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Using Intact Rock Brittleness for Assessing TBM Penetration

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Abstract - The paper reviews the history of the punch penetration test (PPT) as a laboratory method that is conducted on intact sample to measure the rock brittleness (BI). Further, the interaction of rock brittleness and penetration rate of tunnel boring machine (TBM) has been examined and some relationship was developed between the BI and TBM penetration. For this aim, four tunnel projects having various rock types were studied and then four different dataset is examined and discussed herein. It is stated that the test via BI can be perfectly utilized for assessing the boreability of rocks. In conclusion, it is found that there is a relationship between the BI and TBMs penetration with correlation coefficient (r) ranging from 0.34-0.81 for hard rocks condition; however this correlation is quite low (0.34) for weathered and very weak sedimentary rocks categorized as extremely low brittle. The obtained results can be accepted for hard rocks but it should be used with care for weathered weak rocks that does not show brittleness behavior under the indentor.

***Keywords*:** Brittleness, penetration, punch penetration test, rock, tunnelling.

1. Introduction

Rock brittleness that is combination of several rock properties rather than only one rock property is one of the most crucial rock parameters for rock mechanics applications like mechanical tunneling. In spite of that, there is no agreement in rock mechanics community to describe or measure the brittleness. Many researchers [1-10] have indirectly identified the brittleness by using several approaches. Schwartz [1] conducted a series of triaxial tests on intact rock samples of the Indian limestone. He defined that the transition from brittle to ductile behavior appears at a principal stress ratio of approximately 4.3. Hatenyi [2] defined brittleness as the lack of ductility. Ramsay [3] said that as the internal cohesion of rock is destroyed, the rock could be accepted as brittle. Hucka and Das [4] recommended that low values of elongation, fracture failure, higher angle of internal friction, formation of the crack in indentation, and higher ratio of uniaxial compressive to Brazilian tensile strength demonstrate the higher brittleness value. Goktan [5] stated that brittle rock should have a lower specific energy than a less brittle one. Evans and Pomeroy [6] found that impact energy of a cutter pick is inversely proportional to brittleness. Altindag [8] suggested an indirect measured brittleness values obtained from the ratio of uniaxial compressive strength (UCS) and Brazilian tensile strength (BTS) of rock. As seen in the literature, most of the brittleness values are computed via various ratio of uniaxial compressive strength to Brazilian tensile strength of rock. Still several researches are on the way to measure the rock brittleness based on international standard rather than strength ratios.

In late 1960s, punch penetration test (PPT) was developed to provide direct laboratory method to investigate rock behavior under the indenter [11]. Since then, a number of major modification and improvements were made on the test procedures and data evaluations [7, 12-15]. Szwedzick [13] employed the test for measuring rock hardness; Dollinger et al., [14] indicated that the test provides qualitative data for investigating rock behavior under the indenter; Yagiz [7] developed rock brittleness index (BI) based on output of the test. Further, Yagiz and Rostami [16] stated that the observation of the saw tooth behavior of the curve and monitoring of the increases and sudden drop in forces is a good representative of the rock brittleness. In present, the PPT is one of two methods together with Norwegian brittleness test (S_{20}) to measure the brittleness from intact rock sample in the laboratory.

In this study, using various tunnel projects, interaction between the rock brittleness and TBM penetration is evaluated. The relationship between the brittleness index (BI) and rate of penetration (ROP) were developed and discussed herein.

2. Materials and Methods

The PPT can be used for investigating rock properties such as brittleness, toughness, hardness and drillability by using different evaluation techniques [12-17].The test for evaluating the force-penetration graph generated from the outcome of the test is very critical, since the test result characterizes various rock properties rather than only one. The first test apparatus was designed [12]; the test has been used for different purposes and does not specify certain rock properties [7, 13-15]. Standard sample preparation and testing procedure for this test was given previously in the literature [9]; however typical sample set up for testing is re-called (Figure 1).

