PREDICTION OF THE AXIAL CAPACITY OF PILE SOCKETS

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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 cognisance 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. The application of the method to piles socketed into the granites and gneisses of Korea is discussed by way of a case study and by reference to recent direct shear tests on these rocks.

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Design of Pile Shafts in Clay

In the design of piles in clay, many attempts have been made to correlate the available shaft resistance, c_a with the undrained cohesion, c_u. The ratio of c_a to c_u is generally denoted α. Poulos and 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.

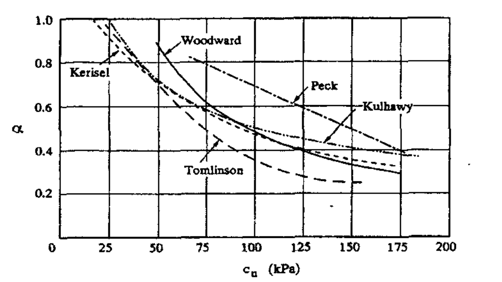


Figure 1. Adhesion factor recommendations for driven piles in clay

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 and 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 and 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 and Phoon' s correlation is expressed in normalized form as follows:

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

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 and Milititsky, 1983). The lateral soil stresses are generally empirically rather than theoretically determined.

Empirical Design of Piles Shafts in Rock

The development of empirical design rules for pile shafts in rock commenced in the 1970' s. The shaft resistances for piles in rock have historically been related to the unconfined compressive strength, qu. Day(1974) and Pells et al.(1978) recommended allowable adhesions in Melbourne mudstone and Sydney sandstone, respectively of 0.05 qu. 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 10 MPa. Williams and 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 shaft friction, τ, to unconfined compressive strength, qu.

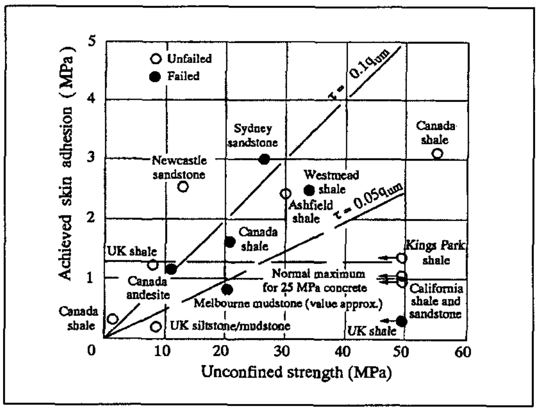


Figure 2. Achieved skin adhesion Vs. rock strength for pile sockets in rock(after Thorne, 1977)

Adopting the earlier convention that(is the ratio of available shaft resistance to undrained cohesion, c_u, and combining Figures 1 and 3 highlights the large discrepancy be-

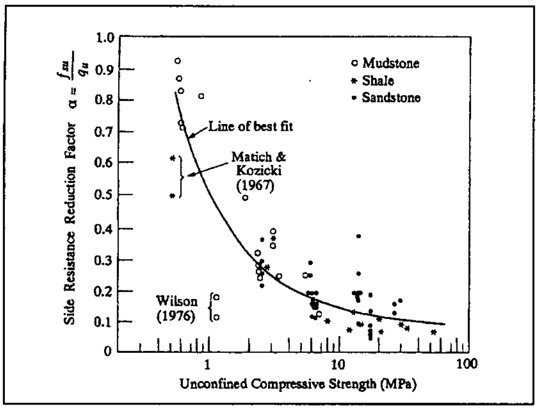


Figure 3. Side resistance reduction factors for pile sockets in rock(after Williams and Pells, 1981)

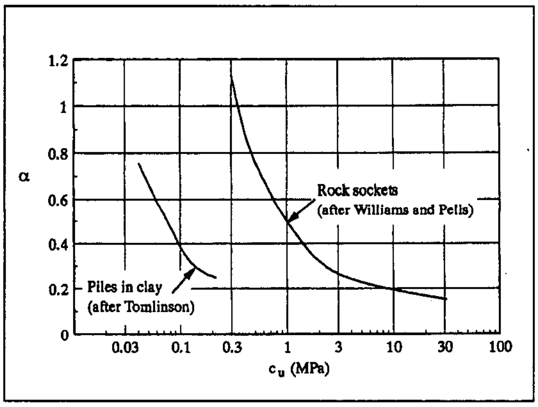


Figure 4. Comparative side resistance reduction factors for pile sockets in clay and rock

