Load Distribution Behaviour of Bored Pile in Various Soil Formation: Rock Socket in Limestone, Schist and Sandstone

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ABSTRACT

Rock socketed bored pile is a solution when the load from the structure is very high and/or accessible bearing surface has an inadequate bearing capacity. The study is based on instrumented bored pile socketing into different types of rock namely. limestone, schist and sandstone at three sites. The result for three (3) test piles namely PTP1, UTP-1 and TP2 shows most of the load are resisted by friction rather than end bearing at the pile working load. The load apportioned to end bearing at higher loads varies for the three test piles. Comparison of observed mobilised skin friction in the rocks with empirical methods indicates that prediction values from Williams and Pells [1] over design for two out of the three test piles and that by Hovarth [2] are under design for two out of the three test piles.

Keywords: Empirical Methods, Instrumented Bored Pile, Rock Socket, Shaft Resistance.

1.0 INTRODUCTION

Pile foundations are used to support heavily loaded structure such as high rise buildings and bridges. Bored piles are commonly used in Malaysia due to its low noise, low vibration and flexibility of sizes to suit different loading conditions and subsoil conditions.

Rock socketed bored pile is a solution when the load from the structure is very high and/or accessible bearing surface has an inadequate bearing capacity. It may be necessary to drill a shaft into the underlying rock and construct a socketed pile. The support provided by socketed bored pile comes from the shear strength around the shaft and the end bearing at the toe of the pile. Many researchers have investigated the behavior of rock socketed bored pile and relate the uniaxial compressive strength (UCS) of intact rock surrounding the pile to the shaft resistance of the pile without considering the rock mass quality (Rosenberg and Journeaux, 1976) [3].

Pile testing is a fundamental part of the pile foundation design. A pile load test is normally carried out to assess the geotechnical capacity of piles in the foundation system and as a tool to check the integrity of constructed pile and prediction of foundation settlements. In design, the concern is over what portion of the capacity is obtained at the pile toe and what is the shaft resistance in the specific soil layers. Therefore, when the purpose of the test is to provide data for design of a piled
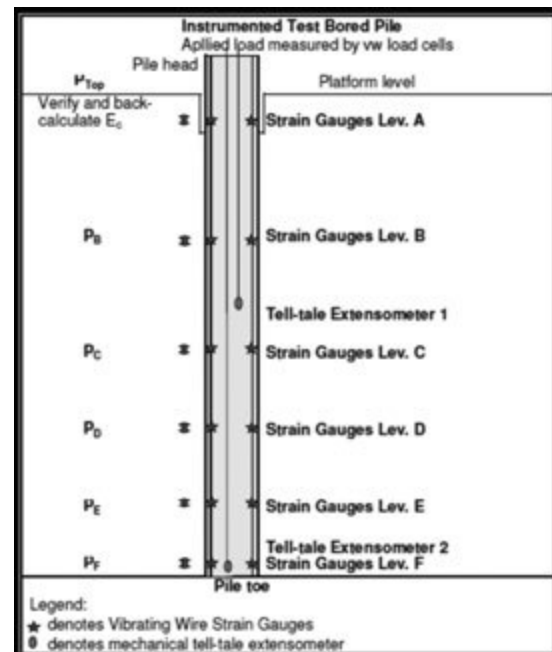

Figure 1: Schematic of Pile Instrumentation.

foundation then the pile must be instrumented in order to determine the load transfer (resistance distribution) such as shown in Figure 1.

The objectives of this study are: -

- To study the behaviour of pile settlement under applied load.
- To determine the bearing capacity of pile and its apportionment into end bearing and shaft friction.
- To compare the behaviour of piles socketing into different type of rocks.

The study is based on case study of three (3) instrumented test bored piles at three actual developments. Data was collected to analyse and compare the behaviour of test pile socketing into different type of rocks in Malaysia. Vibrating wire strain gauges were installed in the test piles to reveal the load transfer behaviour along the pile. Extensometer was installed in test bored piles to observe the pile structural shortening but it is outside the scope of this paper.

