Paper

The axial capacity of pile sockets in rocks and hard soils

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Synopsis

The unrelated empirical relationships developed for soil and rock lead to considerable uncertainty in the design of piles in intermediate materials such as hard soils and soft rocks. A new method for predicting the shaft resistance of piles socketed into rock, and based on fundamental principles is outlined. It is shown that the shaft resistance predictions of this method agree well with the field test data for rock and hard soil, and suggest a transition which links the disparate empirical relationships for soil and rock. Shaft roughness and socket diameter are shown to be critical factors affecting pile socket performance.

Abstract

Empiricism has characterised the traditional methods of pile design; in essence, pile design recommendations are based on the accumulated knowledge of pile behaviour based on the construction and subsequent load testing of piles in soil and rock. In this paper, the traditional approaches to design of piles in soil and rock will be briefly reviewed. It will be shown that the unrelated empirical relationships developed for soil and rock lead to considerable uncertainty in the design of piles in the large class of hard soil or soft rock that are sometimes referred to as intermediate materials. A new method for predicting the shaft resistance of piles socketed into rock, and based on fundamental principles is outlined. It is shown that the shaft resistance predictions of this method agree well with the field test data for rock and hard soil, and suggest a transition which links the disparate empirical relationships for soil and rock. It is demonstrated by way of a limited parametric study that shaft roughness and socket diameter are critical factors in the performance of piles constructed in these materials.

Introduction

In the determination of axial capacity of piles in soil and rock, geotechnical engineers have traditionally relied heavily on empiricism. This is because the pile-soil/rock system was regarded as too complex to understand and to model entirely theoretically. Such an approach leads engineers to consider somewhat arbitrary domains in which empirical relationships are constructed without reference to neighbouring domains. Thus, different empirical formulae are used to design piles in clays and rocks, without taking cognizance that they are part of a continuous spectrum of geomaterials. Similarly, different empirical methods are used to design piles and anchors in rock, although these structural elements are closely related.

This paper examines the axial capacity of piles at the boundary between the soil and rock domains, at which there is an apparent discontinuity in the empirical axial pile capacity predictions. It is demonstrated that a theoretical approach to pile capacity predicts a smooth transition which unifies the limiting and disparate empirical relationships for clays and rock.

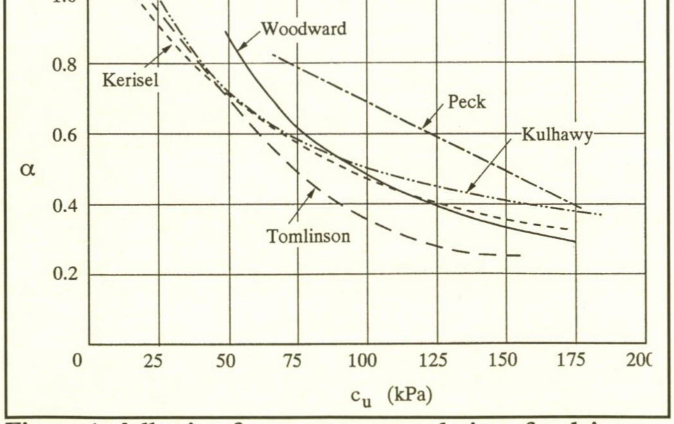

Figure 1: Adhesion factor recommendations for driven piles in clay.

Design of pile shafts in clay

In the design of piles in clay, many attempts have been made to correlate the available shaft resistance, ca with the undrained cohesion, cu. The ratio of c_a to c_u is generally denoted α . Poulos & Davis (1980) compare the correlations for driven piles proposed by Tomlinson (1957), Woodward et al (1961), Peck (1958) and Kerisel (1965), and this comparison is reproduced in **Figure 1**. It should be noted that these correlations are generally determined from databases with wide scatter.

