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Skin friction of driven piles in weak rock

Pile Running Problème et modélisation de grande déformation

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ABSTRACT: Driven piles are regularly used for many applications in various ground conditions, including sites with weak rock. Unlike driven piles in sands and clays, there is limited literature or data on the developed skin friction of driven piles in weak rock. The lack of published data and commonly agreed recommendations and guidance on a method of prediction make it challenging to reliably assess the axial capacity of driven piles in rock. This paper discusses a case study where open-ended steel tubular piles were driven into weak rock. The onsite pile testing data obtained using high-strain dynamic pile testing method (pile driving analyser (PDA) testing) was compared to the theoretical predictions made using a published method of calculation. The dynamic pile tests indicated a large variation in the inferred skin friction achieved in rock, making comparison to predictions difficult. The project data was also compared with the range of skin friction values from driven piles in rock obtained from limited number of past publications. The various potential factors that could impact the results were discussed.

KEYWORDS: driven piles, weak rock, skin friction, piled foundations, pile driving.

1 INTRODUCTION

Driven piles are regularly used for a wide range of applications in various ground conditions, including sites where weak rock is present. On those occasions, piles driven to some depths in the upper weak rock can be required. Common examples are steel tubular piles or steel H-piles being driven some distance into rock.

This paper discusses the axial geotechnical capacity of steel piles driven into weak rock; in particular, the developed skin friction. Unlike bored piles, there is limited literature or data on the skin friction of driven piles in weak rock. This is perhaps due to the less common nature of a driven pile design in rock as mentioned above. The lack of published data and commonly agreed recommendations and guidance on a method to predict skin friction makes it challenging to reliably assess the axial capacity of driven piles in rock.

This paper describes a case study involving open ended steel tubular piles driven into weak rock; and presents the skin friction data obtained from the subsequent pile dynamic load tests (PDA). Estimation of the skin friction in driven piles in rock, using available empirical recommendations, was performed and compared with the measured resistance obtained from the dynamic load tests. Comparisons with published data, and discussions of the results, as well as various relevant rock information will also be presented. The main aim of this paper is to contribute to the existing database and understanding of the topic, the potential limitations and practical considerations in relation to assessment of driven piles in weak rock.

2 CASE STUDY

2.1 Context

The project involved some major upgrade works associated with a port used for freight shipping in Australia, where the existing port infrastructure was required to be modified to accommodate future vessels of up to 210m in length. As a result, construction of new mooring and berthing dolphins was needed as part of the port upgrade. The dolphins were constructed at various locations within the project site; but were within 300 m of each other. Driven steel tubular piles were used for these new dolphin foundations.

Due to the presence of rock at the site (see Section 2.2), rock sockets below the toe of the driven tubular dolphin piles were

designed to provide the required pile geotechnical axial capacities (tension and compression), and lateral pile fixity and capacity. The rock socket was constructed by driving the steel tubular piles to a nominal design length into the weak rock (5m was originally assumed, based on the outcomes of a high-level pile drivability assessment), or refusal; after which the drill-out of the internal soil and rock plug was performed until the design rock socket length (below the steel pile toe level) was achieved. The reinforcement was then installed, and concreting works were conducted to complete the pile construction. As a result of the requirement for the large design pile tension and compression capacities, and the shallow rock depth at the site, large rock socket lengths (well in excess of 5m in cases) were required for the dolphin piles.

Due to the extremely tight deadline for the project, any reduction in the pile socket length would be of significant benefit to the project in terms of both construction time and cost, and minimising geotechnical risks due to having long unsupported drilled rock sockets. As such, there was an interest in understanding of the likely skin friction that could be developed in the section of the driven steel piles in the weak rock, and how its contribution the overall pile axial capacity should be considered in reducing the required rock socket length.

2.2 Ground conditions

The water depths at the dolphin pile locations were generally between 7 m to 12 m. The seabed ground conditions at the site were assessed through intrusive geotechnical investigations performed as part of the project, and historical information from past constructions. The ground below seabed was found to have shallow rock, generally consisting of approximately 2m to 5m of marine sand and stiff clay overlying fine grained phyllite rock of very low to medium strength. The depth to rock varied across the site.