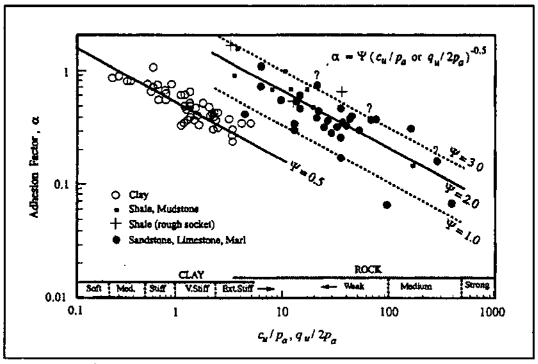


Figure 5. Site averaged adhesion factor vs normalized shear strength(after Kulhaway & Phoon, 1993)

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

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

tween 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, RF, denoted roughness factor.

Rowe and 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 and Phoon(1993) supplemented the data of Rowe and Armitage with 47 additional load tests in Florida limestone after Bloomquist and Townsend(1991) and McVay et al.(1992), as well as that of the pile load tests in clay reported by Chen and Kulhawy(1993).

Kulhawy and Poon presented their data both for individual pile load tests and as site-averaged data, the results of which are shown in Figure 5, in terms of tadhesion factor, α, vs normalized shear strength, (c_u / p_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 and Phoon proposed the following empirical relationships for the rock:

Mean behaviour:

$$\alpha = 2.0[c_u / p_a]^{-0.5} \tag{2}$$

Upper bound (very rough):

$$\alpha = 3.0[c_u / p_a]^{-0.5} \tag{3}$$

Lower bound:

$$\alpha = 1.0[c_u / p_a]^{-0.5} \tag{4}$$

In general, Equations (1) and (2) to (4) can be written in the general form as :

$$\alpha = \Psi[c_u / p_a]^{-0.5} \tag{5}$$

This leads to a general expression for ultimate shaft resistance :

$$\tau = \Psi[c_u \cdot p_a]^{-0.5} \tag{6}$$

Equations (1) and (2) to (4) are superimposed on Figure 5. It can be seen that the empirical relationships for soil and rock form parallel, linear relationships on these log-log plots. The intermediate materials which lie in the region of q_u=400 kPa(hard soils) to 2000 kPa(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 and 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 the bonding at the rock face may contribute to the larger Ψ factors.

It is important to emphasize that the bounding empirical relationships given in Equations (3) and (4) are bounds to site-averaged data, and do not necessarily represent bounds to individual pile behaviour. The coefficient of determination(r^2) for Kulhawy and 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.

Micro-mechanical Approach to the Design of Pile Shafts in Rock

Kodikara et al. (1992) describe a micro-mechanical approach to the design of pile shafts