For bored piles, adhesion factors are commonly assumed to be lower than for driven piles as a result of stress-relief, softening of the socket wall and other construction-related reasons. Golder & Leonard (1954) reported adhesion factors for bored piles varying between 0.25 and 0.7. Nevertheless, on the basis of a series of 127 bored pile load tests to failure in clay formations at 46 sites, Kulhawy & Phoon (1993) proposed a correlation, which is in close agreement with the relationships proposed for driven piles. This relationship is also shown in Figure 1. Kulhawy & Phoon's correlation is expressed in normalised form as follows:

$$\alpha = 0.5[c_u/p_a]^{-0.5} \qquad (1)$$

where p_a is atmospheric pressure (approximated for simplicity to 100 rather than 101.4 kPa).

Theoretical approaches to pile design in clays, based on effective stress approaches have been more recently postulated (Burland, 1973). However, despite the apparent attraction of a fundamental analysis, the difficulties of predicting lateral soil stresses, and accounting for installation effects is an impediment to the universal use of these methods (Clayton & Milititsky, 1983). The lateral soil stresses are generally empirically rather than theoretically determined.

Empirical design of pile shafts in rock

The development of empirical design rules for pile shafts in rock commenced in the 1970s. The shaft resistances for piles in rock have historically been related to the unconfined compressive strength, q_u. Day (1974) and Pells et al (1978) recommended allowable adhesions in Melbourne mudstone

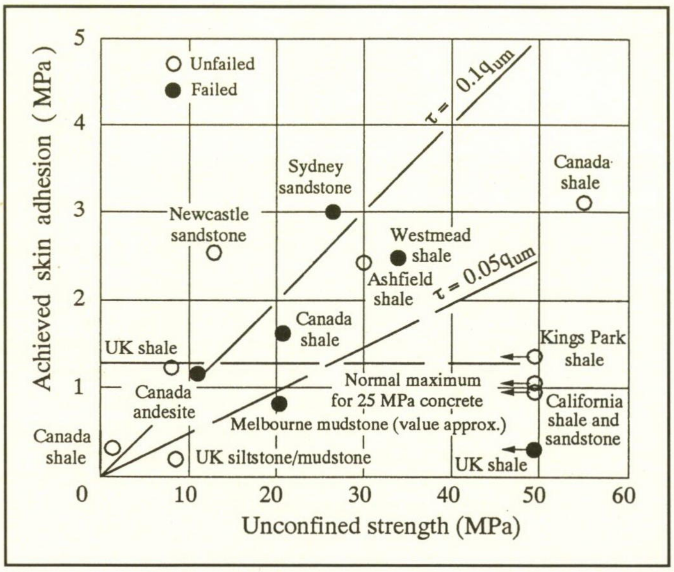

Figure 2: Achieved skin adhesion vs rock strength for pile sockets in rock (after Thorne, 1977).

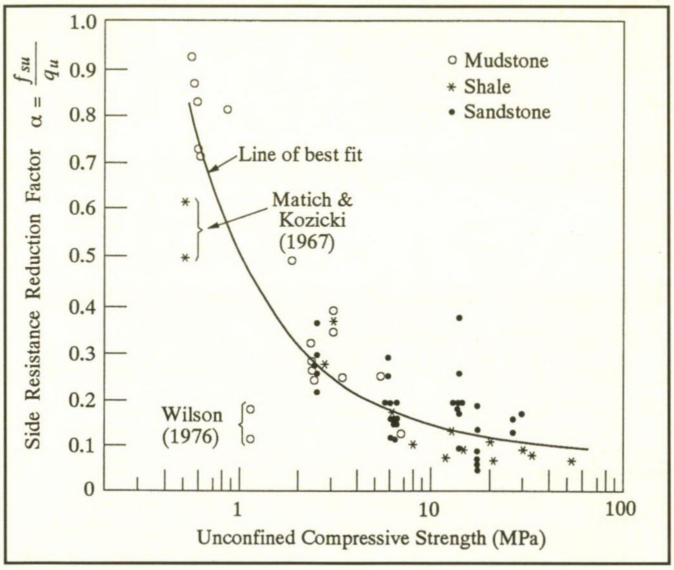

Figure 3: Side resistance reduction factors for pile sockets in rock (after Williams & Pells, 1981).

and Sydney sandstone, respectively of $0.05q_u$. Ultimate shaft resistance values published by Thorne (1977), and reproduced as **Figure 2**, would suggest that these recommended values were not necessarily conservative. This data relates primarily to unconfined compressive rock strengths in excess of 10MPa. Williams & Pells (1981), on the basis of a more comprehensive analysis of pile load tests in soft rocks proposed the relationship between adhesion factor and unconfined compressive strength shown in **Figure 3**. It must be stressed that their adhesion factor is the ratio of achieved side resistance, f_{su}, to unconfined compressive strength, q_u.