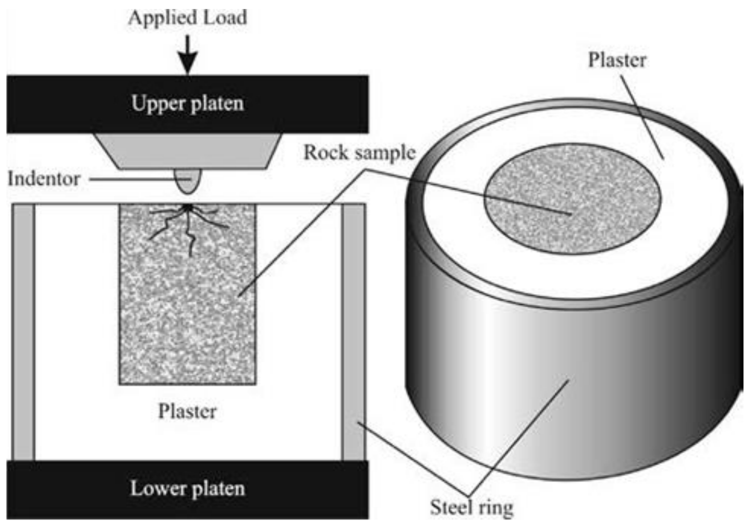

Fig. 1: Schematic drawing of the test apparatus and sample set up [9].

In the earliest method, the test was conducted to examine the boreability of rocks via the slope of the best fit line on force-penetration chart (Figure 2). The slope of the best fit line (kN/mm or lb/in) was named as penetration index (PI) employed for predicting the expected cutter force and corresponding penetration for mechanical excavation [12]. Utilizing the PPT, the obtained force-penetration graph having three distinct phases could be obtained [7, 9 and 13].

In the *first phase*, there is elastic deformation and very fine crushing of rock surface. In the *second phase*, there is crushing of rock fabric as in the *third phase* chips of rock are formed. Elastic deformation and very fine crushing is represented by a linear relation. Crushing in represented by “steps” on the profile and chipping of the rock fragments is represented by peaks (Figure 2). Szwedzicki [13] concluded that it is concluded that the first elastic-linear phase of force-penetration profile, *indentation hardness index* (IHI) could be computed as the ratio of the force, (F) to corresponding penetration, (P) in kN/mm. Yagiz [9] stated that the test can be used for examining the brittleness behavior of rock under the indenter or disc cutters; since the test has three diverse phase of force-penetration profile and represent various rock properties. Consequently, the brittleness index (BI) in kN/mm were computed as using *the slope of whole phase of force-penetration profile*, obtained by drawing line from origin of the chart to the maximum applied force that rock absorbs till the test ended [9]. As a result, the brittleness can be computed as follow;

$$BI = \frac{F_{max}}{P} \tag{1}$$

Where, F_{max} is maximum applied force on a sample in kN, and P is corresponding penetration in mm.

At this graph, the higher brittle rock shows fluctuated force-penetration profile due to the large force drops with large chips; though, moderate brittle rock indicates minor force drop with small chips. As the rock is ductile or low brittle, then there is no force dropping and chipping on the rock but crashing. Further, the based on obtained values and fluctuated force-penetration profile from the test, the rock brittleness could be classified (Table 1).

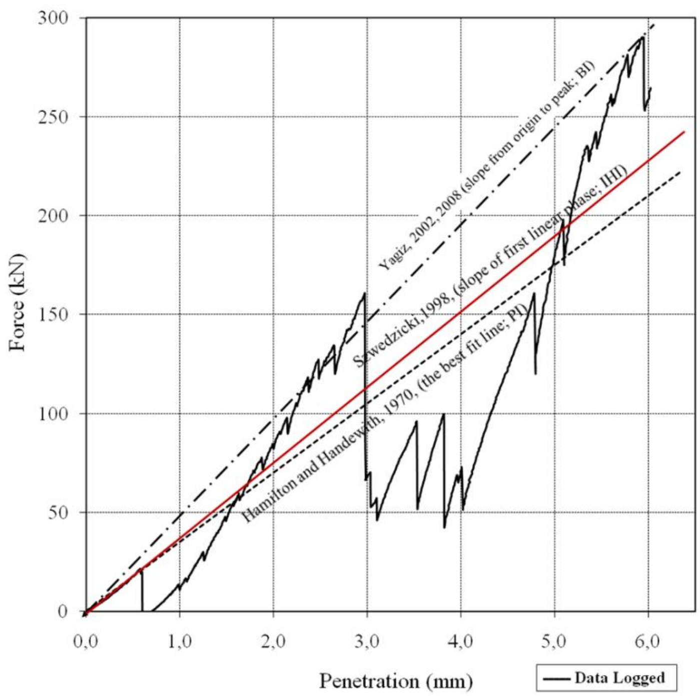

Fig. 2: Various expressions from the fluctuated force-penetration graph of the test [18].