Figure 1 shows an example of the representative overburden soils and the rock conditions encountered onsite, obtained from a geotechnical borehole drilled at the site. In this example, the rock (phyllite) was encountered just above RL -10 mCD (Chart Datum), or approximately 5 m below seabed level. The overlying very stiff clay (the orange coloured material above RL -10 mCD) was inferred as the residual materials derived from the weathering of the underlying phyllite rock.

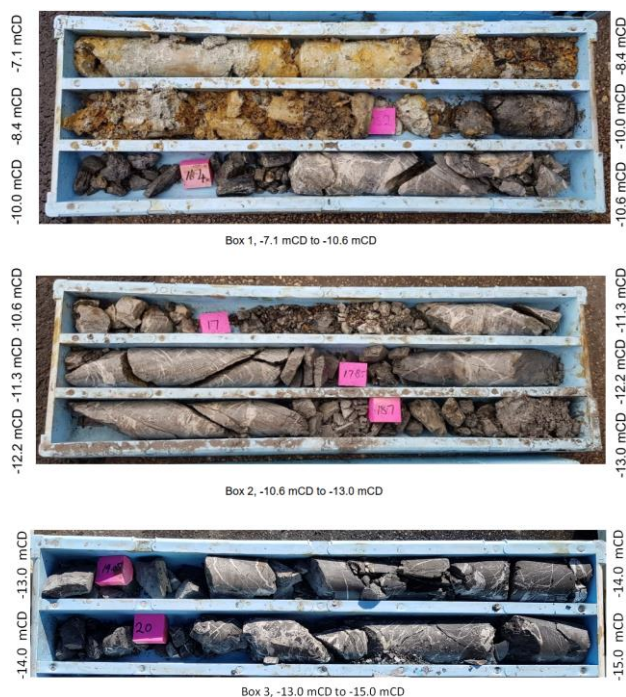


Figure 1. An example of the rock core (phyllite) conditions at the site (the phyllite rock was encountered from approx. RL -10 mCD in this example. Seabed was at approx. RL -5 mCD).

The phyllite was observed as a black and grey rock with varying degrees of weathering, generally highly and moderately weathered to large depth. The phyllite was also encountered as schist and slate. Notable core losses were recorded in the geotechnical boreholes, indicating the low strength and highly fractured nature of the rock, as well as potential for extremely weathered zones and infilled clay seams within the rock mass (as seen in Figure 1).

Some key information on the engineering properties of the encountered phyllite onsite is provided in Table 1.

Table 1. Key rock parameters.

Parameters	Value ⁽¹⁾	Unit
Total Core Recovery (TCR)	0-100 (52)	%
Rock Quality Designation (RQD)	0-71 (20)	%
Point Load (PLI), Is(50)	0.13-0.95 (0.45)	MPa
Unconfined Compressive Strength (UCS)	1.2-5.3 ⁽²⁾	MPa

⁽¹⁾ values in brackets are average.

⁽²⁾ average value not provided due to the limited UCS tests available.

The following notes are made regarding the rock parameters listed in Table 1:

- Based on the measured limited UCS data, the rock can be classified as highly fractured, very low to low strength, and low to medium strength based on PLI data (i.e., as per Australian Standard AS1726:2017 classification).
- The upper 2 m of the rock mass is more relevant for the piles considered herein, based on the inferred pile penetration into rock (discussed in Section 2.4). The properties of the upper rock are generally towards the lower end of the range listed in Table 1 (as the rock conditions were observed to generally improve with depth).

2.3 Pile driving information

Steel tubular piles of 762 mm outer diameter (OD) and 13 mm wall thickness (WT) were used. The design pile verticality varied from being vertical piles to raking piles (at a horizontal to vertical inclination of 1H:4V to 1H:6V).

