and rock strength parameters based on previous studies. (a) Relationships between Ten Percent Fines Value (TFV) and Uniaxial Compressive Strength (UCS); (b) Relationships between Ten Percent Fines Value (TFV) and Point Load Strength Index ($I_{S(50)}$)

both the exponential and logarithmic curve fitting. This result contradicts the low-strength regression of Irfan [88]. The reason could be found in the type of the curve fitting. Presumably, the exponential and logarithmic relationships are closer to reality than the linear relationship.

The possibility that LAAV can be estimated from this Point Load Strength Index was examined by Kahraman and Gunaydin [89]. Igneous, metamorphic and sedimentary rocks were tested and divided according to porosity also. Applying this division, as in the study of Kahraman and Fener [78], it led to a stronger correlation between Los Angeles Abrasion Values and Point Load Strength Index ($I_{S(50)}$). The LAAV was defined on "Grading D" (4.75/9.5 mm) using the method of ASTM C131-66. For the Point Load Tests, regular shape specimens were used and the results were converted to the equivalent diameter of 50 mm by the prescribed ISRM method. During the regression analysis linear, logarithmic and exponential curve fittings were carried out (see Table 1) and supplemented the porosity also. Like in the case of UCS the correlation coefficients of the regression analyses were increased by considering porosity also. According to Kahraman and Gunaydin the best method is to estimate the LAAV is the Point Load Test.

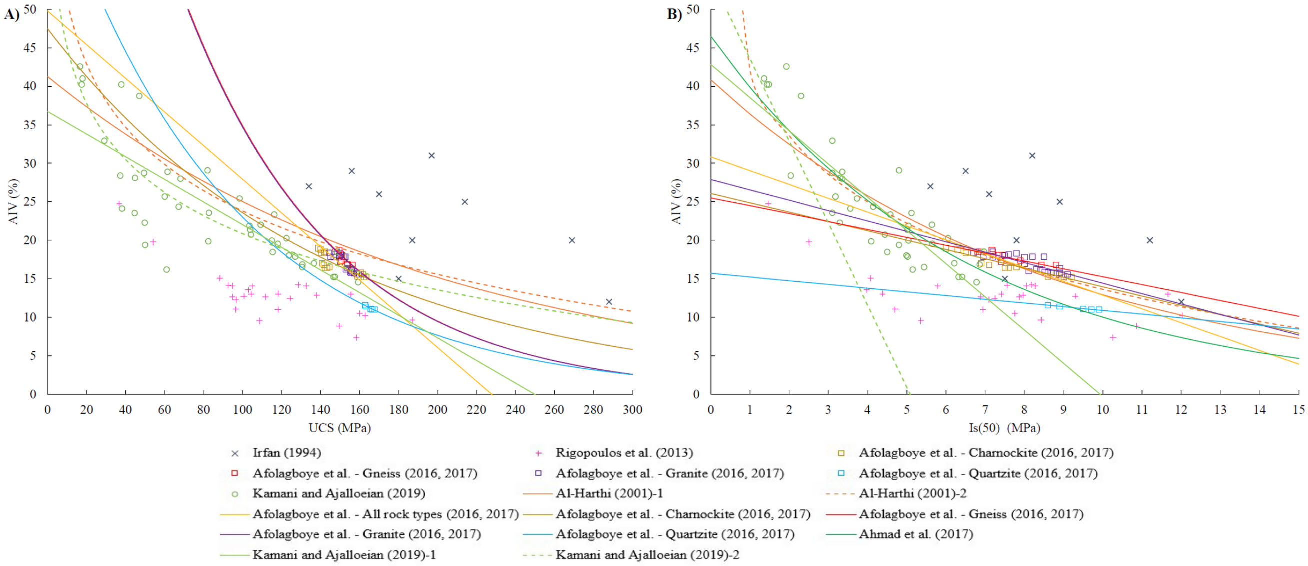

Fig. 4 Aggregate degradation test results and established relationships between aggregate degradation parameters and rock strength parameters based on previous studies. (a) Relationships between Aggregate Impact Value (AIV) and Uniaxial Compressive Strength (UCS); (b) Relationships between Aggregate Impact Value (AIV) and Point Load Strength Index ($I_{S(50)}$)

As mentioned above, Ugur et al. [79] established a relationship (see Table 1) between $I_{S(50)}$ and the LAAV divided by the P-wave velocity (V_p). The Point Load Tests were carried out following the recommendations of Boch and Franklin [90]. LAAV and $I_{S(50)}$ are inversely related.