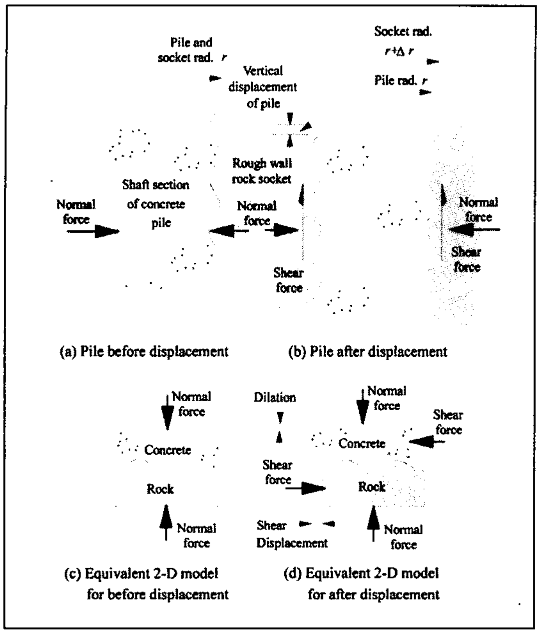
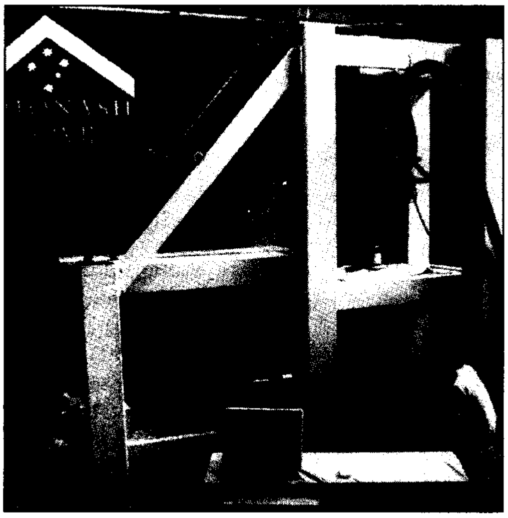


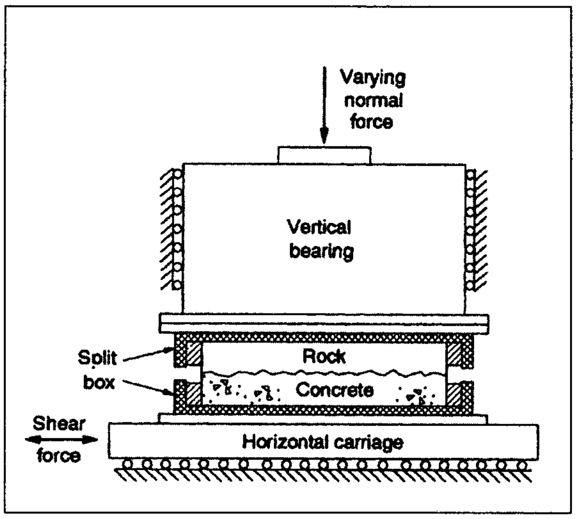
Figure 6. Schematic representation of development of side resistance in rock sockets

in weak rock. This approach was modified as a result of extensive laboratory investigations of concrete–mudstone interface samples in a constant normal stiffness direct shear machine(Johnston et al., 1993; Seidel, 1993; Seidel and Haberfield, 1994). The modified 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.

The behaviour of rock sockets can be modelled in the laboratory using direct shear test with a normal stress which is dependent on dilation, rather than constant. This is indicated in Figure 6 (c) and (d). A substantial laboratory testing program was undertaken by the



(a) Monash CNS rig



(b)Schematic of shear box

Figure 7. The Test equipment

researchers at Monash University in Melbourne, Australia based on direct shear testing of rock-concrete joints tested at large scale. The test equipment is shown schematically and in photograph in Figure 7.

The device can accommodate sample sizes of 600mm length x 200mm width. Shear and normal stresses are applied by servo-controlled Instron hydraulic actuators of 250kN capacity. Testing can be undertaken using load-control or displacement-control, under monotonic or cyclic conditions, with waveforms and periods to simulate realistic loading conditions. The device has PC-based digital control, automatic logging of all displacements and loads, and real-time display. Further details can be found in Johnston et al. (1993).

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 and 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. The complexity of the physical processes and interactions 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 8. A case study of socket construction in Korean granites and gneisses will be discussed later in this paper.

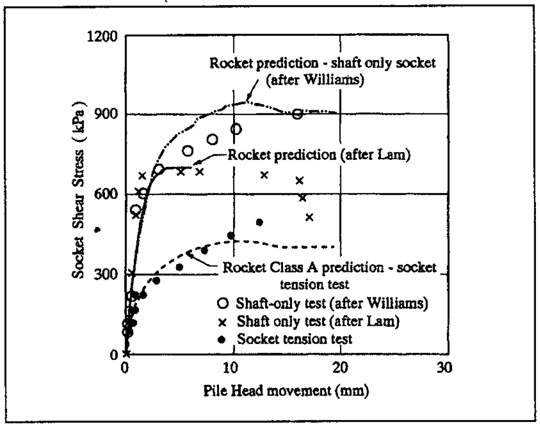


Figure 8. Comparison of pile socket load tests and predictions of Rocket program

Factors influencing rock socket behaviour

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.

