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Bi-directional Pile Loading Tests on Meta-sedimentary Rocks

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ABSTRACT: To determine the geotechnical parameters of sound meta-sedimentary rock for deep foundation design, the bi-directional static load test is a common investigative method. It overcomes the high-load limitations of conventional top-down tests, providing direct data on rock mass deformation and strength characteristics. To support an on-going study by the Geotechnical Engineering Office (GEO) to enhance the foundation design guideline for sound meta-sedimentary rocks in Hong Kong, a large-scale static pile loading test programme adopting bi-direction load cells is being implemented. The test programme comprises over ten test piles strategically located across the north-western and northern parts of Hong Kong, where complex geology has been challenging to the geotechnical design. One challenge is the presence of clastic sedimentary rocks being affected by metamorphism to varying degrees. Coupled with extensive folding, faulting and weathering, the occurrence of meta-sedimentary rocks could be highly variable in nature rendering their engineering behaviours difficult to be generalised. This study advocates the use of a modified Rock Mass Rating (RMR) classification system to register the engineering characteristics of meta-sedimentary rocks. The test piles are rock-socketed piles of sizes ranging from 0.6 m to 1.5 m in diameter. Bi-directional static load testing, where the test load (up to over 30 MN) is applied at the pile base, is adopted. This paper presents key results from bi-directional static load tests on piles socketed in sound meta-sedimentary rock. The tests achieved basal pressures reaching ~16,000 kPa to ~29,000 kPa and average mobilised rock shaft frictions of ~1,200 kPa to ~6,000 kPa. In most cases, the tested basal pressure and the average mobilised rock shaft frictions are limited by the capacity of testing apparatus. The tested rock mass has been classified with modified RMR, from the available data set, rock mass with higher modified RMR generally tending to show stiffer response.

KEYWORDS: Pile foundation, pile loading tests, bi-directional loading, instrumentations, meta-sedimentary rocks.

1 INTRODUCTION

The Northern Metropolis (NM), a new initiative of urbanising the northwestern and northern parts of Hong Kong, has many planned mega-scale infrastructural and building developments in this and next decades. Comparing to the typical granitic or volcanic geology of Hong Kong, NM is situated in a geologically complex sedimentary basin with intense faulting, deep weathering profile and complicated metamorphism. This unique geological setting has led to the formation of meta-sedimentary (MS) rocks (Figure 1).

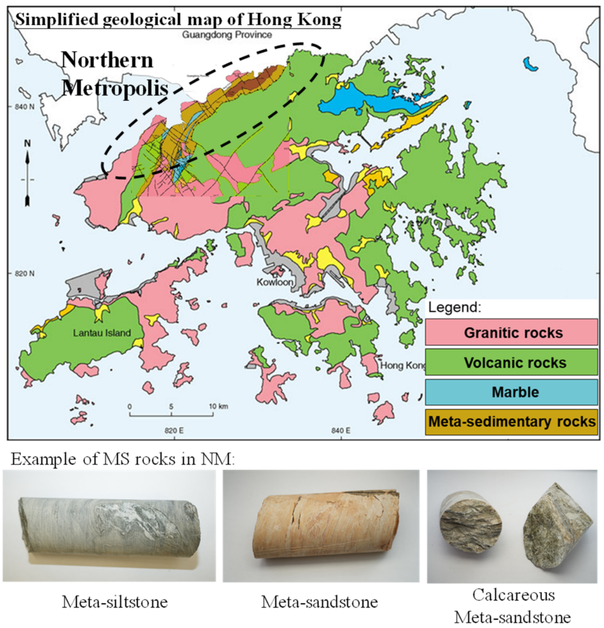


Figure 1. Northern Metropolis and meta-sedimentary rocks.

Due to the undeveloped nature of the NM region in the past and the lack of geotechnical information for MS rocks, the pertinent prevailing foundation design guideline in Hong Kong remains relatively conservative. To support the long-term development of NM, the GEO recently initiated a large-scale pile loading test programme within the NM region to collect essential test data on the end-bearing capacity and the rock socket shaft resistance of piles so as to explore potential enhancement of the foundation design guidelines for MS rocks.

