Vol. 08, No. 2 (2026) 1241-1252, doi: 10.24874/PES08.02B.013

Proceedings on Engineering Sciences

www.pesjournal.net

OPTIMIZATION OF PILE SOCKET DESIGN IN SRI LANKAN METAMORPHIC BEDROCK BY LABORATORY EXPERIMENTS AND PILES' PERFORMANCE

Received 19.12.2025
Revised 08.02.2026
Accepted 07.04.2026

Ayodya Bulathsinhala[1]
Anurudda Puswewala

Keywords:

Pile socket design, Skin friction, Metamorphic rocks, Borehole load testing, Bentonite filter cake, Foundation optimization

A B S T R A C T

Socketing into bedrock is widely practiced in Sri Lanka, yet existing design standards continue to adopt conservative assumptions derived from sedimentary rock conditions. Consequently, rock-socketed bored piles in crystalline metamorphic formations are frequently overdesigned, leading to excessive construction costs. This study evaluates performance-based optimization of skin friction in metamorphic rock sockets by combining theoretical estimation, field load test interpretation, and laboratory-derived shear strength models. Skin friction values were computed for fifteen instrumented load test piles and compared against theoretical predictions adjusted for bentonite filter cake effects. Mobilized skin friction frequently exceeded theoretical design assumptions by factors of 2–7, even in weathered rock. Laboratory results further validated the significantly higher interface shear capacities achievable in crystalline formations. Based on these findings, a revised design approach with a reduced safety factor of 2.5 is proposed, enabling substantial decreases in socket length without compromising safety.

1. INTRODUCTION

Cast in-situ bored (CIB) piles are extensively used in Sri Lanka for high-rise buildings, expressways, and long-span bridges, primarily due to their capability to transfer large loads to deeper competent strata (Wijayasinghe & Samarawickrama, 2013). In many Sri Lankan construction sites, CIB piles are socketed into shallow crystalline metamorphic bedrock, enabling the transfer of load through a combination of base resistance and shaft resistance. Although the contribution from the bedrock–concrete interface is expected to dominate the skin friction component because of the high strength and roughness of crystalline rocks, existing design practices in Sri Lanka continue to rely on conservative recommendations originally derived for sedimentary formations such as sandstone and shale.

A key factor influencing skin friction in CIB piles is the use of drilling fluids, particularly bentonite slurry. Bentonite is widely adopted in South Asia due to its low cost and ability to stabilize borehole walls (Tucker & Reese, 1984). However, the filtration of bentonite leads to the formation of a bentonite filter cake (BFC) along

[1] Corresponding author: Ayodya Bulathsinhala
Email: bulathsinhalau@itum.mrt.ac.lk

the pile shaft, which has been associated with reduced interface shear strength in several studies conducted in sedimentary rock and soil conditions (Johnston et al., 1987; Indraratna et al., 1999; Avşar, 2021). In contrast, the behaviour of bentonite-affected interfaces in metamorphic rocks—characterized by very low permeability, strong interlocking fabric, and anisotropic foliations—remains insufficiently understood.

Current local practice continues to adopt highly conservative values for ultimate skin friction, typically limiting socket resistance to 200 kPa based on Institute for Construction and Training (ICTAD) guidelines (ICTAD, 1997). Additionally, when bentonite is used, a reduction factor of approximately 75% is recommended, leading to the widespread use of safety factors as high as 3–4 in the design of rock sockets (Wyllie, 1991). These assumptions do not adequately reflect the superior engineering properties of Sri Lanka's gneissic and granulitic rocks, nor the performance of piles observed in actual load tests. Recent field evidence indicates that mobilized skin friction in Sri Lankan metamorphic rock frequently exceeds design predictions by factors of up to three or more (Thilakasiri et al., 2015). However, due to limited availability of instrumented load test (IMLT) data and the reluctance of contractors to release borehole logs, the national database remains sparse, limiting the refinement of design approaches.

Internationally, several analytical methods exist for predicting rock socket shaft resistance, including the Williams and Pells (1981) reduction-factor approach, the Hoek–Brown-based models used in the Hong Kong Guidelines (Geotechnical Engineering Office, 2006), and empirical correlations involving RMR and RQD (Bieniawski, 1989; Peck et al., 1974). Although these methods have been calibrated predominantly for sedimentary or heavily jointed rock masses, Sri Lankan practice applies them directly to metamorphic rock without verification. This mismatch results in systematic underestimation of socket capacity, leading to unnecessarily long socket lengths and elevated construction costs.

Recent laboratory studies conducted under both low constant normal stiffness (CNS) conditions demonstrate that metamorphic rock–concrete interfaces exhibit significantly higher shear strengths than commonly assumed, even when bentonite infill is present. These findings highlight the need to integrate experimental evidence with field load test data to re-evaluate design values for Sri Lankan conditions.

The aim of this study is therefore to establish a performance-based framework for optimizing socket length in CIB piles founded in crystalline metamorphic bedrock. This is achieved by combining theoretical predictions using established rock mechanics models, back-calculated mobilized skin friction from fifteen (Instrumented Maintained Load Tests (IMLTs) and Maintained Load Tests (MLTs) conducted in Sri Lanka and laboratory-derived shear strength models that account for bentonite filter cake development.

The above aim was achieved by following objectives.

1. To quantify the discrepancies between current theoretical design assumptions and the mobilized skin friction observed in field load tests.
2. To validate laboratory-derived interface shear models against the field performance of rock sockets constructed in gneissic bedrock.
3. To propose a revised design methodology incorporating an optimized socket length and a rationally reduced safety factor aimed at achieving substantial cost savings while maintaining structural safety.