The piles were driven to refusal on rock using Junttan HHK 3 (3t) and HHK 5 (5t) hydraulic impact hammers. A total of eighteen (18) piles were driven into rock onsite. Seven (7) of the piles were load tested using high-strain dynamic pile testing method (pile driving analyser (PDA) testing). These piles are the focus of the discussion in this paper.

Installation details for the seven PDA tested piles are provided in Table 2. An illustration of the pile set-up is shown in Figure 2.

Table 2. Installation details for the seven PDA tested piles.

Pile	Size: OD × WT (mm)	Total length (m)	Rake (H:V)	Hammer used	Driving set (mm) ⁽¹⁾
1	762 × 13	23.2	Vertical	Junttan HHK 3	2.7
2	762 × 13	23.2	Vertical	Junttan HHK 3	4.7
3	762 × 13	23.2	1:6	Junttan HHK 3	1.6
4	762 × 13	18.3	1:4	Junttan HHK 5	2.7
5	762 × 13	27.8	1:6	Junttan HHK 5	2.0
6	762 × 13	27.8	1:6	Junttan HHK 5	1.8
7	762 × 13	27.8	1:4	Junttan HHK 5	3.1

⁽¹⁾ penetration per blow at 1.2m stroke for HHK 3 hammer, and 1.0m stroke for HHK 5 hammer at pile termination depth.

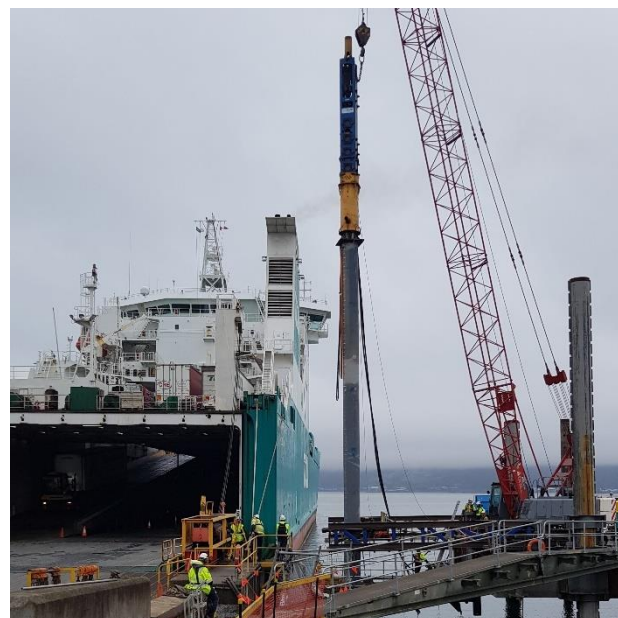


Figure 2. Driven steel tubular pile set-up.

2.4 Determination of rock level

Apart from Piles 1, 2 and 3, where a geotechnical borehole (Figure 1) was available adjacent to the pile location,

geotechnical boreholes were not located in the vicinity of the pile locations. As such, the rock levels at other locations have been inferred based on interpretations of the pile driving records.

Generally, the rock was inferred to be encountered where a notable increase in the measured blow counts versus pile penetration depth (adjusted for the hammer energy used) was noted. A verification of this method of interpretation was performed for Piles 1, 2 and 3, where the inferred rock depth based on the blow counts was cross checked against the adjacent geotechnical borehole data. A good agreement (generally within 0.5m depth) was obtained, as illustrated in Figure 3. It is noted that the blow count in Figure 3 has been adjusted for a 1.2m HHK3 hammer stroke used (by a factor of 1.2/the implemented stroke) to account for the different hammer strokes (drop heights) used during the pile driving for Piles 1 to 3.