The pile loading test programme aims to test the performance of piled foundation founded in sound MS rocks stratum, which with potential high load-bearing capacity (e.g. test load up to 30,200 kN, which equivalent to ~26,000 kPa tested bearing pressure, from a previous case by Littlechild et al. (2000)). In view that the use of top-down loading test may not be viable to achieve the targeted high load at the bearing stratum, static loading test with bi-directional loading system at pile base (Osterberg, 1984; Schmertmann & Hayes 1997) is adopted.

While the trial piling works are still actively on-going, this paper reports the preliminary findings of the pile loading tests on MS rocks, highlighting the potential of enhancing the pertinent foundation design guidelines in local practice.

2 FOUNDATION DESIGN IN META-SEDIMENTARY ROCKS IN HONG KONG

2.1 *Engineering geological characteristics of MS rocks*

The MS rocks in NM belong to Lok Ma Chau Formation and comprise metamorphosed sandstone and carbonaceous siltstone with graphitic interbeds and calcareous content (Tse & Tang, 2024). Their engineering geological characteristics are complicated as the rock is sedimentary in nature and may have been subjected to different degree of metamorphism. Some features are discussed as follows:

- MS rocks are mostly clastic, of which, the material strength and durability are largely controlled by the composition and property of the constituent grains as well as the degree of compaction and cementation.
- The bedding planes may act as planes of weakness where weathering is more intense. As such, MS rocks could be consisting of various properties in lateral and vertical extent, and may change over short distances.

- Dynamic metamorphism due to intense fault movements may result in brecciated and brittled rocks, introducing further structures (i.e. joints and fractures) to the bedding planes originated from the sedimentary process. As a result, the engineering rockhead may not be as easily defined as compared to igneous rocks in Hong Kong.
- Dynamic metamorphism is prominent in the NM region, particularly near San Tin Fault, by way of north or northwest inclined foliation (Sewell et al., 2000.). Foliation is favourable for weathering along the preferred alignment of minerals, leading to rapid disintegration of rock cores. Local variations of foliation orientation will introduce anisotropic material properties relative to the alignment of the fabrics.
- Other metamorphic processes such as recrystallization and hydrothermal alteration would lead to the rock strength varying across the rock mass depending on the type and distribution of the resulting minerals remaining in the rock. For instance, silicification could result in rock strength differing across intercalated layers, with those layers possessing high concentration of silica having a relatively higher strength.

The above mentioned complicated engineering geological characteristic of MS rocks may have more pronounced effects to engineering works if they are exposed to air with time, such as site formation works (e.g. Greenway et al., 1988). For deep pile foundations, the high confinement and minimal exposure to air may have beneficial effects in minimising the adverse impacts of closely-spaced joints on end-bearing capacity of MS rocks. This aspect may be further reviewed based on the pile loading tests conducted in this study.

2.2 *Foundation design practice in Hong Kong*

In Hong Kong, pile foundation design in rocks primarily follows BD (2017), GEO (2025a & 2025b) with the adoption of presumed design values. As can be seen in Table 1, the presumed design values of MS rocks are much lower than that of marble, granitic and volcanic rocks of similar characteristics. It is mainly attributed to the lack of geotechnical information of MS rocks at the time of code publication in 2017. As a result, the required number of pile foundations for a building underlain by MS rocks is usually higher leading to higher construction cost and time, and more congested site arrangement, etc.

Table 1. Presumed allowable vertical bearing pressure.

Description of Rocks	Presumed allowable end-bearing pressure	Presumed allowable rock socket friction*
Category 1(a)-(c): Granitic & Volcanic rocks	7,500 to 12,500 kPa	700 to 1,000 kPa
Category 2: Meta-sedimentary rocks	3,000 kPa	300 kPa
Marble	7,500 to 10,000 kPa	700 to 1,000 kPa

* Design values shown are for piles under compression or transient tension, lower values are prescribed for piles under permanent tension

