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Prediction of the Shaft Resistance of Pile Sockets

암에 근입된 말뚝의 주면저항력 예측

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요 지

기존의 말뚝 설계방법들은 시공 및 재하시험 결과들로부터 축적된 말뚝거동에 대한 경험을 바탕으로 이루어 졌다고 할 수 있다. 이와 같이 만들어진 암에 근입된 말뚝의 설계에 대한 전통적인 방법들에 대해 고찰한 결과, 암에 근입된 말뚝의 경험적인 설계방법들은 설계시 상당한 불확실성을 내포하는 것으로 나타났다. 따라서 본 논문에서는 암에 근입된 말뚝의 주면저항을 예측하는 새로운 방법에 대한 기본원리를 고찰하였다. 이 방법으로 예측한 말뚝의 지지력은 현장에서 측정한 결과와 잘 일치하는 것으로 나타났다. 제한된 변수연구 결과이지만 본 연구를 통해 암의 거칠기와 말뚝의 직경은 암반에 근입된 말뚝의 거동에 중요한 역할을 하는 것을 알 수 있었다. 또한 국내 화강편마암에 대한 현장 사례연구를 통해 이방법의 적용성을 검토하였다.

Abstract

Empiricism has characterized 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 rock will be briefly reviewed. It will be shown that the unrelated empirical relationships developed for rock lead to considerable uncertainty in the design of piles. 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. 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. 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.

Keywords : Constant normal stiffness, Rock socket, Roughness, Shaft resistance, Socket pile

1. Introduction

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, q_u. Day (1974) and Pells et al. (1978) recommended allowable adhesions in Melbourne mudstone and Sydney sandstone, respectively of 0.05 q_u. Ultimate shaft resistance values published by Thorne (1977), and reproduced as Figure 1, would suggest that these recommended values were not necessarily conservative. This data relates primarily

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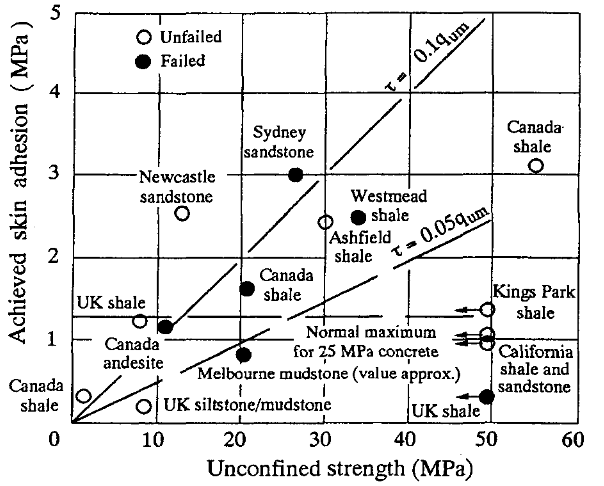


Fig. 1. Achieved skin adhesion vs. rock strength for pile sockets in rock (after Thorne, 1977)

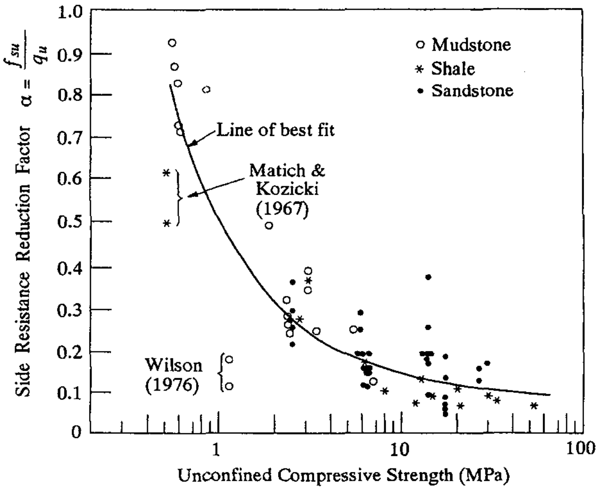


Fig. 2. Side resistance reduction factors for pile sockets in rock (after Williams and Pells, 1981)