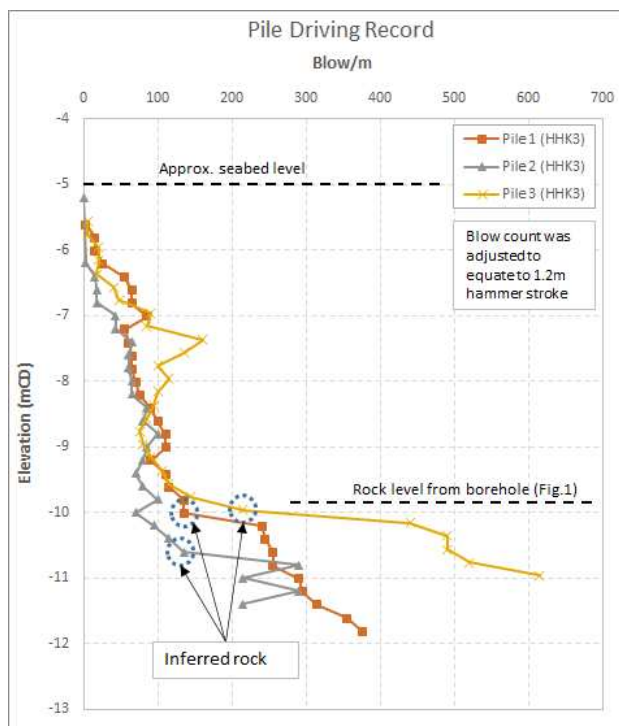


Figure 3. Example of interpretation of rock level.

2.5 Measured skin friction in rock

Seven (7) of the total eighteen (18) piles were load tested using high-strain dynamic pile testing method (pile driving analyser (PDA) testing with CAPWAP analysis (Likins, 1984; Goble and Rausche, 1979)).

All piles were tested within a few hours following the End of Drive (EoD), except for Pile 1, where the PDA testing was conducted at 3 days after the EoD. A summary of the test results is provided in Table 3. The reported ultimate compression skin friction (f_{su}) was the skin friction force derived from the PDA testing divided by the external pile shaft area (external shaft friction only, i.e. a plugged mode).

The ultimate compression skin friction (f_{su}) for the driven pile section within the inferred rock was found to be highly variable, ranging from the low 100's kPa to approximately 500 kPa. This is consistent with the observation of highly variable rock strength and quality (degree of weathering and fractures) across the site. The majority of the results are however within the 200 kPa to 300 kPa range. The single high f_{su} value of 500 kPa is thought to be due to the presence of a stronger rock (i.e., towards medium strength, i.e. UCS = 6 MPa and above) at that particular pile location.

Apart from Pile 1, the achieved penetration length in rock of the other piles has been quite limited (less than 1m). As such, any interpretations of the results may need to consider the smaller pile exposure over a shorter rock contact length of this example. Also, the CAPWAP analysis method has some limitations with respect to separation of shaft resistance and end bearing near the toe of the pile. This may have also contributed to the variability in the skin friction values shown in Table 3.

Table 3. Compression skin friction derived from CAPWAP analysis of pile dynamic testing (PDA).

Pile	Depth to rock (m) ⁽¹⁾	Inferred driven length in rock (m)	EoD to PDA (day)	Approx. f_{su} from PDA (kPa) ⁽²⁾
1	5.7	1.7	3	240-290
2	6.0	0.7	0	160
3	5.0	0.9	0	290-330
4	3.3	0.3	0	260
5	5.1	0.7	0	110
6	5.6	0.9	0	240
7	4.4	0.7	0	500

⁽¹⁾ below seabed level. ⁽²⁾ external shaft friction only (plugged mode).

Dynamic pile testing (PDA) of 800 mm OD steel tubular piles driven into rock at the same project site some 20 years before reported an approximate ultimate compression skin friction value f_{su} value of 175 kPa (external friction only), which is consistent with the range shown in Table 3.

2.6 Comparison with published data

There is limited information in the literature on the measured skin friction for driven piles into weak rock. An attempt was made to compare the project measurements described above to the values reported in some limited past publications. A summary of some published data on skin friction for driven piles in rock is provided in Table 4.

Table 4. Published data of measured ultimate skin friction of driven piles in weak rock.