An alternative design approach is the Rock Mass Rating (RMR) method (GEO, 2006), in which the allowable end-bearing pressure of a pile can be evaluated by assessing the RMR of the founding rock mass. Figure 2(a) shows the score sheet for determining RMR of any jointed rock mass, including MS rocks. This was modified from Bieniawski (1973) and (1989) to suit the purpose of foundation design, and has been adopted in local practice. The allowable end-bearing pressure is directly correlated with RMR with due regard of the local pile loading test data (Figure 2(b)). For example, an allowable end-bearing pressure of 5,000 kPa could be adopted for rock mass with RMR value of 50, and that 7,500 kPa for rock mass with RMR value of 60. In local practice, the RMR method has to be used with on-site verification pile loading tests. For example, Yau & Lau (2024) carried out pile loading tests on MS rocks in Hong Kong and by using the RMR method, they adopted an allowable end-bearing pressure of 5,000 kPa in the foundation design (>60% higher than the presumed design value) leading to significant cost and time saving for a building project. Suen et al. (2025) discussed the details about the RMR method and its potential enhancement for foundation design.

Rating Assigned to Individual Parameters using RMR Classification System (Based on Bieniawski, 1989)

(A) Strength of Intact Rock

Uniaxial compressive strength, σ_c (MPa)	> 250	250 – 100	100 – 50	50 – 25	25 – 5	5 – 1	< 1
Point load strength index, PLI_{50} (MPa)	> 10	10 – 4	4 – 2	2 – 1	σ_c is preferred		
Rating	15	12	7	4	2	1	0

(B) Rock Quality Designation (RQD)

RQD (%)	100 – 90	90 – 75	75 – 50	50 – 25	< 25
Rating	20	17	13	8	3

(C) Spacing of Joints

Spacing	> 2 m	2 m – 0.6 m	0.6 m – 0.2 m	200 – 60 mm	< 60 mm
Rating	20	15	10	8	5

(D) Conditions of Joints

Discontinuity length(1) Rating	2				
Separation	None	< 0.1 mm	0.1 – 1 mm	1 – 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickenside
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Rating	6	5	3	1	0

(E) Groundwater

Rating(1)	7

Notes :

(1) Rating is fixed as the parameter is considered not relevant to the evaluation of allowable bearing pressure of rock mass.
(2) RMR is the sum of individual ratings assigned to parameters (A) to (E).

(a)

Parameters	Rock Mass Rating (RMR)			
	< 40	50	70	88
Allowable bearing pressure, q_a (kPa)	3,000	5,000	10,000	14,500

(b)

Figure 2. Rock Mass Rating (RMR) method for pile design: (a) Calculation of RMR of founding rock mass; and (b) assessment of end-bearing pressure of pile (extracted from GEO (2006)).

3 PLANNING AND SETUP OF PILE LOADING TESTS

3.1 *Planning of test piles*

The use of full-scale pile loading tests has been a promising approach to investigate the pile-rock behavior under high pressure (e.g. Littlechild et al., 2000). Recently, GEO also adopted local pile loading test data to enhance the presumed allowable end-bearing pressure for granitic and volcanic rocks as well as marble (GEO, 2025a & b). As MS rocks have varying properties in terms of rock strength, fractureness, presence of foliation or bedding plane, etc., in this study, it is planned to carry out a systematic pile load test programme targeting on MS rocks of different characteristics in the NM. So far, full-scale loading tests on over ten trial piles at several sites straddling across NM are being undertaken, and data interpretation is on-going. The founding levels of piles are determined by predrills to ascertain the RMR of founding rocks, with a view to covering a range of RMR values from about 34 to 84. Table 2 and Figure 3 summarise the pile loading tests conducted in this study and existing data as well as the brief details of trial piles and RMR of founding rocks. Apart from the pile loading tests, a series of pre-drilling works, post-drilling works, field tests in boreholes, laboratory tests on rock specimens and geophysical survey have been carefully planned to obtain information to support the study. These ground investigation works and laboratory tests are being conducted and the results will be reported later.

Table 2. Summary of pile loading tests on meta-sedimentary rocks in Hong Kong .