1.1 Geotechnical Capacity of Bored Piles

The design of bored pile is normally based on the results of Standard Penetration Test (SPT-N) conducted in the borehole. In designing the pile, the empirical approach of unit skin resistance (f_s) and unit base resistance (f_b) is taken as:

$$f_s = K_s \text{ x SPT-N (in kPa)} \tag{1}$$
$$f_b = K_b \text{ x SPT-N (in kPa)} \tag{2}$$

Where K_s is shaft resistance coefficient and K_b is base resistance coefficient which varies according to soil type.

In current practice, these empirical formulas have been widely used for pile capacity calculation. Both the friction resistance and end bearing resistance are considered in design with an overall factor of safety 2.0 and 3.0 respectively. The design is an estimate thus it is important to understand the actual mobilisation of skin friction and end bearing with the pile movement. The data obtained from the instrumented static load test results can be used to verify the designed piled and the true load transfer behaviour of the bored piles can be observed.

Bored pile socketed in rocks can be expected to have higher pile capacity due to the higher unit friction resistance between the pile and the rock. Table 1 summarizes the typical design socket friction values for various rock formations in Malaysia.

Table 1: Summary of Rock Socket Unit Friction Design Values.

Rock Formation	Allowable Rock Socket Unit Friction	Source
Limestone	300 kPa for RQD < 25%	Neoh [4]
	600 kPa for RQD = 25% to 70%	
	1000 kPa for RQD > 70%	
	The above design values are subjected to 0.05 x minimum of (q_{uc}, f_{cu}) whichever is smaller.	
Sandstone	0.10 x q_{uc}	Thorne [5]
Shale	0.05 x q_{uc}	Thorne [5]

Various other researchers have also developed more systematic approaches in rock socket design [1,3,6]. The following expression is used to compute the rock socket unit friction with consideration of the strength of intact rock and the rock mass effect due to the discontinuities.

$$F_s = \alpha \text{ x } \beta \text{ x } q_{uc} \tag{3}$$

Where q_{uc} is the unconfined compressive strength of intact rock
α is the reduction factor with respect to q_{uc} (Figure 2).
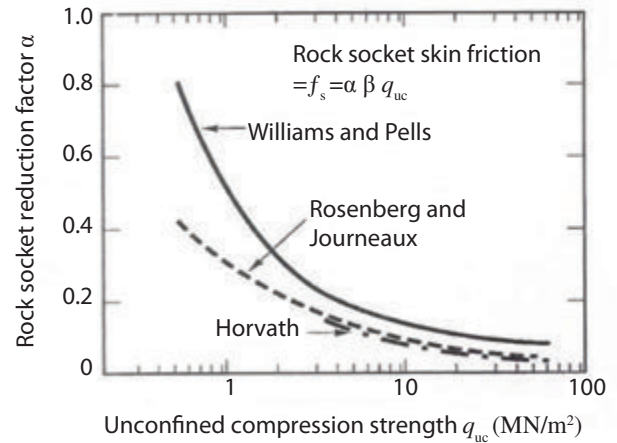β is the reduction factor with respect to the rock mass effect (Figure 3).

Figure 2: Rock Socket Reduction Factor, α versus Unconfined Compressive Strength. (after Tomlinson, [6]).
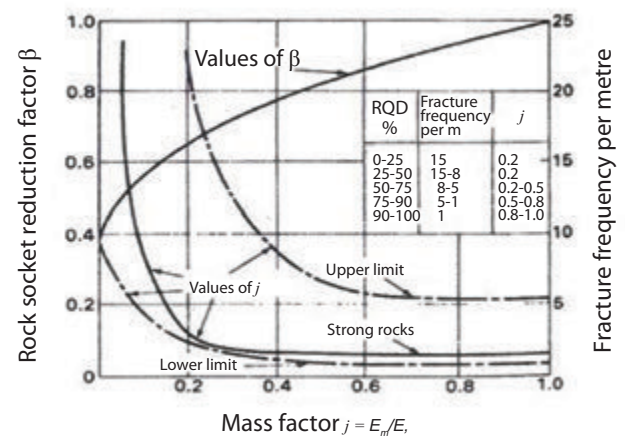

Figure 3: Rock Socket Reduction Factor, β, versus Rock Mass Discontinuity (after Tomlinson [6]).