Study	Reported ult. skin friction f_{su} (kPa)	Rock UCS (MPa)
Tomlinson & Woodward (2008)	45 (calcareous rock) / 127 - 158 (mudstone)	See note below table
Illinois DOT (2011)	570 - 1150	Not reported
Mokwa and Brookes (2009)	60 - 200	0.6 - 12.5
Terente et al. (2017)	100 - 200 (low) / 500 - 1100 (high)	1 - 2 (lower bound)
Matsumoto et al. (1995)	180	< 1 (see notes below)
Project in this paper	110 - 330 (low) / 500 (high)	See Table 1

The following notes are made regarding the skin friction values listed in Table 4:

- Tomlinson & Woodward (2008): Calcareous rock = coral limestone, calcareous sandstone, described as “very weak” (UCS < 2 MPa if inferred by AS 1726:2017 classification). Mudstone = slaty mudstone or Mercia mudstone, one single f_{su} of 28 kPa not included. Described as “very weak to moderately strong”. Reported f_{su} = H-piles, precast concrete and steel tubes. For the steel tubes, unclear if f_{su} was derived for plugged (external shaft friction only) or coring (external and internal) mode.
- Illinois DOT (2011): Rock = limestone, sandstone, shale. The rock strength was not mentioned. Reported f_{su} = H-pile.
- Mokwa and Brookes (2009): Rock = intermediate geomaterials (hard soil to weak and medium strength rock, UCS = 0.6 to 12.5 MPa). Specific rock strength in relation to the reported f_{su} range not reported. Reported f_{su} = derived adopting a plugged pile mode (external friction only) but also inclusive of contribution from overlying soil.
- Terente et al. (2017): Rock = Mercia mudstone, classified as weak (lower-bound UCS = 1-2 MPa). Exact variation in UCS values and rock weathering with depth is unknown. Reported f_{su} = unclear derived for plugged (external shaft friction only) or coring (external and internal) mode.
- Matsumoto et al. (1995): Rock type = residual clay (diatomaceous mudstone), considered to be very weak strength (approximate cone penetration end resistance q_c of 3 MPa). Reported f_{su} = derived from static compression load test on drilled out pile (i.e. external shaft friction only).

Comparison of the above limited data suggests that with the exception of chalk and some coral and calcareous rock (where the very weak or collapsible nature of the rock can result in very low skin friction in driven piles), the measured ultimate skin friction f_{su} values of driven piles in other rock (siltstone and mudstone origin) are generally within the range of approximately 100 to 350 kPa for very low to low strength rock (noting however the uncertainties with some reported f_{su} values, e.g. whether for external friction only, as noted above). This is quite consistent with the range of measured f_{su} values from the project described in this paper. For higher rock strength (e.g. inferred medium strength), the measurements indicate that f_{su} values in excess of 500 kPa are possible.

3 ESTIMATION OF SKIN FRICTION

3.1 Methods of estimation

Unlike the estimation of skin friction in rock for bored piles, there is no recommended method or approach for the assessment of skin friction for driven piles in rock in various design codes. The nature of the interface contact between the driven pile perimeter and rock is highly unknown, thus the uncertainties in the skin friction that can be developed.

Tomlinson and Woodward (2008) suggests that a sand-like skin friction calculation can be adopted for brittle coarse-grained rocks such as sandstones, igneous rocks and some limestones; and a clay-like skin friction can be adopted for mudstone or siltstone weathered to a clayey consistency.

The Tomlinson and Woodward (2008) recommendations are provided in the form below:

Clay-like:

$$f_s = \alpha s_u$$

where

α = adhesion factor

s_u = undrained shear strength

(1)

Sand-like:

$$f_{s(z)} = \beta \sigma'_{vo(z)}$$

(2a)

where

$$\beta = K_s \tan(\delta)$$

(2b)

K_s = coefficient of horizontal stress

δ = soil – pile interface friction angle

No clear recommendation was provided for better quality rock (i.e. rock that is not weathered to a clayey consistency) of mudstone/siltstone origin similar to that herein. Nonetheless, an estimate of the f_{su} value adopting the above recommendations has been performed for the project conditions reported herein. Although the clay-like approach is considered more relevant for the rock type and origin at the project site (phyllite), a sand-like scenario was also assessed for comparison.

