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A Technique to Predict the Side Resistance Behaviour of Rock Socketed Piles

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SUMMARY This paper presents a relatively simple semi-empirical technique to predict the overall stress and displacement characteristics of joints under the constant normal stiffness condition. It is shown how the technique can be used to predict the load-settlement response of side-resistance only rock socketed piles. A comparison of predicted and actual pile performance is presented.

1 INTRODUCTION

The performance of a rock socketed pile is a function of the development of its side and base resistance. The development of the side resistance, to a large extent, depends on the strength and deformation properties of the rock, the roughness of the concrete-rock interface and also the presence or absence of infilled material at the interface. As part of a major research project into the performance of piles socketed in Melbourne mudstone at Monash University, the side shear behaviour of these piles has been studied through a series of laboratory and field tests and theoretical analyses.

The dilating action of the rock walls when a rock socketed pile with rough socket walls is displaced vertically can be simply modelled as radial expansion of a very long cylindrical hole in an infinitely large mass. From elasticity considerations (e.g. Boresi, 1965), it can be shown that the radial expansion of the cylindrical hole, u_r, is given by

$$\frac{\sigma_n}{u_r} = \frac{E}{r(1 + \nu)} \qquad (1)$$

where σ_n = radial or normal stress

r = radius of cylindrical hole

E = Young's modulus

ν = Poisson's ratio

If E, ν and r are constants, then

$$K = \frac{E}{r(1 + \nu)} \qquad (2)$$

is a constant.

Equations (1) and (2) give

$$\sigma_n = Ku_r \qquad (3)$$

On the basis of equations (1) - (3), the side resistance behaviour of rock socketed piles can be studied in the laboratory by testing plane joints under the constant normal stiffness (CNS) condition using a spring to represent the constant stiffness, K (Figure 1).

This paper presents a simple semi-expirical technique, using Ladanyi and Archambault's (1970, 1980) shear equation, to predict the shear behaviour of clean plaster-concrete joints with regular triangular profiles under CNS condition. The application of the technique to predict the behaviour of irregular joint profiles and side resistance rock-socketed pile behaviour (Dight and Chiu, 1981; Chiu and Dight, 1983) will also be discussed. It should be mentioned that it was originally intended to publish this paper before that of Dight and Chiu (1981) and Chiu and Dight (1983). This would then reflect on the sequence of ideas that emerged when the technique was first envisaged.

2 THEORY

2.1 Shear Equation

Various shear equations exist to relate the shear-normal stresses and shear-normal displacements of joints (e.g. Patton (1966), Jaeger (1970), Ladanyi and Archambault (1970), Barton (1971)). Patton's, Jaeger's and Barton's equations do not include the possibility of intact shearing of the joint asperities during sliding and hence the contribution of this mode of failure to the shear strength of the joint is not directly taken into account. Ladanyi and Archambault however, based their equation on contributions to shear strength from sliding friction, dilation and shear through joint asperities. Ladanyi and Archambault proposed

$$\tau = \frac{\sigma_n(1-a_s)(\dot{v} + \tan\phi_r) + a_s S_R}{1 - (1-a_s)\dot{v}\tan\phi_r} \qquad (4)$$

where τ = shear stress

σ_n = normal stress

a_s = the shear area ratio giving the proportion of joint area sheared through asperities.

$\dot{v}$ = rate of dilation, ratio of normal to shear displacement,

ϕ_r = base friction angle

S_R = shear strength of material composing the asperities

For a_s and $\dot{v}$, Ladanyi and Archambault used the following relationships

$$a_s = 1 - (1 - \frac{\sigma_n}{\sigma_T})^{k_1} \quad (5)$$

$$\dot{v} = (1 - \frac{\sigma_n}{\sigma_T})^{k_2} \tan i_o \quad (6)$$

where σ_T = the brittle-ductile transistion pressure (e.g. Mogi, 1966)