2.0 SUBSOIL STRATA, PILE INSTALLATION AND INSTRUMENTATION

2.1 Site A

The site is located at Ipoh, Perak. The area is underlain by an extensive limestone bedrock formation namely the Kinta Limestone. The limestone bedrock rises above the alluvial plains to form limestone hills with steep to vertical slopes. The subsoil strata based on nearest borehole is shown in Figure 4.

PTP1 test pile of 1050mm diameter and 8.8m long is socketed into moderately strong limestone bedrock at depth 4.3m to 8.3m (4.0m length). Based on the nearest borehole data on site, the Rock Quality Designation (RQD) of the rock is between 54%

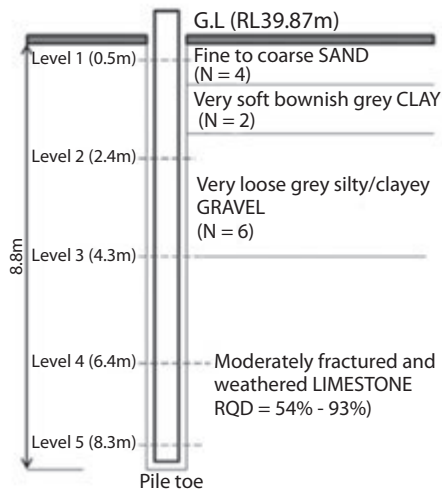*Figure 4: Subsoil Strata and Pile Instrumentation Levels for PTP1.*

Figure 5: Subsoil Strata and Pile Instrumentation Levels for UTP-1.

to 93 % within Unconfined Compressive Strength (UCS) of 35 MPa.

Twenty (20) nos. of Geokon vibrating wire strain gauges (VWSGs) were installed in the test pile to measure strain at nominated locations From Level 1 to Level 5. Each level consists of four (4) nos. of VSWG. There were five (5) nos. of tell-tale extensometers installed at the five (5) levels (one for each level), corresponding to Level 1 to Level 5 from ground respectively. A polystyrene foam soft toe was installed at the base to eliminate end bearing contribution since end bearing was not considered in the design geotechnical capacity due to uncertainty of proper base cleaning during construction.

Maintain Load Test (MLT) was proposed to be carried out in three (3) cycles: first cycle with working load of 750tonnes, second cycle was twice working load of 1500tonnes and the third cycle was 2250tonnes. However, the third cycle was not completed as the pile failed during the step of loading from 1875tonnes (2.5 x working load) to 1950tonnes (2.6 x working load).

2.2 Site B

The proposed development is situated at Mukim Setapak, Daerah Gombak, Selangor where the geological formation consists of schist, phylite slate and sandstone. Soil profile based on nearest borehole is shown in Figure 5.

The test pile UTP-1 was a 1000mm diameter bored pile with embedded length of 16.7m below ground level. The pile was debonded by pre-augering the soil surrounding the the pile up to 13.5m depth. The debonding was conducted in order to observe the load distribution within the socketed depth when no friction resistance is provided by the upper soil.

At depth of 13.5m to 16.5m, the test pile UTP-1 was socketed 3.0m into schist rock. The nearest borehole data shows that RQD of the rock falls between 7% to 17% and the average UCS is 17 MPa.

Pile instrumentation consisted of twenty-four (24) nos. VWSG at six (6) different levels and three (3) nos. of telltale extensometers.

Loading were carried out in three (3) cycles: first cycle with working load of 650tonnes, during the second cycle the maximum
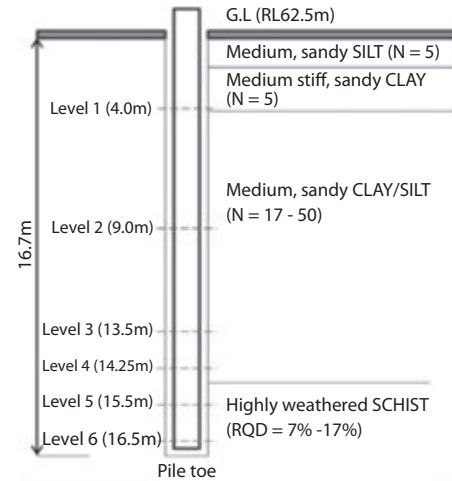
load was 1300tonnes (2.0 x working load) and during the third cycle the maximum load was 1950tonnes (3.0 x working load).

