ESTIMATION OF ULTIMATE PULLOUT RESISTANCE OF ROCK SOCKETED MICROPILES

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ABSTRACT: Traditionally rock socketed bored cast in situ piles are more popular in India to support heavy foundations. However, focusing on the end bearing resistance, usually the skin friction component is ignored due to many uncertainties in the installation procedures and quality control issues. It is observed that in India, many project facilitates; the increase in the concrete weight is adopted as hydrostatic uplift resisting systems. Present study explores the use of rock socketed micro-piles for uplift resistance. The recommendation from Indian national codes and other international standards are reviewed and compared in the present study. Few laboratory tests are also conducted to investigate the ultimate skin friction between the concrete and rock surface thereby estimating ultimate pullout resistance of the micropile. Results of the laboratory testing are compared at the end of paper with recommendations made by various codes and standards

Keywords: Micropiles, pullout resistance, rock socketed, hydrostatic uplift, power projects

INTRODUCTION

Bored cast in-situ piles socketed in rocks are amongst widely used deep foundations in recent years. These piles offer best foundation alternative for heavy loads, and for sub surface conditions such as layer of loose / soft soil overlies bedrock, and under high hydrostatic uplift pressure. Till recent time, it was usual to adopt allowable bearing pressure of 3.0MPa for sound rocks like basalt, and 2.5MPa for weaker rocks like volcanic Braccia and Tuff. During installation, criterion based on chiseling energy (Datye, 1990) is being practiced for pile termination in weathered rocks. These practices appear to be very conservative as they neglect, or assume very low values of the side resistance between pile and rock socket interface (Basarkar, 2004).

Availability of advance information on load displacement behavior of socketed piles would greatly enhance the decision making process in site and increase the confidence in the adopted techniques. In fact, closed form solutions are available (Guo and Randolph, 1998; Kodikara and Johnson, 1994; Randolph and Wroth, 1978) that give the load displacement response of piles. However, these have not been frequently applied to the field situations, since they rely on the parameters and properties that are either difficult to determine or are not a part of the routine geotechnical investigations. Load – displacement analysis using elastic theories are discussed by Mattes and Poulos, 1969; Pells and Turner, 1979. These approaches are best suited at lower values of working loads, where the behavior of the rock sockets are expected to be elastic. In contrast, literature studies and field data have indicated that, load – displacement behavior of rock sockets are non-linear, and hence it is unlikely for the elastic theories to yield a good match, particularly at higher range of loads. Empirical / semi – empirical methods exist that give load – deformation behavior of piles that specify the unit shear and unit base resistance values of rock sockets (Vijayvergiya, 1977; Zhang and Einstein (1998).

UPLIFT RESISTANCE OF MICROPILE

The mechanism of sidewall shear development in rock is very complex, and includes both adhesion and friction (Horvath et al., 1980). The relative contribution depends on the geometry of the pile and socket, and on the rigidities of the pile with reference to the surrounding. According to Kenny (1977), the increase in shear strength at the interface is due to the tendency of the pile to expand the socket laterally:

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first when it is loaded elastically, due to dilatancy effect. Both of these effects tend to mobilize the frictional component intercept by a value depending upon the overall roughness of the pile and the mobilized radial pressure.

ULTIMATE RESISTANCE BASED ON ROCK SOCKETED PILE ANALOGY

In an attempt to predict the ultimate pullout (side resistance) capacity of a micropile in rock, various researchers have proposed correlations based on the rock socketed piles to estimate socket friction. One common approach is to correlate the unconfined compressive strength of rock cores to the ultimate side shear. Table 1 summarizes few popular correlations which are frequently used for estimation of socket friction. Based on load test data and available information, the co-efficient of unit side resistance of socketed piles are compared in the Figure 1.

Benmokrane et al (1994) carried out laboratory investigations of shaft resistance of rock socketed piers using constant normal stiffness direct shear tests. Their study indicates that the side shear resistance at failure of rock socketed piles depends not only on the compressive strength of the weakest material of the pile or the surrounding rock, but also on the relative stiffness and strength of the two material, pile geometry and socket roughness. They also concluded that apart from the strength of the supporting rock, its deformability characteristics also played vital role in shear resistance.