i_o = initial dilation angle

k_1 = 1.5

k_2 = 4.0

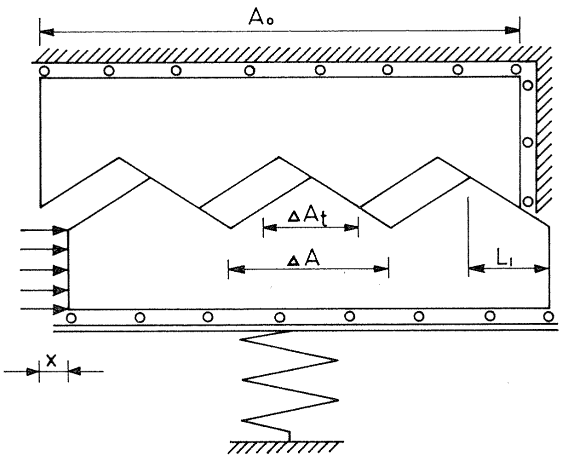

Figure 1 Constant Normal Stiffness Set-Up

Subsequently, with more joint shear tests of various rock types, Ladanyi and Archambault (1980) have suggested other expressions for a_s and $\dot{v}$. For this work, equations (7) and (8) were found to be most appropriate (Chiu, 1981; Dight and Chiu, 1981).

$$a_s = (\frac{\sigma_n}{\sigma_T})^{k_1} \quad (7)$$

$$\dot{v} = (1 - \frac{\sigma_n}{\sigma_T})^{k_2} \tan i_o \quad (8)$$

where k_1 = 0.75

k_2 = 3.0

Also for the shear strength of asperity material, S_R, the Fairhurst's criterion was determined to be most appropriate (Chiu, 1981).

$$S_R = q_u(\frac{m-1}{n})(1 + n\frac{\sigma_n}{q_u})^{\frac{1}{2}} \quad (9)$$

where q_u = uniaxial compressive strength

n = ratio of uniaxial compressive to tensile strength,

$m = (n + 1)^{\frac{1}{2}}$

During sliding with dilation, the true shear area decreases due to a loss of interlock and to account for this Ladanyi and Archambault (1970) included the term η in equation (4) (Figure 2) where

$$\eta = \frac{\Sigma \Delta A_t}{\Sigma \Delta A} = \frac{A_t}{A_o} \quad (10)$$

With equation (9), this gives

$$\tau = \frac{\sigma_n(1-a_s)(\dot{v} + \tan\phi_r) + a_s \eta q_u \frac{m-1}{n}(1 + n\frac{\sigma_n}{\eta q_u})^{\frac{1}{2}}}{1 - (1-a_s)\dot{v}\tan\phi_r} \quad (11)$$

$$a_s = (\frac{\sigma_n}{\eta\sigma_T})^{0.75} \quad (12)$$

$$\dot{v} = (1 - \frac{\sigma_n}{\eta\sigma_T})^{3.0} \tan i_o \quad (13)$$

For the transistion pressure, σ_T, Ladanyi and Archambault (1970) used Mogi's (1966) definition while Goodman (1976) suggested

$$\sigma_T = q_u \quad (14)$$

2.2 Prediction of Shear Behaviour

During a CNS hear test, the joint is at "failure" or yield once shear displacement and dilation have occurred. However, while joint dilation still occurs it can take more shear load. Only when the rate of increase of shear capacity due to dilation is less than the rate of drop of shear capacity due to loss of interlock does the joint become unable to sustain the shear load.

For a regular triangular profile, the degree of interlock (Figure 1),

$$\eta = \frac{A_t}{A_o} = 1 - \frac{x}{L_1} \quad (15)$$

where x = shear displacement

L_1 = projected length of a descending portion of the profile or half wavelength

Using equations (11) - (15), an iterative procedure was developed for the joint to determine the shear stress, normal stress and joint dilation for increments of shear displacement. A listing of the algorithm used is given in Chiu and Dight (1983).

3 REGULAR PLASTER-CONCRETE JOINT PROFILE SHEAR BEHAVIOUR

3.1 Laboratory Tests

To study the behaviour of clean joints in general and investigate the influences of stiffness and roughness, Williams (1980a, 1980b) performed nine direct shear tests on plaster-concrete joints. Six of these had regular 45^o asperities and the remaining three had 12^o asperity angles. The tests were all conducted under CNS condition (Figure 1) on approximately 190 mm long and 45 mm wide samples (Figure 2). Under the CNS condition, the normal force acting across the joint during a test is directly proportional to the magnitude of the constant normal stiffness and the current dilation of the joint. The normal stiffness used varied from about 35 to 1300 kPa/mm and these together with other relevant details are presented in Table 1.