These insights support the development of a more rational, performance-based design framework tailored specifically for Sri Lankan metamorphic rock masses, moving beyond conventional sedimentary-based design paradigms.

2. METHODOLOGY

This study adopts a performance-based approach that integrates theoretical estimation, field load test interpretation, and laboratory-derived shear strength models to evaluate the skin friction capacities of pile sockets in Sri Lankan metamorphic bedrock. The methodology consists of four main components:

1. Development of the pile database.
2. Theoretical estimation of ultimate skin friction using accepted design models.
3. Back-calculation of mobilized skin friction from instrumented and non-instrumented load tests.
4. Comparison with experimental results obtained under controlled laboratory conditions.

Each component is described in the following subsections.

2.1 Pile Load Test Database

Fifteen cast in-situ bored piles were selected from major infrastructure projects across Sri Lanka, including expressways, bridges, and high-rise developments and depicted in Table 1. These piles were chosen based on availability of static load test records (MLT or IMLT) and completeness of borehole logs. The 15 piles included in the study were socketed into metamorphic rocks dominated by Biotite gneiss (fresh to moderately weathered), Quartz-rich gneiss, Charnockitic gneiss in isolated cases. Weathering grades across the sockets ranged from Grade I (fresh) to Grade IV (highly weathered). Most sockets were formed in Grades II and III, which represent the most common subsurface conditions in Sri Lankan projects involving bored piles. The Unconfined Compressive Strength (UCS) of intact rock samples ranged from 30–150 MPa, with fresh rock typically exceeding 100 MPa. Correspondingly, Fracture Indices (FI) ranged between 0.1–1.0, and Rock Quality Designation (RQD) values ranged from 40–100%, indicating widely varying rock mass conditions. Socket lengths varied from 1.9 m to 16.9 m. Longer sockets were typically associated with weathered rock profiles, whereas shorter sockets occurred in high-quality rock where geotechnical designers had assumed conservative parameters.

Table 1. Data obtained from CIB piles driven in Sri Lanka

Pile ID	Project Name & Pile No	Pile Dia. (mm)	Pile Length (m)	Socket length (m)
1	Proposed Port Access Elevated Highway (PAEH)-TP-70A	1200	45.00	16.90
2	Proposed Port Access Elevated Highway (PAEH)-TP-39	1200	10.00	3.50
3	Central Expressway Project (CEP-2)-VD-05-P13-R2	1500	13.33	3.33
4	Central Expressway Project (CEP-2)-VD-06-P7-R2	1200	15.60	3.70
5	Central Expressway Project (CEP-2)-VD-06-P5-L2	1800	7.40	3.40
6	Maga Head Office -T2-TP1	1500	28.28	5.38
7	Prime Residency Ward Place-P-03	1500	39.60	6.60
8	Southern Expressway Project-P4L1	1800	54.39	1.89
9	Southern Expressway Project-P33R1	1800	38.22	9.88
10	Southern Expressway Project-P23L1	1500	14.00	3.00
11	Southern Expressway Project-P17L3	1500	13.00	4.50
12	Colombo Port City Project-P310-SPS01	900	15.20	3.70
13	Kelani Bridge Project-PRC2P17	1500	25.70	3.80
14	Central Expressway Project-(CEP-01)-TP-01	1500	23.90	6.90
15	Apartment complex for Prime Land residencies, Kelaniya	1000	29.40	5.40

Four of the piles (Pile ID 1, 2, 14, 15) were IMLTs, which provided strain gauge data along the pile shaft, enabling direct estimation of axial load transfer and skin friction mobilization. The remaining eleven piles were conventional MLTs, where skin friction had to be inferred from load–settlement relationships. Together, the dataset represents a diverse range of rock conditions—from highly weathered to fresh biotite gneiss—and captures the performance of piles drilled by different contractors using bentonite slurry.

2.2 Theoretical Estimation of Ultimate Skin Friction

The theoretical ultimate skin friction for the rock socket region was estimated using the Williams and Pells (1981) method, which has been previously recommended as the most suitable approach for Sri Lankan metamorphic rocks (ICTAD, 1997). Equation 1 was suggested to find the ultimate skin friction $f_{u,rock}$ in the rock socket region specially piles socketed in sandstone, mudstone or shale as follows.

$$f_{u,rock} = \propto \beta \sigma_c \ \text{MN}/m^2 \qquad (1)$$

Where, $\propto$ is the Reduction factor relating to σ_c, σ_c is UCS value of rock and β is the correction factor related to the discontinuity spacing in the rock mass called mass factor (j), Mass factor (j) is the ratio of the rock mass modulus (E_m) to intact rock modulus (E_i). Further, it can be obtained approximately by means of Fracture Index (FI) or RQD value. For each pile:

1. UCS values were obtained from borehole cores.
2. Rock mass parameters (RQD, FI, weathering grade) were extracted from borehole logs.
3. Layer-by-layer calculations were performed to obtain f_{us} for each segment of the socket.
4. A weighted average ultimate skin friction was obtained for the entire socket.

Since all selected piles were drilled using bentonite slurry, theoretical values were reduced by 25% following Wyllie's (1991) recommendation to account for the presence of a bentonite filter cake (BFC) along the socket wall.

Skin Friction from Overburden Soil Layers must be found to isolate the skin friction mobilized in the rock socket. The contribution from overlying soil layers was first computed. Depending on soil type, one of two standard approaches was used:

- For cohesive soils, the α-method (Tomlinson, 1971) was applied using undrained cohesion $C_u$and adhesion factor α estimated from empirical charts.
- For granular soils, the β-method (Poulos & Davis, 1980) was used, incorporating effective

vertical stress and lateral earth pressure coefficient k.