Pile No.	Pile diameter[1] (mm)	Rock socket diameter (mm)	Pile length (m)	Rock socket length (m)	RMR of founding rock
YLS-P1	813	750	47.2	1.6	84
YLS-P2	813	750	46.9	0.9	67
LB-P1	1500	1350	90.9	1.5	47
LB-P2	1500	1350	74.1	1.5	45
SR-P1	813	750	28.1	3.8	46
SR-P2	813	750	25.6	3.1	42
SR-P3	813	750	40.1	4.2	39
LMC-P1	1500	1350	39.6	1.8	36
LMC-P2	1500	1350	44	1.6	37
FKS-P1	610	550	54.2	2.2	34
FKS-P2	610	550	37	6.4	40
TSW1-P1	1200	1200	40.3	1.5	59
TSW2-P1	813	-	50.4	Minimum	53

(1) Pile diameter above the rock socket portion.
(2) Test locations: YLS-Yuen Long South; LB-Long Bin; SR-Sandy Ridge; LMC-Lok Ma Chau; FKS-Fung Kong Shan; TSW-Tin Shui Wai.
(3) All are bored piles, except FKS-P1/P2 are rock socketed H-pile.
(4) Test data of TSW1-P1 from Littlechild et al. (2000) and TSW2-P1 from Yau & Lau (2024).
(5) Test piles with minimum rock socket provide test data on end-bearing pressure only.

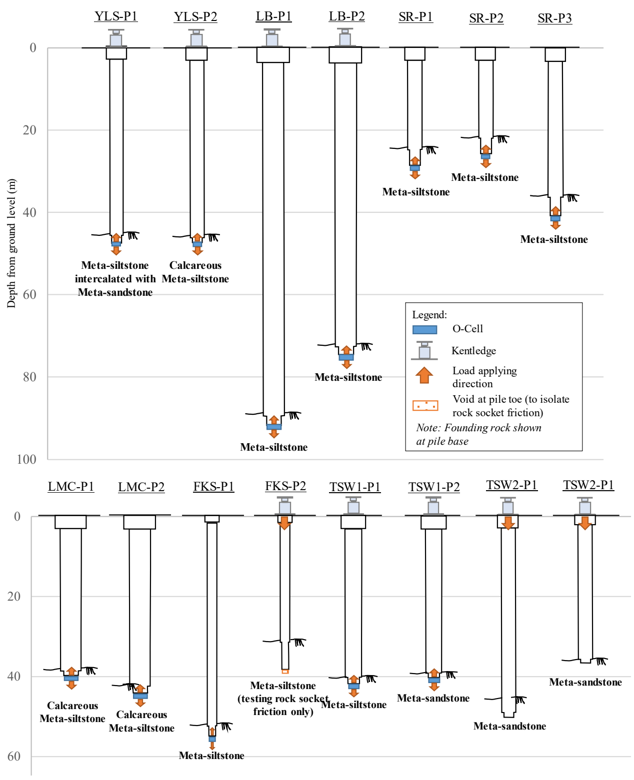


Figure 3. Schematic sectional views of test piles.

3.2 *Pile loading test setup and instrumentations*

As illustrated in Figure 3, a majority of the pile loading tests is conducted using the bi-directional loading system, namely the Osterberg cell (O-cell) (Osterberg, 1984; Schmertmann & Hayes, 1997) installed at the pile base. Kentledge on the ground surface is added if additional reaction force is required. Comparing to conventional top loaded pile loading tests with kentledge, O-cell directly exerts loading on the bearing rock at its base and on the rock socket at its top thereby providing more accurate measurements on the bearing pressure together with the rock socket friction at the same time. This setup overcomes the limitation of the conventional top loading system using kentledge of which the load applied at the top of pile would dissipate along the soil portion (even with sleeving in the soil portion) and the forces being mobilised along the rock sockets and on the pile bases would not only be much smaller than the applied load but also have to be indirectly back-calculated from strain gauges readings. In general, for tests with both O-cell and kentledge, shorter rock socket length (i.e. 0.9 m to 1.8 m) has been adopted, which targeted to test the ultimate rock socket friction. For tests with O-cell but without kentledge, as the reaction to the O-cell is solely provided by rock socket and the soil shaft above, longer rock socket length (i.e. 2.2 m to 4.2 m) is adopted to ensure sufficient reaction is provided to the O-cell for applying load to the bearing stratum. Sleeving has been applied to all test piles in their soil portion, therefore allowing most of the upward load be taken up by the sound rock portion.