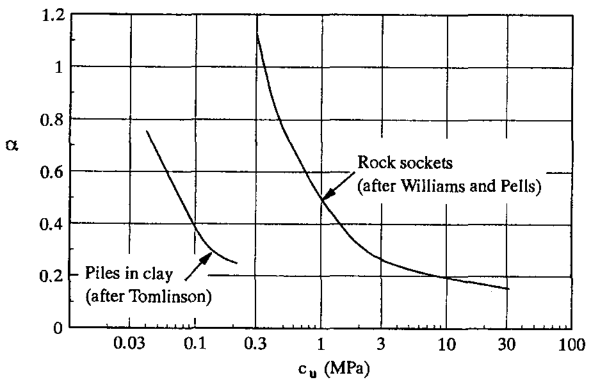


Fig. 3. Comparative side resistance reduction factors for pile sockets in clay and rock

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 2. It must be stressed that their adhesion factor, α, is the ratio of achieved shaft friction, τ, to unconfined compressive strength, q_u.

Adopting the earlier convention that α is the ratio of available shaft resistance to undrained cohesion, c_u, and combining Figure 3 highlights the large discrepancy between the empirical design methods for piles in clay and rock at the boundary between these materials. 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*, denoting roughness factor.

Rowe and Armitage (1984) developed an international database for drilled piles in rock, including 67 load tests to failure on 18 sites. The data were 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 those of the pile load tests in clay reported by Chen

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.

and Kulhawy (1993).

Kulhawy and Phoon presented their data both for individual pile load tests and as site-averaged data, the results of which are shown in Figure 4, 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: $\alpha = 2.0[c_u/p_a]^{-0.5}$ (1)

Upper bound (very rough): $\alpha = 3.0[c_u/p_a]^{-0.5}$ (2)

Lower bound: $\alpha = 1.0[c_u/p_a]^{-0.5}$ (3)

In general, Equations (1) to (3) can be written in the general form as :

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

This leads to a general expression for ultimate shaft resistance :

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

Equations (1) to (3) and empirical equation for soils are superimposed on Figure 4. 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

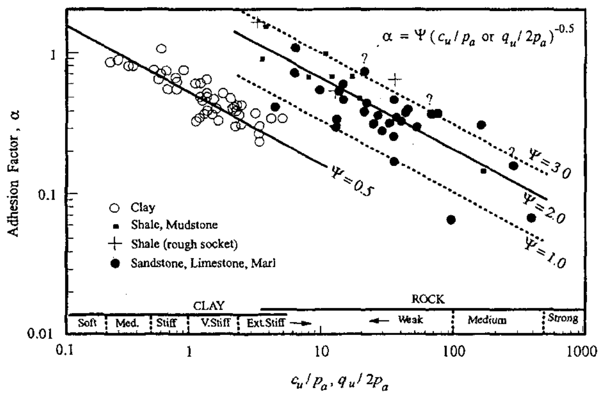


Fig. 4. Site averaged adhesion factor vs normalized shear strength (after Kulhaway & Phoon, 1993)

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.

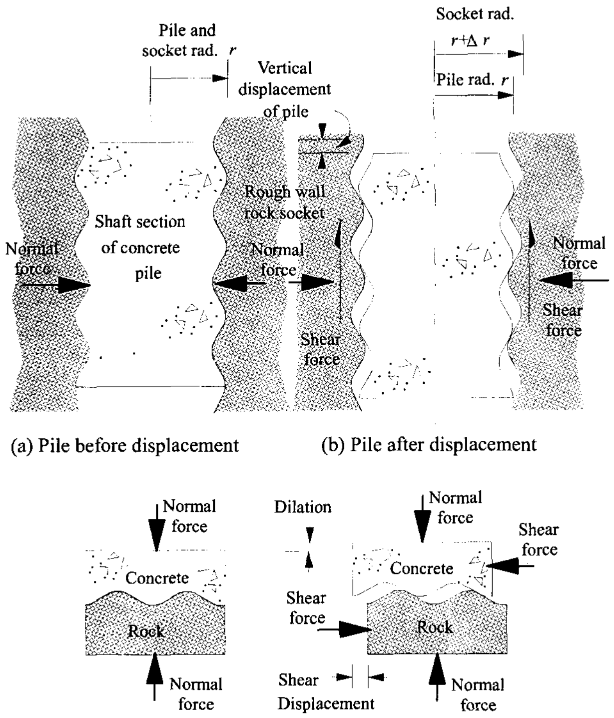
Kulhawy and Phoon (1993) note that sockets in soil are very smooth, and imply that roughness in rock sockets is an important factor in the variable, but greater Ψ factors for rock. They also suggest that bonding at the rock face may contribute to the larger Ψ factors.

