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ORIGINAL PAPER

Relationships between the drilling rate index and physicomechanical rock properties

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Abstract The drilling rate index (DRI) is an important parameter that influences the drillability of rocks. It can easily be used when estimating the economics of any excavation operation. Therefore, in the current study, an attempt was made to investigate the rock properties that govern the DRI. The relationships between the DRI and some physicomechanical rock properties were investigated based on data obtained from experimental work and in situ studies performed in different tunnels. Regression analysis was employed to develop models for estimating the DRI based on physicomechanical rock properties. The derived models were verified based on the behavior of the determination coefficient, the t test, and the F test. The study showed that the DRI decreases with increasing uniaxial compressive strength, point load strength, Brazilian tensile strength, and Schmidt rebound hardness. It was also concluded that the DRI increases with increasing apparent porosity and void ratio. Additionally, modeling results revealed that the proposed models can be successfully used as tools to forecast the DRI.

Keywords Drilling rate index · Physicomechanical rock properties · Regression analysis · Modeling

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Introduction

Drillability is an important factor that affects the drilling rate and tool wear. Therefore, various rock properties should be taken into account when determining the drillability (Thuro 1997). Understanding the individual effects of rock properties on the drillability is essential if we are to improve the planning of underground excavations (Dahl et al. 2012). The drilling rate index (DRI) is by far the most important rock drillability parameter; it is commonly used to predict performance in drill and blast tunneling (Dahl et al. 2010; Zare and Bruland 2013; Yasar et al. 2015). The DRI is also a classification parameter in several models for estimating TBM performance. The Norwegian University of Science and Technology (NTNU) model is among the most commonly used models for estimating TBM performance based on the DRI (Dahl et al. 2007, 2012; Zare and Bruland 2013).

Although many studies (Lien 1961; Howarth and Rowland 1987; Bruland et al. 1995; Ersoy and Waller 1995; Thuro and Spaun 1996; Thuro 1996, 1997; Kahraman et al. 2000; Bilgin and Kahraman 2003; Akun and Karpuz 2005; Singh et al. 2006; Hoseinie et al. 2008; Yaralı 2008; Dahl et al. 2010; Yaralı and Kahraman 2011; Dahl et al. 2012; Yarali and Soyer 2013; Saeidi et al. 2013; Capik et al. 2013; Demirdag et al. 2014) have focused on rock drillability, only a few attempts have been made to determine the relationships between the DRI and rock properties. Recently, Yaralı and Soyer (2013) presented an excellent study of the relationships between the DRI and some mechanical properties. However, in the study described in the present paper, we determined the relationships between the DRI and some physical properties, including the apparent porosity and void ratio. We also obtained new regression models for estimating the DRI from physicomechanical rock properties. These modeling results should prove helpful for the rapid estimation of rock drillability.

Studies

Field studies

This study was carried out in the Cankurtaran and Salmankas tunnels in Turkey. The Cankurtaran Tunnel is a highway tunnel that is being constructed in the Artvin province of Turkey. It will be one of the longest highway tunnels in Turkey (5228.00 m long) when the construction is finished. The tunnel consists of two tubes, with each tube containing two lanes of traffic (one in each direction). The Salmankas Tunnel is another highway tunnel that is currently being constructed; it is located on the border of the Gumushane and Bayburt provinces of Turkey. This tunnel again consists of two tubes, each 4150.00 m in length. The locations of these tunnels are shown in Fig. 1.

Schmidt rebound hardness measurements were taken at the tunnel faces using two test devices (yielding N-type and

