FOUNDATIONS ON SANDSTONE AND SHALE IN THE SYDNEY REGION

P.J.N. PELLS, G. MOSTYN and B.F. WALKER

1 INTRODUCTION

In 1978 a sub-committee of the Australian Geomechanics Society published a paper titled "Design Loadings for Foundations on Shale and Sandstone in the Sydney Region" (Pells et al, 1978). The paper included a classification system for the Sydney shales and sandstones and recommended design parameters for pad and socketed pile foundations. The classification system and guidelines given in the paper were based mainly on data from field tests and on the experience of the five authors. They have been adopted by the design and construction fraternities on a very wide basis over the past 20 years. They have also been found useful outside of the geographic area for which they were intended.

However, the 1978 paper was written before a substantial burst in research activity relating to foundations on rock which occurred from the late 1970's through to the 1980's. The information from this research work, coupled with experience with the 1978 system and developments in limit state design, warrant some revisions to the original guidelines and parameters. This paper summarises relevant research findings over the past two decades, provides modifications to the original classification system to remove ambiguities, and gives revised guidelines for incorporation in limit state design.

2 RELEVANT DEVELOPMENTS

It appears to have been in Canada, and patches of the USA, in the late 1960's and early 1970's that serious research started in relation to foundations on rock which ignited interest in Australia, South Africa, other parts of the USA and some parts of the UK. Early work on the bearing capacity of rock was done by Ladanyi and Roy (1972) and Bishnoi (1968), while Gill (1970) produced the first detailed study of the load distribution in a rock socketed pile using linear and non-linear finite element analyses.

To a substantial extent workers in Australia at Monash University and Sydney University picked up the baton in the late 1970's and undertook substantial field and laboratory testing and theoretical studies (Rowe and Pells, 1980; Pells et al, 1980; Williams, 1980; Williams et al, 1980; Williams and Pells, 1981). In Canada, Horvath (1982) undertook similar field and laboratory testing. Interaction continued with north American workers (eg Kulhawy, 1988 and Rowe, 1984) and culminated in three key publications, namely:

- Structural Foundation on Rock (International Conference held in Sydney in 1980).
- The Design of Piles Socketed into Weak Rock (National Research Council, Ottawa, 1984).
- Analysis and Design of Drilled Shaft Foundations Socketed into Rock (Cornell University, 1987).

Since the peak period of the 1980's research and field testing work has continued particularly at Monash University (Seidel and Haberfield, 1995 and 1999) where a commercial computer program, ROCKET, has been developed which provides a design tool based on analyses incorporating a detailed understanding of the role of sidewall roughness on the performance of socketed piles.

In regard to pad or strip footings the developments contained in the above research have served to provide an understanding of bearing pressures up to which load-deflection behaviour is essentially linear, and the very high bearing pressures which may be deemed to be ultimate capacities. Once these pressures are known, design is trivial.

However, in regard to socketed piles a considerable effort has been extended to understanding factors which control sidewall shear stress versus displacement behaviour, and the applied mechanics of interaction between sidewall and end bearing.

It is considered that the three publications listed above contain adequate information for proper design of foundations on and in rock. They contain several different design methods which appropriately include rock mass properties and the applied mechanics of foundation ground interaction.

3 KEY CONSIDERATIONS IN THE DESIGN PROCESS

There are three key facets which impact on the design process. These are:

1. Construction methodology and quality control.
2. Knowledge of the rock mass properties.
3. Applied mechanics of socket behaviour.

3.1 CONSTRUCTION METHODOLOGY AND QUALITY CONTROL

The over-riding question is whether the foundation is to be designed for load sharing between side and base or whether it is to be a side only or base only design. This decision depends on geometry and the construction method.

If D/b (ie depth in rock over minimum plan dimension) is less than 1, the footing is generally designed for base bearing only.

If the base may not be clean or the concrete might be of doubtful quality, then the footing should be designed for side-shear only. This has important implications in regard to the design safety factor because without the "back-up" of end bearing total reliance rests on the side shear strength.

For deep (D/b > 1) footings, the second construction issue relates to sidewall cleanliness and roughness. The design parameters and methods discussed here presume that the socket sidewalls will be free of crushed and smeared rock. They also presume a knowledge of the sidewall roughness. Ensuring clean sidewalls of appropriate roughness is not a trivial construction problem. Experience with Sydney sandstones and shales, and with the sedimentary foundation rocks of Brisbane's Gateway Bridge, suggests that the easiest way to ensure clean sidewalls is for the socket to be drilled under water. Alternatively the socket hole can be filled with water after drilling and then stirred using the drilling bucket or auger. Another alternative is to use a special tool fitted with sidewall cleaning teeth which is passed up and down the socket a few times after completion of drilling. This latter approach has to be used in rocks which soften or slake significantly when exposed to free water.

Walker and Pells (1998) give practical guidelines for specifications for socketed piles and for appropriate levels of inspection and testing. That paper is based primarily on experience in the Sydney area and alternative approaches may be appropriate in other geological environments.