It is important to emphasize that the bounding empirical relationships given in Equations (1) to (3) 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.

2. 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 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 5, 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



(a) Pile before displacement (b) Pile after displacement

(c) Equivalent 2-D model for before displacement (d) Equivalent 2-D model for after displacement

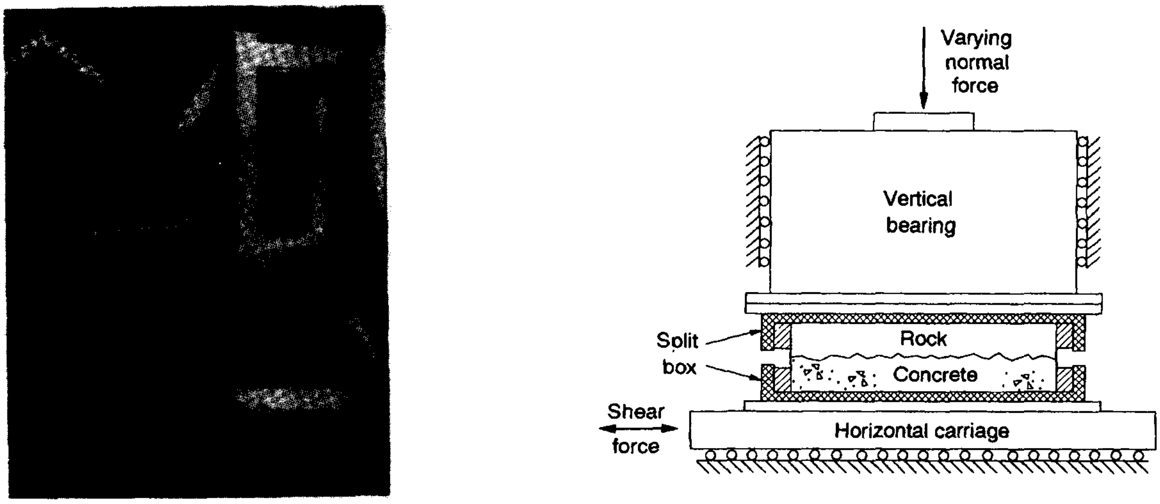
Fig. 5. Schematic representation of development of side resistance in rock sockets

laboratory using direct shear test with a normal stress which is dependent on dilation, rather than constant. This is indicated in Figures 5(c) and (d). A substantial laboratory testing program was undertaken by the 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 6.

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



(a) Monash CNS rig (b) Schematic of shear box

Fig. 6

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 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). A case study of socket construction in Korean granites and gneisses will be discussed later in this paper.

3. 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.

3.1 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:

$$\Delta\sigma_n = \frac{E_m}{(1+\nu_m)}\frac{\Delta r}{r} \qquad (6)$$

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

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

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.

3.2 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). 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).

3.3 Pile Diameter

Equation (7) 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.

3.4 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.

3.5 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.

3.6 Socket Roughness

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 16x16 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).

4. General Parametric Study

4.1 Input Parameters

In order to demonstrate that the *ROCKET* program predicts socket resistances that agree well with international experience, a limited parametric study was undertaken to predict the 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 4.

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, whilst 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 4 compares the adhesion factor with the normalized 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 and Brown,

Table 2. Hoek-brown material constant, m

Rock Type	Strength Range (q_u)	c_u / p_a	Range of m values
Igneous	> 50 MPa	> 250	18
Arenaceous (Sandstone)	10 to 50 MPa	50 to 250	12 to 16
Argillaceous (Mudstone)	3 to 10 MPa	15 to 50	10 to 12
Argillaceous (Mudstone)	1 to 3 MPa	5 to 15	8
Argillaceous (Mudstone)	$\leq$ 1 MPa	$\leq$ 5	6

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 1.0 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 and 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 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 and 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 10 to 30 MPa, a joint frequency of 10 joints/metre reduced the intact modulus values by a factor of approximately 4. 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 and 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 et al. (1983) found that roughness exerts a major influence on the shaft resistance 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