Adopting the earlier convention that f_{su} is the ratio of available side resistance to undrained cohesion, c_u, and combining Figures 1 and 3 highlights the large discrepancy between the empirical design methods for piles in clay and rock at the boundary between these materials (see **Figure 4**). For comparison purposes, it is assumed that $q_u \approx 2c_u$.

The importance of roughness in the shaft resistance of piles in rock was noted by Pells et al (1980) who developed a set of four roughness classes (**Table 1**) based on observation of sockets drilled in Sydney sandstone. In their investigations, sockets in the sandstone were predominantly Class R1 or R2, with some R3 sockets. No R4 sockets were observed, but these would presumably refer to artificially roughened sockets. Horvath et al (1983) proposed a relationship between available shaft resistance and a quantitative measure of roughness, denoted roughness factor, *RF*.

Roughness class	Description
R1	Straight, smooth-sided socket, grooves or indentations less than 1mm deep.
R2	Grooves of depth 1-4mm, width greater than 2mm, at spacing 50mm to 200mm.
R3	Grooves of depth 4-10 mm, width greater than 5mm, at spacing 50mm to 200mm.
R4	Grooves or undulations of depth >10mm, width >10mm at spacing 50mm to 200mm.

Table 1:Roughness Classes after Pells et al (1980).

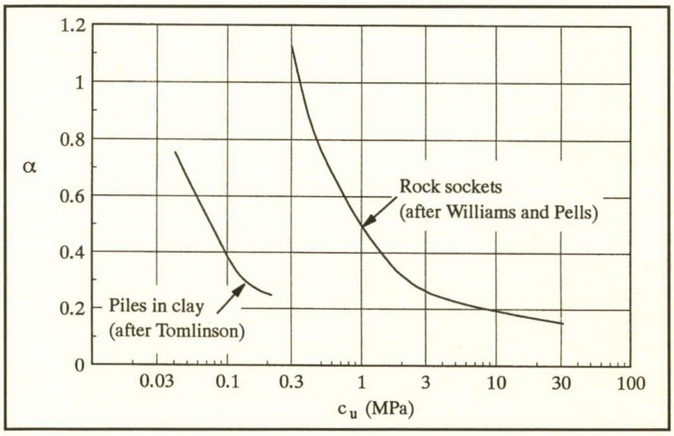

Figure 4: Comparative side resistance reduction factors for pile sockets in clay and rock.

Rowe & Armitage (1984) developed an international data base for drilled piles in rock, including 67 load tests to failure on 18 sites. The data was separated into two categories – sockets with roughness classes R1 to R3, and sockets with roughness R4. Kulhawy & Phoon (1993) supplemented the data of Rowe & Armitage with 47 additional load tests in Florida limestone after Bloomquist & Townsend (1991) and McVay et al (1992), as well as that of the pile load tests in clay reported by Chen & Kulhawy (1993).

Kulhawy & Poon presented their data both for individual pile load tests and as site-averaged data, the latter results of which are shown in **Figure 5**, in terms of adhesion factor, α , vs normalised shear strength, $(c_u / p_a$ or $q_u / 2p_a)$. Understandably, the results of individual load tests showed considerably greater scatter than the site-averaged data. On the basis of the site-averaged data, Kulhawy & Phoon proposed the following empirical relationships for the rock:

Mean behaviour: $$\alpha = 2.0[c_u / p_a]^{-0.5} \quad (5)$$

Upper bound (very rough): $$\alpha = 3.0[c_u / p_a]^{-0.5} \quad (6)$$

Lower bound: $$\alpha = 1.0[c_u / p_a]^{-0.5} \quad (7)$$

In general, equations (1), (5), (6) and (7) can be written in the general form as :

$$\alpha = \Psi\ [c_u / p_a]^{-0.5} \quad (8)$$

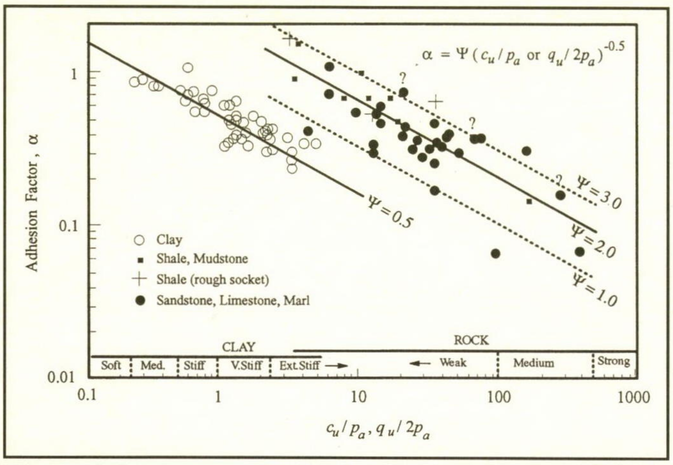

Figure 5: Site averaged adhesion factor vs normalised shear strength (after Kulhawy & Phoon, 1993).

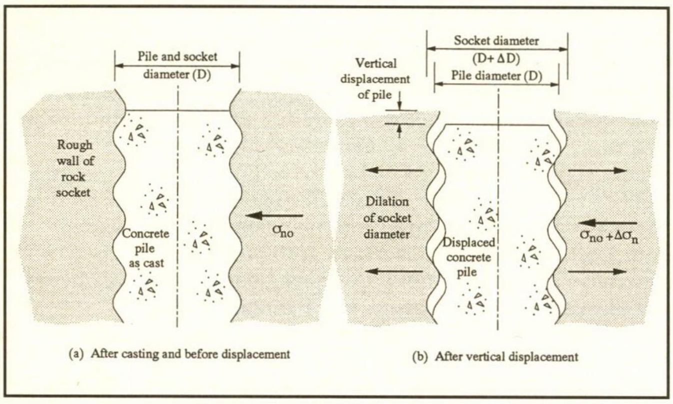

Figure 6: Schematic representation of development of side resistance in pile sockets.

This leads to a general expression for ultimate shaft resistance:

$$\tau = \Psi\ [c_u / p_a]^{-0.5} \qquad (9)$$

Equations (1), (5), (6) and (7) are superimposed on Figure 5. It can be seen that the empirical relationships for soil and rock form a set of parallel lines on these log-log plots. The intermediate materials which lie in the region of q_u = 400kPa (hard soils) to 2000kPa (weak rock) [$2.0 < (c_u / p_a) < 10.0$], represent a large class of materials which could be represented by either of the distinctly different empirical relationships for soils and rocks. It would appear reasonable that in fact, the behaviour of these geomaterials is in some way transitional. However, there is no clear evidence of how this transition occurs, or what causes the transition in behaviour.

Kulhawy & Phoon (1993) note that sockets in soil are very smooth, and imply that roughness in rock sockets is an important factor in the variable, but greater Ψ factors for rock. They also suggest that bonding at the rock face may contribute to the larger Ψ factors.

It is important to emphasise that the bounding empirical relationships given in Equations (6) and (7) are bounds to site-averaged data, and do not necessarily represent bounds to individual pile behaviour. The coefficient of determination (r^2) for Kulhawy & Phoon's rock data sets was approximately 0.71 for the averaged data, but was only 0.46 for the individual data, reflecting the much greater variability of the individual test results.

