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ROCK BEARING RESISTANCE OF BORED PILES SOCKETED INTO ROCK

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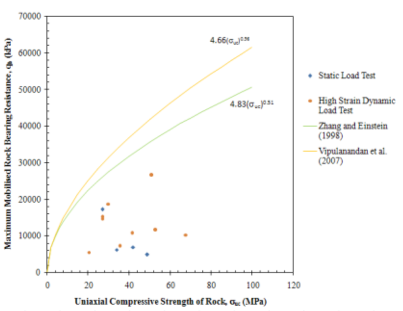
Graphical abstract



Abstract

In view of the large movement required to mobilise the base resistance of bored piles and difficulty in base cleaning, the end bearing resistance often ignored in current design practice that will result in excessive rock socket length. Many attempts have been made to correlate the end bearing resistance with the uniaxial compressive strength of intact rock and the RQD but it is uncertain how applicable they are to rock type in Malaysia. This paper attempts to review the applicability of the formulas from previous studies to rock type in Malaysia. A program of field tests for 13 bored piles with diameter varying from 1000 mm to 1500 mm constructed in granite was conducted to measure the axial response of bored piles, tested using static load test and high strain load dynamic test to verify its integrity and performance. The results were evaluated and compared to the predicted rock bearing resistance. Based on the result obtained, the method by AASHTO gives the best prediction of rock bearing resistance for granite in Malaysia. However the relationship between compressive strength and rock discontinuities with the rock bearing resistance showed scattered results.

Keywords: Rock bearing resistance; uniaxial compressive strength; RQD; granite; bored pile

Abstrak

Memandangkan pergerakan yang besar diperlukan untuk menggerakkan rintangan galas cerucuk terjara dan kesukaran dalam pembersihan dasar, rintangan galas hujung kerap diabaikan dalam amalan rekabentuk asas semasa, yang mengakibatkatkan panjang soket batuan berlebihan berlaku. Banyak percubaan dibuat untuk mengaitkan rintangan galas hujung dengan kekuatan mampatan ekapaksi batuan utuh dan RQD tetapi tidak pasti bagaimana aplikasi kaedah tersebut boleh digunapakai kepada jenis batuan di Malaysia. Kertas kerja ini cuba untuk menyemak semula akan kesesuaian rumus dari kajian terdahulu untuk jenis batuan di Malaysia. Program ujian lapangan telah dijalankan bagi 13 cerucuk terjara dengan diameter yang berbeza dari 1000 mm sehingga 1500 mm yang terletak dalam batuan granit untuk mengukur tindak balas paksi cerucuk terjara yang diuji menggunakan ujian beban statik dan ujian beban terikan tinggi dinamik untuk mengesahkan integriti dan prestasi cerucuk. Keputusan telah dinilai dan dibanding dengan rintangan galas batuan. Berdasarkan keputusan yang diperolehi, kaedah AASHTO memberi ramalan rintangan galas batuan yang terbaik untuk granit di Malaysia. Walau bagaimanapun hubungan antara kekuatan mampatan dan ketidakselanjaran batuan dengan rintangan galas batuan adalah berselerak.

Kata kunci: Rintangan galas batuan; kekuatan mampatan ekapaksi; RQD; granit; cerucuk terjara

1.0 INTRODUCTION

The Klang Valley Mass Rapid Transit (also known as KVMRT) project is a planned for three lines mass rapid transit system to ease the severe traffic congestion in Kuala Lumpur. The proposal was announced in June 2010 and approved by the Government of Malaysia in December 2010, together with the existing light rail transit (LRT), monorail, KTM Komuter, KLIA Ekspres and KLIA Transit systems, will increase the current inadequate rail network and able to serve a corridor with an estimated population of 1.2 million people. The first phase of this project involves the construction of 51 km rail alignment from Sungai Buloh to Kajang with underground tunnel of 9.5 km and 31 stations of which seven will be underground.

The construction of the Sungai Buloh to Kajang line involves the construction of thousands of large diameter bored piles ranging from 1.0 m to 2.8 m diameter to support the structures of viaducts and train stations that will be founded on a wide range of rock types comprising granite, Kenny Hill, limestone and Kajang formations.