2.3 Site C

The site is located at Kuala Lumpur and is underlain by Kenny Hill Formation which is a sequence of clastic sedimentary rocks consisting of interbedded shale, mudstone and sandstones The Kenny Hill material is basically a completely decomposed rock and generally sandy SILT soil. Based on the nearest borehole at the site, the ground profile is shown in Figure 6.

TP2 test pile (900mm diameter) is socketed into sandstone bedrock at depth 10.0m to 15.0m (5.0m length). Based on rock coring and compressive test results from nearest borehole, the Rock Quality Designation (RQD) of the rock falls between 29.3% to 44.6% with UCS of 20 MPa.

Pile instrumentation consisted of twenty-eight (28) nos. VWSG at seven (7) different levels and four (4) nos. of telltale extensometers.

The load test was carried out in four (4) cycles: first cycle with working load of 6000kN, second cycle with maximum load of 7500kN (1.25 x working load). During the third cycle the maximum load was 9000kN (1.5 x working load) and during the fourth cycle the maximum load was 15,000kN (2.5 x working load).
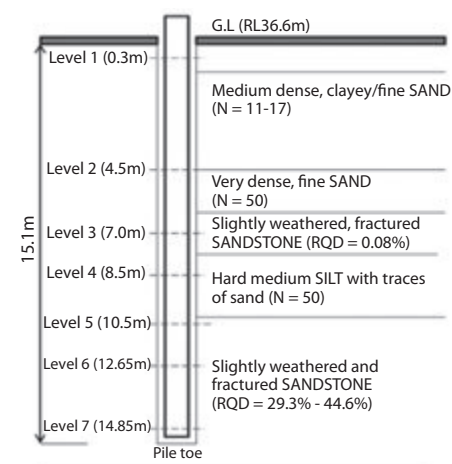

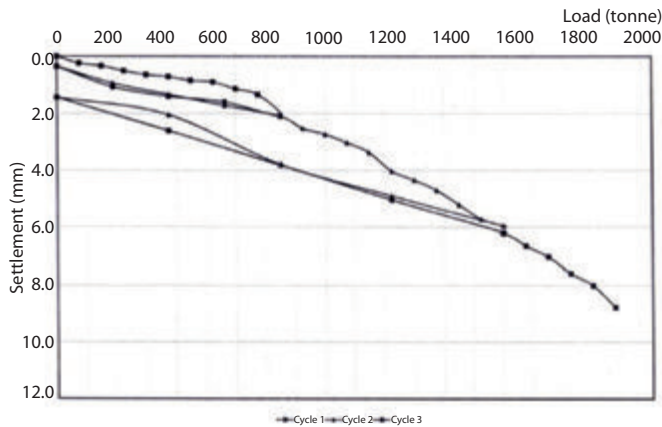*Figure 6: Subsoil Strata and Pile Instrumentation Levels for TP2.*

Figure 7: Load Settlement Curves for PTP1.

3.0 RESULTS AND DISCUSSION

3.1 Site A

Figure 7 shows the Load Settlement Behaviour of the Pile and Table 2 summarises the settlement behaviour.

Table 2: Settlement of Pile Top for PTP1.

Loading Cycle	Max Load (tons)	Max Settlement (mm)	Residual Settlement (mm)	Elastic Rebound (%)
1st	750.00	2.10	0.40	80.95
2nd	1500.00	6.00	1.40	76.67
3rd	1875.00	8.80	N/A	N/A

It can be seen that maximum pile top settlement was recorded at 8.80mm during the 3rd loading cycle when the maximum load of 1875tons was applied. It must be noted that the full program of loading steps for 3rd loading cycle could not be completed as the pile failed during the step of loading from 1875tons (2.5 x Working Load) to 1950tons (2.6 x Working load).