Fig. 1 Locations of the Cankurtaran and Salmankas tunnels

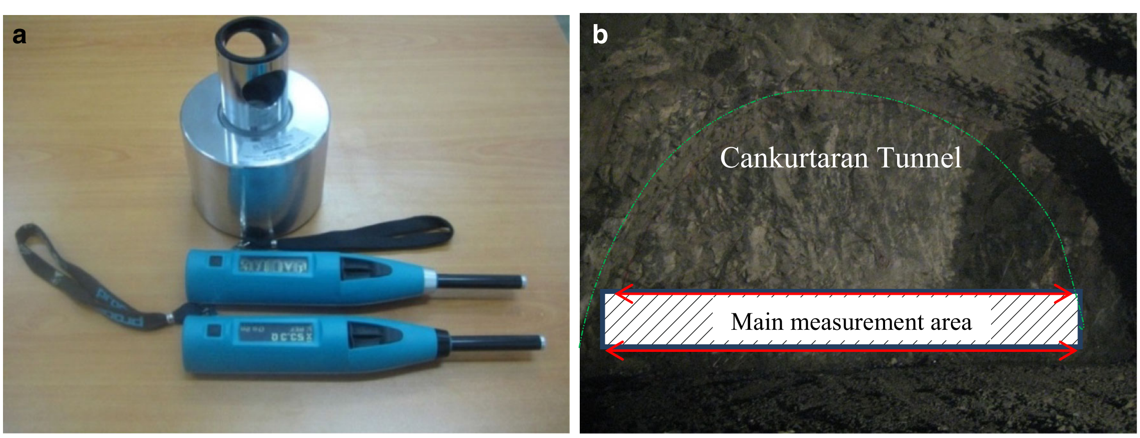

Fig. 2 **a** L- and N-type Schmidt rebound hammers (digital Proceq, Silver Schmidt) and calibration anvil; **b** tunnel area in which the measurements were taken

L-type impact energies of 2.207 and 0.735 Nm, respectively). The tests were performed in accordance with ISRM (1981) and ASTM (2005) suggested methods. The types of Schmidt rebound hammer used in the study and the tunnel area in which the measurements were taken are shown in Fig. 2a and b, respectively.

Table 1 Types and locations of rock samples

Tunnel and tube (m)	Serial no.	Geology/rock type	Distance (m) of the sampling point from the start of the tunnel
Cankurtaran Tunnel (right tube)	CR1	Sandstone	2158.00
	CR2	Porphyritic gabbro/gabbro-porphyry	2312.00
	CR3	Fossiliferous sandstone	2397.00
	CR4	Volcanogenic sandstone	2657.00
	CR5	Biomicritic limestone	2872.00
	CR6	Fossiliferous sandstone	3170.00
	CR7	Marl/micritic limestone	3554.50
	CR8	Diabase	4172.08
	CR9	Marl	4409.00
	CR10	Biomicritic limestone	4803.50
	CR11	Marl/limestone	4951.90
	CR12	Marl	5120.90
	CR13	Marl	5184.70
	CR14	Marl/limestone	5257.50
	CR15	Marl/limestone	5298.50
	CR16	Porphyritic basalt	5615.50
	CR17	Porphyritic basalt	5677.00
Cankurtaran Tunnel (left tube)	CL1	Marl	2094.50
	CL2	Clastic sandstone	2249.40
	CL3	Fossiliferous sandstone	2351.00
	CL4	Volcaniclastic sandstone	2628.00
	CL5	Fossiliferous micritic limestone/marl	2819.00
	CL6	Fine-grained sandstone	3120.00
	CL7	Micritic limestone	3518.90
	CL8	Diabase	4188.53
	CL9	Marl	4395.75
	CL10	Biomicritic limestone	4781.50
	CL11	Siltstone-marl	4939.50
	CL12	Clastic sandstone	5121.00
	CL13	Marl/limestone	5176.80
	CL14	Diabase	5229.00
	CL15	Porphyritic basalt	5586.00
	CL16	Basaltic crystal lithic tuff	5646.00
Salmankas Tunnel	B1	Andesitic crystal lithic tuff	36,760.00
	B2	Andesitic lapilli tuff	36,811.20
	B3	Andesitic crystal tuff	36,920.00
	B4	Basaltic crystal lithic tuff	37,172.00
	B5	Dolerite	37,223.80
	B6	Basaltic tuff	37,392.00
	B7	Marl	37,452.50
	B8	Agglomerated	37,605.00
	B9	Pebble stone	38,100.50
	B10	Andesite	38,897.00