During a test the shear force, normal force, shear displacement and dilation were measured and these have been used to give support to the joint behaviour prediction technique described in the previous section.

TABLE 1 DETAILS OF CNS TESTS ON PLASTER

Test	Initial Area, A_o (m^2)	Stiffness K (kPa/mm)	ϕ_r (deg)	Initial Normal Stress, σ_{n_o} (kPa)
PM4	0.0283	35.3	13.5*	90
PM5	0.0283	77.7	13.5	80
PM6	0.0283	229.7	13.5	80
PM7	0.0283	321.6	13.5	90
PM8	0.0283	1003.5	13.5	90
PM9	0.0283	1314.5	13.5	110
PM12	0.0283	908.1	41.0**	90
PM13	0.0283	134.3	41.0	90
PM14	0.0283	38.9	41.0	90

Plaster q_u = 20000 kPa; q_u/σ_t = 12

σ_t = tensile strength

* with mould release

** without mould release

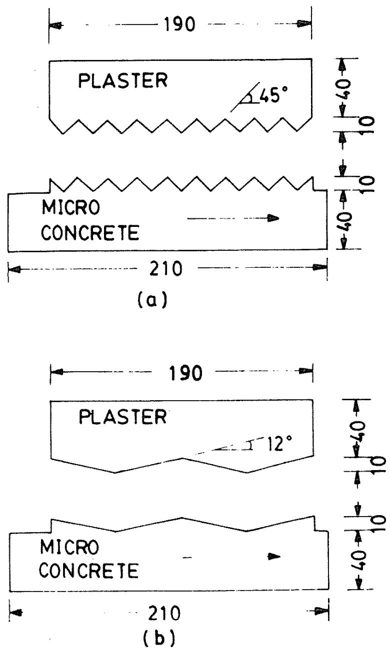

Figure 2 Geometry of Plaster-Concrete Joints

3.2 Comparison of Experimental and Predicted Results

The complete experimental results obtained by Williams (1980a, 1980b) are presented in Figures 3-4 as plots of dilation, shear stress and normal stress against shear displacement and shear stress against normal stress. These relationships as predicted by the shear equation procedure are also plotted in Figures 3-4.

In general, it can be seen that the joint dilation rates and the development of the stresses were reasonably well predicted. This is also evidence in Figure 5 where the predicted and actual normal stress, dilation and shear displacement at peak shear stress are compared indicating good predictions. Figure 5 however, also suggests that the technique tends to predict slightly higher values of displacements and stresses at shear failure. The reasons for the slight disagreements are possibly due to tensile and progressive shear failure of the plaster. Evidence of these has been reported by Williams (1980a).

4 APPLICATIONS

4.1 Laboratory Shear of Mudstone-Concrete Joints

As part of the research into pile side shear behaviour, Williams (1980a) conducted CNS tests on mudstone-concrete samples formed by casting concrete onto mudstone blocks obtained from actual socket walls. The profile of a typical joint sample is shown in Figure 6. By digitizing the irregular joint profile and calculating the statistics of the geometry of the profile, (e.g. average asperity angle, standard deviation of the asperity heights) the joint profile is then modelled as a regular triangular profile which has similar statistical geometrical properties as the joint. This then allowed the technique described in section 2 to be used to predict the joint shear behaviour. As an example, the predicted and observed shear behaviour under CNS condition of the profile shown in Figure 6 have been presented in Figure 7 to demonstrate the applicability of the prediction technique. More details and examples of the procedure to predict the shear behaviour of irregular joint profiles can be found in Dight and Chiu (1981).

4.2 Side-Resistance Behaviour of Rock-Socketed Piles

The prediction technique can also be applied to predict the performance of a side-resistance only rock-socketed pile (see section 1). However, some simplifying assumptions have to be made. Firstly, the lateral stiffness of the rock socket walls is assumed constant with depth and secondly the shear and normal stresses at the pile rock interface are constant for the full length of the pile.