Critical depth (Z_c/d) considerations were included to prevent overestimation of effective stress in deep sand layers. The total skin friction from soil layers was deducted from the total shaft resistance obtained from load test interpretation to isolate rock socket skin friction.

2.3 Interpretation of Field Load Tests

For all piles, the Chin (1978) method was applied to load–settlement data to estimate Ultimate carrying capacity and Mobilized skin friction (from the initial linear region). A plot of settlement/load (S/P) versus settlement (S) was used to obtain the gradient (m_1) as depicted in Figure 1 for Pile ID 14.

$$f_{s,\ mobilized} = \frac{1}{m_1} \tag{2}$$

This method was selected as the primary interpretation technique because previous studies have shown it provides more realistic values for Sri Lankan rock-socketed piles (Thilakasiri, 2009).

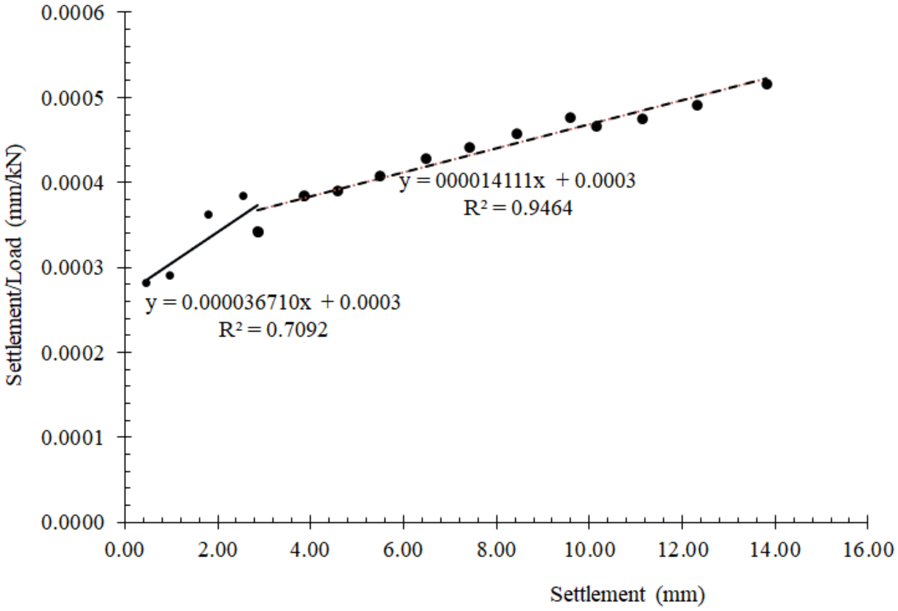

Figure 1. Settlement/load against settlement of pile ID 14. (Bulathsinhala and Puswewala, 2025)

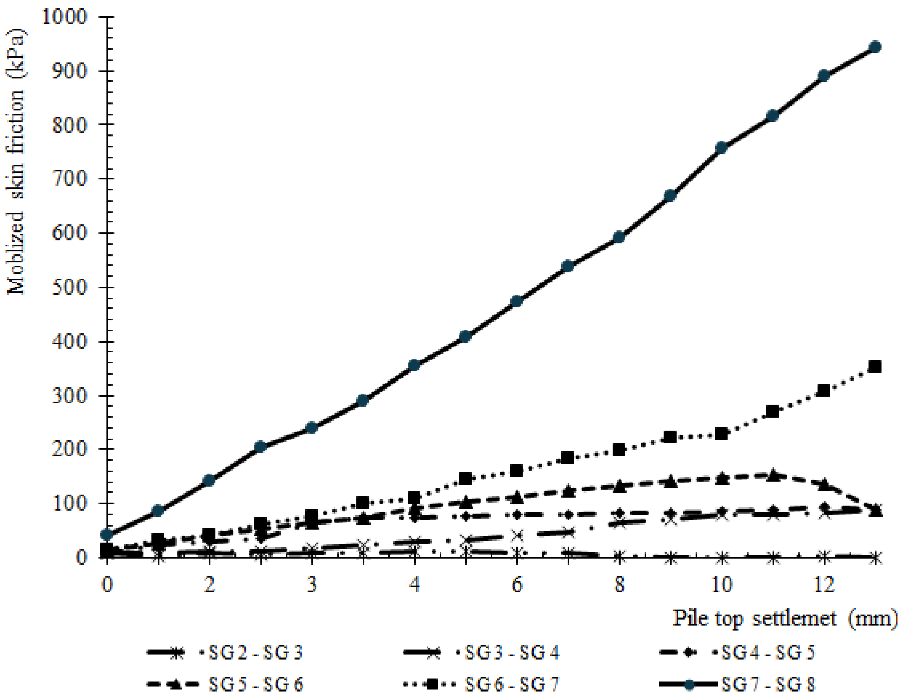

Figure 2. Unit skin friction Vs Pile top settlement of Pile ID 14.

Instrumented Load Tests (IMLTs)
For the four instrumented piles, strain gauges installed along the shaft allowed direct estimation of load transfer in both soil and rock layers. Skin friction at different depths was calculated from differential axial load and the corresponding surface area of each segment. These direct measurements were used to verify the accuracy of values obtained from the Chin (1978) method. Figure 2 depicts the variation of the mobilized skin friction against the settlement of the pile for pile ID 14.