It is noted that a great number of socketed piles are installed using simple auger drilling equipment and under conditions where there is insufficient attention given to adequate base or sidewall cleanliness. For such foundations the modern design parameters and methods are inappropriate and one can simply hope that those responsible for such work adopt very conservative allowable loads.

3.2 KNOWLEDGE OF ROCK MASS PARAMETERS

In essence all the modern design methods require good knowledge of:

1. The equivalent Young's moduli of the rock.
2. The average unconfined compressive strength of the rock.
3. The average roughness of the sidewalls for a deep footing.

Assessment of Items 1 and 2 are a basic part of rock mechanics and there are good guides in many texts for measuring and assessing these parameters. The publication by Rowe and Armitage (1984) is an excellent source of information.

There is no universal classification of roughness. A simple classification system is reproduced in Walker and Pells (1998). It has been found that sockets in sandstone need to be R2, or rougher, to preclude brittle failure of the interface. Sockets of R4 roughness may, in principle, be designed for higher side shear stresses than those of R2 or R3.

Substantial work on the fundamental influence of roughness on side shear behaviour has been completed at Monash University in the past decade. Seidel and Haberfield (1995 and 1999) have developed a roughness model based on the mean asperity angle and the scale (chord length) at which the mean angle is measured. Their research has shown that, at a given displacement, the shear resistance is controlled by the angles of asperities with chord lengths of twice this displacement. Thus if one wishes to compute the likely shear resistance at a displacement of 10mm the key parameter is the average angle of asperities with a chord length of about 20mm.

3.3 APPLIED MECHANICS OF SIDE SHEAR BEHAVIOUR

There are three very important matters of which engineers need to be aware when relying on side shear.

The first is that load will be shared between the sidewall and base according to the relative stiffnesses of footing, sidewall rock and toe rock. It is simply not tenable to arbitrarily ascribe certain portions of load carrying capacity to the sidewalls and the base in accordance with notional allowable values. In other words allowable side shear and allowable end bearing stresses are not additative. For example in a 5m long, 0.6m diameter socket in good quality sandstone it is probable that > 95% of the load will be taken by side shear.

The second is that provided sidewall roughness is R2 or better (see Walker & Pells, 1998) the sidewall stress-displacement behaviour will be non-brittle. Peak shear strength will be mobilised at a small displacement and will then "hang in there" (ie plastic behaviour).

The third point is that peak side-shear resistance is usually mobilised at much smaller displacements than peak end bearing pressures.

These three points mean that in order to mobilise significant base resistance it is necessary to invoke sidewall slip. In other words one has to mobilise full sidewall resistance if one is to make use of the substantial capacity which may be available in end bearing. In turn this means that all the safety margin will be in end bearing.

If there is uncertainty as to the ultimate capacity in end bearing then mobilization of full side slip is not appropriate.

In regard to sidewall slip it should be noted that the progression from first slip, at the location of highest shear stress, to complete slip, takes place over a small interval of displacement (Kulhawy and Carter, 1988; Rowe and Armitage, 1984). Therefore for most practical purposes it is appropriate to ignore the small region of load displacement behaviour representing progressive slip and to assume the relationship to be bilinear (see Figure 1).

4 DESIGN PARAMETERS

A general review of design parameters for rock socketed piles and pad footings on rock is given in a review paper in the 8th Australia New Zealand Geomechanics Conference (Pells, 1999). The present paper restricts consideration to the Triassic sandstones and shales of the Sydney region. However, before considering design side shear and end bearing parameters it is necessary to revisit the 1978 classification system for the shales and sandstones.

4.1 CLASSIFICATION SYSTEM

The experience of the part two decades indicates that the 1978 classification system warrants no significant revision. A minor change is that clays are treated similarly to other seams. The major matter requiring clarification is the definition of defect spacing. The revised classification is given in Table 1.

The classification system is based on rock strength, defect spacing and allowable seams as set out below. All three factors must be satisfied.

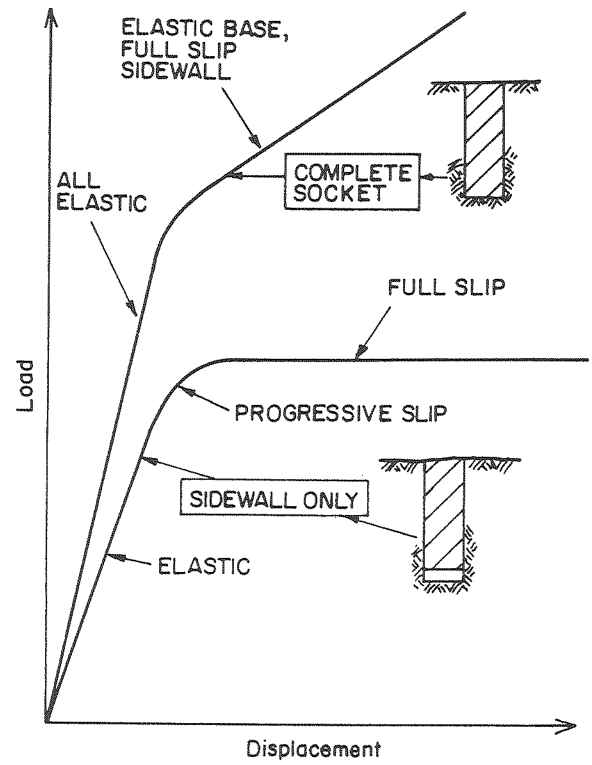

Figure 1 Simplified Load – Displacement Curves

Class	Unconfined compressive strength q_u (MPa)	Defect spacing	Allowable seams
I	>24	>600mm	<1.5%
II	>12	>600mm	<3%
III	>7	>200mm	<5%
IV	>2	>60mm	<10%
V	>1	N.A.	N.A

