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# Determination of Unit Tip Resistance for Drilled Shafts in Fractured Rocks using the Global Rock Mass Strength

Miguel Truzman, David Corley & David Lipka  
Terracon Consultants, Charlotte, NC, USA



## ABSTRACT

Over the last several years, we have been using the AASHTO LRFD Design Specifications to calculate factored resistances for drilled shafts with tips in fractured rocks around the Charlotte Area in North Carolina. These rocks typically consist of granites, diorites and metavolcanics. Most of these rocks are jointed with varying amounts of weathering. The current equation in the Design Specification gives overly conservative unit tip resistance values when the foundation rock has a Rock Mass Rating (RMR) less than 45. This paper evaluates the global rock mass strength equation proposed by Hoek, et. al. (2002) to determine more reasonable tip resistance values for poor to fair quality rock masses with RMR values less than 45. Additionally, we propose a correlation to determine the RMR and the Geological Strength Index (GSI) based on Rock Quality Designation (RQD) values.

## RESUMEN

En los últimos años, los autores han utilizado las Especificaciones de Diseño de la AASHTO LRFD para calcular la resistencia factorizada de pilotes excavados con la punta en rocas fracturadas alrededor del área de Charlotte en Carolina del Norte. Estas rocas típicamente consisten en granitos, dioritas y metavolcánicas y la mayoría de ellas presentan un variado grado de meteorización. Con la actual ecuación recomendada en las Especificaciones de Diseño se obtienen valores unitarios conservadores de la resistencia por punta cuando la roca de fundación tiene una Valoración del Macizo Rocoso (RMR) menor a 45. Este artículo evalúa la ecuación de resistencia global de macizos rocosos propuesta por Hoek et. al. (2002) para determinar valores de resistencia más razonables para macizos con calidades de roca de pobre a mediana con valores del RMR menores a 45. Adicionalmente, se propone una correlación para determinar el RMR y el Índice de Resistencia Geológica (GSI) basado en valores de Designación de la Calidad de Roca (RQD).

## 1 INTRODUCTION

The Load and Resistance Factor Design (LRFD) method was first adopted in 1994 by the American Association of State Highway and Transportation Officials' (AASHTO); and in concurrence with the Federal Highway Administration (FHWA), after October 1, 2007, all states must design new bridges using LRFD.

LRFD incorporates state-of-the-art analysis and design methodologies with load and resistance factors based on the known variability of applied loads and material properties. These load and resistance factors are calibrated from actual bridge statistics to ensure a uniform level of safety.

We have been using the LRFD method for bridges that require DOT design guidelines in the Charlotte Area of North Carolina. The area is part of the Piedmont Geologic Province with rocks consisting of granites, diorites and metavolcanics.

Most of these projects include bridges with end bents supported on driven piles and interior bents supported on drilled shafts.

To develop the required factored resistances for most bridges, drilled shafts typically must bear on Intermediate Geomaterials (IGM) with SPT N-values from 50 to 100 blows per foot (bpf), Partially Weathered Rock (PWR,  $\geq 100$  bpf) and/or rock.

## 2 LRFD PRACTICE FOR BASE RESISTANCE IN ROCK

The generalized strength criterion for jointed rock masses has been defined by Hoek & Brown (1997) as:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left( m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a \quad \text{Eq. 1}$$

where:

$\sigma'_1, \sigma'_3$  = major and minor effective stresses at failure respectively.

$\sigma_{ci}$  = uniaxial compressive strength of the intact rock

$m_b$  = Hoek-Brown constant m for the rock mass

$s, a$  = constants which depend upon the characteristics of the rock mass.

It has been shown by Carter and Kulhawy (1998); and Willie (1999) that a conservative, lower-bound estimate of bearing capacity can be made directly in terms of Hoek-Brown strength parameters by assuming a failure mode approximated by active and passive wedges. This method is known as the Bell solution for plane strain. The failure mass beneath the foundation is assumed to consist of two zones, as shown in Figure 1.