Local guidelines specify that the maximum allowable net settlement under 100% of the design verification load as 12 mm (Construction Industry Development Authority, 2016). According to the strain gauge (SG) readings the skin friction mobilized in the socketed region was obtained.

2.4 Laboratory-Derived Shear Strength Models

Development of equivalent roughness profile for Biotite Gneiss cores
Shear tests were carried out under Constant Normal Loading conditions (CNL) for different metamorphic rock – concrete joints. It was found that the Biotite Gneiss – Concrete Joint is the weakest rock concrete joint of all owing to the presence of Mica in Biotite Gneiss (Bulathsinhala, 2025). Hence the rock cores obtained during the pile drilling of biotite gneiss were traced for the surface profiles by a profilometer and details were collected as depicted in Table 2 The surface which had the lowest Joint roughness coefficient (JRC) value (2.5) was chosen for further analysis

Table 2. Roughness details of biotite gneiss rock cores obtained in Sri Lanka (Bulathsinhala, 2025)

Location	WG	JRC from Standard Profile	Fractal Dimension (D)	JRC Barton and Choubey (1977)	Centerline Roughness (Z_2) (mm)	Roughness Amplitude (mm)
Malabe	MW	4-6	1.070	5.5	1.5253	3.05
	F	8-10	1.125	9.0	2.2435	4.49
	F	6-8	1.160	6.3	1.9700	3.93
Pattiwila	F	2-4	1.025	2.5	1.7480	3.50
University of Sri Jayawardenepura	MW	6-8	1.120	6.0	1.5205	3.04
	MW	6-8	1.030	6.0	1.5425	3.09
	MW	4-6	1.150	5.5	1.4670	2.93
	F	8-10	1.105	9.0	1.7990	3.60
	F	4-6	1.127	5.8	1.3347	2.67
	F	8-10	1.195	9.0	2.2070	4.41
	F	8-10	1.040	9.0	2.1970	4.39

Above rock cores had been traced for the surface profiles by a profilometer and the JRC ranges had been estimated by comparing with the standard roughness profiles and later deduced to single values by following the Barton and Choubey (1977) method. The profile shown in Figure 3(a) which had the lowest Joint roughness coefficient (JRC) value (2.5) was chosen for further analysis. Thereafter for the chosen surface, the fractal dimension (D = 1.072) and the roughness amplitude (Z = 3.5mm) were obtained and converted to an equal triangular asperity surface profile as depicted in Figure 3(b) having the asperity angle (i) 8^0 and asperity height (a) 3.5 mm by trial-and-error method.

Constant Normal Stiffness (CNS) direct shear tests
A Laboratory study was conducted a weakest metamorphic rock – concrete joint encountered to find out the shear behaviour of rock sockets. Accordingly, Biotite Gneiss Samples were shaped into saw-toothed profiles having the asperity angle 8^0 as depicted in Figure 3(b) and sheared against Grade 30 (M30) concrete samples by the direct shear apparatus at University of Wollongong Australia shown in Figure 4.

During the shearing different Bentonite infill thicknesses varying from 0.5 mm to 5.5 mm were employed between Biotite gneiss -concrete samples and a low constant normal stiffness of 8.5 kN/mm was employed to represent the highly jointed rock layers filled with infill materials. The shear behaviour was modelled using the Equation 3 (Johnston and Lam, 1989).

$$\tau_{\mathrm{sl}}^{r} = (\sigma_{\mathrm{no}} + \Delta\sigma_{\mathrm{n}})\, tan\,(\Phi_b + i) + \frac{\eta c_{sl}}{2\pi \cos^2 \mathrm{i}(1 - \tan \mathrm{i} \tan \emptyset_b)} \cos^{-1}\left(1 - \frac{4\tau_{\mathrm{sl}}^{r} \sin \mathrm{i} \cos \mathrm{i}}{\eta q_{\mathrm{u}}}\right) \quad (3)$$

Where: τ_{sl}^{r} is Residual shear stress for subsequent sliding, σ_{no} is initial normal stress, ϕ_b is the base residual friction angle (sliding), i is the asperity angle, C_{sl} is the cohesion of rock for asperity sliding, η is the interlocking factor, q_u is uniaxial compressive strength of rock, $\Delta\sigma_n$ is change in the normal stress.