Theoretical approach to the design of pile shafts in rock

Kodikara et al (1992) describe a mechanistic approach to the design of pile shafts in weak rock. This approach has recently

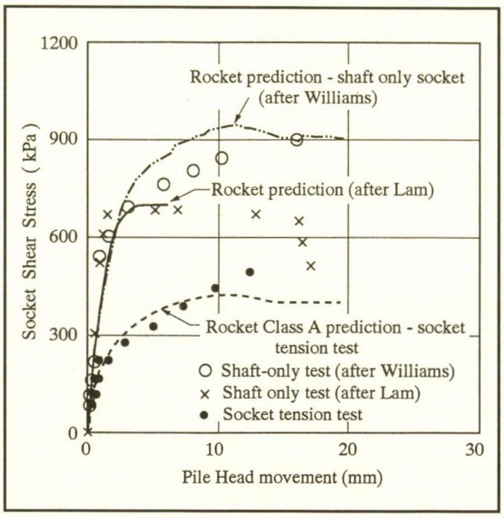

Figure 7: Comparison of pile socket load tests and predictions of Rocket program.

been modified as a result of an extensive laboratory investigation of concrete-mudstone interface samples in a constant normal stiffness direct shear machine (Johnston et al 1993; Seidel 1993; Seidel & Haberfield 1994). This analytical approach is based on modelling the physical processes of load transfer from the concrete pile to the surrounding rock socket. A fundamental aspect of this approach is the modelling of the dilation of the rough concrete/rock interface by a constant normal stiffness boundary condition (Johnston et al, 1987). As shown in **Figure 6**, as the pile undergoes axial displacement due to imposed load, the socket roughness forces dilation and increasing normal stress (and shear resistance) at the socket boundary.

Analytically, the processes of sliding of concrete over rock asperities and failure of the rock asperities when local contact stresses exceed the rock strength are modelled using the drained shear strength parameters for the intact rock, and the residual sliding friction angle of the concrete/rock interface. Local contact stresses are greatly influenced by redistribution of stresses that result from the elasticity of both the rock and concrete. Advanced models of roughness, based on concepts of fractal geometry (Seidel & Haberfield, 1995) are incorporated in the analytical model, and have been verified experimentally.

The analytical models are able to simulate the complete shear stress/displacement behaviour of pile sockets. Numerical studies have shown that the available peak shear resistance of rock sockets is a complex interaction of the following parameters – initial normal stress, intact rock strength, the residual friction angle of the rock, pile diameter (influencing the constant normal stiffness), rock mass modulus, Poisson's ratio, and socket roughness. This complexity necessitates computer, rather than manual solution, and the models have therefore been incorporated in a computer program called Rocket developed at Monash University (Seidel, 1995). Rocket, which is a full Windows application, requires the user to input these parameters, including a statistical representation of the socket roughness. On-line help is available to the user for all aspects of data input, and guidance is given for suggested parameters where only simpler strength data, such as unconfined compressive strength is available. Rocket then predicts the complete shear stress-displacement response of the pile shaft. The program has been verified against an existing data set of field load tests including in Melbourne mudstone by Williams (1980) and in Hong Kong granite by Lam et al (1991). Typical comparisons are presented in **Figure** 7.

Parametric study input parameters

The program Rocket was used in a limited parametric study to predict the expected variation of ultimate shaft resistance with unconfined compressive strength for a range of rock strengths. Although the program was developed from an experimental investigation of concrete/mudstone (weak rock) interfaces, the program has been applied to a much wider range of rock strengths in this study.

As noted previously, the peak shear resistance of rock sockets is a function of many variables, and to make an accurate prediction of an individual socket response, these variables must each be assessed on data relevant to the specific socket. Nevertheless, in order to establish general trends in socket resistance, it is reasonable to use typical, rather than specific parameters. This is consistent with the use of site-averaged data in Figure 5.

In this study, the effect of only two variables was investigated – socket roughness and socket diameter. In the first part of the study, socket roughness was varied between typical, rather than absolute minimum and maximum values, while all other parameters were held constant. An assumed socket diameter of 900mm was used. In the second part of the study, the socket diameter was varied between 450mm and 1800mm, which covers the normal range of socket diameters used in practice. For this study, only average roughness values were adopted. The following sections detail the choice of input parameters used in the parametric study.

Intact strength parameters

Figure 5 compares the adhesion factor with the normalised undrained shear strength of the soil/rock (c_u / p_a). The asperity failure model, however, is based on the drained Mohr Coulomb strength parameters of the rock. In order to establish appropriate strength parameters, the Hoek-Brown rock failure criterion (Hoek & Brown, 1980) was adopted, and Mohr Coulomb strength parameters determined after the method of Hoek (1990) using only the unconfined compressive strength of the rock, and appropriate values of the parameters *s* and *m*. In all cases, the material constant, *s*, was taken as 1 which is appropriate for intact rock. The material constant, *m*, which is dependent on the geological origin of the rock, was varied as shown in **Table 2**, and generally in accordance with the recommendations of Hoek & Brown. The intact strength parameters were held constant for the parametric study.