Elastic Properties of the Rock Mass

The dilation of the rock socket can be approximated to the expansion of an infinite cylindrical cavity in an infinite elastic space of modulus, E_m, and Poisson's ratio, ν_m. Accordingly, the increase in normal stress, $\Delta\sigma_n$, for a socket of radius, r, can be related to interface dilation, Δr, as:

$$\Delta\sigma_n = \frac{E_m}{(1+\nu_m)} \frac{\Delta r}{r} \tag{7}$$

This expression can be rearranged to compute the normal stiffness, K, as follows:

$$K = \frac{\Delta\sigma_n}{\Delta r} = \frac{E_m}{(1+\nu_m)r} \tag{8}$$

The normal stiffness condition is extremely important to the work strengthening behaviour of rock socketed piles, as progressive slip displacements of the pile prior to peak resistance cause increasing normal stresses, and therefore increased interface strength.

Initial Normal Stress

Since the shear stress developed at the pile-rock interface is frictional and depends on the normal stress acting on the interface, an estimate of the initial normal stress is required. The initial normal stress, σ_{no} is primarily a function of the depth of concrete cast continuously above the socket. The concrete is assumed to act hydrostatically against the walls of the socket, with a pressure proportional to the total height of concrete poured, and the density of the concrete. A maximum effective height of 10m is usually adopted. In reality, the normal stress applied to the socket is a complex function which is dependent on the rate of placement of the concrete, arching effects of the concrete aggregate, the rate of hardening, the degree of compaction, and any setting shrinkage of the cement(Taylor, 1965; Clayton and Milititsky, 1983). It is noted that the performance of drilled shafts in rock may be improved by the use of expansive concretes, which can substantially increase the initial normal stress(Haberfield et al., 1994).

Pile Diameter

Equation (8) indicates that the normal stiffness is inversely proportional to the pile radius and diameter. The larger the pile diameter, the lower the normal stiffness, and the smaller the normal stress increases due to socket dilation.

Residual Friction Angle

On application of an axial load to the pile, the pile and the rock will displace elastically until such time as the shear strength at the pile-rock interface causes slip. The interface can be idealized as a rock joint containing a number of irregularly shaped asperities. The shear strength of the interface depends on the roughness of the asperities and the sliding friction angle between the concrete and the rock. This sliding friction angle(ϕ_{sl}) has been shown to be very close to the residual friction angle of the rock.

Intact Rock Strength

After the initiation of interface slip, the contact area between the concrete and the surrounding rock gradually reduces from full contact area, to smaller contact areas as shear displacement progresses. Local normal stresses increase both as a consequence of the reduced contact area, and as a result of the interface dilation in combination with the constant normal stiffness condition. A critical normal stress is reached at which the asperity can no longer sustain the loading, and individual asperity shear failure occurs. The stress at which this occurs depends on the geometry of the asperity(as defined by the roughness of the socket) and on the shear strength properties of the intact rock.

Socket Roughness

As mentioned above, the socket roughness plays an important role in the development of shaft capacity. Seidel and Haberfield(1995) describe the use of fractal geometry in the characterization of rough profiles, and determine the relationship between the so-called fractal dimension and the more common statistics of standard deviation of asperity length and angle. It is shown that these statistics are not independent and that the scale dependence of these parameters can be predicted using fractal geometry. It is beyond the scope of this paper to describe this work in detail. However, in essence the roughness of asperity sockets is modelled using a quasi-probabilistic approach; the standard deviation of asperity angle defines a probability density function for asperity angles. In the model, this probability density function is implemented as a deterministic approximation, with asperity angles randomly assigned to a grid of 16×16 asperity patches. The net response of the highly complex surface roughness is determined by combining the interacting responses of the many simple triangular asperities which make up the complete surface(Seidel and Haberfield, 1994).

