Principles of Side Resistance Development in Rock Socketed Piles

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SUMMARY An understanding of the principles associated with the side resistance of rock socketed piles has been developed by considering the pile-rock interface as a confined joint. The approach indicates that relative displacement between the pile and rock will occur by either sliding of one surface on the other or by shearing through the concrete or rock asperities. In either case, a dilation has been shown to occur across the interface which has given rise to significant stresses normal to the pile. The side resistance has been found to be a function of the normal stresses and the strength properties of the concrete and rock.

The validity of the concepts developed has been investigated by field testing several piles 1 to 1.2 m diameter and 2 to 2.5 m long in highly to moderately weathered mudstone. The stresses acting normal to each pile have been calculated from measurements of strain in the pile segments, and the socket dilations have been measured by displacement transducers installed in the rock mass. These measurements showed agreement with the postulated principles and have allowed investigation and design procedures to be rationally developed. Further, the developed principles simply explain the work strengthening or work weakening observed during field testing of side resistance piles.

1 NOTATION

- A area
- c_i cohesion (bond) of the concrete-rock interface
- c_o apparent cohesion of the joint
- c_s cohesion of the rock substance
- E_i Young's modulus of intact rock
- E_m Young's modulus of a rock mass
- f_{su} peak side resistance
- f_{sr} residual side resistance
- i asperity angle relative to the overall shear direction
- j mass factor $= E_m/E_i$
- N normal force
- q_a unconfined compressive strength of rock
- R radius of a rock socket
- S shear force
- S_H standard deviation of asperity height above a line running through the roots of the asperities
- S_i standard deviation of the asperity angle
- α side resistance reduction factor reflecting changes in the strength of intact rock $= \dfrac{f_{su}}{q_a}$ (for intact rock)
- β side resistance reduction factor reflecting changes in the mass modulus $= \dfrac{f_{su}}{\alpha q_a}$ (for jointed rock)
- δ_n normal displacement (dilation)
- ΔR change in the radius of a rock socket
- ϕ_i friction angle of the concrete-rock interface
- ϕ_o apparent friction angle of the joint
- ϕ_r residual friction angle of the rock resistance
- ϕ_s peak friction angle of the rock substance
- σ normal stress
- τ shear stress
- ν Poisson's ratio of a rock mass.

INTRODUCTION

The foundations for high and concentrated loads frequently consist of large diameter piles socketed into rock. Such piles may be designed to carry their load in side resistance only, in base resistance only or in both side and base resistance, according to the construction methods and local practice. The allowable values of side or base resistance are often determined on the basis of guidelines contained in Codes of Practice, previous experience or special investigations designed to suit the need of the particular project.

Investigations concerning the side resistance of rock socketed piles have usually involved the construction and load testing of small or full size test piles. Typical investigations, e.g. Rosenberg and Journeaux (1976), Hvorath (1978) and Williams *et al.* (1980a) have measured the peak side resistance f_{su}, and the residual side resistance f_{sr}, and related these capacities to the unconfined compressive strength, q_a, of the rock. As a result, useful empirical correlations now exist between side resistance and rock strength. Unfortunately, investigators reporting the results of load tests have not proceeded with their various analyses to obtain an understanding of the principles associated with the development of side resistance, which makes it difficult to apply data obtained from one set of conditions to a different situation. For example, the importance of socket roughness does not appear to have been fully considered, and no attention appears to have been given to the effects of jointing or the rock mass compressibility. These and other factors need to be considered if a proper understanding of side resistance is to be achieved and sensible pile designs are to be produced.

This paper develops the principles governing the development of side resistance by considering the pile-rock interface as a joint and by applying the technology established for rock-rock joints to the pile-rock joint. The concepts developed have been supported by laboratory tests designed to simulate the action of side resistance and by small and large size field pile tests, with particular attention being given to the effects of the following:

(i) roughness of socket walls

(ii) construction under bentonite

(iii) the modulus of the rock mass including the effects of jointing.

3 PILE-ROCK INTERFACE AS A JOINT

3.1 Shear Strength of Joints

The shear characteristics of rough joints which are applicable to the present discussion may be considered initially in terms of the bilinear joint model (Patton, 1966) and the semi-empirical strength criteria of Barton (1976). The bilinear model used by Patton (1966) to describe the behaviour of the regular joint shown in Figure 1 indicates that sliding will occur along the asperities according to Equation 1 and that shear will then occur through the asperities according to Equation 2.

$$\tau = \sigma \tan(\phi_i + i) \qquad (1)$$

$$\tau = c_o + \sigma \tan \phi_o \qquad (2)$$

Equation (1) indicates that sliding on the interface will not occur if $\phi_i + i \geq 90°$, which, for typical values of $\phi_i = 30°$ to $40°$, implies that sliding will not occur if the asperity angle, i, is greater than about $50°$ to $60°$. Asperity angles of this order seldom occur in natural joints, however, they may be common in the case of rock socketed piles, particularly if the socket has been hand excavated or if the socket has been specially grooved. Further, it may be seen from Equations (1) and (2) that if $\phi_i + i$ is greater than about $75°$ to $80°$, i.e. if i is greater than about $35°$ to $50°$, only a small normal stress will preclude failure according to Equation (1), and failure will therefore occur by shearing through the roots of the asperities according to Equation (2).