Bored piles are commonly used in Malaysia as foundation to support heavily loaded structures such as high-rise buildings and bridges, considering its ability to carry very high load, minimum noise and vibration, and the ability to be constructed in tight tolerance in unstable and difficult soil condition. Such attributes are especially favoured in urban areas where strict restrictions with regards to noise and vibration are imposed by relevant authorities which restricted the use of other conventional piling system, for example the driven piles. Although there are significant advantages in designing the end bearing resistance component, the end bearing resistance is often ignored in current design practice [1-3]. According to Crapps and Schmertmann [2], the most common reasons cited by designers for neglecting end-bearing resistance in design include settled slurry suspension, reluctance to inspect the bottom, concern for underlying cavities, and unknown or uncertain end-bearing resistance. Obviously, neglecting the end-bearing resistance in design will result in excessive rock socket lengths. Due to the high cost of shaft construction in rock, an overdesign of socket length will lead to increased cost. It was suggested that to account for end bearing resistance in design and using appropriate construction and inspection techniques to ensure quality base conditions is a better approach than neglecting end-bearing resistance [2].

In Malaysia, bored pile design in rocks is heavily based on semi-empirical method. Generally, the design of rock socket friction is the function of surface roughness of rock socket, unconfined compressive strength of intact rock, confining stiffness around the socket in relation to fractures of rock mass and socket diameter, and the geometry ratio of socket length-to-diameter. The roughness between pile and rock in rock socketed pile will influence its bearing mechanism significantly. It is an important factor in rock socket pile design due to its significant effect on the normal contact stress at the socket interface during shearing. The level of dilation resulting from the increasing of socket friction due to increasing of normal contact stress is mostly governed by the socket roughness. The intact rock strength governs the ability of the irregular asperity of the socket interface transferring the shear force, otherwise shearing through the irregular asperity will occur due to highly concentrated shear forces from the socket.

The rock quality designation, RQD, was initially proposed by Deere *et al*. [4] as an index of assessing rock quality quantitatively describing whether a rock mass provided favourable tunneling conditions. RQD has since then been the topic of various assessments, mainly for civil engineering projects. Its application has been used for over 20 years and an index of rock quality also been quickly extended to other areas of rock mechanics, and it has become a fundamental parameter in geotechnical engineering [4-9]. The success of the RQD is due, in large part, to its simple definition, which is a modified core recovery percentage in which all the pieces of sound core over 100 mm (4 in.) long are summed up and divided by the length of the core run.

RQD generally are among the earliest data obtained from a site study recorded on the logs of the exploratory borings. The percentage of core recovery, the RQD measurements, and the geologic descriptions of the cores are determined at the drilling site by the field engineering geologist within minutes of recovery of the cores. RQD value shall be read in conjunction with geologic mapping to provide early project information on the distribution of rock types, degree and depth of rock weathering and zones of rock weakness and fracturing. This information may be used by the engineers in evaluating the required depths of excavation for founding the structures and of any potential problems of bearing capacity, settlement, or sliding.

The end bearing resistance is often ignored in current design practice in Malaysia due to difficulty in obtaining references and consistent base cleaning during the construction of bored piles. Many attempts have been made to correlate the end bearing resistance with the unconfined compressive strength of intact rock and RQD. A few methods have been proposed for predicting the end bearing resistance of bored piles. Of these different methods, empirical and semi-empirical relations have been most widely used. The method used in previous studies correlates the maximum rock bearing resistance with respect to rock compressive strength and the RQD. The strength and RQD are obtained from laboratory test results conducted on the intact rock core samples. However, there is still no such study conducted in Malaysia. Also, it is still uncertain on how applicable these methods to various rock types, specifically in Malaysia.