For the clay-like estimate, the s_u values were taken as half of the rock UCS ($s_u = \text{UCS}/2$). UCS values of 1 MPa to 3 MPa were considered the likely range for the upper rock at the project site. Assessments for UCS values in the order 4 MPa to 6 MPa, i.e. the higher end of the encountered rock strength at the project site, were also made. The recommendation of API RP 2GEO (American Petroleum Institute, 2014) was used to calculate the adhesion factor (α) in Equation (1). It should be noted that UCS values were not available at all test locations and over all the relevant test depths. Due to the often fractured, weathered and weak nature of the rock, samples of appropriate length required for UCS testing were quite not readily available. As such the inferred UCS values have been based on the limited UCS measurements, the interpretations from the measured PLI values, observations of core materials from boreholes and records of the pile driving process..

For the sand-like estimate, the skin friction factor (β) in Equation (2b) was made based on the recommendation of API RP 2GEO.

The above calculations were performed for each of the seven (7) piles tested, and for the depth range presented in Table 3. The estimates made using the above approach are provided in Table 5.

Table 5. Estimated of f_{su} values adopting Tomlinson and Woodward (2008) recommendations.

Scenario	UCS range (MPa)	Assumed density ⁽¹⁾	Calculated f_{su} (kPa)
Clay-like	1 - 3		130 - 355
	4 - 6		360 - 595
Sand-like		L-M	10 - 20
		M-D	15 - 25

⁽¹⁾ L-M = loose to medium, M-D = medium to dense.

The results in Table 5 show a marked difference in the predicted skin friction of the driven piles in rock adopting clay-like mechanism and sand-like mechanism. Further discussions of the results are provided below.

3.2 Discussion

The skin friction values from the dynamic pile load tests (PDA) shown in Table 3 suggest that the unit skin friction for driven piles can vary significantly (by a few hundred kPa) even for nearby piles (e.g. Pile 2 compared with Piles 1 and 3). Skin friction estimates based on the clay-like or sand-like approaches of Tomlinson and Woodward (2008) resulted in marked differences in the calculated f_{su} values.

The predicted f_{su} values adopting a clay-like skin friction mechanism appear to be in consistent with the measurements

obtained from the PDA pile load tests. In particular, a range of 130 kPa to 355 kPa was calculated for piles 1 to 7 for the lower strength rock (UCS = 1 to 3 MPa), which compares well with the measured range of 110 kPa to 330 kPa (Table 3). Calculated f_{su} values of 360 kPa to 595 kPa for higher rock strength (UCS = 4 to 6 MPa) also appear to be in good agreement with the measured f_{su} of 500 kPa in Table 3, albeit the latter being a single test result.

On the other hand, the prediction adopting sand-like mechanism, with the skin friction calculated as per API RP 2GEO recommendations, grossly under-predicted the measured skin friction by an order of magnitude for the piles considered herein. It is clear that a high skin friction in rock, as observed, cannot be developed in a low overburden environment under a sand-like mechanism unless significantly higher lateral pressure is used (see further discussions below). In addition, the rock type lends itself to a more clay-like approach than sand-like.

The Tomlinson and Woodward (2008) approach to predict the likely skin friction of driven piles in weak rock, while yielding reasonably good comparison with the pile test measurements in this study if a clay-like approach was used, appears rather simplistic, and does not consider a number of key rock properties and characteristics that may influence the results. These include in-situ lateral rock pressure, fracture spacing, joint infill materials, bedding orientation, geological strength index (GSI) etc. As such, an improved prediction approach would be required to improve the calculated results.