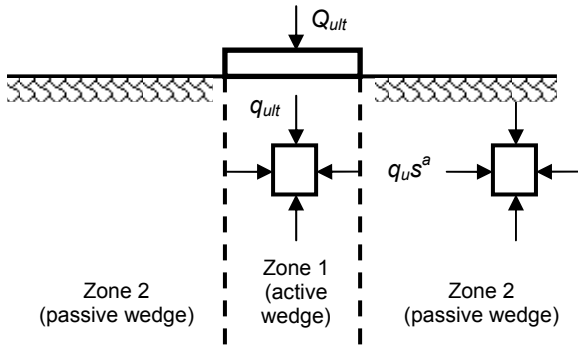


Figure 1. Bearing Capacity Analysis for Rock

The active zone (Zone 1) is subjected to a major principal stress ( $\sigma'_1$ ) which is the nominal bearing resistance at failure ( $q_{ult}$ ) and a minor principal stress ( $\sigma'_3$ ) that satisfies equilibrium with the horizontal stress in the adjacent passive failure zone (Zone 2). In Zone 2, the minor principal stress is conservatively assumed to be zero; and the major principal stress, acting in the horizontal direction, is the ultimate strength according to the Hoek-Brown criterion.

For Zone 2, setting the vertical stress  $\sigma'_3 = 0$  and solving Eq. 1 for  $\sigma'_1$  yields

$$\sigma'_1 = \sigma'_H = q_u s^a \quad \text{Eq. 2}$$

where:

$\sigma'_H$  = horizontal stress in Zone 2

To satisfy equilibrium, the horizontal stress given by Eq. 2 is set equal to  $\sigma'_3$  in Zone 1. Substituting  $\sigma'_3 = q_u s^{0.5}$  into Eq. 1 and considering that  $\sigma'_1 = q_{ult}$  yields

$$q_{ult} = q_u [s^a + (m_b s^a + s)^a] \quad \text{Eq. 3}$$

For most cases,  $a = 0.5$  (or very close to it). Therefore, we can simplify Eq. 3 and set the tip resistance  $q_p$  as equal to  $q_{ult}$  and we get the LRFD equation in Section 10.8.3.5.4c.

$$q_p = \left[ \sqrt{s} + \sqrt{(m_b \sqrt{s} + s)} \right] q_u \quad \text{Eq. 4}$$

where:

$q_p$  = unit tip resistance

$s, m_b$  = fractured rock mass parameters

$q_u$  = unconfined compressive strength of rock (ksf)

The values of the Hoek-Brown constants  $s$  and  $m$  are listed in the LRFD Design Specification in Table 10.4.6.4-4 based on the type of rock. These values were calculated based on the following equations for disturbed rock masses:

$$m_b = m_i e^{\left( \frac{RMR-100}{14} \right)} \quad \text{Eq. 5}$$

$$s = e^{\left( \frac{RMR-100}{6} \right)} \quad \text{Eq. 6}$$

### 3 LIMITATIONS OF LRFD RECOMMENDATIONS FOR UNIT TIP RESISTANCE CALCULATION IN ROCK

The following two examples are included to demonstrate that the current LRFD Design Specification equation gives unrealistically low values when designing shafts in poor to fair quality rock. First, equation 10.8.3.5.2c-2 of the LRFD Design specifications to calculate unit tip resistance in IGM is:

$$q_p = 0.59 \left[ N_{60} \left( \frac{p_a}{\sigma'_v} \right) \right]^{0.8} \sigma'_v \quad \text{Eq. 7}$$

Where:

$N_{60}$  = average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

$p_a$  = atmospheric pressure (= 2.12 ksf)

$\sigma'_v$  = vertical effective stress at the tip elevation of the shaft (ksf)

Using the Eq. 7 for an IGM with 50 to 100 blows per foot (bpf), the unit tip resistance values will be on the order of 40 to 60 ksf.

In accordance with Eq. 1, if we have a coarse-grained igneous rock ( $m_i = 32$ ), a  $q_u = 1,000$  ksf and a RMR values of 20 to 45, the range of unit tip resistance values will be on the order of 10 to 55 ksf.

It is obvious that even poor to fair quality rock – a material that can only be excavated with rock coring techniques and is intact enough to provide relatively modest unconfined compression test results- should have higher unit tip resistance value than an IGM that can be drilled and sampled with a standard SPT device. But that is not the case in using the equations in the LRFD Design Specification.