Hence, this paper presents the results of the study carried out with the objectives to review the available design relationship addressing bearing resistance of piles socketed to rock, to validate the established empirical designs relationship with respect to field pile load test results, and to identify the trends in the behaviour of rock discontinuities and unconfined compressive strength with respect to maximum rock bearing resistance. The significance of this study is to ensure that the correlations obtained by previous works, are adopted in the design of end bearing resistances, are satisfactory and can be implemented in Malaysia. This study provides better understanding on the trends of rock discontinuities particularly of RQD and rock compressive strength with respect to maximum rock bearing resistance. The scope of this study focusses on the prediction of end bearing resistance rather than socket shaft resistance.

2.0 METHODOLOGY

The data for this study was acquired from MMC-Gamuda KVMRT (PDP) Sdn. Bhd. These includes the Soil Investigation Reports, bored piling records and pile load testing results. In total, 13 pile testing results which consists of 4 numbers using static load tests and 9 numbers using dynamic load tests were reviewed and evaluated. Only the rock bearing resistance of bored piles with diameter varying from 1000 mm to 1500 mm which were constructed in granite formation has been considered in this study. All the bored piles were socketed from 1 m up to 3.8 m into the rock and tested with pile load testing.

The data was compiled, summarised, processed and transferred into tables and graphs. The soil data of the ground stratifications, types of rock, RQD, rock strength test results and total core recovery (TCR) were obtained from the soil investigation works carried out at bored pile locations or within the footprint of pile cap. The as-built rock socket lengths were reviewed based on the bored piling records of varied pile diameters. The corresponding pile lengths and rock socket lengths were assessed based on the surveyed level of the pile cut-off level, pile toe level and rock reduced level. The mobilized rock bearing resistances were assessed from the load testing results either by static load test or high strain dynamic load test. The information from the Pile Load Testing Reports are used for comparison with the predicted rock bearing resistances.

Analysis of data was carried out by predicting the rock bearing resistance based on formula from previous study proposed by Zhang and Einstein [10-16]. The predicted rock bearing resistances were then compared with the mobilised rock bearing resistances, assessed from load testing results, to validate the establish empirical design relationship with respect to field pile load test result. An additional plot of RQD and uniaxial compressive strength of rock, mobilised rock bearing resistance and pile settlement, maximum mobilised rock bearing resistance and RQD and maximum mobilised rock bearing resistance and unconfined compressive strength of rock were plotted to determine if there is any trend behaviour between rock discontinuities and unconfined compressive strength with respect to maximum rock bearing resistance. Conclusion is made based on the results of analysis. Some recommendations are proposed for future development to further validate and enhancement of this study.

3.0 RESULTS AND DISCUSSION

The summary of tested bored pile is as shown in Table 1. The length of tested pile ranges from 6.6 m to 30.1 m with rock socket length varying from 1.0 m to 3.8 m. The settlement at the mobilised rock shaft resistance is also tabulated to assess the mobilisation load. From Table 1, it can be concluded that the maximum mobilised rock bearing resistance is between 4875 kPa to 26526 kPa. The lowest rock bearing resistance was encountered at piles V6/PTP1. This might be due to this pile was installed with the shortest rock socket length of 1.0 m while the TCR is 100 % and RQD is 32 %. The same range of TCR and RQD reading was recorded at V6/P138a-1 with slightly higher maximum mobilised rock bearing resistance. From Table 1, the highest rock bearing resistance was encountered at pile V6/P70-12 with TCR of 100 % and RQD of 81%. Pile V6/P138a-1 exhibits the lowest RQD and unconfined compressive strength of 28 % and 21 kPa, respectively resulting with the value of maximum mobilised rock bearing capacity of 5438 kPa. This pile has the longest rock socket length of 3.8 m.

The mobilised rock bearing resistances obtained from the load test result were incorporated in Figure 1 and Figure 2. From Figure 1, it is observed that all the rock bearing resistances lie below the value predicted using Zhang and Einstein method [10-16] and from Figure 2, all of the rock bearing resistances lie below Zhang method [12].

The plot of maximum rock bearing resistances, $q_{b(max)}$ versus the pile top settlement is shown in Figure 3. It is observed that the rock bearing resistance started to be mobilised after 5 mm movement encountered at the highest $q_{b(max)}$ of 26526 kPa. Pile V3/PTP-2 mobilised about 67 mm to achieve the maximum rock bearing resistance of 17234 kPa. From Figure 3, four numbers of piles had moved more than 25 mm to achieve the maximum rock bearing resistance from 4875 kPa to 17234 kPa. One should note that these values may need to be evaluated in terms of the permissibility of pile movement to ensure the satisfactorily of the design.