In addition to UCS, Ozcellik [80] also examined the relation between LAAV and volume weight, apparent porosity, tensile strength, shore hardness, and Point Load Strength Index, which one is indicated in Table 1. He found that the abrasion of aggregates was greatly affected by these properties. It can be concluded that comprehensive, research is his.

As previously it was discussed, extensive research was conducted to determine whether degradation parameters can be estimated from the rock strength properties by Afolagboye et al. [83,84]. Within these studies, point strength was also measured. For the combined lithology and also the separated rock classes the regression analysis resulted in inverse linear relationships between $I_{S(50)}$ and LAAV (see Table 1), ACV (see Table 2), AIV (see Table 4), while direct linear with TFV (see Table 3), except for gneiss. The variation of these obtained correlations changed on a wide scale (from poor to very strong).

Capik and Yilmaz [85] carried out beyond Uniaxial Compression Test and Point Load Strength Test also (according to ISRM). It was investigated whether a relationship could be generated to estimate micro-Deval Coefficient (MDE) from the Point Load Strength Index ($I_{S(50)}$). The best relationship can be found in Table 5 and depicted in Fig. 5(b). The exponential curve fitting proved to be the best in the regression analysis.

According to the typology of Ahmad et al. [91], the strength of the aggregates can be indicated with Point Load Strength Index ($I_{S(50)}$), Aggregate Impact Value (AIV) and Aggregate Crushing Value (ACV), while the durability of crushed stones can be evaluated with Los Angeles Abrasion Value. The purpose of the study was to establish interrelations between the different parameters. Four types of igneous rocks (basalt, granite, gabbro and diorite), three types of sedimentary rocks (sandstone, limestone and dolomite) and four types of metamorphic rocks (quartzite, schist, phyllite and amphibolite) were tested according to Indian Standards. The results of the regression analysis are summarized in tables (Table 1, Table 2, Table 4) and plotted in graphs (Fig. 1(b), Fig. 2(b), Fig. 4(b)).

In the study of Fotev and Angelova [92] railway ballast aggregates were examined in Bulgaria. Igneous and sedimentary origin crushed stones were tested. The abrasion resistance of the materials was determined by micro-Deval and Los Angeles Tests according to the European standards (EN 1097-1 and EN 1097-2), while the strength of the rock was characterized by the Point Load Strength Index expressed by following the ASTM method. They found an inverse relationship between the abrasion values and $I_{S(50)}$. While the regression analysis resulted in a strong correlation between LAAV and $I_{S(50)}$ (see Table 1), the correlation between MDE and $I_{S(50)}$ was comparatively low (see Table 5). The authors noted the Point Load Test is the best empirical method for predicting the Los Angeles Abrasion Value.

Table 5 Comparison of the correlations between micro-Deval Coefficient (MDE) and rock strength parameters – Uniaxial Compressive Strength (UCS) and Point Load Strength Index ($I_{S(50)}$) – developed in previous studies

Equations	R^2	Rock types	Standards	References
$MDE = 20.19 - 3.54 \times \ln(UCS)$	0.66	Sedimentary, Igneous	ASTM D 6928-10 (2010) ASTM 70122-D7110 (2010)	Capik and Yilmaz [85]
$MDE = 34.18 \times \exp(-7.32E - 03 \times UCS)$	0.76	Andesite	EN 1097-1:2012 ASTM D7012-14e1	Czinder and Török [86]
$MDE = 34.18 \times 0.99^{UCS}$	0.76	Andesite	EN 1097-1:2012 ASTM D7012-14e1	Czinder [87]
$MDE = 9.55 \times \exp(-0.13 \times I_{S(50)})$	0.62	Sedimentary, Igneous	ASTM D 6928-10 (2010) ISRM (Ulusay and Hudson, 2007)	Capik and Yilmaz [85]
$MDE = 73.11 \times I_{S(50)}^{-1.22}$	0.41	Igneous, Sedimentary	EN 1097-1:2011 ASTM D 5731-08	Fotev and Angelova [92]