Application to pile design

The key factors described in the previous section are modelled in the ROCKET program so that the shear-displacement response at rough pile-rock socket interfaces can be predicted. Multi-layer systems can be analysed, and combined in a t-z analysis with the pile end-bearing, which can be linear elasto-plastic, hyperbolic, or based on the findings of Williams et al. (1980).

Although the procedure may seem complex, with the need to input key material strength and mass deformation parameters, these parameters can often be estimated with sufficient accuracy based on local correlations. Of

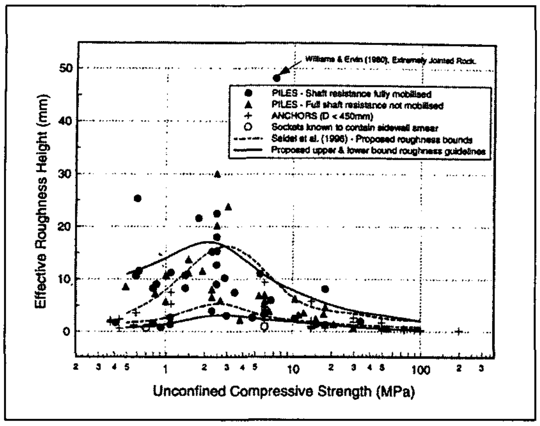


Figure 9. Roughness recommendations after Seidel and Collingwood (2001)

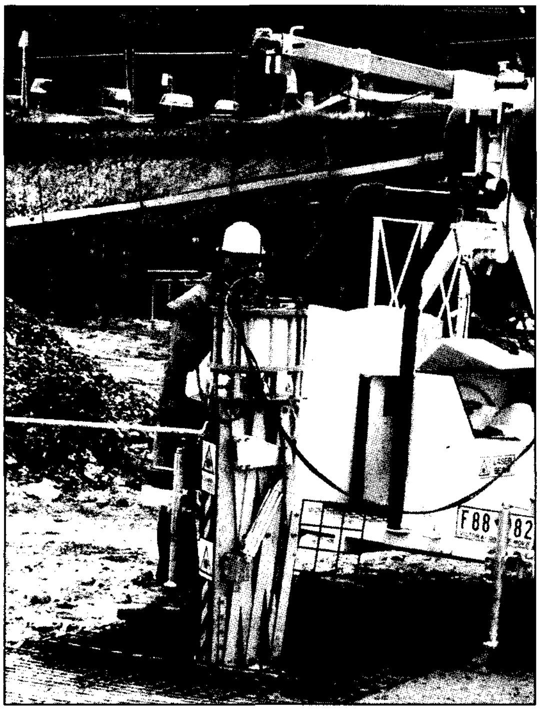


Figure 10 . SocketPro

course, high-level laboratory and field testing to determine the key parameters is preferred.

One factor which requires particular consideration is socket roughness. Roughness can have a profound effect on socket capacity, and should be taken into account in any socket design. The researchers at Monash have taken two approaches. The first is the development of an international socket roughness database. Figure 9 summarizes some roughness recommendations which are discussed in more detail in Seidel and Collingwood, 2001. Further work must be done, and development of local measurements and recommendations is encouraged. The second approach is direct measurement of socket roughness. The SocketPro device developed by Monash is a laser-based remote roughness measurement tool which can be lowered down sockets to a depth of 60m. The device shown in Figure 10 has been used for both research and contract purposes.

The analysis method is often used in order to undertake parametric studies to determine the degree of sensitivity of design to such factors as pile diameter, or socket roughness. The method can also be used to extrapolate the results of test-sockets to sockets of different geometry or material properties(e.g. zones of brecciated rock).

Case Study

A carefully controlled and executed full-scale socket test program was conducted by Jeon(2000). A large test program of tension and compression testing was undertaken, however, for the purpose of this case study, only the results of two compression pile tests will be reviewed. The test program was analysed by Piletech et al(2002) and compared with the predictions of some empirical methods nominated in various Korean codes of practice, and with the predictions of the Rocket program.