Table 1 Database of bored pile load testing under review

Pile Ref.	Diameter (mm)	As-Built Pile Length (m)		TCR at base (%)	RQD at base (%)	σ_{uc} (MPa)	Pile Testing	$q_{b(max)}$ (kPa)	Settlement (mm)
		Total	Rock						
V2-PTP2	1200	28.5	1.5	100	66	34	SLT	6169	32
V3-PTP2	1350	30.1	1.8	100	46	27	SLT	17234	67
V6-PTP1	1000	22.0	1.0	100	32	49	SLT	4875	43
V6-PTP2	1200	27.7	1.2	100	71	42	SLT	6873	28
V6/P19-4	1200	7.5	3.4	100	56	27	HSDLT	14589	9
V6/P20-1	1200	13.2	2.5	100	56	27	HSDLT	15031	18
V6/P23-1	1200	9.0	3.0	100	82	36	HSDLT	7074	5
V6/P31-3	1200	11.3	3.4	100	83	42	HSDLT	10610	10
V6/P41-3	1200	26.5	2.2	100	82	53	HSDLT	11495	21
V6/P62-4	1500	18.8	3.5	100	76	68	HSDLT	10186	11
V6/P70-12	1200	6.6	2.8	100	81	51	HSDLT	26526	5
V6/P138a-1	1200	16.0	3.8	100	28	21	HSDLT	5438	12
V6/P140a-2	1200	12.0	3.2	100	49	30	HSDLT	18656	6

Denotation:
TCR – Total core recovery
RQD – Rock quality designation (within the rock socketed length of pile)
SLT – Static load test
HSDLT – High strain dynamic load test
σ_{uc} – Unconfined compressive strength of rock
$q_{b(max)}$ – Maximum mobilised rock bearing resistance

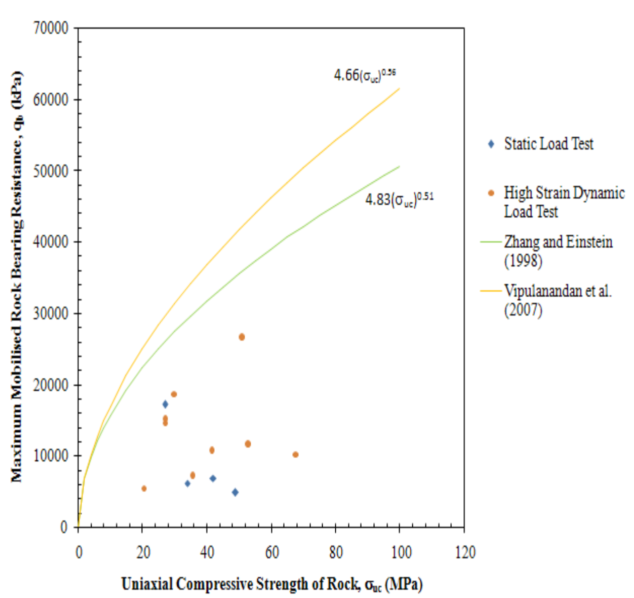


Figure 1 Correlation between maximum mobilised rock bearing resistance with uniaxial compressive strength