Using Table 1, rock can be classified according to the brittleness values and then BI can be used as an input for estimating TBM penetration in various models, such as Colorado School of Mines (CSM) or Modified CSM Model.

Table 1: Brittleness classification based on laboratory test; updated from [9].

BI	Classification
>45	Extremely high brittle
40-44	Very high
35-39	High
30-34	Medium
25-29	Low
20-24	Very low
<19	Extremely low

The PPT tests equipment used for measuring the brittleness of rocks cannot be found in every institute or laboratory, since it is expensive and preparing the test sample is not easy and time consuming. In order to overcome this problem, Yagiz [9] introduced empirical equation for estimating rock brittleness (BI) as a function of rock strengths and density for various rocks including sedimentary through metamorphic as below:

$$BI = 0.198 \times \sigma_c - 2.174 \times \sigma_t + 0.913 \times \rho - 3.807 \tag{2}$$

In this equation, the σ_c, σ_t, and ρ refers to uniaxial compressive strength, Brazilian tensile strength and density of rock respectively. With this equation, the brittleness may be estimated around 10% of average relative error. In the followings, cases and obtained dataset with analysis introduced for evaluating the brittleness and TBM penetration interaction.

2.1. Case Studies

In order to examine the relationship between the rock brittleness and TBM performance, various tunnel projects are evaluated in accordance with TBM performance obtained in the field and intact rock behavior under the indenter in the laboratory. Following projects have been highlighted for this purpose.

2.1.1. Queens Tunnel

The Queens Water tunnel was constructed to improve distribution of freshwater throughout the City of New York, especially in district of Queens [7]. The Queens Water tunnel was located in New York City where the geological formation was highly complex and composed of different type of metamorphosed igneous rock with shear zones, faults, joints and other local fractures. The combination of mineral assemblage and rock texture demonstrated that the rock had several episodes of high degree metamorphic recrystallizations. The tunnel site composed of rhyodacite dyke, granitic gneiss intermixed with orthogneiss, gneiss, amphibolite, pegmatite, and the combination of gneiss-schist [7, 17 and 19].

2.1.2. Manapouri Tunnel

The second tailrace tunnel of the Manapouri hydro project that is an underground hydroelectric power station located in the Fiordland area of southwestern New Zealand. The objective of adding the tailrace tunnel was to increase the overall cross-sectional area of flow, thereby reducing the flow velocities and associated frictional head losses [20]. The Paleozoic age metamorphic rocks dominate the region of the Fiordland National Park, which is bounded by the tectonic boundary between the Australian and the Pacific crustal Plates in New Zealand. The principal rock types were: gneiss with varying degrees of intrusion by pegmatite and other granitic rocks; gabbro; diorite and minor amounts of mixed meta-sediments like marble, quartzite and calc-silicates [20-23].

2.1.3. Zagros Tunnel

Zagros long tunnel (lot 2) with a length of 26 km and about 6.73 m diameter is located in Kermanshah Province in the west of Iran where the tunnel has been designed to transfer water from the Sirwan River to the tropical plains. Geological formations along the tunnel route mainly composed of dark gray shale, shaly limestone, and limestone masses [24]. Arabian plate compressional tectonic forces have created several faults and thrust faults with NW-SE trend in the study area. The tunnel strike is NESW, so that cuts vertically the trend of region structures [24]. The area is located in the Zagros Fold- Thrust Belt [25].

2.1.4. Karaj Tunnel

The Karaj–Tehran water conveyance tunnel with 30 km length is located northwest of Tehran in Iran, between Karaj and Tehran, and was designed for transferring 16 m^3/s of water from Amir–Kabir dam to Tehran [26]. The Karaj Water tunnel is divided into two sections: Data collected from first section of the tunnel that is 16 km long at the southeast end is used for this study. The lithology of this section of the tunnel consists of a sequence of Karaj formations and is composed of variety of pyroclastic rocks, often interbedded with sedimentary rocks [27-30].