The results show that the settlement was 2.1mm (0.2% of the pile diameter) at pile working load and 6.0mm (0.57% of the pile diameter) at two times working load. It also shows that at working load the pile gives an elastic rebound of 80.95%.

Readings from the strain gauges were analysed to determine the load distribution behaviour and the mobilised unit friction and unit end bearing during the sequence of loading. The results are shown in Table 3 and Figure 8.

It is noted that the rock socket start from depth 4.3m to 8.3m. Table 3 tabulates the load distribution along the pile shaft and pile base. It shows that only about 3tons to 6tons (0.32% to 0.4%) of the applied load was carried by end bearing throughout

Table 3: Summary of Load Distribution for PTP1.

Pile depth (m)	Load taken by surrounding soil (tons)		
	1.0 x Working Load (750 tons)	2.0 x Working Load (1500 tons)	2.5 x Working Load (1875 tons)
0-0.5	0.0	0.0	0.0
0.5-2.4	112.0	164.0	195.0
2.4-4.3	23.0	70.0	248.0
4.3-6.4	487.0	1123.0	1265.0
6.4-8.3	125.0	138.0	162.0
End Bearing	3.0	5.0	6.0

the whole range of applied load. The small amount of load at the pile base is probably due to the installation of polystyrene foam soft toe. The soft toe was installed as to minimise the load interference from the pile base (the end bearing was neglected in design consideration).
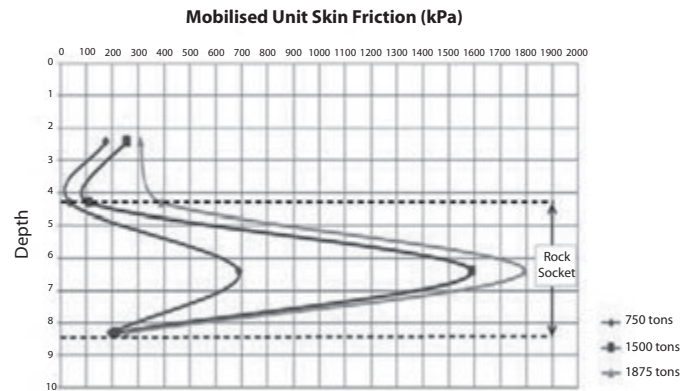

Figure 8: Mobilised Unit Skin Friction for PTP1.

Based on Figure 8, the chart shows that the maximum mobilised skin friction are at Level 3 to Level 4 with maximum value of 689kPa (1st loading cycle), 1590kPa (2nd loading cycle) and 1790kPa (3rd loading Cycle). Since, this pile was tested to fail, the maximum mobilised skin friction of 1790kPa is considered as ultimate value for the limestone of fair to good rock quality.

3.2 Site B

The Load Settlement Behaviour of the Test Pile is shown in Figure 9 and the settlement is summarised in Table 4.
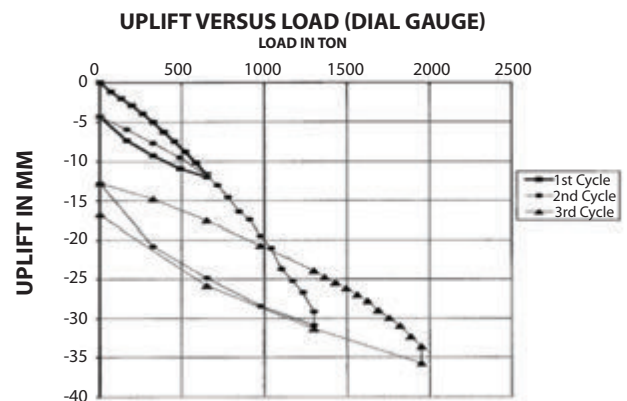

Figure 9: Load Movement Curves for UTP-1.

Table 4: Settlement of pile top for UTP-1.