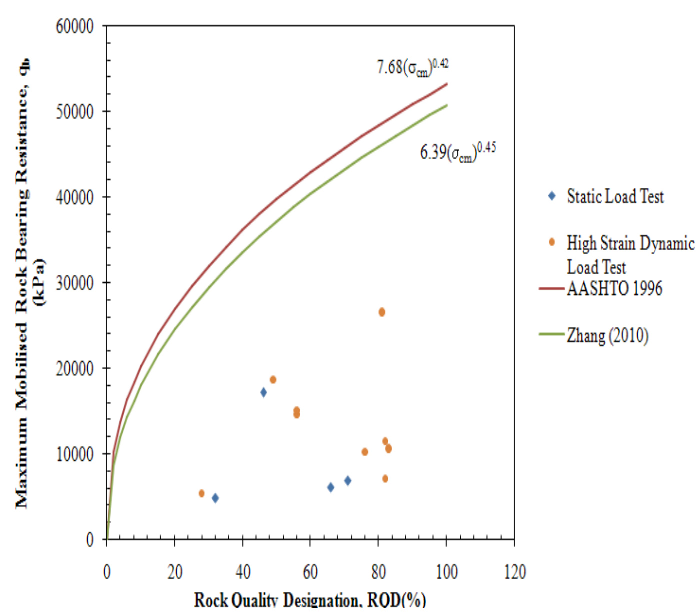


Figure 2 Correlation between maximum mobilised rock bearing resistance with rock quality designation

The predicted rock bearing resistance computed from various methods were compared with the maximum measured rock bearing resistances from load test results and shown in Figure 4 and Figure 5. Corresponding lines of 1:1 that represent perfect prediction (i.e., Factor of Safety, FS=1), -50% that represent non conservative (i.e., FS=0.5) and +50% which represent conservative (FS=1.5) were incorporated to evaluate which methods gave the best prediction of rock bearing resistance for granitic rock.

Based on the plotted Figure 4 to Figure 5, comparison between predicted rock bearing resistances from interpretation of load test results indicates that 70% of the measured values of $q_{b(max)}$ were under-predicted based on the method by Zhang and Einstein [16] and Vipulanandan *et al.* [10]. The percentage of non-conservative prediction of rock bearing resistance of 70% of total tested piles lie below the -50% line. However, the method used by AASHTO from Figure 5 indicates a good agreement of $q_{b(max)}$ compared to the measured rock bearing resistances. This is shown by at least 30% of total tested piles lie above the 1:1 line and 54% of the total test piles lie below the -50% line.

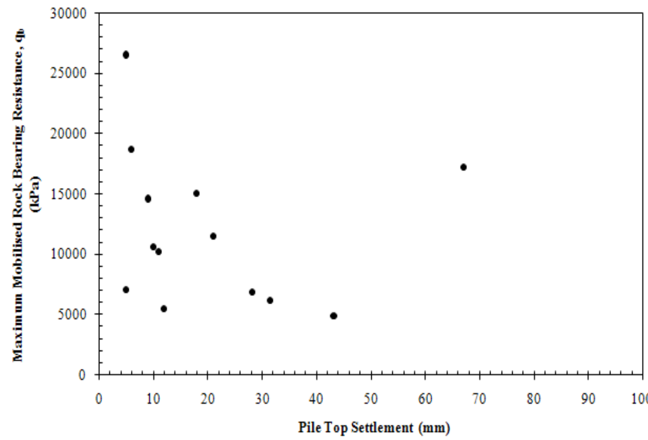
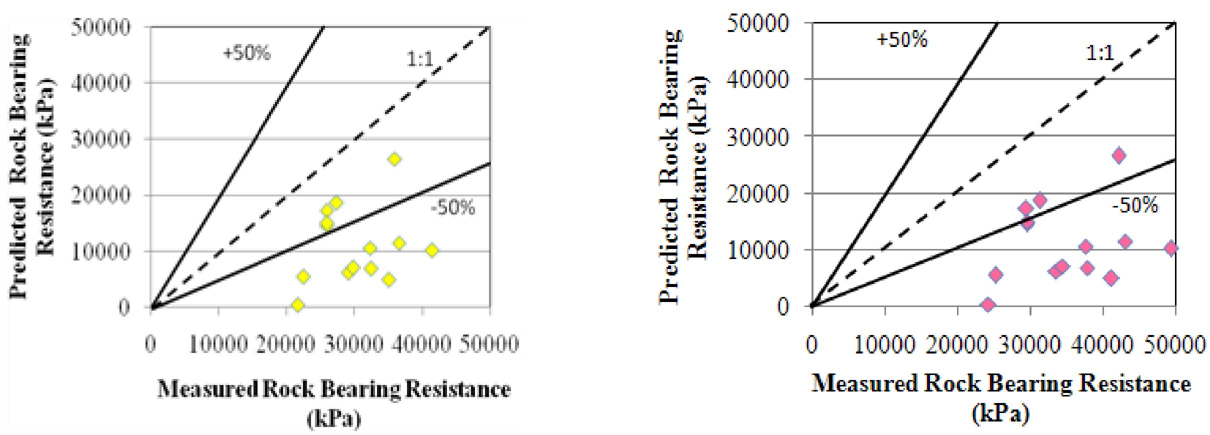