All trial piles are heavily instrumented for both force and deformation measurements, as shown schematically in Figure 4. For deformation measurement, the O-cell is equipped with a set of linear vibrating wire displacement transducers (LVWDT) (see pink arrow in Figure 4) to measure its stroke. A series of extensometers (see green arrow in Figure 4) are installed at the top and bottom plates of O-cell, rockhead level and pile top, aiming to measure the gross movement and shortening of the pile as well as the deformation of the bearing rock. For force measurement, a dual strain measurement scheme is adopted where the forces mobilised along the pile shaft are concurrently captured by both strain gauges (see yellow arrow in Figure 4) and fibre optics (see red arrow in Figure 4). Apart from point-wise measurements by strain gauges, this study adopts distributed fibre optic sensing technique to provide a continuous profile of strain measurements results making up the missing data between individual strain gauges. The optic frequency domain reflectometry (OFDR) technology is adopted achieving higher spatial resolution of strain measurements with higher sensing frequency (Lin et al., 2023). Similar technique was also effectively used for monitoring diaphragm wall panels during bulk excavation works in Hong Kong, successfully revealing the hoop stress development (Li et al., 2022).

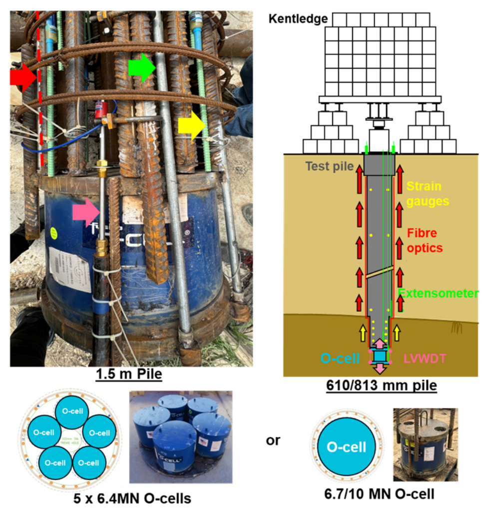


Figure 4. Instrumentation and bi-directional loading system.

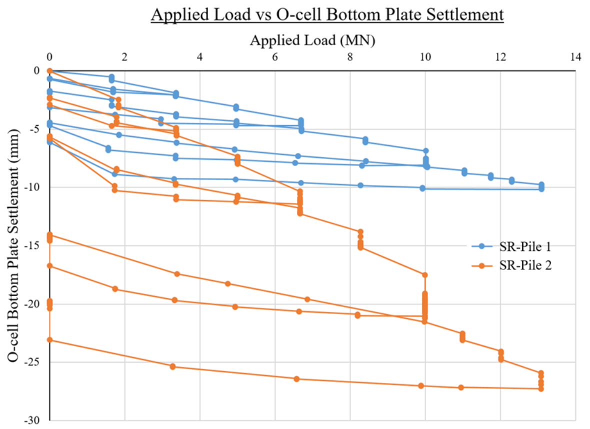
4 PRELIMINARY RESULTS AND DISCUSSIONS

While the pile loading tests and data analysis are still on-going, selected preliminary test results are presented hereafter.

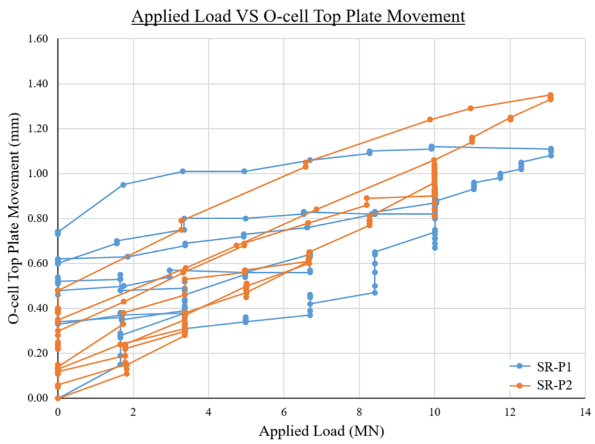
4.1 *Loading-unloading behavior*

In this study, each test pile was subjected to multiple loading-unloading cycles. Preliminary results from two test piles at Sandy Ridge are presented to illustrate the typical loading-unloading behavior of the piles, reflecting both bearing rock behavior under compression (Figure 5a) and pile-rock frictional behavior along the socket length (Figure 5b).