Table 3. Socket wall roughness for sydney sandstone and melbourne mudstone

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

drilled in sandstone (10 MPa < q_u < 50 MPa *or* 50 < c_u / p_a < 250), suggests 4 classes of roughness which have been characterized as smooth, medium, rough and very rough, with corresponding roughness heights. Also shown in Table 3, are ranges of roughness heights suggested by Kodikara et al. (1992) on the basis of roughness measurements for sockets drilled in Melbourne mudstone (1 MPa < q_u < 10 MPa *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 7 mm or more than 20 mm. 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 7 shows the variation of typical (rather than absolute) minimum and maximum mean roughness heights with normalized shear strength used in this study. It has been constructed on the basis of the roughness categories proposed by Pells and 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

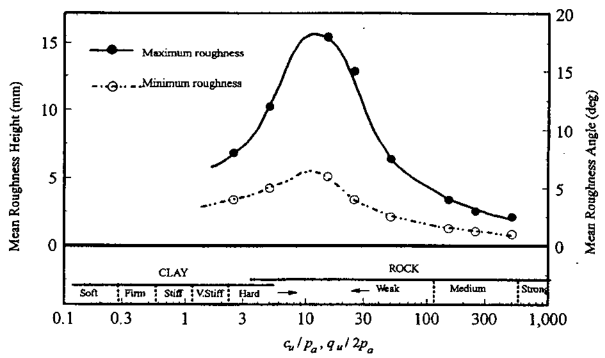


Fig. 7. Effect of socket diameter on socket adhesion

constitute the maximum roughness height in the model. For a normal distribution, the standard deviation of asperity height is equal to $(\pi/2)^{0.5}$ 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 7.

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 socket diameter is to vary 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 450 mm and 1800mm were analysed whilst maintaining a constant (average) socket roughness.

4.2 Predicted Responses

Based on the parameters noted previously, ROCKET was used to predict the variation of upper and lower

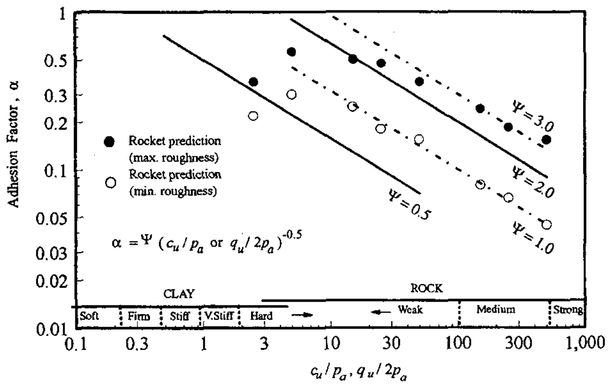


Fig. 8. Effect of roughness on socket adhesion factor

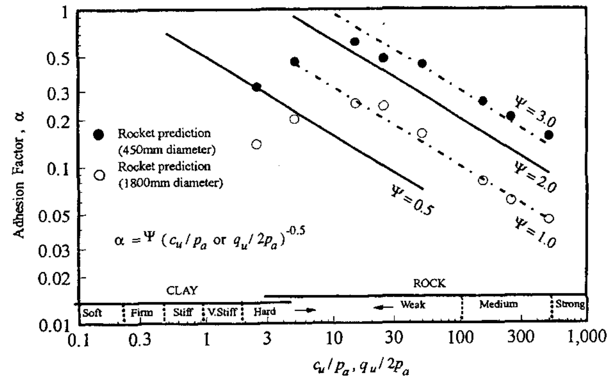


Fig. 9. Effect of socket diameter on socket adhesion factor

bound shaft resistance with normalized shear strength. The results of the first part of the study, in which socket roughness was the only parameter varied, are shown in Figure 8 in terms of predicted adhesion factor, α, *vs* normalized shear strength. Equations (1) to (3) and equation for soil (α=0.5), which were shown in Figure 4, are also superimposed on Figure 8 for reference.

The results of the second part of the study, in which socket diameter was the only parameter varied, are shown in Figure 9, again in terms of predicted adhesion factor, α, *vs* normalized shear strength. As for Figure 8, the equations are superimposed on this figure.