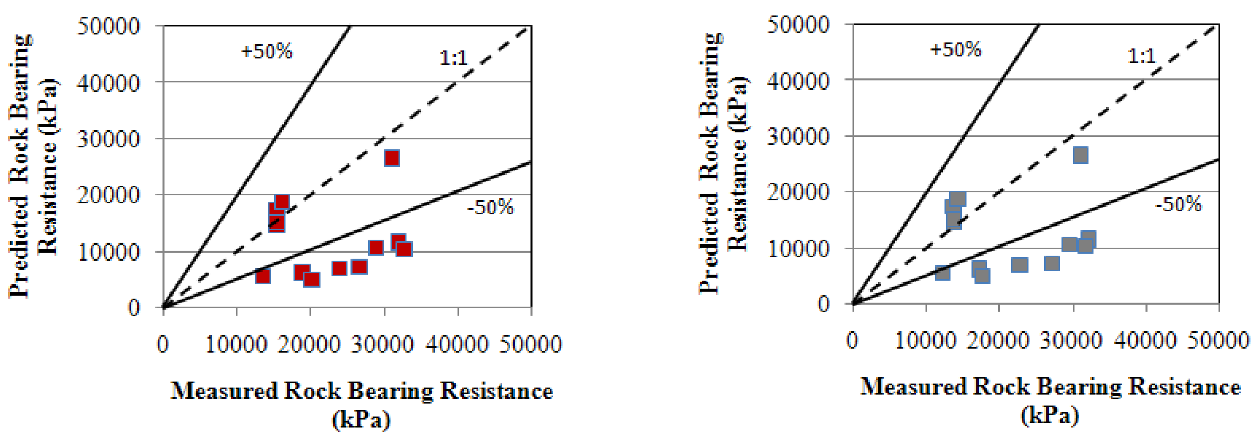
Figure 3 Maximum mobilised rock bearing resistance versus pile top settlement



(a) Zhang and Einstein [16], $q_{b(max)}=4.83(\sigma_{uc})^{0.51}$

(b) Vipulanandan *et al.* [10], $q_{b(max)}=4.66(\sigma_{uc})^{0.56}$

Figure 4 Comparison of rock bearing resistances considering the uniaxial compressive strength



(a) Zhang [12], $q_{b(max)}=7.68(\sigma_{cm})^{0.42}$

(b) AASHTO [11], $q_{b(max)}=7.68(\sigma_{cm})^{0.42}$

Figure 5 Comparison of rock bearing resistances considering the rock quality designation

To identify whether any consistent trend is encountered between rock strength and maximum rock bearing resistance, the data of uniaxial compressive strength is plotted against the $q_{b(max)}$ as shown in Figure 6. It is observed that there is no clear trend on the distribution of the uniaxial compressive strength and rock bearing resistance and it is generally scattered. Eight piles indicate maximum rock bearing resistance of higher than 5438 kPa with uniaxial compressive strength value varied between 20 MPa to 40 MPa. In general, it is observed that the rock bearing resistances started to mobilise with uniaxial compressive strength value of more than 21 MPa.