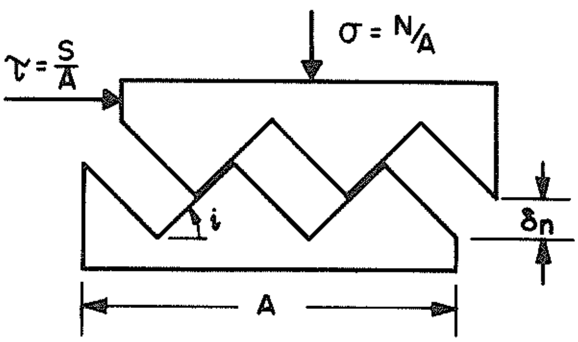

Figure 1. Model of a joint with regular asperities (after Patton, 1966)

Patton's model has treated the interface as a friction surface; however, if the concrete is cast against a clean socket wall, a significant bond will develop. The presence of such a bond, or cohesion, will influence the initial mode of failure only, because it will be destroyed if any sliding on the interface occurs. It is therefore appropriate to modify Equation (1) to include the interface cohesion, using a static analysis, as shown in Equation (3) for the condition before shear displacement has occurred.

$$\tau = \frac{c_i}{2 \cos i \, (\cos i - \sin i \tan \phi_i)} + \sigma \tan(\phi_i + i) \qquad (3)$$

In order to determine whether failure will occur initially along the interface or through the roots of the asperities, Equations (2) and (3) may be combined, then if the normal stresses are considered to be negligible before the initial failure, the criterion for initial failure by sliding is given by Equation (4).

$$2 \frac{c_o}{c_i} \cos i(\cos i - \sin i \tan \phi_i) > 1 \qquad (4)$$

In the case of a rock socketed pile observations indicate that it may be assumed that $c_o = c_i$, and Equation (4) then indicates that failure ill occur by sliding only if $(\phi_i + i) < 60°$ to $65°$. ne effect of a concrete-rock bond therefore has the effect of reducing the value of i, below which sliding may occur, to about 20° to 35°.

The foregoing analysis is simplistic and it has been included to simply illustrate the commonly accepted failure mechanism for rock joints and the importance of the interface bond and asperity angle. It is more practical and realistic to consider the shear characteristics of an irregular joint in terms of Barton's (1976) strength criteria, viz.:

$$\frac{\tau}{\sigma} = \tan \left[\text{JRC} \log_{10} \left(\frac{q_a}{\sigma}\right) + \phi_r\right] \qquad (5)$$

where JRC is an empirical joint roughness coefficient.

The ratio q_a/σ effectively provides for a sliding type of failure when the normal stress is low compared with the compressive strength of the rock, and for a progressive increase in shearing through the asperities as the normal stress increases.

The joint roughness coefficient, JRC, has been determined empirically by analysing the strength of various joints with different roughness profiles. In order to use Barton's equation, the JRC must be estimated, and this is often done by comparing typical standard roughness profiles with the profile in question. This method of determining the JRC is rather subjective and often difficult in practice. It has been found practicable to describe the roughness of rock sockets in terms of the statistical shape of the asperities. The statistical shape parameters found to reflect shear characteristics by Meyers (1962) for metal surfaces and by Krahn and Morgenstern (1979) for rock surfaces may be translated to the standard deviation of asperity height above the root line of the asperities, S_H, and the standard deviation of the asperity angle, S_i, although both Meyers (1962) and Krahn and Morgenstern (1979) found that S_i alone adequately represented the roughness from the shear strength viewpoint. A statistical analysis of the typical roughness profiles considered by Barton (1976) has produced the correlation between the JRC and the asperity angle S_i as shown in Figure 2, which tends to confirm the ability of S_i to adequately represent the roughness of joints subjected to a constant normal force.

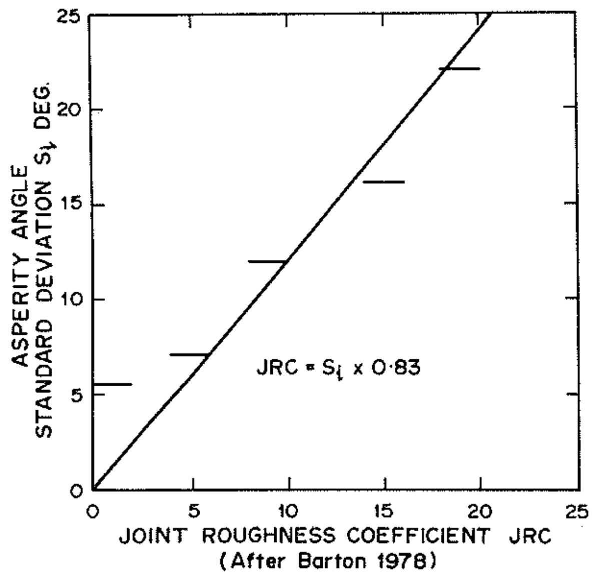

Figure 2 The correlation between Barton's (1978) joint roughness coefficient and the asperity angle standard deviation.