To check the applicability of the above, the predicted load-settlement performance of a side resistance only pile in Melbourne mudstone tested by Williams (1980a) was compared with the observed behaviour. Figure 8 shows the geometry of the pile and the regular triangular profile (purpose cut) used for the pile-rock interface while Figure 9 presents the predicted and observed pile settlement behaviour. It can be seen that the correspondence between predicted and observed pile behaviour is good.

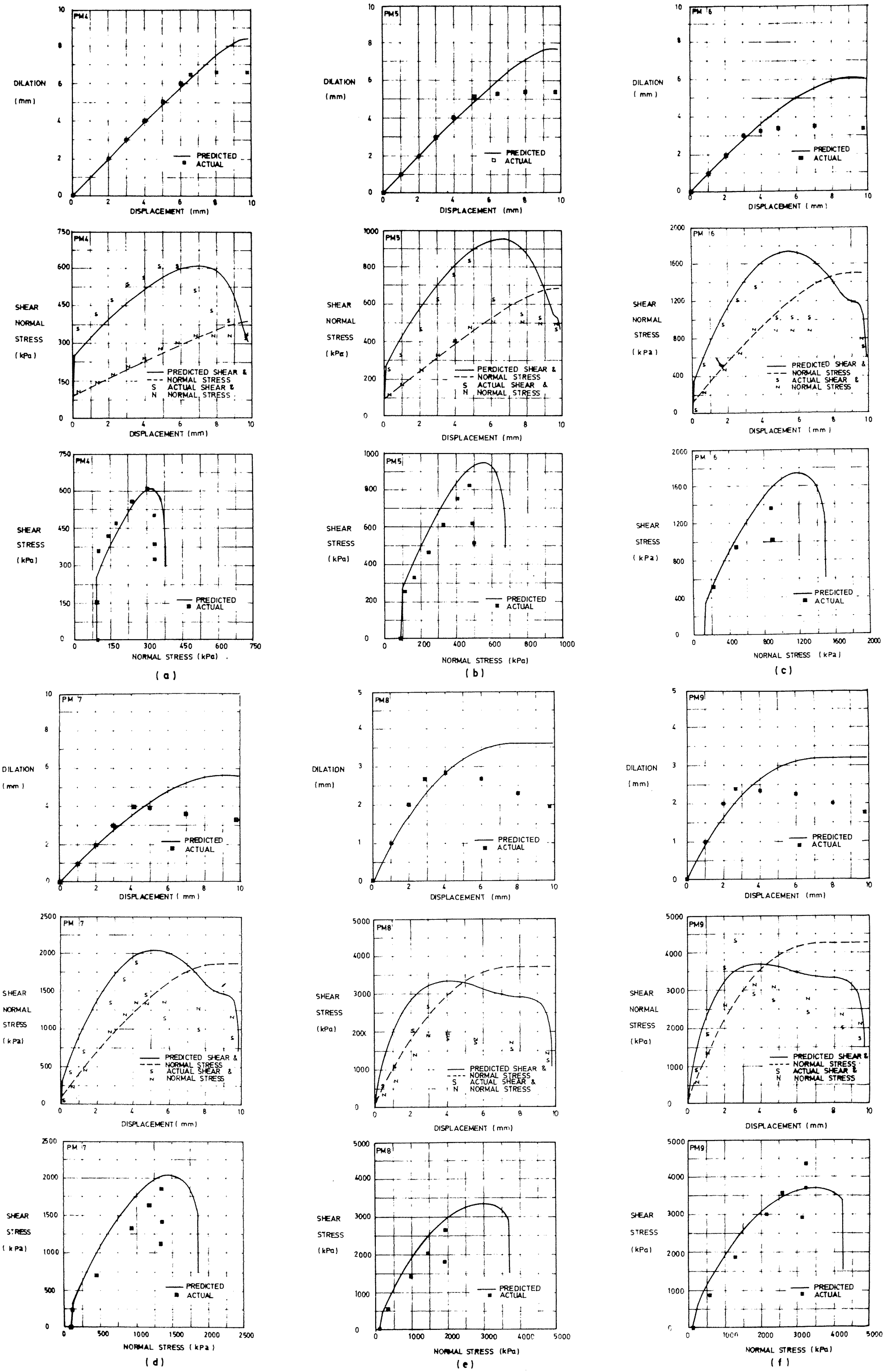

Figure 3 Predicted and Actual Shear Displacement, Dilation, Shear Stress and Normal Stress For Plaster-Concrete Joints with 45° Asperities.