Table 1. Coefficient of unit side resistance of socketed piles

Sr. No.	Reference	Empirical Correlations
1	IRC 78	$\alpha = 0.225(q_u)^{-0.5}$
2	Rowe and Armitage (1987)	$\alpha = 0.45(q_u)^{-0.5}$
3	Horvath et al. (1980)	$\alpha = 0.25(q_u)^{-0.5}$
4	Rosenberg and Journeaux (1976)	$\alpha = 0.375(q_u)^{-0.5}$
5	Zang and Einstein (1998) - smooth sockets	$\alpha = 0.4(q_u)^{-0.5}$
6	Zang and Einstein (1998) - rough sockets	$\alpha = 0.8(q_u)^{-0.5}$
7	Horvath and Kenney (1979)	$\alpha = 0.67(q_u)^{-0.5}$
8	Carter and Kulhawy (1988)	$\alpha = 0.63(q_u)^{-0.5}$

Note : q_u= unconfined compressive strength of rock cores.

Basarkar (2004) concluded that from the field load test the value of unit side shear parameter is found to be vary between 0.345 to 0.67. These values confirm the range, 0.4 to 0.8 as recommended by Zang and Einstein (1998).

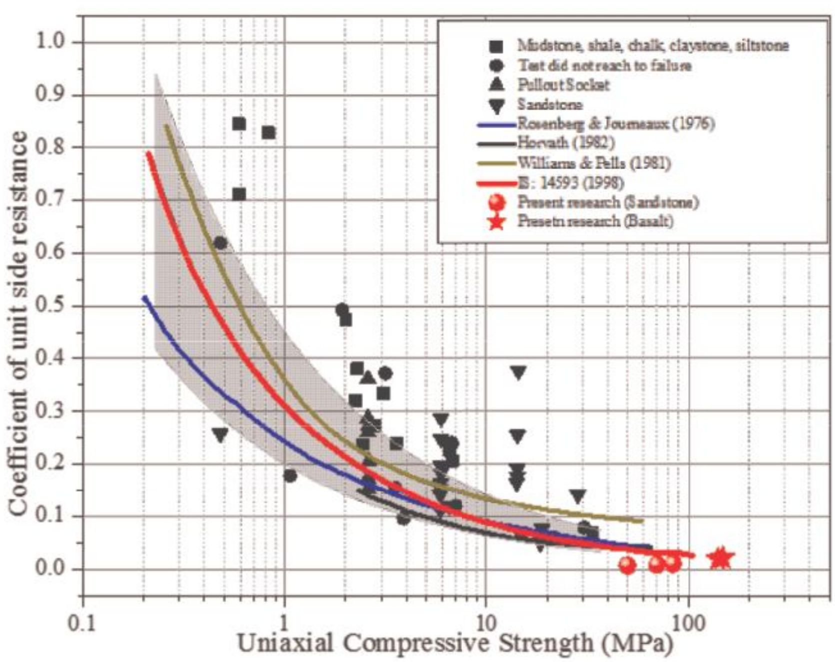

Figure 1. Coefficient of unit side resistance of socketed piles.

INDIAN RECOMMENDATIONS

IS: 14593 (Indian Standard code) suggest unit shear resistance considering uniaxial compressive strength of rock to be modify by rock socket side resistance factor α, and rock socket correction factor β which depends on mass reduction factor. Figure 1 describes the variation of rock socket side resistance factor with respect to uniaxial compressive strength of rocks recommended by IS: 14593. The recommended co-efficient of unit side resistance of rock socketed pile as per IRC: 78 is presented in Table 1 for comparison. However it is important to note that IRC: 78 limits the maximum allowable side resistance to 5 Mpa.

ULTIMATE RESISTANCE BASED ON ROCK ANCHOR ANALOGY

Rock anchors have been used successfully in the past for wider range of applications in the retaining structures and rehabilitations. Type A micropile (FHWA NHI-05-039) is similar to bored piles, while, grouted micro pile i.e. Type D can be considered similar to grouted rock anchors. In any of the case, drilling of micropile differs from the construction of the bored pile and matches more with drilling for rock anchors. Considering the fact that present study is focused toward the estimation pull-out resistance, the resistance largely governed by the interface shear strength between rock and grout (sometimes referred as ultimate bond strength between rock and grout). For design purpose, various values of bond strengths are proposed. Table 2 summarizes few recommendations popularly adopted in design offices.