The second example is presented in Figure 2. This figure was created by using Eq. 4 for a range of RMR values for different types of rock. For an RMR value less than 45, the Bearing Capacity Ratio ( $N_{cr}$ ) is less than about 0.06. That is, the recommended unit tip resistance is only about 6% of the unconfined compressive strength of that rock. This is another obviously unrealistically conservative value.

Clearly, an alternative approach is needed to evaluate poor to fair rock. We are suggesting a more reasonable method to calculate unit tip resistance for drilled shafts in rock with an RMR of 45 or less.

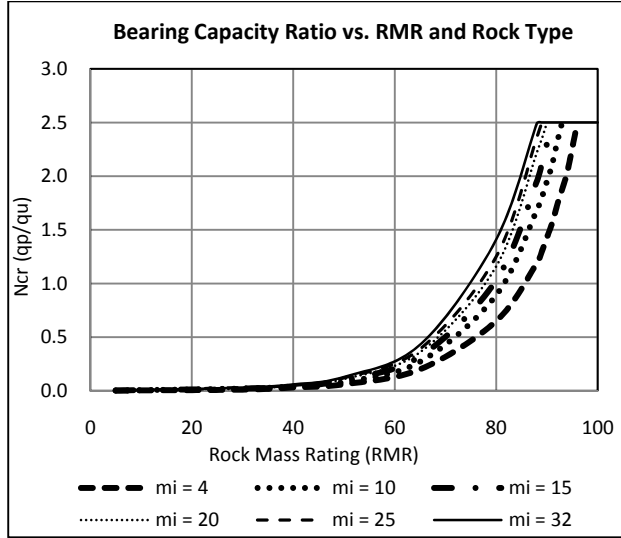


Figure 2. Bearing Capacity Ratio vs. RMR and Rock Type based on Eq. 4 recommended by LRFD.

#### 4 PROPOSED ALTERNATE METHOD TO CALCULATE UNIT TIP RESISTANCE FOR DRILLED SHAFTS

Hoek, Carranza-Torres and Corkum (2002) defined the uniaxial compressive strength of the rock mass ( $\sigma_c$ ) by setting  $\sigma'_3 = 0$  in Eq. 1, giving:

$$\sigma_c = \sigma_{ci} \cdot s^a \quad \text{Eq. 8}$$

They indicated that the failure of a rock mass initiates at the boundary of an excavation when  $\sigma_c$  is exceeded by the stress induced on that boundary. The failure propagates into a biaxial stress field and it eventually stabilizes when the local strength, defined by Eq. 1, is higher than the induced stresses  $\sigma'_1$  and  $\sigma'_3$ . Most numerical models can follow this process of fracture propagation and this level of detailed analysis is very important when considering the stability of excavations in rock and when designing structures such as tunnel support systems.

However, there are times when it is useful to consider the overall behavior of a rock mass rather than the detailed failure propagation process described above, and this is the case of the excavation of a drilled shaft. This leads to the concept of a "global rock mass strength" ( $\sigma'_{cm}$ ) and Hoek and Brown (1997) proposed that this could be estimated from the Mohr-Coulomb relationship:

$$\sigma'_{cm} = q_p = \sigma_{ci} \cdot \frac{(m_b + 4s - a(m_b - 8s)) \left(\frac{m_b}{4} + s\right)^{a-1}}{2(1+a)(2+a)} \quad \text{Eq. 9}$$

where for a undisturbed rock masses:

$$m_b = m_i e^{\left(\frac{GSI-100}{28}\right)} \quad \text{Eq. 10}$$

$$s = e^{\left(\frac{GSI-100}{9}\right)} \quad \text{Eq. 11}$$

$$a = \frac{1}{2} + \frac{1}{6} \left( e^{-GSI/15} - e^{-20/3} \right) \quad \text{Eq. 12}$$

Equations 10, 11 and 12 incorporate the Geological Strength Index (GSI) to calculate the constant parameters of Hoek and Brown instead of RMR.