The loads applied from the O-cell during the two tests are plotted against the O-cell bottom plate settlements in Figure 5a. Each test pile was subjected to four loading-unloading cycles up to the maximum loading capacity of the O-cell. In each cycle, the piles were incrementally loaded to a designated test load and then unloaded to zero load. Increasing O-cell bottom plate settlements were observed during the loading increments, while rebound movements occurred during unloading. Residual settlements were noted in each loading cycle, probably due to closures of joints of the founding rock. For each pile, the tangent modulus showed comparable values across the different loading cycles indicating largely elastic behaviour. For these two test piles, the RMR of the rock mass beneath the base of O-cell at pile P1 is 46, while that beneath the base of O-cell of pile P2 is 42. Given the lower RMR, it is not unexpected that the load-settlement curve for P2 exhibited a lower deformation modulus and thus large O-cell bottom plate settlement. Despite the larger settlement observed, the load-settlement curves for pile P2 are comparable to those of pile P1, as both remained relatively linear throughout the testing.



(a)



(b)

Figure 5. Typical loading-unloading behavior of test piles: (a) O-cell bottom plate settlement; and (b) O-cell top plate movement.

The loads applied from the O-cell during the two tests are plotted against the O-cell top plate movements in Figure 5b. Both pile loading test results indicate that the O-cell top plate movements typically increase with the applied load in each loading cycle and rebound during unloading. O-cell top plate movements reflect the behavior of the piles subjected to rock socket shaft friction. Notably, as there was no sudden and continuous upward movement under a constant applied load, no failure of rock socket shaft friction was observed in either of the two pile tests. As shown in the Figure 6, pile P2 exhibited higher O-cell top plate movements and rates of movement in each loading cycle when compared to pile P1. This can be attributed to the differences in rock mass conditions between the two piles.

4.2 *Mobilised end-bearing pressure*

While the pile loading tests and data analysis in this study are still on-going, available results for four test piles from two sites (i.e. Yuen Long South and Sandy Ridge) are selected for discussion. The maximum mobilised end-bearing pressure and the maximum pile base settlement (represented by the lowest point of the pile testing system contacting with the rock, i.e. O-cell bottom plate or pile base) of these tests are summarised in Table 3. Based on literature review, existing pile loading tests for MS rocks (located at Tin Shui Wai) have been retrieved from Littlechild et al. (2000) and Yau and Lau (2024), and the corresponding results (i.e. TSW1-P1 and TSW-P2) are included in Table 3 for comparison.

In brief, all test piles successfully achieved the designed test load with minimal pile settlements (most of which less than 1.3% of the pile base diameter). Higher pile base settlement was observed at SR-P2 which founding rock has the lowest RMR of about 42. The maximum mobilised end-bearing pressures varied between approximately 16,000 kPa and 29,000 kPa. However, these values do not represent the ultimate end-bearing capacity of the test piles as their overall loading behaviors are still within the linear range (see Figure 6). In fact, it is important to note that the maximum test load was constrained by the capacity of the testing equipment.

Table 3. Summary of mobilised end-bearing pressure of selected pile loading tests on MS rocks in Hong Kong.

Pile No.	Mobilised end-bearing pressure	Maximum pile base settlement	RMR of founding rock
YLS-P1	~27,000 kPa	~4 mm (0.5% dia.)	84
YLS-P2	~29,000 kPa	~8.3 mm (1.1% dia.)	67
SR-P1	~29,000 kPa	~10 mm (1.3% dia.)	46
SR-P2	~29,000 kPa	~27 mm (3.6% dia.)	42
TSW1-P1	~26,000 kPa	~14 mm (1.2% dia.)	59
TSW2-P1	~16,000 kPa	~9 mm (1.2% dia.)	53

By plotting the mobilised end-bearing pressure at each loading increment against the corresponding pile base settlement, the pile loading behaviors of the above-mentioned tests are compared in Figure 6. As can be seen, the load-settlement behavior of the test piles in this study, similar to other existing test results on MS rocks, remains within a nearly linear range, showing no signs of plastic failure. In general, the higher the RMR value of the founding rock mass, the stiffer is the pile loading behavior. Also plotted on the same graph are the loading test results of piles founded on Granite and Tuff, both retrieved from Littlechild et al. (2000). While the pile loading behaviors on MS rocks appear to be more variable, they do not have notable difference with that on Granite and Tuff, which are two dominant types of rock in Hong Kong.