Whilst the estimation of the pullout capacity of rock anchor are usually estimated based on crude cone or wedge mechanisms whereby the system is equated to the weight of a specified rock cone (Figure 2), the capacity of micropile is estimated using the circumferential interface friction along the rock-micropile contact similar to pile analogy. However for shallow grouted micropiles (i.e. of 3 to 5m length), the rock cone mechanism is observed to be conservative and widely practice for design. In case of rock cone mechanism also, while the shape of the rock cone is widely agreed, its position with respect to the socket varies (Figure 2) i.e. IS: 10270 specifies 90^0 as included angle of apex cone whereas BS: 808, AS 4678 recommends to use rock strength as deciding criteria (Figure 2).

Table 2. Summary of typical interface strength between grout and rock

Type of rock	Average Ultimate interface strength (MPa)		
	PTI (2004)*	IS : 10270*	FHWA-NHI-05-039**
Granite and Basalt	1.7-3.1	0.5-0.7	1.38-4.2
Khaondolite / Charnokite		0.3-0.5	
Dolomite Limestone	1.4-2.1		
Soft Lime Stone	1-1.4		1.035-2.07
Slate and Hard Shales	0.8-1.4		0.515-1.38
Soft Shales	0.2-0.8	0.3	0.205-0.55
Sandstones	0.8-1.7	0.3	0.52-1.7
Quartzite		0.3	
Weathered Sandstones	0.7-0.8	0.25	
Chalk	0.2-1.1		
Jointed Quartzite		0.35	
Grey Chioritic Schist		0.35	
Weathered Marl	0.15-0.25		
Concrete	1.4-2.8		

* For rock anchors;

** for Type A micropile; see Annexure –A for further information

With regard to uplift capacity of micropile, no experimental or practical evidence substantiates the method currently used to calculate the ultimate resistance of pullout of individual micropile. However, it is reassuring to note that most designs are likely to be conservative in adopting a cone method in which no allowance for the shear strength of the rock mass has been made.

EXPERIMENTAL SETUP

Piles are casted in the rock chunk after drilling hole of 52 mm of specified socket length. The piles are casted using M30 (compressive strength 30 N/mm2) concrete up to the depth of 2D to 3D (D= dia of rock core) from the top. The bottom portion of the drilled hole is filled with soft material or kept open to relieve any end bearing mobilization.

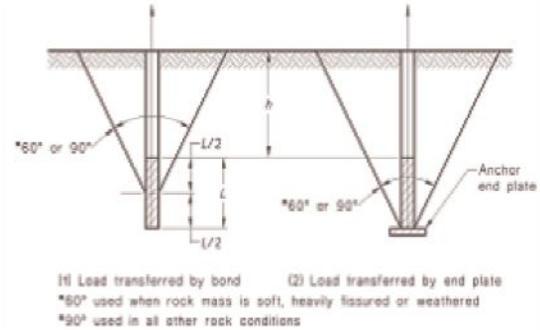

Figure 2. Geometry of cone for uplift capacity estimation (after Littlejohn and Bruce, 1977)

After sufficient curing of rock socketed pile model, it is placed under the load frame of 25 ton capacity. The experimental setup is schematically illustrated in Figure 3.

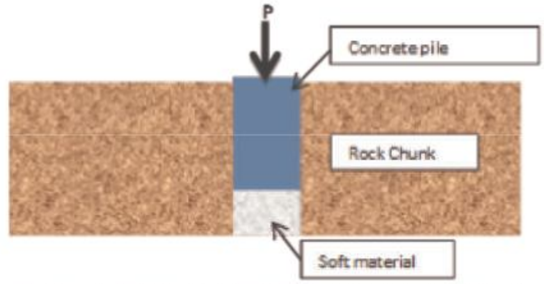

Figure 3. Schematic laboratory experiment setup to calculate maximum sock friction capacity

Table 3. Estimated and observed coefficient of uniform side resistance for rock specimens