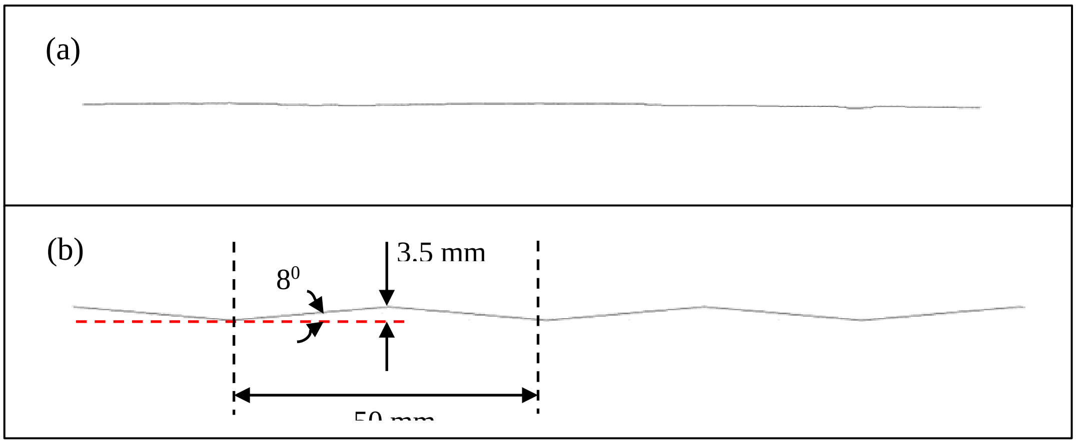

Figure 3. (a) Actual surface profile (b) Equivalent triangular profile

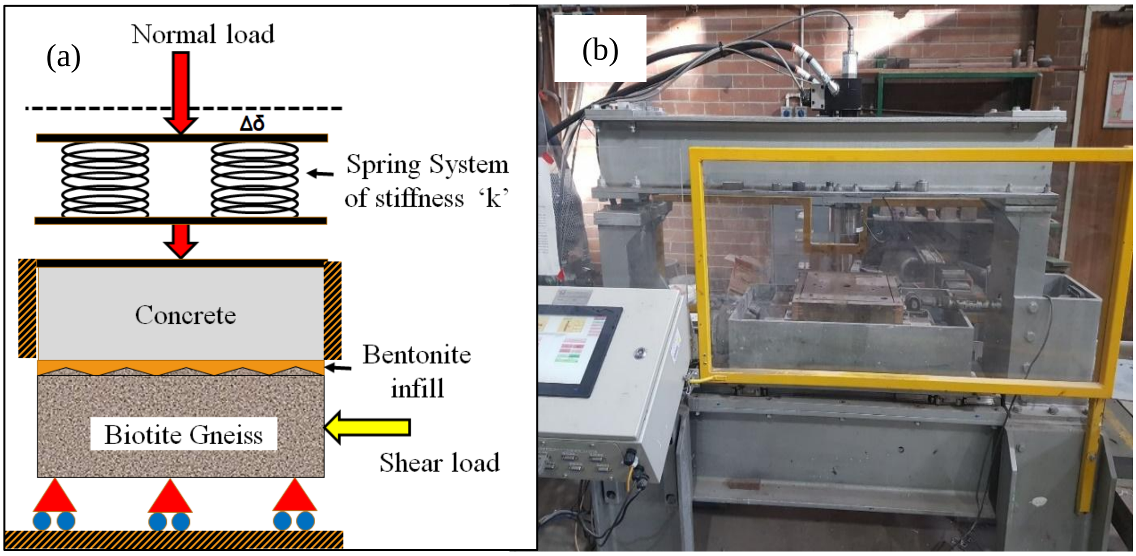

Figure 4. Direct shear apparatus; (a) Force diagram (b) Apparatus at University of Wollongong, NSW, Australia

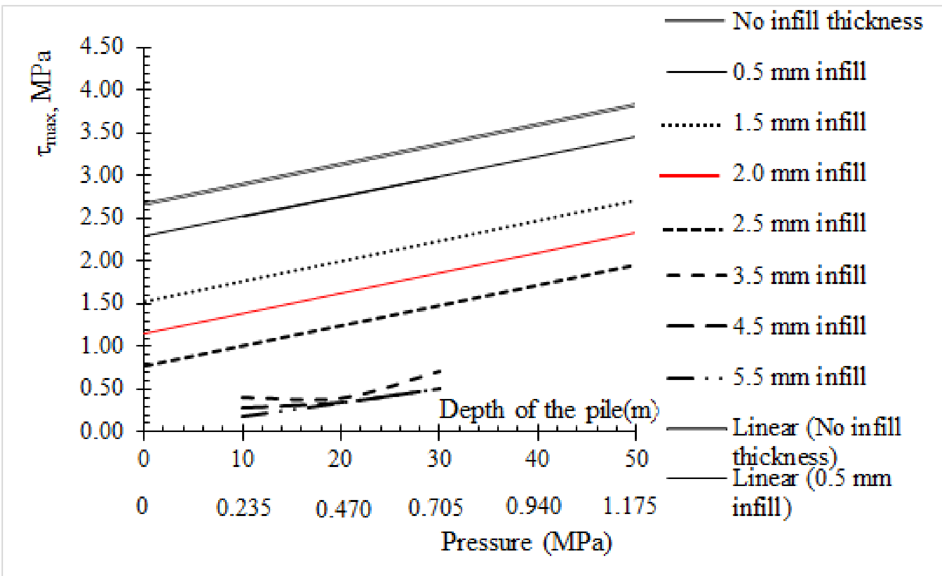

Figure 5. Variation Ultimate Theoretical Skin friction obtained for Biotite Gneiss – Concrete interface from modified Johnston and Lam model (Bulathsinhala and Puswewala, 2025)

From equation 3 the curves were generated to estimate ultimate theoretical skin friction capacity as depicted in Figure 5. According to the recent findings 2mm thick Bentonite filter cake is formed over fresh crystalline metamorphic after 12 hours (Bulathsinhala and Puswewala, 2024). Hence the skin friction values correspondent to 2 mm thick Bentonite infill thickness obtained from Figure 5 were compared against the mobilized skin friction.

2.5 Comparative Analysis and Development of Design Recommendations

The theoretical, field-derived, and laboratory-derived shear strengths were compared across all piles to evaluate discrepancies between predicted and mobilized skin friction, the degree of conservatism in conventional design assumptions, and opportunities for reducing safety factors and optimizing socket length. A performance-based design recommendation was then developed, incorporating observed mobilized values, experimental limits, and acceptable safety margins for Sri Lankan metamorphic rock conditions.

3. RESULTS

This section presents the results from theoretical estimations, field load test interpretations, and laboratory-derived interface shear strengths. Mobilized skin friction values are compared across the 15 case study piles to evaluate discrepancies with conventional design assumptions and to establish evidence for performance-based optimization of pile sockets.