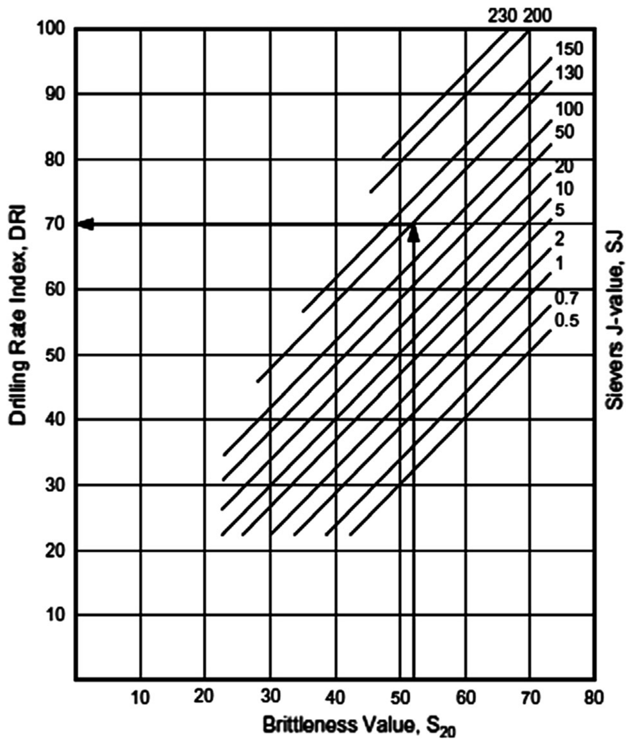

Fig. 3 Diagram showing the DRI assessment (Dahl 2003)

Table 2 Classification scheme used for the DRI (Dahl 2003)

Category	DRI
Extremely low	≤25
Very low	26–32
Low	33–42
Medium	43–57
High	58–69
Very high	70–82
Extremely high	≥83

Rock samples from 43 different areas of the tunnels were obtained for laboratory tests. Detailed information on the rock samples are provided in Table 1.

Laboratory studies

Drillability tests

The DRI was assessed on the basis of two laboratory tests, the Sievers J-miniature drill test and the brittleness test (Bruland 1998; Dahl et al. 2007). A diagram of the DRI assessment is shown in Fig. 3. The classification scheme used for the DRI values of rocks is shown in Table 2.

The Sievers J-miniature drill test was originally developed by Sievers in the 1950s. The Sievers J-value is a measure of rock surface hardness or resistance to indentation. It is defined as the mean value of the drill hole depth (in units of 0.1 mm) after 200 revolutions of a 8.5-mm miniature drill bit under a weight of 20 kg. The SJ test is normally performed as 4–8 drillings. The number and placement of the drill holes is determined by the heterogeneity and the variations in the texture of the rock sample. The standard procedure is to use a pre-cut surface of the sample which is perpendicular to the foliation of the rock. The Sievers J-value is hence measured parallel to the foliation (Dahl 2003; Dahl et al. 2012). An outline of the Sievers J-value test and the test equipment used is shown in Fig. 4.

The brittleness (S_{20}) test method was originally developed in Sweden by Matern and Hjelmer in 1943. The original test was initially intended as a means to determine strength properties of aggregates, but several modified versions of the test have since been developed for various purposes. The