Loading Cycle	Max Load (tons)	Max Settlement (mm)	Residual Settlement (mm)	Elastic Rebound (%)
1st	650.00	11.95	4.31	63.92
2nd	1300.00	30.88	12.75	58.71
3rd	1950.00	35.77	16.77	53.12

The test pile UTP-1 did not fail after loading up to three (3) times the working load. The pile top settlement was recorded at 11.95mm (1.2% of pile diameter), 30.88mm (3.1% of pile

diameter) and 35.77mm (3.6% of pile diameter) at test load of 650tons, 1300tons and 1950tons respectively.

Table 5: Summary of Load Distribution for UTP1.

Pile depth (m)	Load taken by surrounding soil (tons)		
	1.0 x Working Load (650 tons)	2.0 x Working Load (1300 tons)	3.0 x Working Load (1950 tons)
0-4.0	8.6	8.7	8.3
4.0-9.0	29.2	15.7	15.3
9.0-13.5	27.2	26.8	27.5
13.5-14.25	278.8	292.7	292.7
14.25-15.5	247.6	521.6	521.5
15.5-16.5	39.6	312.1	422.5
End Bearing	18.9	122.4	662.2

The load transfer distribution and mobilised skin friction and end bearing is shown in Table 5 and Figure 10 respectively.

It is noted that the rock socket start from depth 13.5m to 16.5m. Based on Table 5, it can be deduced that from 0m to 13.5m depth of pile, only a small amount of applied load which are 8.3tons to 27.5tons was distributed to the surrounding soil, due to the debonded section. Therefore, smaller load was recorded at depth up to 13.5m. At depth 13.5m and below, most of the load was taken by the rock socket.

It also shows that some percentage of loads was distributed to the pile base. The load distribution for end bearing was 18.9tons (2.9%) at normal working load, 122.4tons (9.2%) at two times working load and 662.2tons (34.0%) at three times working load. The trends of linearly increasing load transfer along the shaft and base resistance during maximum loading (three times of working load) indicates that ultimate shaft and base resistance were not fully mobilised at working load and that a settlement of 3.6% pile diameter was required to mobilised the end bearing to a significant value. This justify the practice of ignoring the end bearing in geotechnical capacity estimation.
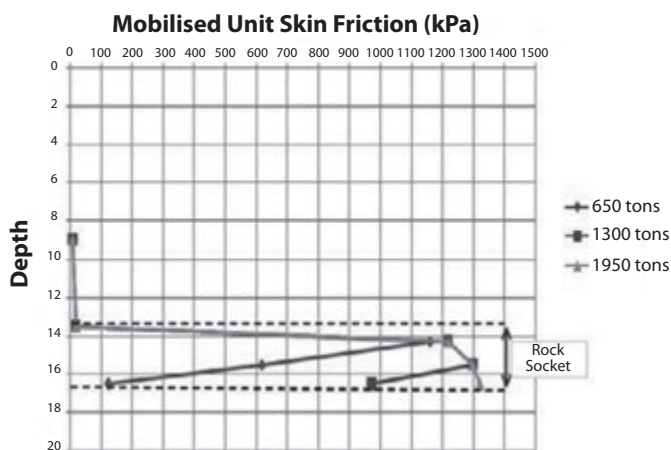

Figure 10: Mobilised Unit Skin Friction for UTP-1.

Figure 10 shows that the maximum mobilised skin friction is at 3rd loading cycle with maximum value of 1220kPa (Level 3 to Level 4), 1300kPa (Level 4 to Level 5) and 1320kPa (Level 5 to Level 6). It can be suggested that a value of 1300kPa may be considered as ultimate unit friction value for this very poor quality schist.

3.3 Site C

Readings from the strain gauges were analysed to determine the load distribution behavior and the mobilised unit friction and unit end bearing during the sequence of loading. The results are shown in Table 6 and Figure 11.
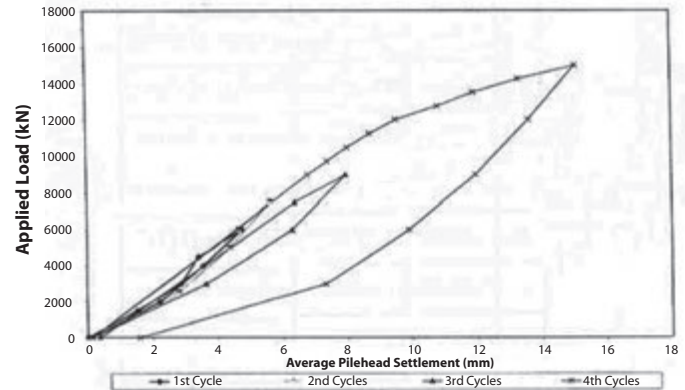

Figure 11: Load Movement Curves for TP2.