The calculation of  $m_b$  is also dependent on the constant  $m_i$ . This constant can be obtained in Table 1 where Hoek (2001) determined the value for different rock types.

Table 1. Values of the constant  $m_i$  for intact rock, by rock group (Hoek, 2001). Note that values in parenthesis are estimates.

Rock Type	Class	Group	Texture			
			Coarse	Medium	Fine	Very Fine
SEDIMENTARY	Clastic		Conglomerates (21 ± 3)	Sandstones 17 ± 4	Siltstones 7 ± 2	Claystones 4 ± 2
			Breccias (19 ± 5)		Greywackes (18 ± 3)	Shales (6 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
	Organic				Chalk 7 ± 2	
METAMORPHIC	Non-Foliated		Marble 9 ± 3	Hornfels (19 ± 4)	Quartzites 20 ± 3	
				Metasandstone (19 ± 3)		
	Slightly Foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
	Foliated*		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite (20 ± 5)	Dolerite (16 ± 5)		
			Granodiorite (29 ± 3)			
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
Volcanic	Lava		Rhyolite (25 ± 5)	Dacite (25 ± 3)	Obsidian (19 ± 3)	
			Andesite 25 ± 5	Basalt (25 ± 5)		
	Pyroclastic		Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

\* These values are for intact rock specimens tested normal to bedding or foliation. The Value of  $m_i$  will be significantly different if failure occurs along a weakness plane.

Truzman (2007) after more than 1,200 rock mass characterizations in tunnels excavated in igneous-metamorphic rocks indicated that the GSI and the RMR are practically equal for the GSI greater than 25.

The following shows the classification table of the GSI for igneous-metamorphic rock used to characterize rock masses in different excavation projects.

Table 2. GSI Table for Igneous-metamorphic rocks (Truzman, 2007).

CHARACTERISTIC OF THE ROCK MASS TO ESTIMATE THE GEOLOGICAL STRENGTH INDEX (GSI)		SURFACE CONDITIONS				
From the description of structure and surface conditions of the rock mass, select the appropriate interval of this chart. Estimate the average value of the Geological Strength Index (GSI) from the interval. Do not attempt to be too precise. Guiding a range of GSI from 36 to 42 is more realistic than stating that GSI = 36. It is also important to recognize that the Hoek-Brown criterion should only be applied to rock masses where the size of the individual blocks or pieces is small compared with size of the excavation under consideration. When individual block sizes are more than approximately one quarter of the excavation dimension, failure will generally be structurally controlled and the Hoek-Brown criterion should not be used.		DECREASING SURFACE QUALITY				
STRUCTURE		INCREASING PRESENCE OF FOLIATED ROCKS OR DECREASING INTERLOCKING OF ROCK PIECES				
NON-FOLIATED ROCK MASS	FOLIATED ROCK MASS	VERY GOOD Very rough, closed and fresh unweathered surfaces	GOOD Rough, slightly weathered, aperture < 1 mm, hard fillings	FAIR Fairly rough, moderately weathered, aperture 1-5 mm, hard and soft fillings	POOR Poor, highly weathered, aperture > 5 mm, predominant soft fillings	VERY POOR Slabbed, highly weathered, surfaces, aperture > 5 mm, soft fillings
<b>INTACT OR MASSIVE</b> Intact or massive rocks in situ rock with few discontinuities widely spaced		90			N/A	N/A
<b>NON FOLIATED/FRACTURED</b> rock mass little perturbed consisting of cubical blocks formed by three orthogonal systems of discontinuities, very well interlocked.	<b>LITTLE FOLIATED</b> rock mass partially fractured formed by three discontinuity systems. Massive intervals can have some thin layers of foliated rocks	80				
<b>NON FOLIATED/FRACTURED</b> rock mass partially disturbed consistent of angular blocks interlocked, formed by four or more discontinuity sets	<b>MODERATELY FOLIATED</b> rock mass fractured formed by intervals of foliated or non-foliated rocks in similar proportions	70				
<b>NON FOLIATED/DISTURBED</b> rock mass folded and/or faulted with angular blocks formed by many intersecting discontinuity system	<b>FOLIATED</b> rock mass some foliated and/or faulted very fractured, foliated rocks prevail with occasionally non foliated rock intervals	60		50		
<b>NON FOLIATED/DISTURBED</b> poorly interlocked, heavily broken rock mass with a mixture of angular and rounded rock pieces	<b>VERY FOLIATED</b> rock mass folded, highly fractured, formed only by foliated rocks. Can be present some faults	N/A		40		
<b>NON FOLIATED/DISTURBED</b> rock mass totally fractured and tectonic sheared, resulting in complete lack of blocks	<b>FAULTED/SHEARED</b> rock mass very folded and faulted, tectonically disturbed with respect of breccia.	N/A		30		
		N/A	N/A	20		
		N/A	N/A	10		