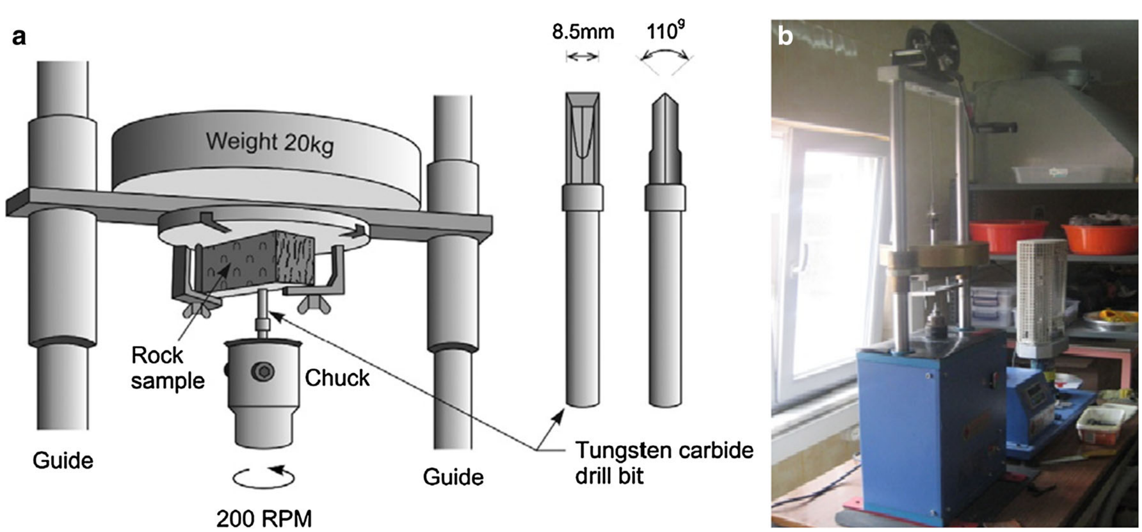

Fig. 4 **a** Outline of the Sievers J-value (SJ) miniature drill test (Dahl 2003); **b** Sievers J-value test equipment

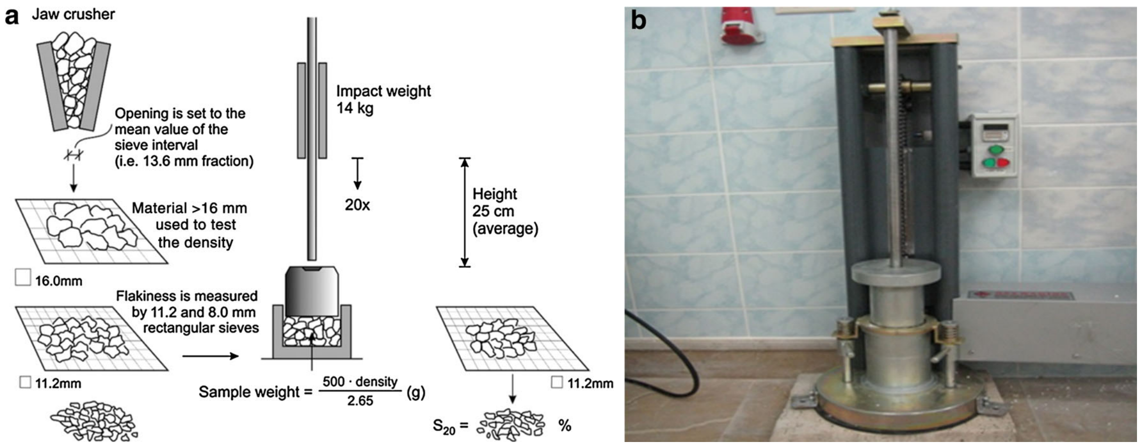

Fig. 5 **a** Outline of the brittleness (S_{20}) test (Dahl 2003), **b** brittleness test equipment

version of the S_{20} test developed for the determination of rock drillability has been used since the end of the 1950s. The test is normally performed on three equal extracts from the 11.2–16.0 mm fraction. For a measured specimen density of 2.65 g/cm^3, each extract should have a mass of 500 g. An outline of the equipment used in the brittleness test is shown in Fig. 5 (Dahl 2003; Dahl et al. 2012).

Physicomechanical properties

Uniaxial compressive strength (UCS)

The UCS tests were performed on prepared core samples with length-to-diameter ratios of 2–2.5. The applied stress rate was 0.5–1.0 MPa/s, and five core samples from each rock were subjected to the UCS test. The tests were carried out using an electro-hydraulic servo-controlled stiff press testing machine. The tests were carried out according to ISRM (1979) and ASTM (2010a) suggested methods.

Brazilian tensile strength

The Brazilian tensile strength test method consists of loading a disc of the rock until failure occurs across the diametric axis. The disc was prepared from 54-mm diameter (NX) core samples with a length-to-diameter ratio of 1:2. A loading rate of 200 N/s was applied. The tests were carried out using an electro-hydraulic servo-controlled stiff press testing machine. The test was conducted on ten samples of each rock type and the results were averaged. The tests were performed in accordance with ISRM (1981) and ASTM (2010a) suggested methods.

Point load strength

The point load strength test is intended as an index test for the strength classification of rock materials. The test was performed on core samples with a length-to-diameter ratio of 1:2. The load was steadily increased such that failure occurred within 10–60 s. The test was invalidated if the fracture surface passed through only one loading point. The point load strength test was repeated at least ten times for each rock type, and the average value was recorded as the point load strength. The tests were carried out according to ISRM (1985) suggested methods.