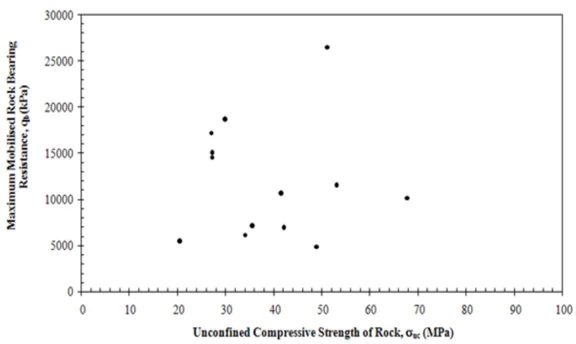


Figure 6 Maximum rock bearing resistance versus unconfined compressive strength

The data of rock quality designation is also plotted against the $q_{b(max)}$ to study whether there is any consistent trend encountered. As shown in Figure 7, it can be seen that there is no clear or consistent trend between rock quality designation and maximum mobilised rock bearing resistances. One pile indicated a maximum rock bearing resistance of 17234 kPa with 46 % RQD value. At the same location, the maximum rock bearing resistance is 7074 kPa even with high RQD value of 82 %. In general, it is observed that the rock bearing resistances started to mobilise with RQD value of more than 28 %. However, even though there is no significance variation of the maximum rock bearing resistance by the rock discontinuities, this may not be excluded in the consideration during the rock socket design as other factors such as confinement stiffness may also affected by the rock discontinuities criteria.

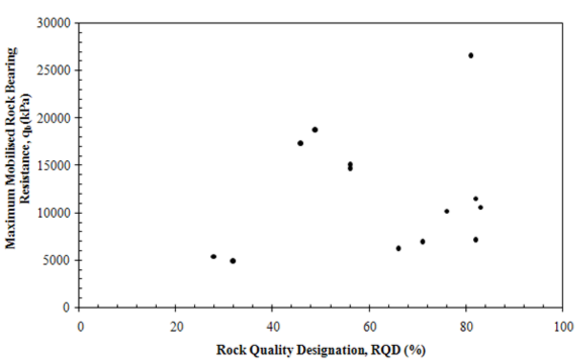


Figure 7 Maximum rock bearing resistance versus rock quality designation

There is also no specific trend of rock discontinuities with uniaxial compressive strength of rock as shown in Figure 8. The highest value of uniaxial compressive strength is observed with RQD value of 76 %. However at some location that exhibited high RQD value of 82 %, the unconfined compressive strength value was only 53 MPa. In general, the increased in rock compressive strength seemed to develop proportionally with the increased of RQD.

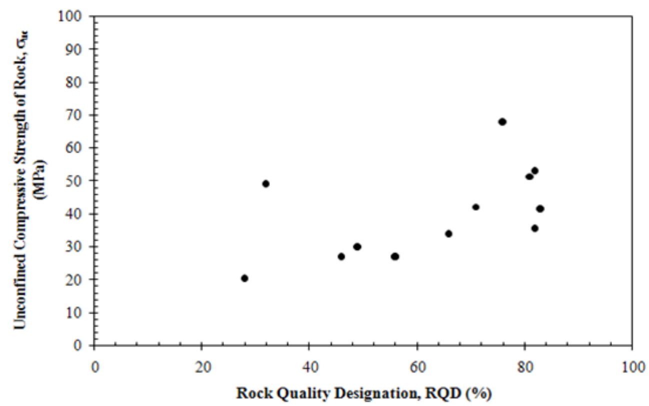


Figure 8 Unconfined compressive strength of rock versus rock quality designation

4.0 CONCLUSION

Based on the results obtained, the following conclusions are drawn:

(a) The available test field data on rock socketed bored piles indicated lower bearing resistances than predicted by all methods considering unconfined compressive strength. However, the study shows that the maximum bearing capacity is best predicted by using AASHTO method that considers rock quality designation. This is shown by at least 30 % of total tested piles lie above the 1:1 line and 54 % of the total test piles lie below the -50 % line.

(b) There is no consistent trend observed in the unconfined compressive strength and rock discontinuities with respect to maximum rock bearing resistance.

(c) Further review on the 13 nos of pile testing results does not reveal any discrimination of rock strength along the depth or alignment nor any clear trend with the rock quality designation.

(d) No strong correlation can be made between the socket length, diameter and the pile movement. However, in general the rock bearing resistance were observed to mobilise more than 4800 kPa with pile displacement ranging from 5 mm to 67 mm.

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