Using Eq. 9, we calculated the Bearing Capacity Ratio ( $N_{cr}$ ) of different types of rocks and GSI values. The results of this calculation are depicted in Figure 3.

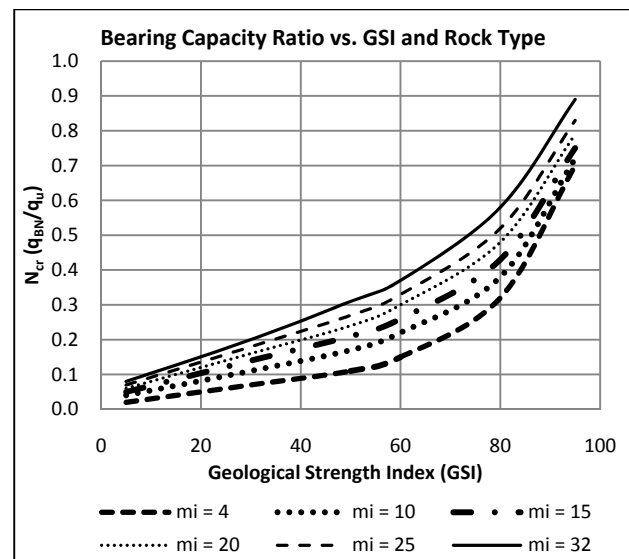


Figure 3. Bearing Capacity Ratio vs. GSI and Rock Type based on the “global rock mass strength”.

Based on the same example used in Section 3 for a coarse-grained igneous rock mass ( $m_i = 32$ ), a  $q_u = 1,000$  ksf and  $RMR = GSI$  values between 20 to 45, and using equation 8 of the “global rock mass strength,” the range of the unit tip resistances will be on the order of 150 to 270 ksf. We consider that these results are more reasonable and reflect actual load test data available for our review. They also are greater than the values obtained for the tip in IGM as shown in Section 3.

This concept of ‘global rock mass strength’ may be used to calculate the nominal unit tip resistance of a drilled shaft in rock based on the LRFD Design Specifications for a range of values of the GSI between 5 to 45. It is important to note, that no practical field tests have been performed by the authors to demonstrate the reliability of this equation at the time of this paper.

However, we compiled some load testing data of drilled shafts in rock performed in the past by others where the proposed “global rock mass strength” equation could be used. We found the following results:

Table 3. Comparison of unit tip resistance measured in the field by others and the method described herein.

Test Performed By and Location	Rock Type	$q_u$ (ksf)	RQD (%)	GSI <sup>1</sup>	$q_p$ measured (ksf)	$q_p^2$ calculated (ksf)
Loadtest (2006) Charlotte NC	Granite	335	15	20 - 25	57 - 159	59
Gunnink & Kiehne (1998) Missouri	Limestone	900 - 1540	78 - 100	60 - 85	190 - 478.8	224 - 433
Axtel, Thompson & Brown (2009) Missouri	Shale	115 - 540	88 - 100	70 - 85	275	60 - 290

<sup>1</sup> GSI calculated based on correlations with RQD

<sup>2</sup>  $q_p$  calculations based on Eq. 9

The results summarized in Table 3 suggest that the proposed equation to calculate the unit tip resistance of the drilled shaft in rock may be used for rocks with GSI values greater than 45 as well.