3. Results and Discussions

At present, the ratio of the compressive strength and Brazilian tensile strength of rock are mostly used for estimating the rock brittleness but it is just the ratio and not represents the rock behavior under the indenter or cutters. The brittleness that can be computed via punch penetration test is one of the main rock parameters to be used for estimating TBM penetration rate in rock mass. The punch penetration is the only rock test where the rock surface is actually penetrated by a tool, causing crushing and chipping in a fashion similar to that which occurs with cutters on a TBM. Because this test actually penetrates the rock, it provides the capability to reveal some important rock boreability features that the compressive and the tensile strength tests may fail to show. In other words, an examining the force-penetration graph obtained from the test, the rate of penetration may be computed since the indenter penetrates the rock in the laboratory and TBM penetrates the rock in the field has similar fashion.

In this paper, four tunnel projects were reviewed to examine the relationship between the ROP and BI. The revised data including ROP and BI were given in Table 2 for Zagros and Manapouri Tunnels and Table 3 for Queens and Karaj Tunnels. For further information relevant to each project, please see references given in the Tables. According to the dataset, it is observed that the BI values ranges from around ductile rocks such as sedimentary and shales, to hard brittle rocks like granite. Each tunnel case includes different rock type and also machine used for excavation (Table 2-3) because of that variation of the data ranges and obtained results are likely.

Table 2: Field ROP and BI for each station along the Zagros and Manapouri Tunnels.

Zagros Tunnel [24]	BI (kN/mm)	ROP (m/hr)	*Manapouri Tunnel* (Unpublished data)	BI (kN/mm)	ROP (m/hr)
Shale, marly limestone	6.99	17.7	Calc silicate gneiss	38.88	1.36
Shale, marly limestone	6.2	17.4	Hormblend bitite gneiss	28.37	1.09
Marly limestone	6.68	16.7	Biotite hormblend gneiss	31.17	1.08
Dark marly limestone and shale	6.61	17.25	Biotite hormblend gneiss	21.89	0.52
Shale marly limestone	4.9	14.73	Biotite hormblend gneiss	29.42	0.75
Shale marly limestone	4.37	21.67	Biotite hormblend gneiss	29.42	0.61
Shale limy shale	4.95	18.24	Biotite hormblend gneiss	26.10	0.99
Shale limy shale	4.57	19.09	Biotite hormblend gneiss	35.75	1.35
Shale, shaly lime, sandstone	5.96	17.95	Hormblend biotite gneiss	42.11	1.61
Limestone. shaly limestone	3.24	33.28	Hormblend biotite gneiss	34.58	1.51
Limestone	2.86	27.48	Biotite hormblend gneiss	33.62	1.89
Limestone	3.34	24.82	Biotite hormblend gneiss	24.63	0.83
Limestone	3.53	21.27	Calc silicate	31.17	1.20
Limestone, marl	4.65	19.96	Calc silicate	17.34	1.02
Limestone, shale, gypsum	4.45	21.66	Hormblend biotite gneiss	19.61	0.69
Limestone	2.45	24.6	-	-	-

Table 3: Field ROP and BI for each station along the Karaj and Queens Tunnels.

	BI (kN/mm)	ROP (m/hr)	*Queens Water Tunnel* [17]	BI (kN/mm)	ROP (m/hr)
	3.23	3.49	Granitoid (felsic) gneiss and orthogneiss	55	2.19
	3.26	4.88	Granitoid (felsic) gneiss and orthogneiss	55	2.12
	3.48	5.73	Mafic- to mesocratic orthogneiss	55	1.88
	3.88	5.02	Mafic- to mesocratic gneiss. amphibolite	56	2.81
	3.40	2.99	Mafic- to mesocratic gneiss. amphibolite	56	2.20
	3.71	3.32	Granitoid (felsic) gneiss and orthogneiss	58	2.37
	1.22	2.02	Mafic- to mesocratic orthogneiss	58	2.34
	2.08	3.35	Granitoid (felsic) gneiss and orthogneiss	58	2.90
	2.68	3.89	Granitoid (felsic) gneiss and orthogneiss	57	3.04
	2.20	1.89	Granitoid (felsic) gneiss and orthogneiss	57	3.07
	2.58	4.63	Massive garnet amphibolite and mafic	54	3.04
	2.26	1.97	Massive garnet amphibolite and mafic	54	2.95