3.1 Comparison of Theoretical Skin Friction method with Mobilized skin friction

Using the Williams & Pells (1981) approach, adjusted for rock mass quality and reduced by 25% for bentonite filter cake effects the theoretical estimation is obtained. The mobilized skin friction obtained from Chin (1978) method is also compared with theoretical estimation and shown the Table 3.

Table 3. Theoretical ultimate skin friction capacity Vs Mobilized skin friction

Pile ID	Theoretical Skin Friction Capacity (MPa) (Williams and Pells Method, 1981)	Factor of Safety = 4.0 (Wyllie, 1991)	Mobilized skin friction (MPa), (Chin (1978) Method)	% of Mobilized Skin Friction to Wyllie's (1991) Theoretical Estimation
1	0.777	0.194	0.728	375%
2	3.583	0.896	2.650	296%
3	1.282	0.321	1.393	435%
4	1.365	0.341	1.159	340%
5	1.405	0.351	0.708	202%
6	1.765	0.441	2.046	464%
7	0.796	0.199	1.295	651%
8	0.975	0.244	0.006	2%
9	0.537	0.134	0.678	505%
10	1.069	0.267	0.694	260%
11	1.369	0.342	1.253	366%
12	1.985	0.496	1.490	300%
13	1.109	0.277	0.728	263%
14	1.406	0.352	0.803	228%
15	1.306	0.327	0.837	256%

Except the outlier Pile (Pile ID 8), almost all the piles have the performance varying between 200% to 700% of their theoretical estimation. It is evident that dividing the factor of safety 4 as per the recommendation of Wyllie (1991) is highly conservative and hence the different safety factors (FoS) were tried out until the values were within the safe limits (Table 4).

As shown in Table 4, the design skin friction capacity of pile sockets can be optimized by reducing the FoS to 2 from the existing FoS 4. Hence the socketing length can be reduced at least by half and cost of rock drilling can be saved at least by half. However, Williams and Pells (1981) method does not account the normal stress exerted on the pile due to the wet concrete column at the piles casting stage. Further, it doesn't account the roughness values of crystalline metamorphic rocks into the calculation of skin friction capacity.

Table 4. Comparison of different safety factors with the ultimate theoretical skin friction values obtained by Williams and Pells (1981) method

Pile ID	Skin Friction (MPa) (Williams and Pells Method, 1981)	Mobilized skin friction (MPa), (Chin (1978) Method)	Design Skin Friction Capacity (MPa)					
			FoS =1.0	FoS =1.5	FoS =2.0	FoS =2.5	FoS =3.0	FoS =4.0 (Wyllie, 1991)
1	0.777	0.728	0.777	0.518	0.389	0.268	0.222	0.194
2	3.583	2.650	3.583	2.389	1.792	1.236	1.024	0.896
3	1.282	1.393	1.282	0.855	0.641	0.442	0.366	0.321
4	1.365	1.159	1.365	0.910	0.683	0.471	0.390	0.341
5	1.405	0.708	1.405	0.937	0.703	0.484	0.401	0.351
6	1.765	2.046	1.765	1.177	0.883	0.609	0.504	0.441
7	0.796	1.295	0.796	0.531	0.398	0.274	0.227	0.199
8	0.975	0.006	0.975	0.650	0.488	0.336	0.279	0.244
9	0.537	0.678	0.537	0.358	0.269	0.185	0.153	0.134
10	1.069	0.694	1.069	0.713	0.535	0.369	0.305	0.267
11	1.369	1.253	1.369	0.913	0.685	0.472	0.391	0.342
12	1.985	1.490	1.985	1.323	0.993	0.684	0.567	0.496
13	1.109	0.728	1.109	0.739	0.555	0.382	0.317	0.277
14	1.406	0.803	1.406	0.937	0.703	0.485	0.402	0.352
15	1.306	0.837	1.306	0.871	0.653	0.450	0.373	0.327

3.2 Comparison of Theoretical Skin Friction obtained by curves obtained by laboratory experimental setup up with Mobilized skin friction

Theoretical skin friction values were obtained from the curves shown in Figure 5 assuming that 2 mm thick Bentonite filter cake is formed over crystalline metamorphic rocks. Accordingly, values were compared with mobilized skin friction capacities and, the different safety factors were tried out with the theoretical skin friction capacities obtained by the curves generated by direct shear tests carried out under low CNS conditions as depicted in Table 5.

Table 5. Comparison of different safety factors with theoretical skin friction obtained from newly developed model

Pile ID	Mobilized skin friction (MPa), (Chin (1978) Method)	Theoretical Skin Friction Capacity (MPa) obtained by the curves generated by direct shear tests carried out under low CNS conditions	FoS =1.5	FoS =2.0	FoS =2.5	FoS =3.0
1	0.728	2.01	1.34	1.01	0.69	0.57
2	2.650	1.34	0.90	0.67	0.54	0.45
3	1.393	1.42	0.95	0.71	0.57	0.47
4	1.159	1.47	0.98	0.74	0.59	0.49
5	0.708	1.28	0.86	0.64	0.51	0.43
6	2.046	1.75	1.17	0.88	0.70	0.58
7	1.295	2.00	1.34	1.00	0.80	0.67
8	0.006	2.33	1.55	1.16	0.93	0.78
9	0.678	1.93	1.29	0.97	0.67	0.55
10	0.694	1.44	0.96	0.72	0.58	0.48
11	1.253	1.40	0.93	0.70	0.56	0.47
12	1.490	1.46	0.98	0.73	0.59	0.49
13	0.728	1.71	1.14	0.85	0.68	0.57
14	0.803	1.63	1.09	0.82	0.65	0.54
15	0.837	1.81	1.20	0.90	0.72	0.60

Accordingly, the FoS 2.5 is the most appropriate FoS as the values are well below the mobilized skin friction capacity.