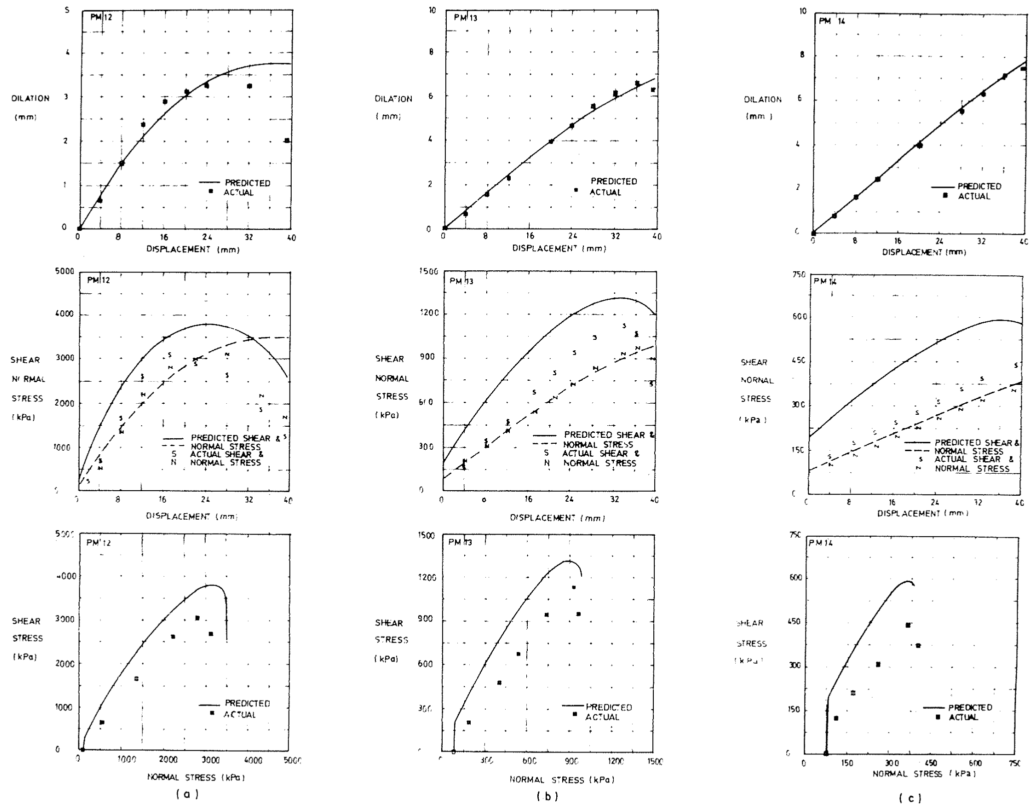

Figure 4 Predicted and Actual Shear Displacement, Dilation, Shear Stress and Normal Stress For Plaster-Concrete Joints With 12° Asperities.