Sliding friction angle

The concrete-rock (residual) sliding friction angle for the argillaceous material was uniformly adopted as 24° for the parametric study; similarly, a sliding friction angle of 35° was adopted for the arenaceous and igneous materials.

Rock type	Strength range (q_u)	c_u / p_a	Range of m values
Igneous	>50MPa	>250	18
Arenaceous (Sandstone)	10-50MPa	50-250	12-16
Argillaceous (Mudstone)	3-10MPa	15-50	10-12
Argillaceous (Mudstone)	1-3MPa	5-15	8
Argillaceous (Mudstone)	1MPa	5	6

Table 2: Adopted Hoek-Brown material constant, *m*.

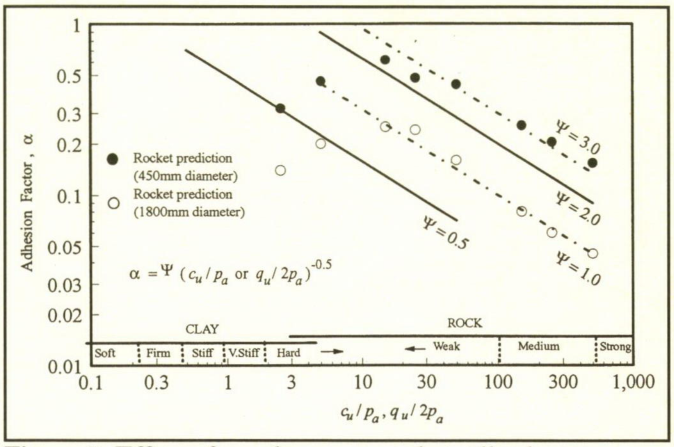

Figure 8: Effect of roughness on socket adhesion factor.

Rock mass modulus

The rock modulus required for analysis is the rock mass modulus. Deere (1968), suggests intact modular ratios, (E / q_u) of between 1:100 (low), 1:200 (average), and 1:500 (high) for intact rocks. These values will represent the upper limit for the rock mass modular ratio, as any defects and jointing intersecting the rock mass will act to reduce the mass modulus. Hobbs (1974) in a study of the deformation of shallow footings on rock suggested rock mass modular ratios (E_m / q_u) varied from 1:50 to 1:200, and averaged 1:100 for a wide range of materials varying from normally consolidated clays, weathered and unweathered argillaceous rocks and arenaceous sedimentary rocks, covering a compressive strength range similar to that investigated in this parametric study.

Williams & Ervin (1980), in their study of the effects of jointing on rock mass modulus, found that in Silurian mudstone, with unconfined compressive strengths in the range of 10MPa to 30MPa, a joint frequency of 10 joints/metre reduced the intact modulus values by a factor of approximately four. From their investigations, they suggested (E_m / q_u) ratios varied from 1:45 to 1:190, with an average value of 1:107.

For the purpose of this parametric study which is evaluating average performance, a constant rock mass modular ratio of 1:100 was adopted. Variations in modular ratio (and therefore rock mass modulus) would contribute to differences in shear resistance, which would be reflected in greater variability of individual socket responses.

Roughness

As noted previously, Kulhawy & Phoon (1993) determined that sockets in soil are very smooth. Similarly, sockets which are machine-drilled in hard rocks also tend to be very smooth. Thus, at either end of the spectrum of geomaterials, sockets exhibit minimal roughness. In the central portion of this spectrum, however, socket roughness has been observed to be very important. Pells et al (1980) and Horvath (1983), found that roughness exerts a major influence on the shaft resistance

Roughness class	Roughness heights (mm)	
	Sydney sandstone after Pells et al (1980)	Melbourne mudstone after Kodikara et al (1992)
Smooth (R1)	<1	1-4
Medium (R2)	1-4	4-20
Rough (R3)	4-10	20-
Very Rough (R4)	>10	-80

Table 3: Socket wall roughness for Sydney sandstone and Melbourne mudstone.