Physical properties

The physical properties of the rock, such as its apparent porosity and void ratio, were determined in accordance with ASTM (2010b). The apparent porosity and void ratio were evaluated for core samples with a diameter of 54 mm and a length-to-diameter ratio of 1:2 (Franklin et al. 2007).

Results and discussion

Linear relationships between the DRI and physicomechanical rock properties are depicted in Fig. 6. The figure shows that there are high correlations between the DRI and the UCS, point load strength, and Brazilian tensile strength. The DRI decreased with increasing UCS, point load strength, Brazilian tensile strength, and L-type and N-type Schmidt rebound hardness. The DRI also increased with the increasing apparent porosity and void ratio. It should also be noted that the correlation coefficients for the DRI with $RL_{\mathrm{ASTM(2005)}}$ and $RN_{\mathrm{ASTM(2005)}}$ were greater than those for the DRI with $RL_{\mathrm{ISRM(1981)}}$ and $RN_{\mathrm{ISRM(1981)}}$.

Using the relationships between the DRI and the physicomechanical rock properties, the following models (Eqs. 1–9) were developed for estimating the DRI. The determination coefficients (R^2) of the models ranged from 0.67 to 0.84. The models based on point load strength and

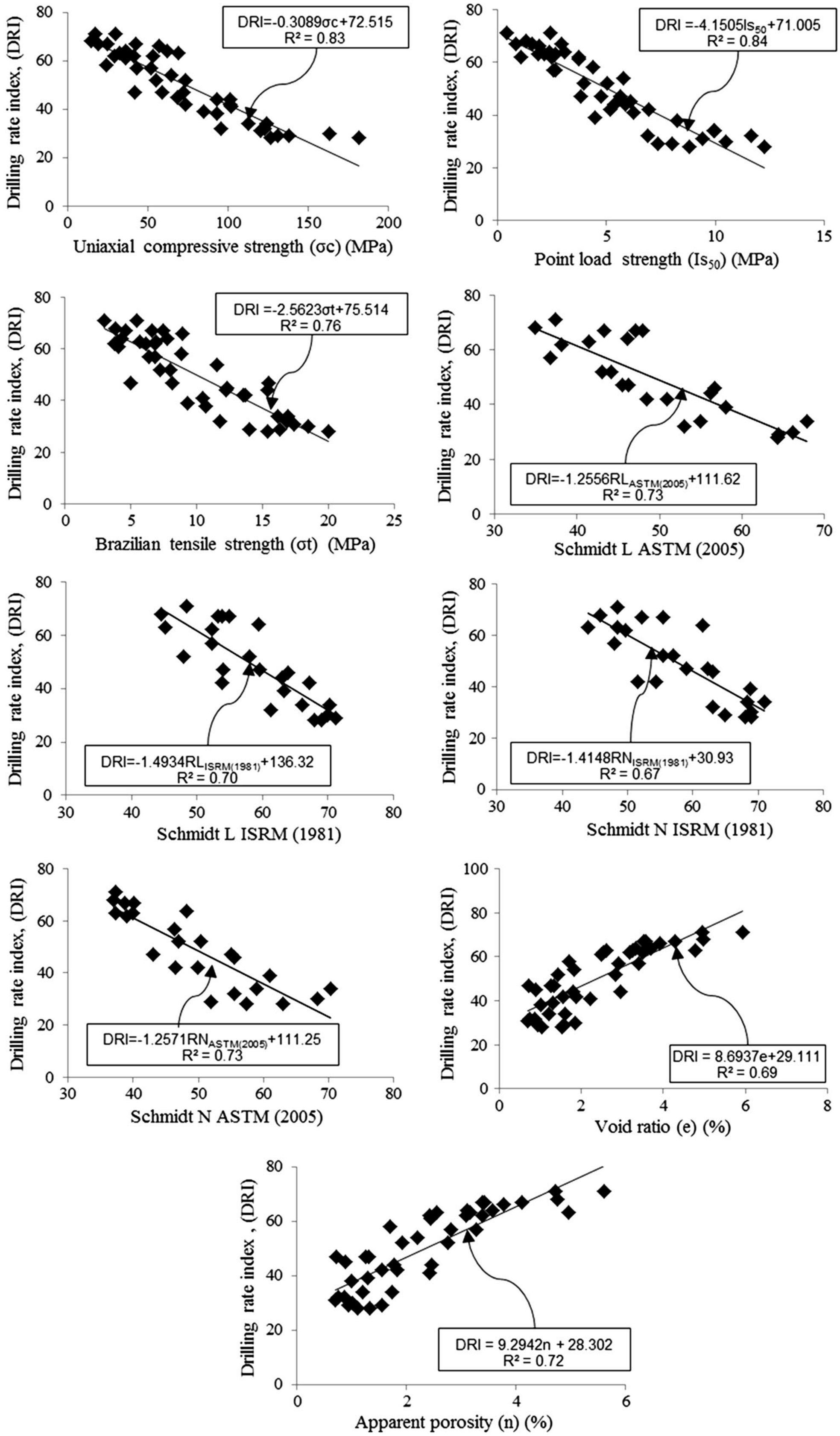

Fig. 6 Relationships between the DRI and physicomechanical properties of rock

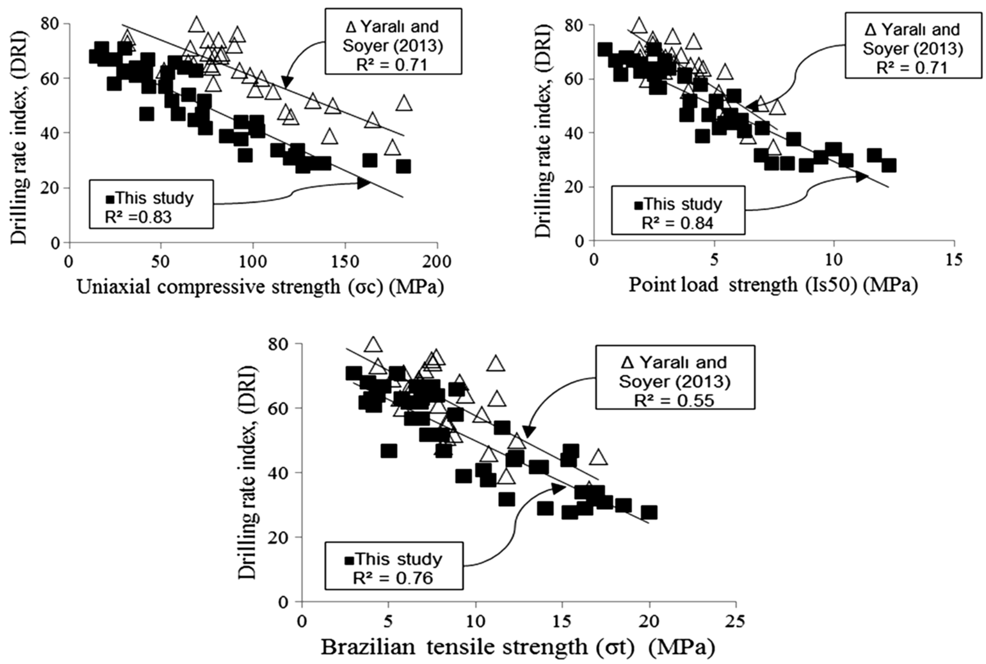

Fig. 7 Comparison of the results of this study with the results of Yarali and Soyer (2013)

UCS showed better forecasting performance than the other models.

$$\mathrm{DRI} = -0.3089\sigma_c + 72.515 \quad (R^2 = 0.83) \tag{1}$$

$$\mathrm{DRI} = -4.1505\mathrm{Is}_{50} + 71.005 \quad (R^2 = 0.84) \tag{2}$$

$$\mathrm{DRI} = -2.5623\sigma_t + 75.514 \quad (R^2 = 0.76) \tag{3}$$

$$\mathrm{DRI} = -1.4934RL_{\mathrm{ISRM(1981)}} + 136.32 \quad (R^2 = 0.70) \tag{4}$$