Sockets were 400mm in all cases, and were constructed by percussion drilling with air flushing of the excavated soil and rock. SB-1 and SB-2(shown as open circles) are the two test piles that are the subject of this case study. The closed circles are reaction piles. Boreholes were advanced at the location of piles SB-1 and SB-2, as well as at adjacent positions N-1 and N-2(shown as crosses).

The rock into which the piles were socketed is a highly weathered gneiss. Extensive pressuremeter testing was performed in the gneiss. SPT testing in the overlying unconsolidated sediments indicate soft to firm clays and loose sands and gravels, that would provide negligible uplift resistance.

The results of the pressuremeter tests in the adjacent boreholes N-1 and N-2 are shown in Table 2.

Both piles SB-1 and SB-2 were tested in compression, however, the base resistance of SB-1 was eliminated by use of a Styrofoam plug placed at the pile base prior to concreting. The significant properties of the two piles are shown in Table 3.

The ROCKET analyses undertaken by Cho (2002) were based primarily on derivation of input parameters based on the results of the site investigation. However, the values of peak shearing angle, the roughness height, and the roughness length were determined by back-analysis of the measured load-displacement responses. The peak shearing angles are consistent with the descriptions of the rock type and weathering. The roughness parameters are consistent with a very smooth socket, but cannot be confirmed against measurement or observation. The complete list of input parameters adopted for the ROCKET analyses are shown in Table 4.

On the basis of these parameters, Figure 11 and 12 shown the comparative measured load-settlement responses and Rocket-predicted load-settlement responses for piles SB-1(shaft only) and SB-2(shaft and base resistance).

The Korean design codes-e.g. Foundation Engineering Design Manual(Korean Geotechnical. Society, 1997), Design Code of Bridge Foundation(Korea Society of Civil Engineers, 2001), and Design Code of Rail Road(Korea Rail Road Office, 1999) are all based on overseas codes(e.g. AASHTO, FHWA, DM7 etc). These are in turn based on the results of a limited number of research papers(Horvath and Kenny, 1979 and Carter and Kulhawy, 1987).

The table shows that when applied to highly weathered gneiss rocks, the code methods

Table 2. Pressuremeter test results in N-1 and N-2

Borehole	Depth (m)	Modulus (MPa)	SPT N/cm	TCR (%)	Material
N-1	5.3	4.36	3/30~2/30	-	Soil
N-1	8.3	185.08	50/13~50/7	80	HWR
N-1	9.6	369.94	50/7~50/3	95	HWR
N-1	10.9	115.59	50/3 ~50/2	75	HWR
N-1	12.9	173.12	50/2	80	HWR
N-1	14.3	113.93	-	75	HWR
N-2	8.1	94.10	-	50	HWR
N-2	9.1	152.27	50/7	77	HWR
N-2	10.6	387.15	50/3	95	HWR
N-2	12.1	336.28	50/2	92	HWR
N-2	13.4	149.30	-	85	HWR
N-2	14.5	70.68	-	63	HWR
N-2	15.6	247.18	-	90	HWR

Table 3. Details of Piles SB-1 and SB-2

Pile	Diameter (mm)	Total length (m)	Length in soil(m)	Length into HWR(m)	Resistance type
SB-1	400	9.9	6.9	3.0	Shaft
SB-2	400	13.2	7.2	6.0	shaft + base

Table 4. Rocket input parameters for piles SB-1 and SB-2

Pile No	SB-1	SB-2
pile modulus	22,431 MPa	22,431 MPa
pile base elastic modulus	369.71 MPa	149.2 Mpa
ultimate base stress	0 MPa	9.38 MPa *
rock mass modulus (shaft)	184.97 MPa	361.5 Mpa
sliding angle	15	25
shearing angle	38 *	45 *
cohesion	0.1 MPa	0.1 Mpa
poisson's ratio	0.25	0.25
initial normal stress	0.081 MPa	0.141 Mpa
segment height	0.9mm	1.3mm
segment length	40mm *	33mm *
layer thickness	3.0m *	6.0m *
* back-figured from measured pile responses. all other parameters determined independently from site investigation.		