	Experimental Observations				
	1	2	3	4	5
Type of rock	Basalt	Basalt	Sandstone	Sandstone	Sandstone
Description	Fresh dark greyish black colour fine grained porphyritic Basalt		Fresh pinkish red colour ferrogenous Sandstone with fine to medium grained cementing material		
UCS of rock (MPa)	140	148	84	50	70
Dia of Socket (mm)	52	52	52	52	52
Length of Socket (mm)	156	130	104	130	130
Observed Ultimate load (kN)	68.88	67.5	15	7.5	13
Observed Ultimate unit side resistance (MPa)	2.703	3.178	0.883	0.353	0.612
Coefficient of unit side resistance	0.019	0.021	0.011	0.007	0.009
Specimen Failed (Yes/No)	yes	yes	yes	yes	Yes
Estimation of coefficient of unit side resistance using bored pile analogy					
IS: 14593	0.01	0.01	0.04	0.08	0.05
IRC 78	0.019	0.018	0.025	0.032	0.027
Rowe and Armitage (1987)	0.038	0.037	0.049	0.064	0.054
Horvath et al. (1980)	0.021	0.021	0.027	0.035	0.030
Rosenberg and Journeaux (1976)	0.032	0.031	0.041	0.053	0.045
Zang and Einstein (1998) - smooth sockets	0.034	0.033	0.044	0.057	0.048
Zang and Einstein (1998) - rough sockets	0.068	0.066	0.087	0.113	0.096
Horvath and Kenney (1979)	0.057	0.055	0.073	0.095	0.080
Carter and Kulhawy (1988)	0.053	0.052	0.069	0.089	0.075
Estimation of coefficient of unit side resistance using rock anchor analogy					
PTI (2004)	0.017	0.016	0.015	0.025	0.018
IS : 10270	0.004	0.004	0.004	0.006	0.004
FHWA-NHI -05-039	0.020	0.019	0.013	0.022	0.016

The load was applied using screw jack and was measured using 20 ton proving ring. The load was applied gradually in increment of 2.5 tons and corresponding settlements were measured. The next incremental load was applied after the settlement ceased under the applied load. The load increment and corresponding settlement observation continue till the model pile fails by loss of friction. Based on observed failure load, coefficient of unit side resistance is computed. The coefficient of unit side shear resistance as obtained experimentally is shown in Figure 1. Estimated values from the various approaches are compared with the observed values and present in Table 3.

SUMMARY OF OBSERVATIONS AND RECOMMENDATIONS

There are no elaborative procedures for designing micropile socketed in rock. Their capacities are estimated either on the basis of structural design guidelines or based on the conventional bored pile analogy. Present research compares the two different analogy for estimating ultimate pullout resistance of the micropile socketed in rock. It is observed that very few historical data available for the rocks having high uniaxial strength i.e. more than 60 MPa. It is observed that for the two Basalt samples, (except IRC 78; Horvath et al., 1980), the prediction based on the bored pile analogy overestimate the co efficient of unit side friction and on the other hand, the rock anchor based estimation results on the conservative sides even for other samples also. Based on the experimental results, it is observed that for higher rock strength class (i.e. Basalt), the estimation from rock anchor analogy closely predicts the failure load. However, for lower strength class (i.e. Sandstone), recommendations may over predict the failure load. Similarly, IRC 78, Horvath et al. (1980) closely predict the ultimate failure load for higher strength class rocks and over predict the failure load for lower strength class rocks. Comparatively, the ultimate failure load based on the bored pile analogy predicts the higher failure load compared to the load estimated from rock anchor analogy. This may be due to the difference in the method of installations.

Mayo et al. (2003) conducted pullout load test on Gneiss having compressive strength ranging from 76 MPa. The pull test was carried out on 9 m long, 75 mm dia anchor and ultimate socket friction around 0.4 MPa was reported. The back calculated coefficient of unit side resistance is in the range of 0.005 however, the test did not reach to failure. Singh and Chopra (1986) conducted field testing in moderately weathered soft mica chloride schist and phylllites with 65 mm grouted rock anchors. Working side friction was observed to be more than 0.55 MPa which are in agreement with the the estimation of unit side resistance based on anchor analogy. However, it is important to note that usually in design, ultimate pullout estimation using the wedge cone analogy (Figure 2) results very conservative estimate. Robertson (through Littlejohn and Bruce, 1977) tested 12 m to 15 m long, 100 mm dia grouted anchors. in jointed and bedded qurtzitic sandstones and observed failure between grout and rock bond. For shallow depth fixed anchors, the ultimate side friction is reported to be 0.5 MPa. Based on these discussions following recommendations can be made for estimation of ultimate pullout resistance of micro piles.