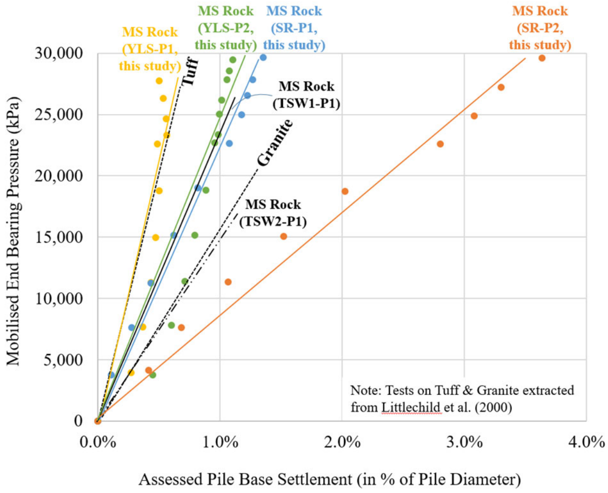


Figure 6. Load-settlement behavior of test piles and comparison with existing data on different types of rocks.

The maximum mobilised end-bearing pressure of the MS rocks is plotted against the RMR at the bearing stratum in Figure 7, together with the presumed allowable end-bearing pressure for MS rocks (i.e. 3,000 kPa) and the design line based on the RMR method in prevailing local practice. Despite that ultimate end-bearing pressure was not reached in any of the pile loading test, it is worth to note that the maximum allowable end-bearing pressures mobilised in the tests are at least 5 times higher than current presumed design value. Based on the test results obtained so far, there is apparently room for enhancing the prevailing design guideline, subject to more pertinent test results in the future.

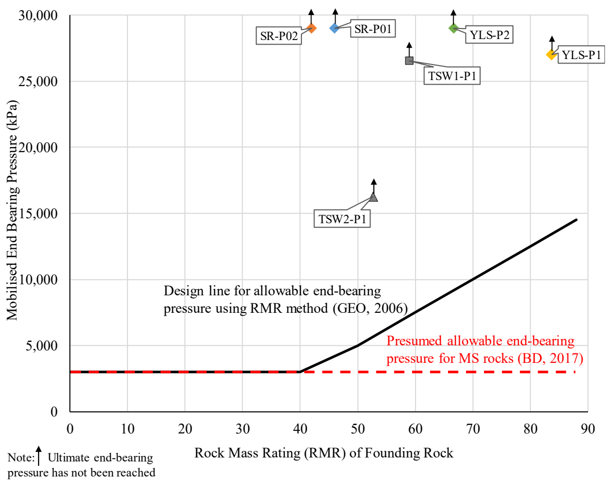


Figure 7. Maximum mobilised end-bearing pressure plotted against RMR of the founding rock mass and comparison with prevailing design guideline.

Since the ground investigation and laboratory tests associated with the pile loading tests are still in progress, the results involving different rock mass properties (such as fracture state, RMR, rock strength, etc.) will be analyzed further and presented separately in the future.

4.3 *Mobilised average rock socket shaft friction*

Apart from end-bearing pressure, bi-directional loading test conducted at the pile base also allows the shaft friction mobilised along the rock socket be obtained. Table 4 outlines the preliminary findings for the four selected test piles from Yuen Long South and Sandy Ridge. In addition, two previously conducted pile loading tests for MS rock and Tuff (Littlechild et al., 2000) are included for comparison. The mobilised average rock socket shaft friction along the rock socket of these tests, which are derived from strain gauge readings, are also plotted against their socket movement in Figure 8.