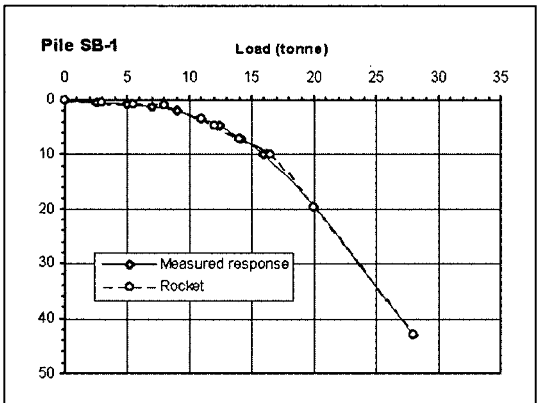


Figure 11. Measured vs predicted load-settlement responses for Pile SB-1

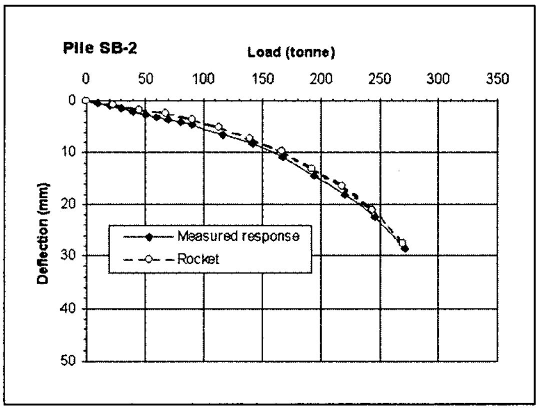


Figure 12. Measured vs predicted load-settlement responses for Pile SB-2

(FHWA, NAVFAC, CFEM and GEO) all substantially can over-predict the ultimate shaft resistance. This highlights the inherent problems with extrapolating empirically-based design methods to geologies and rock conditions which differ from the database on which the empirical method has been established.

The ROCKET analysis provides very close comparison with the measured values because it incorporates all the relevant site-specific information into the analysis.

It is also noted that the measured shaft resistance values are considerably less than those that would be predicted by Kulhawy

Table 5. Comparison of Code Methods and ROCKET with results for Piles SB-1 and SB-2

Pile	Quantity	UCS(MPa)	FHWA	NAVFAC	CFEM	GEO	Rocket	Measured
Pile1	Ult.Cap (kN)	22.7	3728	3430 (α=2.3)	3600 (β=0.63)	3208	184	215
Pile1	Unit Res (kPa)	22.7	990	910 (α=2.3)	956 (β=0.63)	851	50	57
Pile2	Ult.Cap (kN)	21.9	7338	6750 (α=2.3)	7080 (β=0.63)	6337	1285	1651
Pile2	Unit Res (kPa)	21.9	973	895 (α=2.3)	940 (β=0.63)	841	171	219

and Phoon for this strength rock. The range of ultimate shaft resistances predicted by their method are equivalent to 841Pa to 2680 kPa. The lower limit corresponds well to the other code methods. Clearly, however, empirical values should be applied with considerable caution to other countries.

Table 5 indicates that the ROCKET predictions (confirmed by actual measurement) are dramatically lower than would normally be predicted. However, recent direct shear testing of granite/concrete interfaces by Piletech et al.(2002) indicates that under appropriate boundary conditions, and with sufficient roughness, shaft resistances of 1000 kPa and greater can be expected from pile sockets in Korean granites. Given the sensitivity of shaft resistance to the particular geotechnical conditions, appropriate site investigation and testing to allow characterization of the foundation and pile construction is strongly advocated.

Concluding remarks

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. 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 10 MPa. Further work is required to build a database of socket roughness measurements.

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. This is demonstrated by a case study for piles socketed into highly weathered gneiss in Korea. Good correlation was obtained between predicted and measured responses for the two piles studied.

The case study further demonstrates the limitations of applying general empirical methods to the design of pile sockets without considering all the factors which affect the development of shaft resistance, and without having an adequate site investigation which allows these parameters to be determined.