1. Cone analogy may not predict correct pullout capacity for rocks having considerable strength i.e. more than 50 MPa.
2. The working bond strength can be conservatively estimated through recommendations, however estimation based on the rock anchor analogy is preferred over bored pile analogy.
3. Bored pile analogy may over predict the ultimate pullout strength for rocks having more than 50MPa compressive strength.

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Annexure A : Rock/Grout values recommended in practice (After Littlejohn and Bruce, 1977)

Rock Type	Working Bond (N/mm^2)	Ultimate Bond (N/mm^2)	Factor of Safety	Source
IGNEOUS				
Medium hard basalt		5.73	3 - 4	India - Rao [1964]
Weathered granite		1.50 - 2.50		Japan - Suzuki et al [1972]
Basalt	1.21 - 1.38	3.86	2.8 - 3.2	Britain - Wycliffe Jones [1974]
Granite	1.38 - 1.55	4.83	3.1 - 3.5	" - " " "
Serpentine	0.45 - 0.59	1.55	2.6 - 3.5	" - " " "
Granite & basalt		1.72 - 3.10	1.5 - 2.5	U.S.A. - P.C.I. [1974]
METAMORPHIC				
Manhattan schist	0.70	2.80	4.0	U.S.A. - White [1973]
Slate & hard shale		0.83 - 1.38	1.5 - 2.5	U.S.A. - P.C.I. [1974]
CALCAREOUS SEDIMENTS				
Limestone	1.00	2.83	2.8	Switzerland - Losinger [1966]
Chalk - Grades I-III		0.22 - 1.07	1.5 - 3.0	Britain - Littlejohn [1970]
Tertiary limestone	0.83 - 0.97	2.76	2.9 - 3.3	Britain - Wycliffe-Jones [1974]
Chalk limestone	0.86 - 1.00	2.76	2.8 - 3.2	" - " " "
Soft limestone		1.03 - 1.52	1.5 - 2.5	U.S.A. - P.C.I. [1974]
Dolomitic limestone		1.38 - 2.07	1.5 - 2.5	" - P.C.I. "
ARENACEOUS SEDIMENTS				
Hard, coarse-grained sandstone	2.45		1.75	Canada - Coates [1970]
Weathered sandstone		0.69 - 0.85	3.0	New Zealand - Irwin [1971]
Well cemented mudstone		0.69	2.0 - 2.5	" " - " "
Bunter sandstone	0.40		3.0	Britain - Littlejohn [1973]
Bunter sandstone (U.C.S. > 2.0 N/mm^2)	0.60		3.0	" - " "
Hard find sandstone	0.69 - 0.83	2.24	2.7 - 3.3	Britain - Wycliffe-Jones [1974]
Sandstone		0.83 - 1.73	1.5 - 2.5	U.S.A. - P.C.I. [1974]
ARGILLACEOUS SEDIMENTS				
Keuper marl		0.17 - 0.25	3.0	Britain - Littlejohn [1970]
Weak shale		0.35		Canada - Golder Brawner [1973]
Soft sandstone & shale	0.10 - 0.14	0.37	2.7 - 3.7	Britain - Wycliffe Jones [1974]
Soft shale		0.21 - 0.83	1.5 - 2.5	U.S.A. - P.C.I. [1974]
GENERAL				
Competent rock (where U.C.S. > 20 N/mm^2)	U.C.S. ÷ 30 (up to a maximum value of 1.4 N/mm^2)	U.C.S. ÷ 10 (up to a maximum value of 4.2 N/mm^2)	3.0	Britain - Littlejohn [1972]
Weak rock	0.35 - 0.70			Australia - Koch [1972]
Medium rock	0.70 - 1.05			
Strong rock	1.05 - 1.40			
Wide variety of igneous and metamorphic rocks	1.05		2.0	Australia - Standard CA35 [1973]