Terente *et al.* (2017) stated similar limitations and presented a number of suggestions. In particular, for the clay-like method, $\alpha = 0.11UCS^{-0.5}$ was proposed based on the lower bound curve fitting of limited test data. Estimates adopting this recommendation resulted in f_{su} much lower (less than half) of the measured f_{su} values herein. This is perhaps due to the “lower bound” nature of this formulation. For the sand-like mechanism, K_s of 1 to 8 was noted from the example in their study. This together with a peak $\tan(\delta)$ range of 0.6 to 0.8 for rock-steel interface in their paper results in a $\beta = K_s \tan(\delta)$ factor of 0.6 to 6.4, which would result in predicted f_{su} values closer to the project measurements herein, albeit still with a very large range of predicted values.

In any prediction that involves the use of rock UCS values, the difficulties with obtaining and assessing rock UCS data should be born in mind. This is because the penetrable weak rock can be highly fractured and has notable degree of weathering, making it quite often not feasible to be able to obtain an adequate rock sample size for UCS testing. Reliance on PLI values may present a better practical solution in those cases, although the large variations of PLI and UCS correlations for a specific rock can also impact the results.

4 COMPARISON WITH BORED ROCK SOCKET

Unlike for driven piles in weak rock, information and recommendations for skin friction in bored (drilled and grouted) rock sockets are more widely available. An attempt is made herein to compare the developed skin friction in the two cases.

As part of the project pile design verification, a static pull-out test was performed within a borehole drilled in the vicinity of Piles 1, 2 and 3 (Figure 1). The purpose was to evaluate the grout-rock skin friction to verify the pile socket design. The drilled hole diameter was a 130 mm. A 3 m length section of the phyllite rock, from RL -11.5 mCD to -14.5 mCD (Figure 1) was grouted with a steel bar in the center, and subsequently tested in the static pull-out of the steel bar.

The results of the rock socket static pull-out test and the calculated grout-phyllite skin friction are presented in Figure 4. These results indicate that an average skin friction well in excess of 700 kPa was measured for the tested rock socket (note: due to

the high length/diameter ratio of the test socket, its skin friction is discussed as “average” value since the mobilized skin friction can vary along the socket). This skin friction value is in general agreement with estimation by, say Rowe and Armitage (1987) where $f_{su-bored} = 0.45-0.6\sqrt{UCS}$, for say UCS = 1 to 5 MPa (measured UCS range for the project phyllite rock).

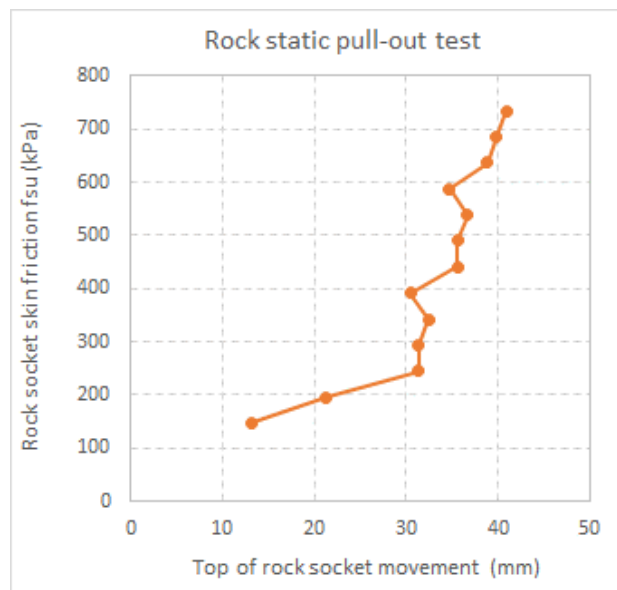


Figure 4. Ultimate tension skin friction for drilled and grouted rock anchor.

Comparison with the skin friction results in Table 3 for the driven steel piles at the same location (Piles 1, 2 and 3) suggests that the skin friction for the driven steel piles are significantly less, generally lower than 20% to 45% of the value for bored socket, i.e. $f_{su-driven} < 20-45\% f_{su-bored}$. Observations of the rock cores over the embedment depths of the nearby driven Piles 1 to 3 (approximately RL -10 mCD to -12 mCD) and the bored socket (RL -11.5 mCD to -14.5 mCD) suggests that the general rock conditions appear similar, although the rock fracture seems to reduce for the latter.