4. DISCUSSION

Usually, the MLTs are performed for problematic piles where the weak strata are present or for piles that may be suspected of as defective piles. Hence, most of the bed rocks found in MLT locations are not sound rocks and sometimes highly weathered. Therefore, the rocks found in MLT locations do not represent the entire bed rock of the site but may represent the weakest rock in the site. The theoretical skin friction values obtained by Williams and Pells (1981) method for these 15 piles underestimate the actual skin frictions mobilized during the application of the load and hence the skin friction is neglected in most designs. In designing of the piles, when the Bentonite is used as the drilling fluid, the theoretical skin friction capacity is divided by 4 (Wyllie, 1991) to obtain design load. However, these values are too conservative and hence end up with huge costs in pile construction.

Around 2 mm thick BFC is formed at 30 m depth if the borehole is idle for 12 hrs. between the drilling termination and concreting (Bulathsinhala and Puswewala, 2024). Hence considering the worst-case scenario, where a 2 mm thick BFC forms, the shear strength of the rock joint is 1.84 MPa according to the newly developed model for 30 m deep socket. Further it is evident that the design loads of the piles are too conservative as both laboratory experiments and the mobilized skin frictions obtained by back calculations of the load test results show significantly higher values compared to the design loads (allowable loads) calculated by various methods. Accordingly, the safety factors used to obtain the design loads can be reduced by 2 by keeping the existing Willams and Pells (1981) as it is used to obtain the ultimate skin friction capacity. However, this method does not count the surface roughness and the lockdown pressure exerted on the pile socket during the concreting. However, it still can be used with FoS 2 by reducing the socketing length, time and cost by Half.

The new model was developed through serious direct shear tests done for weakest roughness profiles. Further the Bentonite infill thickness varied during the experiments to develop more realistic models to obtain the ultimate theoretical skin friction capacity. Obtained from the new model through laboratory experiments discussed in section 2.4.2, these curves provide the Ultimate theoretical skin friction capacity ($\boldsymbol{\tau_{max}}$) capacities corresponding to the average socket depth ranges starting from 10 m to 30 m for metamorphic rocks. However, using the developed model, this range was extended up to 50 m depth starting from 0 m depth for infill thicknesses up to 2.5 mm.

Table 5 compares the skin friction values obtained from MLT data with the experimental data obtained under Low CNS conditions. Accordingly, it shows the values obtained after applying different FoS for the values obtained from experimental models. Experimental data showed that tests carried out under low CNS conditions well represent the half-mobilized skin frictions obtained by Chin (1978) method after dividing FOS 2.5. Accordingly, the design load (DL) (given in units of load per unit area of socket perimeter surface) accounted by the rock can be expressed as follows for metamorphic rocks.

$$DL\,rock = \frac{\begin{array}{c}Experimental\ \tau_{max}\ obtained\ from\\ the\ model\ developed\ for\ low\ CNS\ conditions\end{array}}{2.5} \quad (4)$$

Hence as a suggestion it can be proposed to use the $\boldsymbol{\tau_{max}}$ obtained from the model devised from the tests conducted at low CNS boundary conditions. This suggestion is not highly economical since it was devised from just 15 MLT data. No pile had reached failure criteria even after loading 4 times of design verification load. The safety factor could have been reduced more if any of the piles had loaded up to the failure points. In other piles skin friction in the socketed region had not been mobilized fully. Nevertheless, this solution is far less conservative and saves the socketing cost by half compared to the old method of dividing the ultimate theoretical skin friction capacity which was obtained accounting to Williams and Pells (1981) and modified by Willie (1991) by using a FOS 4 for the rock sockets which have the influence of Bentonite slurry.

5. CONCLUSIONS

In this study 15 piles were used with Maintained Load Test (MLT) data along with the corresponding borehole log data which enabled the authors to perform a comprehensive analysis. After analyzing results, this study recommends decreasing the FoS which is used to find the load component by Williams and Pells method accounted for by the socketed region of the piles from 4 to 2 for any metamorphic rock.

Further, the shear strength values obtained by the laboratory experiments under lower constant normal stiffness (CNS) conditions had always higher values even at most unfavorable conditions where 2 mm thick Bentonite infill was present compared to theoretical estimation and the mobilized values. By considering these facts, this study recommends using the FoS 2.5 to find the load component accounted for by the socketed region of the piles driven in metamorphic rock. This will lead to a decrease in the socketing length and cost by at least half. If these outcomes are used in future cast in-situ bored CIB pile designs construction cost of piles will be reduced significantly.

Between the two approaches evaluated for estimating theoretical skin friction capacity—the Williams and Pells analytical method and the shear strength values predicted using curves derived from direct shear tests conducted under low CNS conditions—the laboratory-generated curves provide a far more realistic representation of field performance. This is primarily because the experimental curves incorporate the most adverse conditions expected in Sri Lankan crystalline metamorphic rocks, including the lowest practical surface roughness and the maximum bentonite infill thickness observed during drilling operations. By directly capturing the interface behaviour under these conservative yet realistic parameters, the laboratory-derived curves reflect the actual shear transfer mechanisms more accurately than the generalized empirical reduction factors used in the Williams and Pells method. Consequently, the shear capacities obtained from low-CNS testing offer a more reliable basis for performance-based design of rock sockets, supporting more optimized yet safe socket lengths in practice.