3.2 Application of Joint Analysis to Rock Sockets

The discussion above has been based on the concept of the common direct shear test in which the normal stress (or force) is maintained at a constant value until a peak, and sometimes residual, strength is indicated. Such test results are obviously applicable to the analysis of slope stability problems for example, where the normal force is reasonably constant.

In cases where the joints are confined, as in rock sockets and in many other underground situations, the normal stress develops as a result of dilation normal to the joint as sliding or shearing occurs and as a result of elastic effects, and as such it is not constant. The dilation normal to the joint is usually small enough to allow elastic theory to be used to relate joint dilation to normal stress. In the case of a rock socket the normal stress caused by joint dilation may be estimated according to the expanding infinite cylinder theory, e.g. Boresi (1965), as indicated by Equation (6)

$$\sigma = \frac{\Delta R}{R} \frac{E_m}{1 + \nu} \tag{6}$$

The normal stress which results from Poisson's ratio effects in the pile and surrounding rock mass, and which should be added to that given by Equation (6), may be estimated from elastic finite element analysis, however, preliminary work by Williams *et al.* (1980b) has found this to be small compared with the normal stresses discussed in Section 5.

4 LABORATORY DATA

It is possible to determine the shear characteristics of a confined joint by carrying out a direct shear test in which the normal stress is determined automatically according to the normal stress-dilation relationship. Such a test may be termed a "constant normal stiffness direct shear test", (CNS direct shear test), because the ratio of normal stress to dilation, σ/δ_n, is constant. In the case of a rock socketed pile the normal stiffness may be determined according to Equation (6) for the particular socket size being considered.

A direct shear machine has been designed by the author and built at Monash University to enable tests to be made with a constant N/δ_n, which provided a practical approximation to the constant σ/δ_n concept. The CNS machine was designed to test specimens nominally 150 mm wide x 200 mm long.

Initial laboratory tests were made on plaster-concrete samples with regular 12° and 45° asperities to provide simple models on which to develop an understanding of the CNS direct shear test. A typical result from a test on 45° asperities made with a normal stiffness of 1017 kPa/mm is shown in Figure 3.

The initial dilation angle of 45° shown in Figure 3(b) corresponds to the asperity angle which indicates that the initial linear portion of the curves corresponds to sliding along the plaster-concrete interfaces. An abrupt decrease in shear resistance is apparent at a displacement of 3 mm when failure occurred through the asperities, however, the normal stress was apparently unaffected by the asperity failure. The abrupt decrease in shear resistance may therefore be attributed to the loss of cohesion of the asperity, with the subsequent behaviour becoming largely frictional as the remaining roughness was worn away. The results of several tests on plaster-concrete joints with 12° and 45° asperities are summarized in Figure 4.

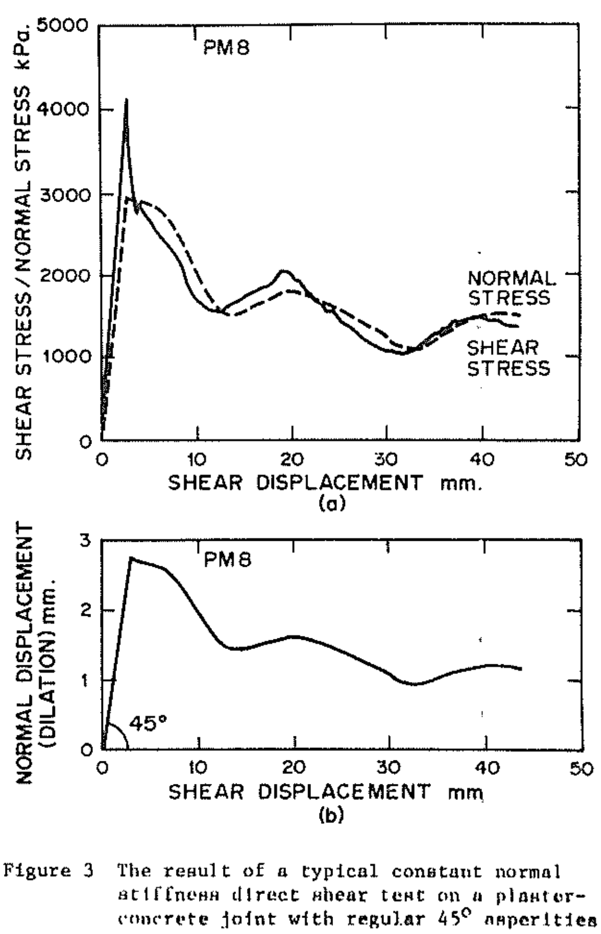

Figure 3 The result of a typical constant normal stiffness direct shear test on a plaster-concrete joint with regular 45° asperities.