This comparison has not considered any potential difference between compression skin friction (Table 3) and tension skin friction (Figure 4) in rock. Where a lower tension skin friction is adopted (e.g. AS 2159:2009 Clause 4.4.2), the tested $f_{su-driven}$ will be reduced further below 20-45% $f_{su-bored}$ for the project example described herein. The different sizes of the bored test socket (130mm) and the driven piles (762mm) can also contribute to variations in the developed skin friction.

There is a much larger database and better developed predictive methods associated with the pile skin friction from a bored rock socket construction, yet it is still difficult to accurately predict rock socket shaft resistance for design purposes. The comparisons given herein are based on limited test data but illustrate the significant variability in shaft resistance outcomes. A much large database of driven steel tube skin friction than is currently available in the literature is required in order to provide meaningful guidance to designers of this pile type.

5 CONCLUSIONS

This paper has described a case study where the skin friction of steel tubular piles driven into weak phyllite rock was presented. Comparisons with other (limited) published data from driven piles in weak rock have been made. In addition, comparison with the skin friction from the drilled and grouted (bored) socket at the same project site was also made. Estimation of the skin

friction adopting clay-like and sand-like mechanisms as per Tomlinson and Woodward (2008) was performed. The main findings of this study are summarised below:

- The ultimate skin friction (f_{su}) developed in the open-ended steel tubular piles (762 (OD) x 13 (WT) mm) driven from 0.3m to 1.7m into weak phyllite rock, obtained from the pile dynamic load testing (PDA) of seven piles at various locations on the project site, was in the range of 110 kPa to 330 kPa, with one result being 500 kPa. The latter single result was inferred to occur in the higher range of rock strength at the site (UCS was thought to be approximately 4 MPa to 6 MPa); and the former range was inferred for the lower upper rock strength, where the rock UCS was inferred to be 1 MPa to 3 MPa.
- Comparison with past limited publications of skin friction of driven piles in weak rock (f_{su}) appears to suggest a good consistency with the project measurements given herein; where f_{su} values are generally within the range of approximately 100 kPa to 350 kPa for very low to low strength rock (UCS < 6 MPa, PLI < 0.3 MPa), and in excess of 500 kPa for higher rock strength.
- The prediction of the f_{su} values for driven piles in weak rock adopting the clay-like mechanism as per Tomlinson and Woodward (2008) recommendations were generally consistent with the measured data from the project example herein (refer to the results in Table 3 and Table 5).
- The prediction of the f_{su} values adopting a sand-like scenario and the API RP 2GEO recommendations in calculating the skin friction, however, grossly under-predicted the developed driven pile skin friction in the project example herein by an order of magnitude. It is clear that a high skin friction in rock, as observed herein, cannot be developed in a low overburden environment under a sand-like mechanism, unless significantly higher lateral pressure is used.
- Comparison with the skin friction developed from the drilled and grouted rock socket constructed at the project site in this study suggests that the ultimate skin friction developed from driven piles in weak phyllite rock was 20% to 45% of that from the drilled and grouted socket. The discussion in Section 4 should be noted in relation to the interpretation of this result.

Based on the results of the study herein, and in lieu of an improved prediction method, the following suggestions are made in assessing the likely skin friction of driven piles in shallow weak rock:

- Assess the theoretical $f_{su-driven}$ value using the Tomlinson and Woodward (2008) clay-like mechanism, especially for sites with shallow rock depths.
- Compare the calculated values with past publications and experiences (some examples are provided in this paper). This paper notes that $f_{su-driven}$ is in the order of 100 kPa to 350 kPa for very weak to weak rock. For higher strength rock (medium strength, UCS > 6 MPa), $f_{su-driven}$ in excess of 500 kPa is possible.
- Specify test piles (e.g. with PDA) early in the design process where project cost and program allow, and amend design assessment accordingly; or verify the design assumption with pile testing during installation.

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