$$\mathrm{DRI} = -1.2556RL_{\mathrm{ASTM(2005)}} + 111.62 \quad (R^2 = 0.73) \tag{5}$$

$$\mathrm{DRI} = -1.4148RN_{\mathrm{ISRM(1981)}} + 30.93 \quad (R^2 = 0.67) \tag{6}$$

$$\mathrm{DRI} = -1.2571RN_{\mathrm{ASTM(2005)}} + 111.25 \quad (R^2 = 0.73) \tag{7}$$

$$\mathrm{DRI} = 9.2942n + 28.302 \quad (R^2 = 0.72) \tag{8}$$

$$\mathrm{DRI} = 8.6937e + 29.111 \quad (R^2 = 0.69), \tag{9}$$

where σ_c is the uniaxial compressive strength (MPa), Is_{50} is the point load strength (MPa), σ_t is the Brazilian tensile strength (MPa), n is the apparent porosity (%), e is the void ratio (%), $RL_{\mathrm{ISRM(1981)}}$ and $RL_{\mathrm{ASTM(2005)}}$ are the L-type Schmidt rebound hardness according to the ISRM (1981) and ASTM (2005) suggested methods, and $RN_{\mathrm{ISRM(1981)}}$ and $RN_{\mathrm{ASTM(2005)}}$ are the N-type Schmidt rebound hardness according to the ISRM (1981) and ASTM (2005) suggested methods.

The study results were compared with the results of Yarali and Soyer (2013), who tested 32 sedimentary, igneous, and metamorphic rocks and suggested that there were linear correlations between the DRI and mechanical rock properties. Similar trends were observed between the DRI and rock properties in both studies, as depicted in Fig. 7. However, the higher determination coefficients of the models proposed in the current study indicate that they give better prediction performance for the DRI.

The proposed models were validated using the F and t tests in SSPS 20.0. The F and t tests were carried out to check the validity of the whole model and the independent variable involved in the model, respectively (Aydin et al. 2013a). If the t value calculated in SPSS is greater than the tabulated t value (obtained from a t distribution table), the independent variable in the model is considered to be significant. If the F calculated by SPSS is greater than the tabulated F value (obtained from a F distribution table), the model is accepted as valid (Berman and Wang 2011; Aydin et al. 2013b; Aydin 2014). As can be seen from Table 3, at the 95% confidence level, the computed t values are greater than the tabulated t values, suggesting that the developed models are statistically valid. Also, at the 95% confidence level, the computed F values are greater than the tabulated F values, indicating the correctness of the models.

Table 3 Validation of the developed models

Physicomechanical property	Correlation coefficient	$F_{computed}$	$t_{computed}$	$F_{tabulated}$	$t_{tabulated}$
UCS	−0.91	195.95	−14.05	1.68	2.02
Point load strength	−0.92	210.86	−15.30	1.68	2.02
Brazilian tensile strength	−0.87	132.27	−11.30	1.68	2.02
Apparent porosity	0.85	107.17	10.33	1.68	2.02
Ratio void	0.83	91.85	9.53	1.68	2.02
$RL_{ISRM(1981)}$	−0.84	55.28	−7.42	1.98	2.07
$RL_{ASTM(2005)}$	−0.85	60.02	−7.74	1.98	2.07
$RN_{ISRM(1981)}$	−0.82	45.58	−6.72	2.02	2.07
$RN_{ASTM(2005)}$	−0.86	60.80	−7.90	2.02	2.07

Conclusions

The DRI is a reliable measure of rock surface hardness. The following conclusions can be drawn from the present study:

- The DRI decreases with increasing UCS, point load strength, Brazilian tensile strength, and Schmidt rebound hardness. It was also found to increase with increasing apparent porosity and void ratio.
- The modeling results showed that the models that included point load strength and UCS give the best DRI-forecasting performance.
- The derived models can be successfully used to predict the DRI.

Further studies could focus on the mineralogical properties of rocks such as granite in order to determine their effects on the DRI. The mineralogical properties that most strongly influence the DRI could be determined, and the DRI could be modeled as a function of these properties. The estimation of the DRI could also be investigated with multiple regression analysis, neural networks, or another metaheuristic search. The results of the different methods could be compared with simple regression analysis to gauge the performance of the proposed models.

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