Figures 4(a) and (b) indicate a significantly greater shear displacement and a greater dilation at the overall peak shear stress for the 12° asperities, however, Figure 4(c) suggests that the overall peak shear stress is the same for the 12° and 45° asperities.

The shear tests on plaster-concrete joints were followed by similar tests on mudstone-concrete joints. The mudstone samples were obtained from the sides of 1 m to 1.2 m diameter sockets drilled in mudstone adjacent to the test piles discussed in Section 5. The CNS direct shear tests were therefore designed to model the behaviour of the test piles.

In the first series of tests samples were obtained from 1 m diameter sockets drilled in highly weathered mudstone. One socket had been drilled normally without any special regard being given to the socket roughness, while a second socket was drilled normally and then specially roughened by adding an extra tooth to the auger. The roughness parameters obtained are represented by the parameters quoted for the test piles S3 (roughened) and S5 (normal) in Table II. The result of a CNS direct shear test on a specimen from a normally drilled rock socket is shown in Figure 5. Similar results were obtained for specimens from roughened sockets. Figure 5(a) indicates that an abrupt loss of shear resistance does not occur as progressive shearing of the asperities occurs. This is presumably because of the dilation which occurs and the normal stress which develops during the shearing of a confined, irregular, rough joint. The normal stress and normal displacement are seen to remain reasonably constant after the main asperity shearing, while the shear resistance is seen to decrease gradually to a residual value. The results of the tests on specimens from normally drilled and roughened sockets in highly weathered mudstone are summarized in Figure 6.

A second series of CNS direct shear tests was made on specimens obtained from a 1 m diameter socket drilled normally into moderately weathered mudstone. The roughness parameters pertaining to these specimens were similar to those for test piles M1 and M3 as listed in Table II. The results of the tests were similar to those for the highly weat ered mudstone, and are summarized in Figure 7.

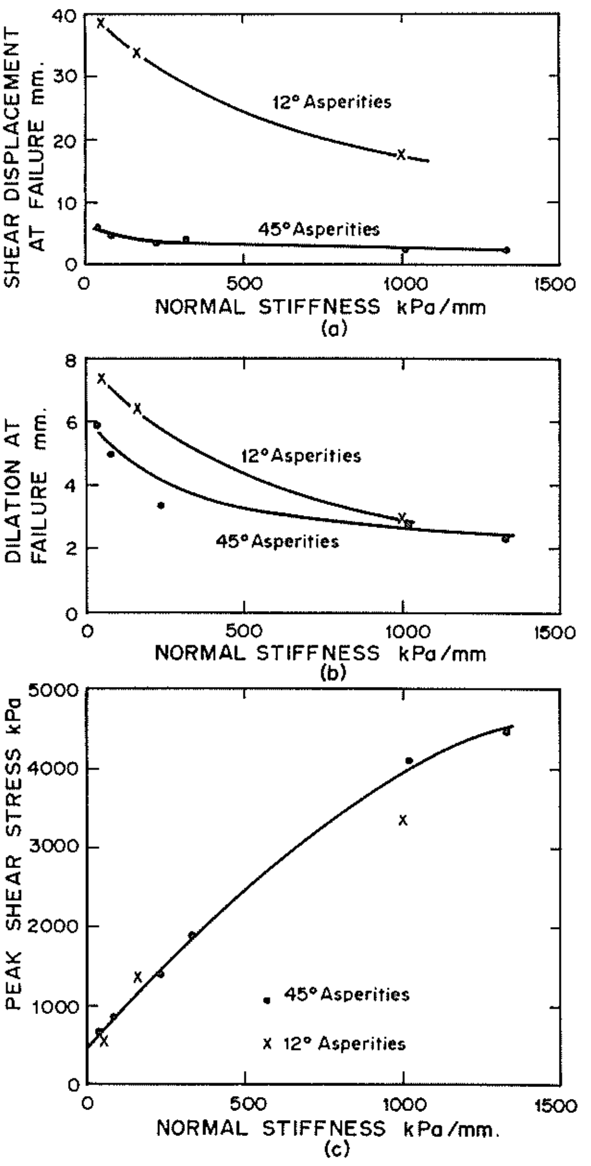

Figure 4 Summary of constant normal stiffness direct shear tests on plaster-concrete joints with regular 12° and 45° asperities.