Acknowledgement: The authors wish to express their sincere gratitude to Eng. M.N.C. Samarawickrama, Senior Lecturer, Open University of Sri Lanka; Eng. R.S. Ruwanpathirana, Geotechnical Engineer, Knight Piésold Pty. Ltd.; and Eng. Arjuna Gunatilake, Managing Director, San Piling (Pvt) Ltd., for their invaluable technical guidance and support in compiling field load test data. The authors also gratefully acknowledge Dr. Pabasara Wanniarachchige and Prof. Jayan S Vinod of the School of Civil, Mining, and Environmental Engineering, University of Wollongong, NSW, Australia, for providing access to laboratory facilities and technical assistance during the experimental program.

References:

Avşar, E. (2021). An experimental investigation of shear strength behavior of a welded bimrock by meso-scale direct shear tests. *Engineering Geology, 294,* 106321.

Bieniawski, Z. T. (1989). *Engineering Rock Mass Classification*. John Wiley, New York.

Barton, N., & Choubey, V. (1977). The shear strength of rockfill. *Rock Mechanics, 10*(1-2), 1-54. https://doi.org/10.1007/BF01237819

Bulathsinhala, A. U. V. B., & Puswewala, U. G. A. (2025, October). Performance-based optimization of pile socket design in Sri Lankan metamorphic bedrock. In *the Proceedings of the 4th International Research Conference of the Institute of Technology, University of Moratuwa (ITUM-IRC 2025).* (p. 1).

Bulathsinhala, A. U. V. B., & Puswewala, U. G. A. (2024). Formation of bentonite filter cake over crystalline metamorphic rocks by means of a pressure chamber: A laboratory study. *Journal of Testing and Evaluation, 52*(1), 364–377. https://doi.org/10.1520/JTE20220322

Bulathsinhala, A. U. V. B. (2025). *Ultimate shear resistance of rock sockets in the presence of bentonite filter cake* (PhD thesis). University of Moratuwa.

Chin, F. K. (1978). Introduction to rock mechanics (Vol. 9). The Fifth Southeast Asian Conference on Soil Engineering.

Construction Industry Development Authority, Sri Lanka. (2016). *Specification for bored and cast in-situ reinforced concrete piles.* CIDA /SP/101.

Geotechnical Engineering Office. (2006). Foundation Design and Construction, Civil Engineering and Development, The Government of the Hong Kong Special Administrative Region (Vol. Geo Publication No.01).

Indraratna, B., Haque, A., & Aziz, N. (1999). Shear behaviour of idealized infilled joints under constant normal stiffness. *Géotechnique, 49*(3), 331–355. https://doi.org/10.1680/geot.1999.49.3.331

Institute for Construction Training and Development, Sri Lanka. (1997). Guidelines for Interpretation of Site Investigation Data for Estimating the Carrying Capacity of Single Piles for Design of Bored and Cast In-situ Reinforced Concrete Piles, (ICTAD /DEV/15).

Johnston, I. W., Lam, T., & Williams, A. (1987). Constant normal stiffness direct shear testing for socketed pile design in weak rock. *Geotechnique, 37*(1), 83-89. https://doi.org/10.1680/geot.1987.37.1.83

Johnston, I. W., & Lam, T. S. (1989). Shear behavior of regular triangular concrete/rock joints—analysis. *Journal of geotechnical engineering, 115*(5), 711-727. https://doi.org/10.1061/(ASCE)0733-9410(1989)115:5(711)

Peck, R. B., Hanson, W. E., & Thornburn, T. H. (1974). *Foundation engineering* (2nd ed.). John Wiley & Sons.

Poulos, H. G., Davis, E. H. (1980). *Pile Foundation Analysis and Design*. John Wiley & Sons.

Thilakasiri, H. S. (2009). *Construction & Testing of Piles* (01 ed.). Sarasavi Publishers, Sri Lanka.

Thilakasiri, H. S., Abeyasinghe, R. M., & Peiris, T. A. (2015). Optimization of the designing of bored and cast in-situ piles in Sri Lanka. In *Proceedings of the International Conference on Geotechnical Engineering* (pp. 253-256). Sri Lanka Geotechnical Society.

Tomlinson, M. J. (1971). Some effects of pile driving on skin friction. In Behaviour of piles (pp. 107-114). Thomas Telford Publishing, 107-114.

Tucker, K. L., & Reese, L. C. (1984). The effect of bentonitic slurry on drilled shafts (Research Report 351-1F). Centre for Transportation Research, The University of Texas.

Wijayasinghe, W. P. S. S., & Samarawickrama, M. N. C. (2013). An investigation on actual soil skin friction capacity of CIB piles: Case study of a proposed forty-two storied building project, Colombo 03. *Journal of Engineering and Technology of the Open University of Sri Lanka (JET-OUSL)*, 1(2).

Williams, A. F., & Pells, P. J. N. (1981). *Side resistance of rock sockets in sandstone, mudstone, and shale. Canadian Geotechnical Journal*, *18*(4), 502–513. https://doi.org/10.1139/t81-061

Wyllie, D. C. (1991). Foundations on Rock (1 ed.). E & FN Spon.

Ayodya Bulathsinhala
Institute of Technology University of Moratuwa
Diyagama, Homagama,
Sri Lanka
bulathsinhalau@itum.mrt.ac.lk
ORCID: 0000-0002-8459-2773

Anurudda Puswewala
Sri Lanka Institute of Information Technology ,
Malabe,
Sri Lanka
anuruddha.p@sliit.lk
ORCID: 0000-0002-1611-1210