It can be seen from Figures 8 and 9 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 and 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. Firstly, as noted by Kulhawy and Phoon, individual socket performance does vary considerably more than is suggested by an analysis of site-averaged data. Secondly, 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 and 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.

For Korean granites and gneisses with UCS values between 20 and 80 MPa, the Kulhawy limits would suggest ultimate shaft resistances of between 1000 and 3000 kPa for 20 MPa rock, and between 2000 and 6000 kPa for 80 MPa rock. Experience indicates that these values are much higher than would normally be considered in design of pile shafts in Korean granites and gneisses of these strengths. Large diameter sockets, constructed by reverse circulation drilling are common in Korean foundation practice. Reverse circulation drilling generates very smooth socket walls relative to other drilling methods. In addition, the weaker classes of rock are closely jointed, and this results in low modular ratios. All these factors will reduce the expected socket shaft resistance.

5. 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. The results of Piletech et al (2002)'s study are presented in this section.

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 are the two test piles that are the subject of this case study. Boreholes were advanced at the location of piles SB-1 and SB-2, as well as at adjacent positions N-1 and N-2.

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 indicates 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 4. In Table 4, TCR and HWR stand for total core recovery and highly weathered rock, respectively.

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 5.

The ROCKET analyses undertaken by Cho et al.(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 is shown in Table 6.

Table 4. 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 5. 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 6. 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
Poissons 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.

On the basis of these parameters, Figures 10 and 11 show the comparatively 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(1996), FHWA, NAVFAC 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).

Table 7 shows that when applied to highly weathered gneiss rocks, the code methods (FHWA (1988), NAVFAC (1982), Canadian Foundation Engineering manual; CFEM (CGS, 1992) and Geotechnical Engineering Office; GEO (1996)) all substantially 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 and Phoon for this strength rock. The range of ultimate shaft resistances predicted by their method are equivalent to 1065 kPa to 3195 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 7 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, unpublished) 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 charac-

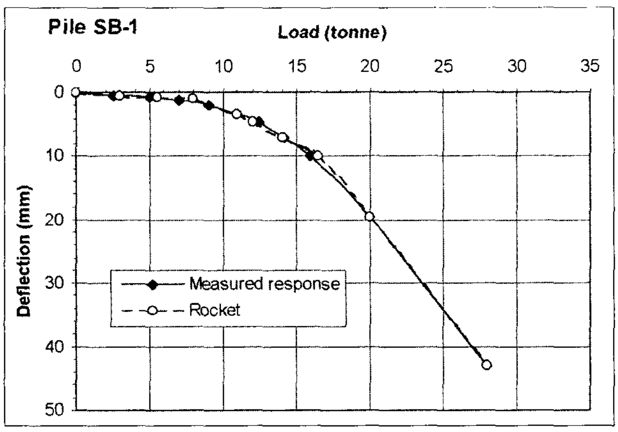


Fig. 10. Measured vs predicted load-settlement responses for Pile SB-1

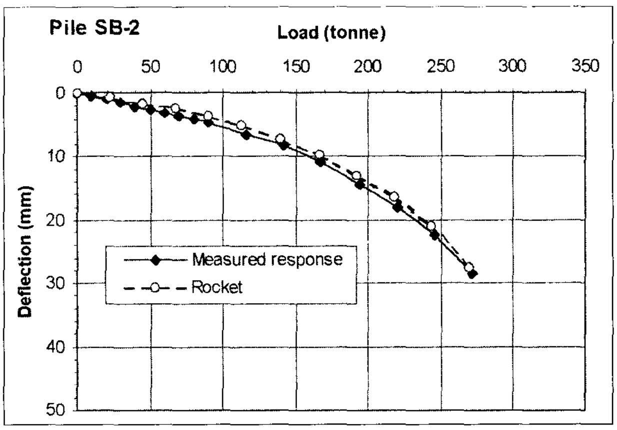


Fig. 11. Measured vs predicted load-settlement responses for Pile SB-2

Table 7. Comparison of code mthods and rocket with results for piles SB-1 and SB-2

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

terization of the foundation and pile construction are strongly advocated.

6. Conclusions

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 and Phoon.

The roughness of rock sockets is a critical parameter determining available socket resistance. 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. 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.

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