Table 4. Summary of mobilised average rock socket shaft friction along the rock socket of selected pile loading tests on MS rocks in Hong Kong.

Pile No.	Mobilised average rock socket shaft friction	Maximum socket movement	Rock socket length
YLS-P1	~3,000 kPa	~6 mm (0.7% dia.)	1.5 m
YLS-P2	~4,700 kPa	~7 mm (1.0% dia.)	0.9 m
SR-P1	~1,200 kPa	~1 mm (0.2% dia.)	3.8 m
SR-P2	~1,800 kPa	~1 mm (0.2% dia.)	3.1 m
TSW1-P1	~6,000 kPa (ultimate capacity)	~10 mm (0.8% dia.)	1.5 m
Test pile on Tuff	~3,900 kPa	~4 mm (0.3% dia.)	2.0 m

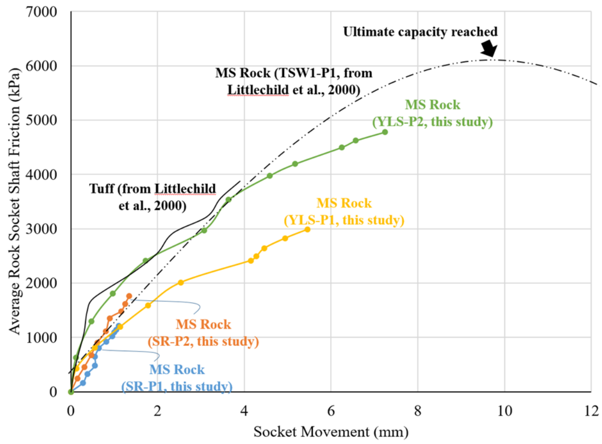


Figure 8. Mobilised average rock socket shaft friction of test piles and comparison with existing data on different types of rocks.

As shown in Figure 8, upon exertion of loading by the O-cell at the pile base, the mobilised average rock socket shaft friction tends to increase with greater upward socket movement during loading increments. For the four test piles (conducted by this study) presented, no ultimate average rock socket shaft friction was reached. Based on literature review, there is one test pile socketed in MS rock (TSW1-P1), has achieved its ultimate average rock socket shaft friction of about 6,000 kPa. Upon further load increment, a noticeable decline in rock socket friction is observed alongside a significant increase in socket movement. Based on the test results, the maximum mobilised average rock socket shaft friction for these test piles in MS rocks varied between approximately between 1,200 kPa and 6,000 kPa (such large variation is mainly due to the provision of different rock socket lengths). Again, the maximum test load was limited by the capacity of the testing equipment. The rock socket load-movement behaviors are comparable with that exhibited by a test pile founded in Tuff (Littlechild et al., 2000). The preliminary results of this study indicate that an average rock socket friction of at least 1,200 kPa can well be attained in the pile loading tests, while existing literature shows that average rock socket friction in MS rock can reach up to 6,000 kPa. This suggests a potential to improve the presumed allowable rock socket friction values of 300 kPa (for compression or transient tension) and 150 kPa (for permanent tension) as recommended in in prevailing design guideline in Hong Kong.

5 CONCLUSIONS

A comprehensive program of full-scale pile loading tests is currently being carried out at various locations within the NM of Hong Kong. This region is rich in MS rocks which exhibit a range of engineering geological characteristics as a results of the complex effects of sedimentation and metamorphism. Preliminary findings from pile loading tests at Yuen Long South and Sandy Ridge, combined with existing local test data, indicate the potential for enhancing the current foundation design guidelines for MS rocks. This includes increasing the design values for allowable end-bearing pressure and allowable rock socket shaft friction. Given the heterogeneous nature of MS rocks, and in light of forthcoming results from other pile loading tests as well as the associated field and laboratory testing results of which the data interpretation and analysis are underway, there is a significant opportunity to enhance the foundation design guidelines for MS rocks. Such improvements could yield considerable savings in both cost and time for foundation projects, while also contributing to material conservation and overall sustainability. In fact, some outcomes from the pile loading tests conducted in this study have already been utilised in specific housing development projects to enhance their foundation designs, highlighting the immediate advantages of this study.

6 ACKNOWLEDGEMENTS

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