[28] Karaj Tunnel; The formation mostly composed of altered thin bedded shale, siltstone and sandstone; weak rocks Ductile!	1.36	2.43	Massive garnet amphibolite and l mafic	53	2.66
	2.84	1.52	Massive garnet amphibolite and mafic	52	2.30
	3.07	1.63	Massive garnet amphibolite and mafic	52	2.17
	2.62	1.48	Massive garnet amphibolite and mafic	46	2.87
	3.04	2.31	Mafic- to mesocratic gneiss. amphibolite	45	2.48
	3.86	6.55	Granitoid (felsic) gneiss and orthogneiss	43	2.02
	2.80	1.49	Granitoid (felsic) gneiss and orthogneiss	39	1.87
	2.53	2.57	Granitoid (felsic) gneiss and orthogneiss	39	2.00
	2.60	2.35	Granitoid (felsic) gneiss and orthogneiss	39	2.45
	2.66	1.50	Granitoid (felsic) gneiss and orthogneiss	42	2.18
	2.35	2.41	Mafic- to mesocratic gneiss. amphibolite	42	2.17
	2.81	5.73	Mafic- to mesocratic orthogneiss	46	2.09
	2.35	2.57	Mafic- to mesocratic orthogneiss	46	2.10
	2.39	1.61	Mafic- to mesocratic orthogneiss	46	2.05
	2.60	3.13	Massive garnet amphibolite and mafic	42	2.60
	1.79	2.38	Mafic- to mesocratic orthogneiss	43	2.46
	1.72	2.45	Mafic- to mesocratic orthogneiss	42	2.20
	1.70	4.04	Mafic- to mesocratic orthogneiss	41	2.14
	4.78	1.97	Mafic- to mesocratic orthogneiss	41	2.78
	1.22	1.67	Massive garnet amphibolite and mafic	40	2.03
	2.66	2.41	Massive garnet amphibolite and mafic	39	2.05
	1.72	1.87	Mafic- to mesocratic gneiss. amphibolite	39	2.47
	3.07	2.38	Granitoid (felsic) gneiss and orthogneiss	34	2.17
	2.80	2.31	Mafic- to mesocratic gneiss. amphibolite	34	1.65
	2.06	1.99	Mafic- to mesocratic gneiss. amphibolite	32	1.75
	2.17	1.49	Massive garnet amphibolite and mafic	32	1.87
	3.48	5.11	Massive garnet amphibolite and mafic	32	1.78
	3.71	1.80	Massive garnet amphibolite and mafic	32	1.78
	2.08	2.20	Massive garnet amphibolite and mafic	31	2.39
	3.48	2.12	Mafic- to mesocratic orthogneiss	31	2.13
	1.70	2.01	Mafic- to mesocratic gneiss. amphibolite	30	1.71
	3.28	4.47	Granitoid (felsic) gneiss and orthogneiss	30	2.04
	4.70	1.87	Mafic- to mesocratic gneiss. amphibolite	30	2.39
	2.21	5.37	Mafic- to mesocratic gneiss. amphibolite	30	2.14
	4.10	5.13	Mafic- to mesocratic gneiss. amphibolite	30	1.88
	3.89	4.12	Granitoid (felsic) gneiss and orthogneiss	34	2.12
	4.56	3.49	Granitoid (felsic) gneiss and orthogneiss	35	2.47

The established datasets compiled from the literature are used for examining the relationship between the ROP and BI. In order to develop an equation between BI and ROP, simple statistical correlations were conducted and the achieved results are demonstrated in Figure 5 for each tunnel case. So, the ROP and BI have relation with regression coefficient ranges from 0.34 to 0.81. It is found that there is a relationship between the ROP and BI; however, these relations and its rank is various from project to project; since each project has different rock including brittle through ductile (Table 2-3).

4. Conclusion

In this paper, the relationships between brittleness obtained via punch penetration test and penetration rate of TBM were were statistically examined using the several dataset obtained from Queens, Manapouri, Karaj and Zagros tunnel route. It is It is found that there is acceptable correlation between the penetration rate of TBM and the brittleness for Queens, Manapouri Manapouri and Zagros tunnel (correlation coefficient of 0.58, 0.71 and 0.81 respectively); however, this relationship is quite quite low for Karaj tunnel that is composed of ductile "*sedimentary*" rocks like altered thin bedded shale, siltstone and sandstone with correlation coefficient of 0.34.