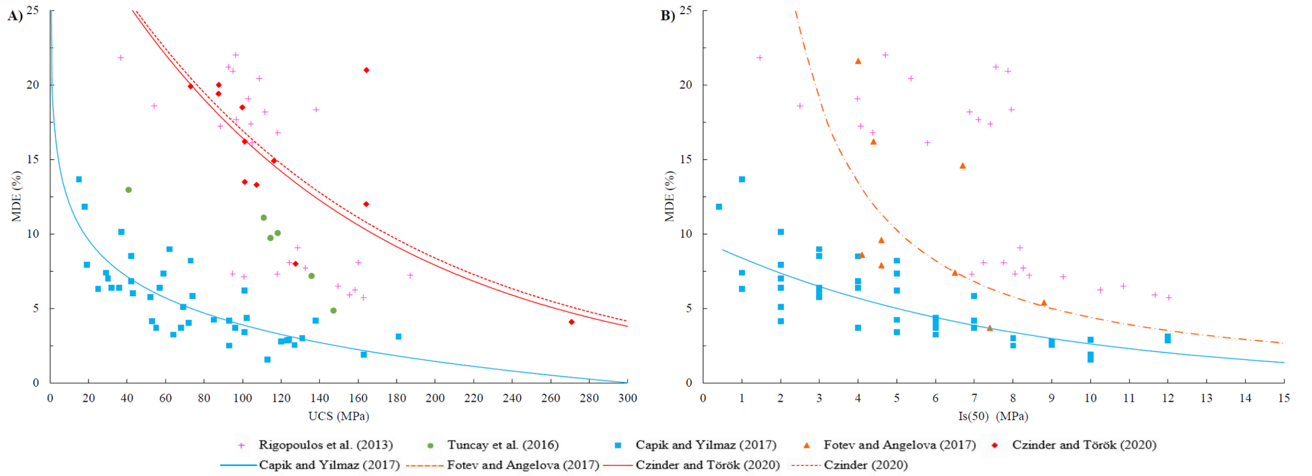

Fig. 5 Aggregate degradation test results and established relationships between aggregate degradation parameters and rock strength parameters based on previous studies. (a) Relationships between micro-Deval Coefficient (MDE) and Uniaxial Compressive Strength (UCS); (b) Relationships between micro-Deval Coefficient (MDE) and Point Load Strength Index ($I_{S(50)}$)

The results of Kamani and Ajalloeian [2] indicated the fact that aggregate degradation is better estimated from the Point Load Strength Index ($I_{S(50)}$) than from the Uniaxial Compressive Strength (UCS) in the case of samples with a lower strength (especially for AIV). In addition to this, the prediction is more accurate on specified rock types or groups than on unclassified rocks. Generally strongly inverse relationships were obtained between $I_{S(50)}$ and the degradation parameters (Table 1, Table 2, Table 4). In the case of AIV and ACV the quadratic models gave the best results, while the correlation coefficient of the linear function of LAAV and $I_{S(50)}$ was the lowest. Using a non-linear regression, the correlation coefficient increased, but the results inferred that $I_{S(50)}$ and LAAV are the least related.

4 Conclusions

Generally, it can be concluded that the extent of aggregate degradation depends on the rock strength. The results show that well-defined equations allow a good estimation of the degradation values. The relationships between TFV and the rock strength parameters were the weakest, while according to previous studies MDE and the rock strength parameters are also less related compared to other degradation values. The non-linear function fits lead to a more accurate result between the different types of degradation parameters and the rock strength parameters based on the international literature.

However, the previous studies show that the rock strength is affected by several geological feature. As a result, those research led to better regression where the evaluation of degradation and rock strength properties based on specific rock types or groups that share the same geological features [2, 80, 83, 84]. Thus, the established relationships cannot be used universally. From this, it can be deduced it is worth examining rock groups where the classification of the tested materials is carried out on the basis of geological features like texture, mineral composition, weathering, origin, etc.

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