Table 6: Settlement of pile top for TP2.

Loading Cycle	Max Load (tons)	Max Settlement (mm)	Residual Settlement (mm)	Elastic Rebound (%)
1st	600.00	4.74	0.14	97.05
2nd	750.00	5.62	0.37	93.42
3rd	900.00	7.96	0.13	98.37
4th	1500.00	15.07	1.58	89.52

The pile top displacement (settlement) were recorded at 4.74mm (0.5% of diameter pile) at test load 600tons, 5.62mm (0.6% of pile diameter) at applied load 750 tons, 7.96mm (0.88% of pile diameter) and 15.07mm (16.7% of the pile diameter) at applied load of 900tons and 1500tons respectively. Table 6 also shows the higher percentage of elastic rebound is between 89.52% (applied load of 1500tons) to 97.05% at applied load of 600tons. The test pile TP2 was loaded up to 2.5 times working load and did not fail and the settlement was only 15.07mm. It indicates that the pile still can behave well if imposed load is more than that.

Pile Depth (m)	Load taken by surrounding soil (tons)			
	1.0 x Working Load (600 tons)	1.25 x Working Load (750 tons)	1.5 x Working Load (900 tons)	2.5 x Working Load (1500 tons)
0-0.3	0.0	0.0	0.0	0.0
0.3-4.5	68.2	28.0	39.2	99.8
4.5-7.0	100.3	185.3	206.4	356.3
7.0-8.5	71.7	108.6	125.9	199.3
8.5-10.5	118.3	185.4	217.0	330.5
10.5-12.65	205.2	227.0	264.5	435.9
12.65-14.85	17.8	12.4	13.7	28.6
End Bearing	18.5	28.5	33.3	49.6

It is noted that rock socket is from 10.5 to 14.85m depth. As shown in the table, only a small portion of applied loads about 18.5tons to 49.6tons (3.1% to 3.8%) were transferred to the pile base and most of the load was distributed to the surrounding soil and rock socket shaft. The ultimate shaft and base resistance were not fully mobilised at the pile working load as the load transfer along the shaft and the base still shows the trend of linearly increasing during maximum loading (2.5 times working load).
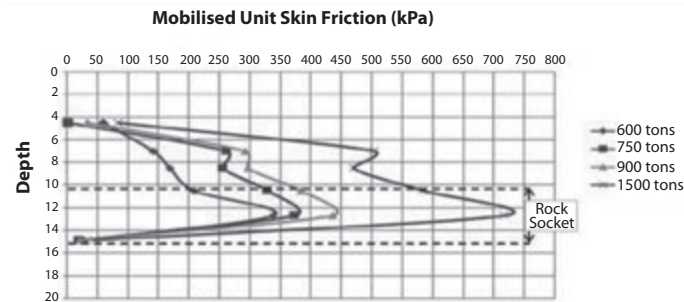

Figure 12: Distribution of Mobilised Unit Skin Friction for TP2.

Figure 12 shows that the maximum mobilised skin friction is Level 5 to Level 6 with maximum value of 338.0kPa (at 600tons), 373.0kPa (at 750tons), 435.0 kPa (at 900tons) and 717.0 kPa (at 1500tons). Based on these results it may be taken that the ultimate unit friction in the poor quality sandstone is around 700kPa.

3.4 Prediction of Ultimate Unit Skin Friction in Rock Socket

Various researchers have proposed numbers of empirical and semi-empirical design methods on rock socketed piles, most of them compute the ultimate unit skin friction based on average unconfined compressive strength (UCS) of the rock mass and applying reduction and correlation factors. In order to examine the applicability of these methods, their prediction values of ultimate unit skin friction in the rock socket are compared with the observed maximum unit skin friction values obtained from Site A (PTP1), Site B (UTP-1) and Site C (TP2).