### 5 NEW CORRELATION BETWEEN GSI AND RQD

One of the most important processes to calculate the unit tip resistance of a drilled shaft in rock is the calculation of the GSI at the tip elevation based on rock cores. Marinos, Marinos and Hoek (2005) mentioned that rock cores are one of the most common sources of information for the estimation of the GSI values of a rock mass.

It has to be recognized that it is necessary to extrapolate the two-dimensional information provided by the core to the three-dimensional in situ rock mass. However, this is a problem common to all rock core investigations, and most experienced engineering

geologists are comfortable with this extrapolation process. Multiple rock cores and inclined rock cores can be of great help in the interpretation of rock mass characteristics at depth.

As another tool to calculate the GSI at depth, we have obtained a correlation from the measurement of the Rock Quality Designation (RQD) of the rock mass of the rock cores. This correlation is based on more than 1,000 surveys in different tunnel excavations and slopes in igneous-metamorphic rocks.

$$GSI = 18.7e^{0.0152RQD} \quad \text{Eq. 13}$$

We recommend that this correlation be used for rock cores of 5 feet or longer. Figure 4 presents the results of the data compiled and previously published by Truzman from 1997 to 2009 in different rock excavation projects. It can be seen that Eq. 13 has a reliability of 70%. From this figure, we also recommend that a range of  $\pm 15$  be applied to the GSI to increase the probability of a more reliable value.

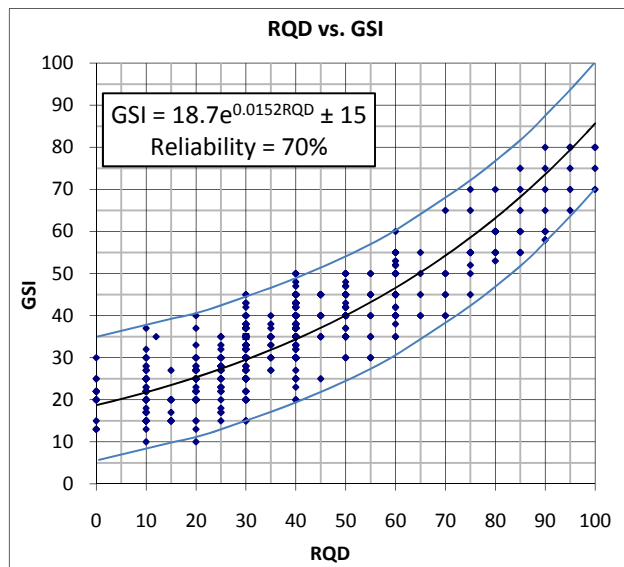


Figure 4. Correlation between GSI and RQD

## 6 CONCLUSIONS

The method proposed in the LRFD defined in section 10.8.3.5.4c gives  $q_p$  values for RMR of 45 or less unrealistically low. A method is recommended to calculate the nominal unit tip resistance ( $q_p$ ) of drilled shafts in rock based on the concept of the GSI and the "global rock mass strength" ( $\sigma'_{cm}$ ).

Based on previous investigations, the GSI and the RMR are practically the same for values greater than 25. This method improves the results of the  $q_p$  for the range of GSI between 5 to 45.

Currently the use of the GSI is not a common engineering practice in the United States to evaluate rock masses for foundations. For the authors, the evaluation of the GSI is easier than the RMR to determinate the  $q_p$  of the drilled shaft.

According to Marinos, Marinos and Hoek (2005) the GSI has considerable potential for use in rock engineering through enhancing geological logic and reducing engineering uncertainty. Another advantage of the index is that it allows adjustments of its rating to cover a wide range of rock masses and conditions but it also allows us to understand the limits of its applications.

We consider that the use of the "global rock mass strength" concept to calculate the nominal unit tip resistance of a drilled shaft in rock is applicable due to the overall behavior of a rock mass in an excavation rather than the detailed failure propagation process of a biaxial stress field.

The authors recommend that load testing of drilled shafts should be performed in different types of rock together with the GSI evaluations and unconfined compressive strength tests to develop comparisons between the field measurements of the unit tip resistance and the method recommended in this paper.

A new correlation between the GSI and RQD is proposed based on an extensive investigation in excavations in different type of rock masses. This correlation may be used as a simple tool to evaluate the GSI in rock cores.

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