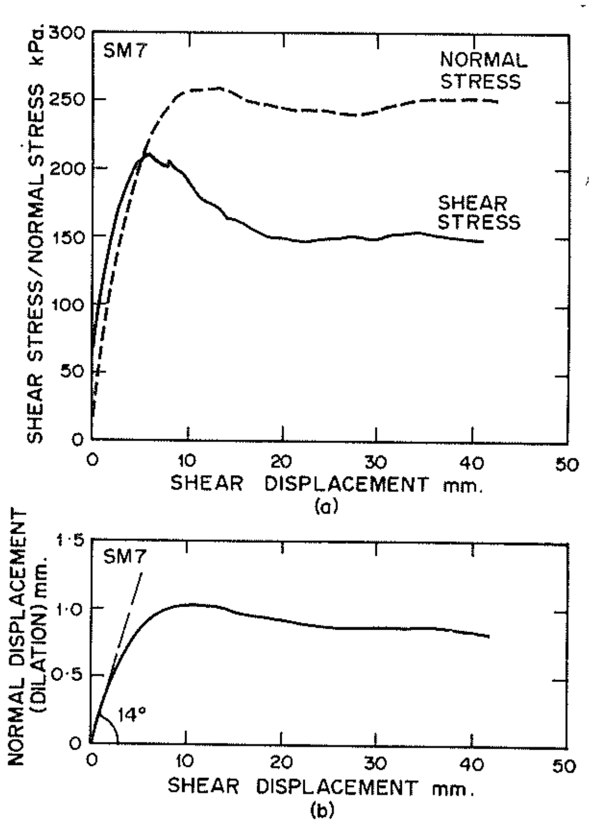

Figure 5 The result of a typical constant normal stiffness direct shear test on a mudstone-concrete joint. The mudstone was obtained from a normally drilled socket in highly weathered mudstone.

Figures 6(c) and 7(c) indicate the effect of normal stiffness, i.e. rock mass modulus, on the peak side resistance. This effect is particularly relevant to the design of piles socketed into a jointed rock mass in which the mass modulus has been significantly reduced by jointing (e.g. Deere *et al.*, 1966). In order to quantify the effect of modulus changes in a general form, it is necessary to normalize the curves contained in Figures 6(c) and 7(c) in terms of the side resistance factor β, which represents the decrease in side resistance relative to that in an intact rock, and the mass factor j, which represents the decrease in mass modulus relative to that of an intact rock. The result of normalizing the curves is shown in Figure 8, which indicates that the peak side resistance decreases at a much slower rate than the rock mass modulus.

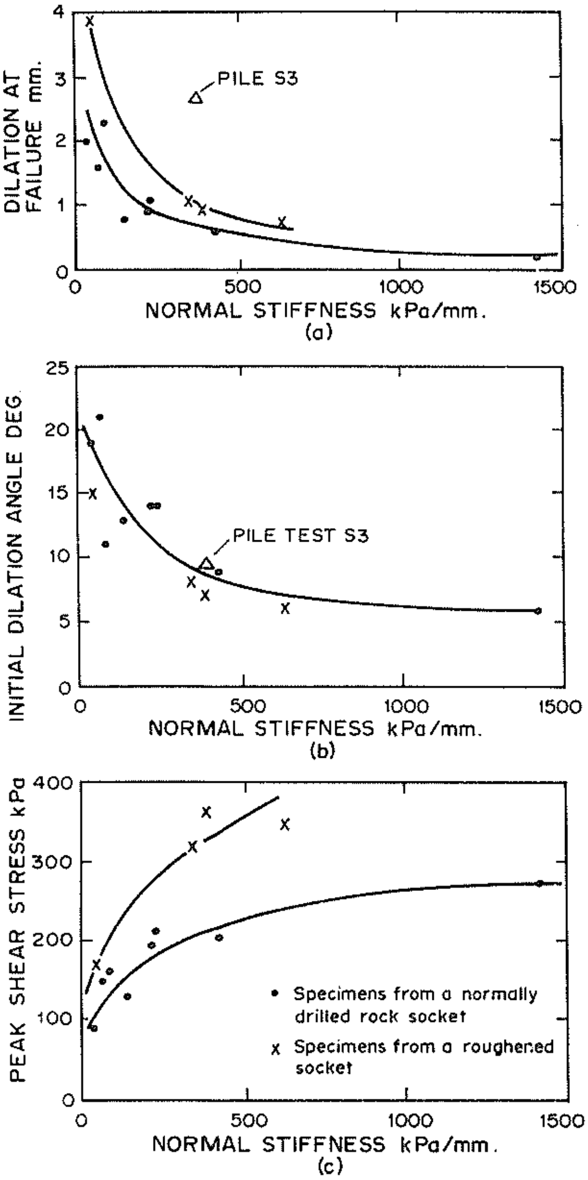

Figure 6 Summary of constant normal stiffness direct shear tests on mudstone-concrete joints. The mudstone was obtained from normally drilled and roughened rock sockets in highly weathered mudstone. The results of pile test S3 are also shown.