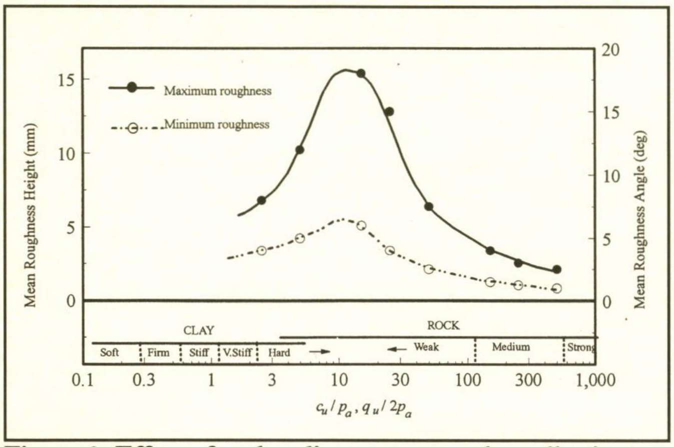

Figure 9: Effect of socket diameter on socket adhesion factor.

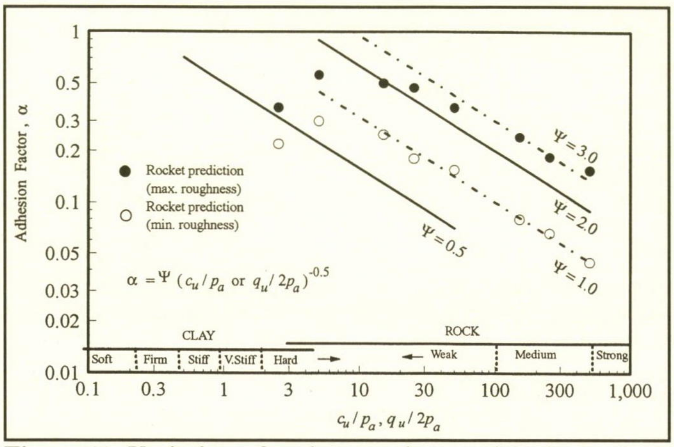

Figure 10: Variation of socket roughness with clay and rock strength.

of socketed piles in the rocks they investigated. The roughness of rock sockets will vary with the drilling technique, rock jointing and rock strength. As this study is dealing with average behaviour, the effects of drilling technique cannot be examined on an individual basis. Rock jointing may be reflected in a range of roughness for any given rock strength.

Table 3, based on observations by Pells of sockets drilled in sandstone (10MPa < q_u < 50MPa or 50 < c_u / p_a < 250), suggests four classes of roughness which have been characterised as smooth, medium, rough and very rough, with corresponding roughness heights.

Also shown in Table 3, are a range of roughness heights suggested by Kodikara et al (1992) on the basis of roughness measurements for sockets drilled in Melbourne mudstone (1MPa < qu < 10MPa or 5 < c_u / p_a < 50).

It should be noted, that the limited data set of Williams (1980) used by Kodikara did not actually include roughness heights less than 7mm or more than 20mm. It is evident from this table, that the qualitative descriptions of smooth and rough are subjective, and are governed by the normal range of drilled socket surfaces observed.

Figure 8 shows the variation of typical (rather than absolute) minimum and maximum mean roughness heights with normalised shear strength used in this study. It has been constructed on the basis of the roughness categories proposed by Pells & Kodikara, and the constraints of sockets in soils and hard rocks being very smooth. In the analytical model, roughness is represented as a set of triangular elements (asperities) of varying inclination. As the roughness model is based on a normal distribution of inclinations, the mean roughness heights do not constitute the maximum roughness height in the model. For a normal distribution, the standard deviation of asperity height is equal to $\sqrt{(\pi/2)}$ times the mean asperity height. A length of 50mm was adopted for the asperity sides for this study. Equivalent asperity angles are also indicated on Figure 8.