Therefore, in order to determine the prediction value of each researchers noted in Table 8, the average Rock Quality Designation (RQD) and UCS from the nearest borehole data were used in the estimation. The value of Rock Socket Reduction Factor, α and Rock Socket Correlation Factor, β can be obtained from Figure 2 and Figure 3 respectively. Table 8 present the

Table 8: Prediction of Ultimate Shaft Friction by Various Researchers.

Pile	RQD %	UCS (kPa)	Ultimate Unit Skin Friction, f_s (kPa)			Observed maximum unit shaft friction
			Rosenberg and Journeaux (1)	Horvath (2)	Williams and Pells (3)	
PTP1	54-93	35,000	1505.0	1204.0	2408.0	1790.0
UTP-1	7-17	17,000	884.0	773.5	1326.0	1320.0
TP2	29-45	20,000	1040.0	780.0	1430.0	717.0

(1), (2), (3) f_s is calculated from $f_s = \alpha \times \beta \times q_{uc}$ where α, β from respective charts

summary of the comparison between predictions with the observed maximum value of rock socket friction on site.

It can be deduced that for test pile PTP1, the observed maximum unit shaft friction of 1790 kPa was an ultimate resistance since the pile is loaded to failure. Rosenberg and Journeaux [3] method gives the nearest ultimate value of 1505 kPa.

With regard to UTP-1, method proposed by Williams and Pells [1] gave the nearest accurate ultimate skin friction of 1326.0kPa compared to observed value of 1320.0kPa. The other methods, gave quite lower value compared to the observed skin friction. Since the estimated skin friction is lower than the actual friction of the in situ rock, it can be assumed that those predictions by Rosenberg and Journeaux [3], and Horvath [2] methods are under design of skin friction, fs.

Whilst for TP2, Williams and Pells [1] method gave an ultimate value of 1430.0kPa, and Rosenberg and Journeaux [3] method gave a value of 1040.0kPa. Method by Horvath [2] gave the nearest ultimate skin friction (780kPa) compared to observed shaft friction value (717kPa).

It can be seen that for each test pile certain method over design, under design or predict closely the observed values.

It is noted that the ratio of ultimate mobilised unit friction for TP2 (sandstone) over maximum load is lower than those for UTP-1 (schist formation) even though the RQD and UCS is much better than those for the schist formation. It indicates that type of rock affects the friction at the shaft interface.

4.0 CONCLUSION

The performance of test pile PTP1, UTP-1 and TP2 shall be deemed to have satisfied the requirements of the JKR Standard Specification for pile head settlement where at design working load, the total settlement of the test piles did not exceed 12.5mm and when loaded to twice working load, the total settlement of the pile head did not exceed 38mm or 10% of the pile diameter whichever is lower. After removal of the designed working load, the residual settlement did not exceed 6.5mm and after removal of the test load at twice working load, the residual settlement did not exceed 20mm.

The test piles mainly utilised the frictional resistance to support the design capacity of pile with factor of safety at least 2.0. End bearing resistance is only mobilised from two to three times working load.

In most bored pile design, base resistance of bored pile is usually ignored due to uncertainties of base cleaning. The results in this study show that even if base cleaning were properly done very little end bearing resistance is utilised at pile working load. This could be a technical justification to disregard end bearing resistance for bored pile.

Comparison of rock skin friction from various methods with the observed values on site shows that lower value than actual skin friction is considered as under design. While the higher value than actual skin friction is considered as over design. This means that prediction of ultimate values from Horvath [2] is most conservative and that by William and Pells [1] is most liberal for those three (3) test piles.

The trend of mobilised skin friction and end bearing is similar for all test piles indicating that it is not affected by type of geological formation however the magnitude is dependent on the type of rock, strength and quality. ∎

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PROFILES

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In 2007 her career came full circle when she joined the Faculty of Civil Engineering, Universiti Teknologi MARA as a Lecturer. Now she enjoys teaching and working with students.

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