5 FIELD DATA

A series of pile tests was made on 660 mm to 1300 mm diameter piles constructed in highly and moderately weathered mudstone. The tests were designed primarily to measure the peak and residual side resistances, however, the opportunity was taken to obtain field data concerning the dilation of pile-rock interfaces and the development of normal stresses during shearing.

The dilation of the pile-rock interface was assessed from the results of displacement transducers installed in the rock to measure the radial diplacement of the rock between a point near the interface and a point 1 m from the interface. The displacement measured over the gauge length was adjusted to provide an estimate of the interface dilation relative to an infinite boundary (Boresi, 1965). A typical result is shown with the load-settlement curve for pile S3 in Figure 9, where it is seen that the

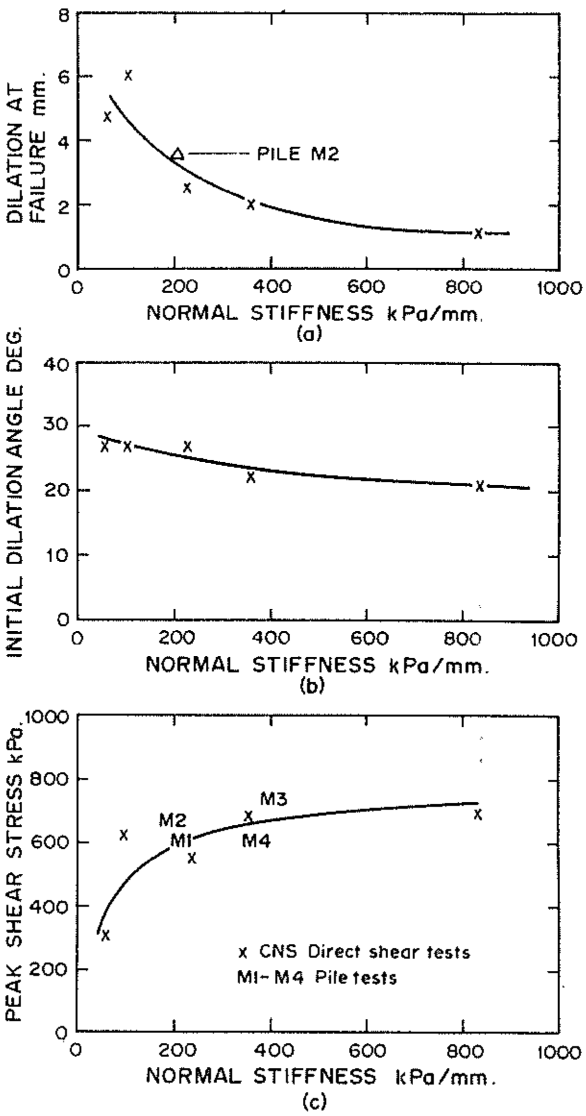

Figure 7 Summary of constant normal stiffness direct shear tests on mudstone-concrete joints. The mudstone was obtained from normally drilled sockets in moderately weathered mudstone. The results from pile tests in the same mudstone are also shown.

TABLE I

	TEST PILE S3	S5	M2	M8
Pile dimensions mm — diameter	1170	1120	1300	660
length	2510	2590	2000	1800
Socket roughness S_H mm	8.3	2.3	10.2	-
S_i deg	18	11	25	-
Rock unconfined compressive strength kPa	570	620	2300	2000
Residual interface dilation from transducers in rock mm	2.6	2.5	3.3	-
Residual normal stress from strain gauges kPa	1190	990	-	1570
Residual side resistance from load test	505	479	632	804
Residual side resistance from dilation gauges[1]	760	670	740	-
Residual side resistance from strain gauges[2]	460	380	-	770
Residual side resistance from laboratory tests[3]	340	220	600	-

Notes:

1 The normal stress has been calculated according to Equation (6) and residual side resistance has been calculated from $f_{sr} = \sigma \tan \phi_r$ with $\phi_r = 21^o$ for piles S3 and S5, 25^o for pile M2 and 26^o for pile M8 as indicated from residual direct shear tests.

2 The residual side resistance was calculated according to $f_{sr} = \sigma \tan \phi_r$ with ϕ_r as above.

3 Values selected from Figure 6(c) for piles S3 and S5 and from Figure 7(c) for pile M2 according to the rock modulus indicated by the pile tests.