For the study on the effect of socket roughness on peak socket shear stress, a constant socket diameter of 900mm was used. For the study on the effect of socket diameter, the average socket roughness applicable to each soil/rock strength was adopted.

Socket diameter

The diameter of bored piles drilled into soil or socketed into rock varies enormously from 100mm or less to in excess of 3000mm. Most bored piles, however, vary in diameter from 450mm to 1800mm, which is the range of socket diameters that was investigated in this study. The effect of varying socket diameter is to change the constant normal stiffness imposed on the pile by the surrounding rock mass. The effect of a greater constant normal stiffness is to increase the normal stresses and available shear resistance at the pile/rock interface. Constant normal stiffness is inversely proportional to socket diameter.

As noted previously, for the study on the effect of socket roughness on peak socket shear stress, a constant socket diameter of 900mm was used. For the study on the effect of socket diameter, diameters of between 450mm and 1800mm were analysed while maintaining a constant (average) socket roughness.

Parametric studies – predicted responses

Based on the parameters noted previously, Rocket was used to predict the variation of upper and lower bound shaft resistance with normalised shear strength. The results of the first part of the study, in which socket roughness was the only parameter varied, are shown in **Figure 9** in terms of predicted adhesion factor, α, vs normalised shear strength. Equations (1), (5), (6) and (7), which were shown in Figure 5, are also superimposed on Figure 9 for reference.

The results of the second part of the study, in which socket diameter was the only parameter varied, are shown in Figure 10, again in terms of predicted adhesion factor, α, vs normalised shear strength. As for Figure 9, equations (1), (5), (6) and (7), are superimposed on this figure.

It can be seen from Figure 9 and **Figure 10** that Rocket predicts a range of shaft resistances that agree well with observed load test results. The upper and lower bound responses can be attributed either to the effect of socket roughness, or socket diameter. Furthermore, in both cases, Rocket predicts a transition from hard soils to rocks that links the empirical relationship for soils and rocks postulated by Kulhawy & Phoon.

The fact that both socket roughness and socket diameter individually cause variations in socket shear stress approximately equal to the range of observed behaviour suggests that their combined effect would cause a far greater variation than that typically observed. Two points should be noted in this regard. First, as noted by Kulhawy & Phoon, individual socket performance does vary considerably more than is suggested by an analysis of site-averaged data. Second, socket diameter and socket roughness are most likely codependent variables – roughness generally increases with socket diameter. As increasing roughness increases the peak shear stress, and increasing diameter decreases peak shear stress, the effects of roughness and socket diameter will in general balance rather than compound.

It is important to stress that the limited parametric studies presented here only relate to typical behaviour, and do not attempt to demonstrate the possible range of behaviours of individual pile sockets. This is consistent with Kulhawy & Phoon's observation of a much larger spread of individual test results. Individual analysis would require a rigorous analysis of the parameters appropriate to each particular socket, in particular the strength parameters and modular ratio.

Conclusions

It is concluded that the analytical methods which are used in the computer program Rocket provide a rational basis for the prediction of rock socket behaviour in geomaterials varying from hard soils to strong rock. It would appear from the limited parametric study conducted that predictions of the program are in general agreement with international databases on pile socket load testing.

Rocket predicts a transition from hard soils to rocks that links the empirical relationship for soils and rocks postulated by Kulhawy & Phoon. Furthermore, Rocket enables the effect of the critical parameters governing socket resistance to be modelled.

The roughness of rock sockets is a critical parameter determining available socket resistance. Socket roughness is very low for the extremities of geomaterial strengths (soils and hard rocks), but increases for intermediate geomaterials. The range of normal socket roughnesses is highest for rocks with unconfined compressive strengths less than 10MPa. Further work is required to build a database of socket roughness measurements.

Pile diameter is also a critical factor in determining the available shear resistance of rock sockets, due to the inverse dependence of confining stiffness on pile diameter. The confining stiffness results in a work hardening behaviour of pile sockets.

Rocket has been developed specifically for predicting the behaviour of piles sockets in Melbourne mudstone, and has been demonstrated to predict the load-deflection response of piles socketed into this material. There is evidence that the program can be used to predict the behaviour of piles socketed in a wide range of geomaterials.

Acknowledgments

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