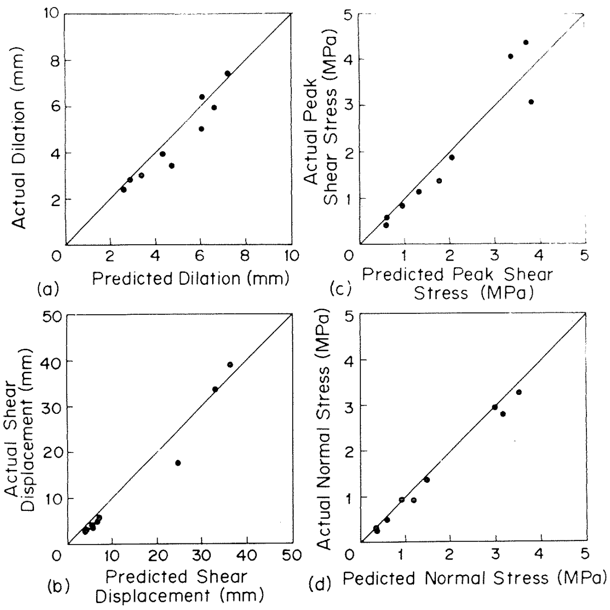

Figure 5 Comparison of Predicted and Actual Displacements and Stresses

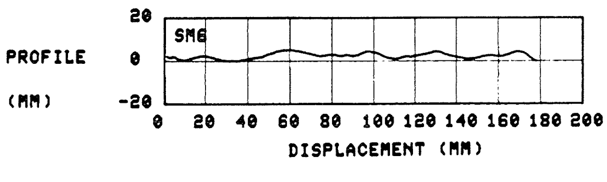

Figure 6 Digitized Mudstone-Concrete Joint Profile

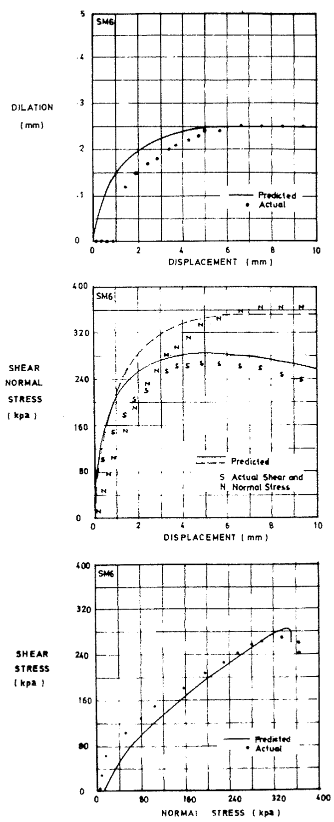

Figure 7 Predicted and Actual Displacements and Stresses For Mudstone-Concrete Joint

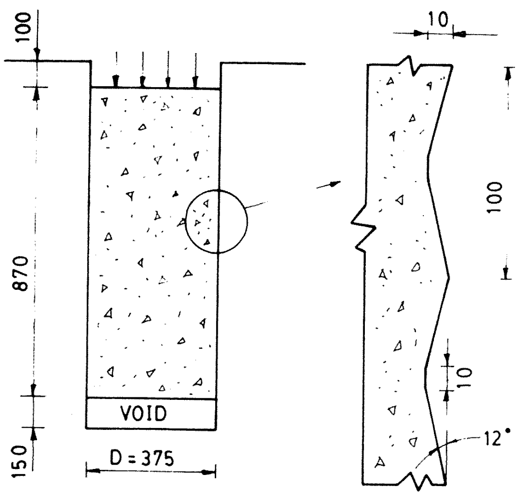

Figure 8 Geometry of Side-Resistance-Only Pile, S15

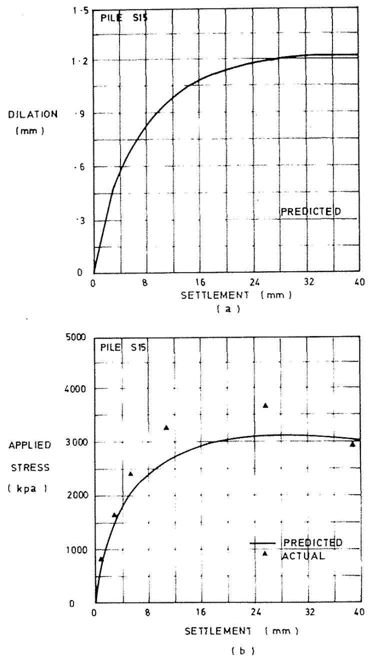

Figure 9 Predicted and Actual Pile Load - Settlement Relationship

Using the triangular profile model as described in section 2.2, the performances of side-resistance only piles with irregular pile-rock interfaces can also be predicted. Details of this and justification of the assumptions employed have been discussed and presented by Dight and Chiu (1981) and Chiu and Dight (1983).

5 CONCLUDING REMARKS

A simple technique using Ladanyi and Archambault's shear equation was developed to predict the shear behaviour of joints under the constant normal stiffness condition. The validity of the technique was demonstrated by the reasonably good comparisons between predicted and observed shear behaviour of clean plaster joints with regular triangular asperities. It was then shown how the technqiue could be used to predict the shear behaviour of joints with irregular profiles and also the load-settlement performance of side-resistance only rock socketed piles.

7 ACKNOWLEDGEMENTS

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