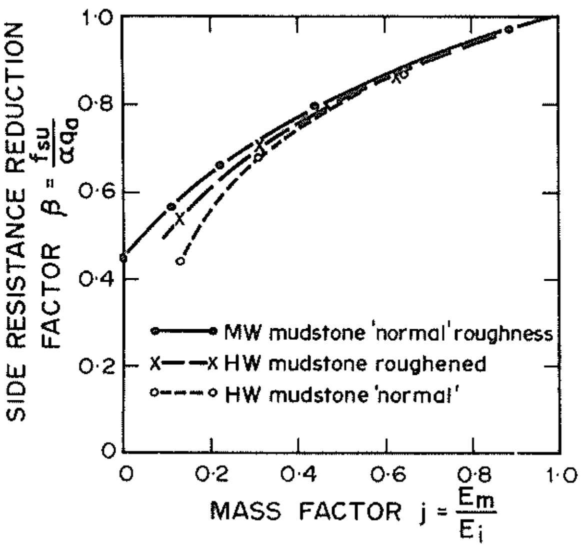

Figure 8 The effect of a reduction in the modulus of a rock mass on the peak side resistance, for sockets drilled in highly or moderately weathered mudstone.

dilation increases quickly as the pile load increases and then the dilation angle decreases to a very small value as the pile apparently reaches a residual condition.

The development of normal stress was determined for the test piles from the results of resistance wire strain gauges embedded into microconcrete briquettes and cast into the test piles. The strain gauges were placed to measure vertical and radial strain at various locations within the piles. The vertical and radial stresses in the pile were calculated on the basis of elasticity, e.g. Zienkiewicz (1971), to provide the distribution of vertical stress and radial (normal) stress with depth.

The results obtained from the test piles have been analysed with respect to the residual side resistance and summarized in Table I.

Although there is some scatter in the estimation of the residual side resistance according to the various measurements, the estimates are all of the same order and thus support the suggestion that the residual resistance depends largely on the development of normal stresses due to dilation of the pile rock interface.

6 DISCUSSION

The preceding paragraphs have demonstrated that the behaviour of the pile-rock interface of a side resistance pile is similar to that of a natural

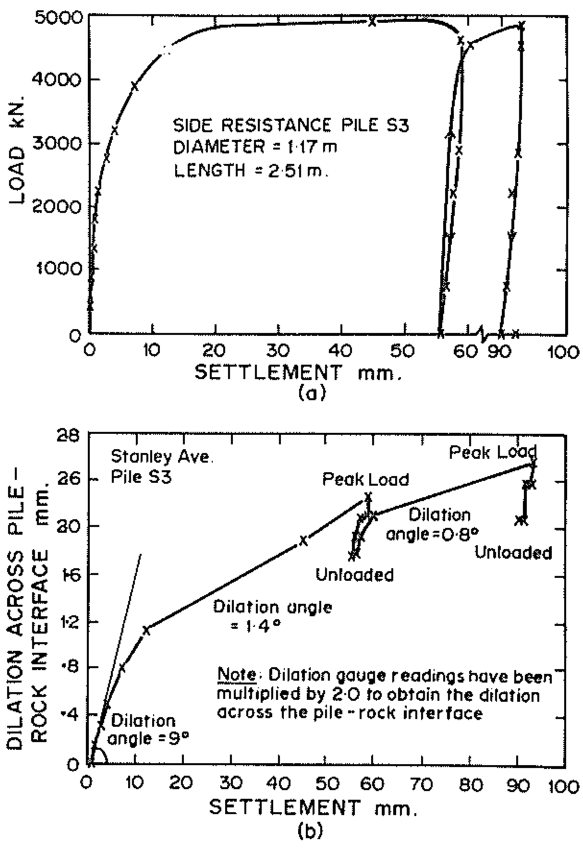

Figure 9 The load-settlement curve for a side resistance test in a roughened mudstone socket and the corresponding measured socket dilation.

TABLE II

THE IMPORTANCE OF ROUGHNESS TO SIDE RESISTANCE

Pile No.	Rock Description	Socket Size mm	Socket Roughness S_H mm	Socket Roughness S_i deg	q_a kPa	f_{su} kPa	f_{sr} kPa	$\frac{f_{su}}{q_a} = \alpha$	$\frac{f_{sr}}{f_{su}}$
S3	HW Siltstone (Melbourne mudstone)	1170 dia. x 2150	6.4	18	570	505	505	0.89	1.0
S5	" "	1120 dia. x 2590	3.3	13	620	485	479	0.78	0.98
S12	" "	335 dia. x 900	< 1 mm	-	620	412	275	0.44	0.67
M2	MW Siltstone (Melbourne mudstone)	1300 dia. x 2000	10.2	25	2300	640	632	0.28	0.99
M3	" "	1230 dia. x 2000	3.1	14	2300	710	710	0.31	1.0
M1	MW Siltstone (Melbourne mudstone)	1220 dia. x 2000 (bentonite)	2.3	13	2460	609	609	0.25	1.0
M4	" "	1350 dia. x 2000 (bentonite)	12	27	2340	617	590	0.26	0.96
Pells (1979) 6 tests	Sydney sandstone	160 to 710 dia.	smooth		6000	1000	640	0.17	0.64
Pells (1979) 2 tests	" "	210 dia.	2 mm grooves at 10 mm pitch		6000	1030	920	0.17	0.89
Pells (1979) 3 tests	" "	210 to 315 dia.	7.5 mm deep grooves at 10 mm pitch		6000	1240	1160	0.21	0.94

rock joint. There appears to be an initial phase during which dilation and normal stress increase rapidly in a manner similar to the sliding phase of the bilinear model. The rate of dilation appears to diminish to a negligible value as major shear planes develop approximately parallel to the direction of shear, and the behaviour enters a second phase which is similar in effect to the shearing of the asperities of the bilinear model. There appears to be a gradual transition from the first phase to the second as observed for natural rough joints by Barton (1976) and Ladanyi and Archambault (1970).