It is found that the correlations between the ROP and the BI are fairly good especially for hard rock conditions; however, those relations could change from site to site due to excavated rock type and utilized machine specification. So, the obtained results should be used for similar rock types with care. Further, since the test penetrates the rock in the laboratory condition, it provides the capability to reveal some important rock boreability features that the rock strength tests may fail to identify. As a result, the test has the potential of becoming a standard to be used for rock mechanics and tunneling.

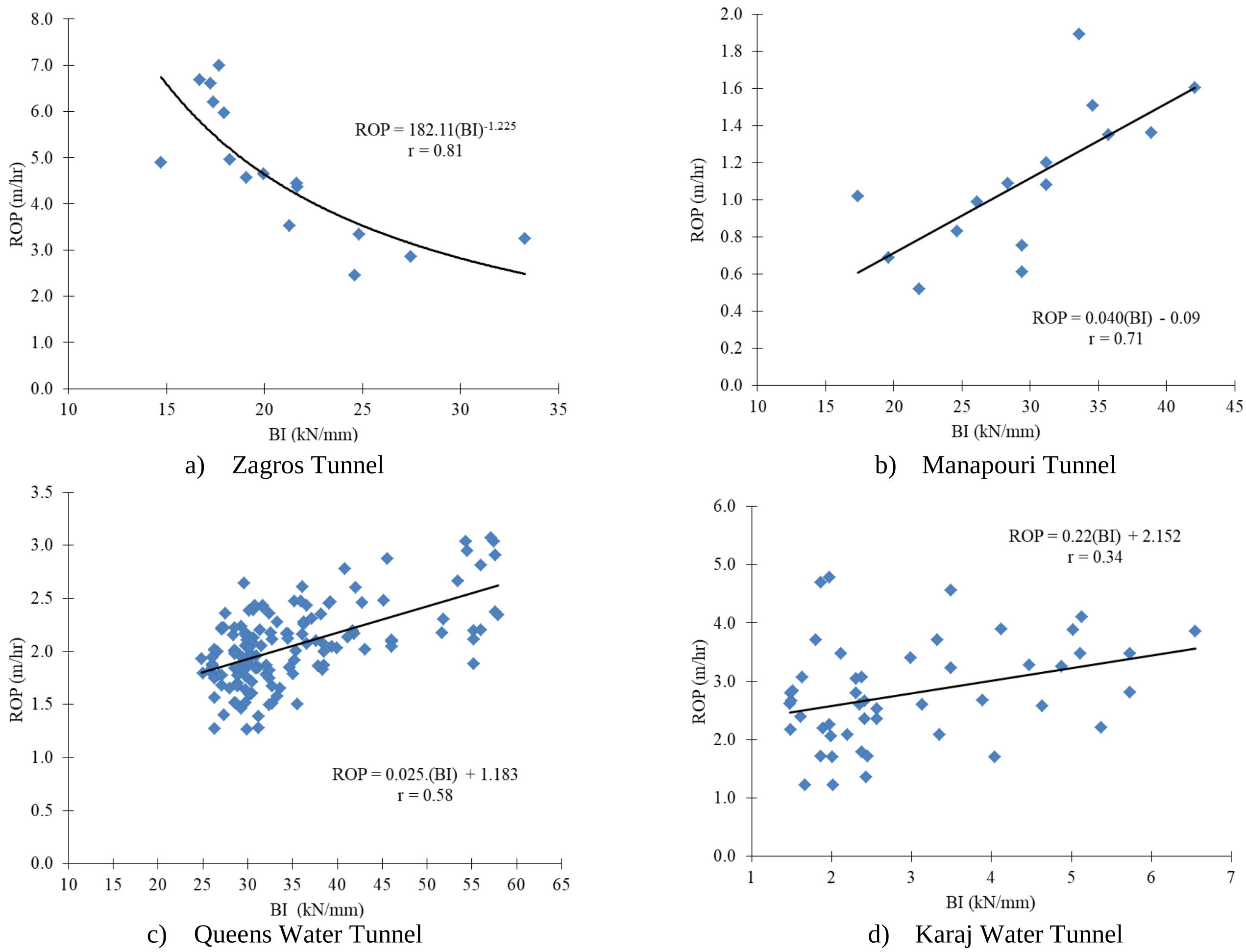

Fig. 5: Relationships between the ROP and BI for different tunnel cases.

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