The peak side resistance may be regarded as a function of the normal stress, the bond between the concrete and rock, the strength of the intact rock, and the shape of the asperities. However, the residual side resistance will depend only on the normal stress and the residual friction angle of the rock, with the normal stress being determined primarily by the dilation due to shearing and elastic effects. The implications of this are apparent when the results of tests on smooth and rough sockets are compared for the results of pile tests shown in Figure 10.

The peak side resistance of a pile cast into a smooth sided socket will depend on the interface cohesion, or bond, plus a small normal stress due to Poisson's ratio effects. Such a pile will experience an abrupt loss of capacity when the bond is broken and will have a residual capacity which is significantly lower than the peak capacity. Pile behaviour of this type is illustrated in Figure 10 for pile C2 in a smooth sandstone socket and for pile S12 in a relatively smooth mudstone socket. Because the peak side resistance of a smooth socket depends largely on the concrete-rock bond, any smearing of remoulded rock or bentonite which may reduce the bond will also affect the peak side resistance. This effect was noted by Pells (1979), who found that failure to remove a thin coating of remoulded sandstone reduced the peak resistance by 60%, and that the casting of a pile under bentonite reduced the peak resistance by 76%.

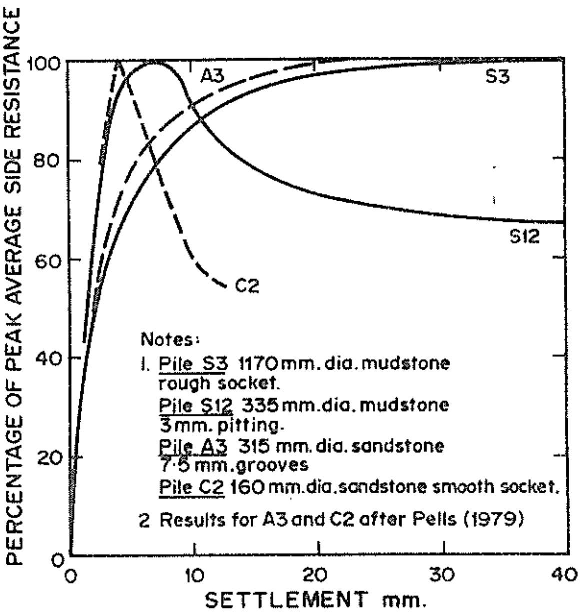

Figure 10 The effect of socket roughness on sockets in sandstone and mudstone

The behaviour of piles constructed in sockets with some degree of roughness has been found to be distinctly different to the smooth socket piles, as indicated in Table II and illustrated in Figure 10 for the relatively smooth socket tests C2 and S12 and the rougher socket tests A3 and S3. The provision of 7.5 mm deep grooves in 210 to 315 mm diameter sandstone sockets (Pells, 1979) and S_H of more than about 3mm in 1.1 to 1.2 m diameter mudstone sockets was found to increase the peak side resistance by 60% to 80%. The provision of roughness was also found to increase the residual side resistance to at least 90% of the peak value, as indicated in Table II. Further, the construction of piles under bentonite was found to reduce the capacity of rough sockets by less than 10%.

7 CONCLUSIONS

The mechanism of side resistance in rock socketed piles has been shown to be consistent with the concepts developed for the shear strength of rock joints.

In the case of smooth sockets, the failure mechanism has been found to consist initially of breaking the concrete-rock bond which results in a sharp loss of side resistance, followed by the development of a small normal stress due to minor interface dilation and Poisson's ratio effects. The peak side resistance of such sockets has been shown to be sensitive to the presence of any smear on the socket surface which could reduce the concrete-rock bond. The residual side resistance of smooth sockets has been shown to be much lower than the peak side resistance.

In the case of rough sockets, the failure mechanism has been found to consist of a sliding and progressive shearing of asperities, which produces a significant dilation across the pile-rock interface. The side resistance has been shown to approach the residual condition slowly, generally with little loss of side resistance because the interface dilation and associated normal stress are maintained naturally at a relatively high level. Although the provision of roughness on a rock socket has been shown to increase the peak side resistance, it appears that the peak side resistance does not increase significantly further for asperities rougher than about $S_H = 3$ mm and $S_i = 12^o$ (JRC = 10). The peak side resistance has also been shown to depend on the modulus of the rock mass, which follows from the relation between the interface dilation and the resulting normal stress.

8 AKCNOWLEDGEMENTS

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