Table 1a Classification for sandstone

Class	Unconfined compressive strength q_u (MPa)	Defect spacing	Allowable seams
I	>16	>600mm	<2%
II	>7	>200mm	<4%
III	>2	>60mm	<8%
IV	>1	>20mm	<25%
V	>1	N.A.	N.A.

Table 1b Classification for shale

4.1.1 DEFECT SPACING

Pells et al (1978) adopted a scale for "degree of fracturing" presented in McMahon et al (1975). This scale was neither exhaustive nor mutually exclusive and, therefore, it was difficult to apply unambiguously. In fact many organisations ignored it when applying Pells et al (1978). It is considered that some scale capturing degree of fracturing is important. The Australian Standard for Geotechnical Site Investigations (AS1726-1993) does not include such a scale but the draft International Standard for Identification and Descriptions of Rock (ISO/DIS 14689) and ISRM suggested methods have the scale given in Table 2 relating to the spacing of natural fractures in N or H sized core. The boundaries in this scale have been adopted in the revised scheme.

4.1.2 ALLOWABLE SEAMS

Seams include clay, fragmented, highly weathered or similar zones, usually sub-parallel to the loaded surface. The limits suggested in the table relate to a defined zone of influence. For pad footings, the zone of influence is defined as 1.5 times the least footing dimension. For socketed footings, the zone includes the length of the socket plus a further depth equal to the width of the footing. For tunnel or excavation assessment purposes the defects are assessed over a length of core of similar characteristics.

Defect spacing mm	Terms used to describe defect spacing[1]
>2000	Very widely spaced
600-200	Widely spaced
200-600	Moderately spaced
60-200	Closely spaced
20-60	Very closely spaced
<20	Extremely closely spaced

[1]After ISO 14689 and ISRM

Table 2 Defect Spacing

4.2 SIDEWALL SHEAR RESISTANCE

Two paths have been taken in regard to the development of sidewall shear strength parameters.

By far the most commonly used approach is the development of empirical relationships between sidewall shear strength ($\tau_{ave\ peak}$) and the rock substance unconfined compressive strength (q_u), see Williams & Pells (1981), Horvath (1982) and Rowe & Armitage (1984). The relationship is simply:

$$\tau_{ave\ peak} = \alpha\, q_u \qquad (1)$$

Figure 2 gives the results of field and laboratory tests on mudstones and sandstones as evaluated by Williams & Pells (1981).

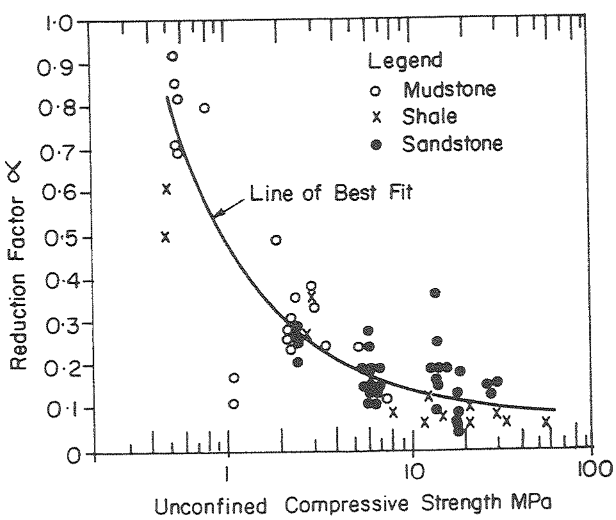

Figure 2 Side Shear Reduction Factor

Williams & Pells (1981) noted that the stiffness of the surrounding rock mass affects the side shear resistance and proposed a modification to Equation 1 to include a reduction factor for the influence of rock mass stiffness. Hence:

$$\tau_{\text{ave peak}} = \alpha\beta q_u \qquad (2)$$

where

β = modulus reduction factor which can be estimated from Figure 4.

It should be noted that the above equations do not represent lower bounds to all data points but are close to the best fit equations and represent correlation coefficients of greater than 80%.

One of the problems with putting test data from all over the world in one basket is that there is a large scatter; geological differences and differences in construction methodology are lost. For example Figure 3 shows the relationship between $\tau_{\text{ave peak}}$ and q_u for sockets in Hawkesbury Sandstone. It can be seen that for sockets of roughness R2 or better, $\alpha \geq 0.2$. This is substantially higher than would be obtained from the line of best fit from Figure 2.

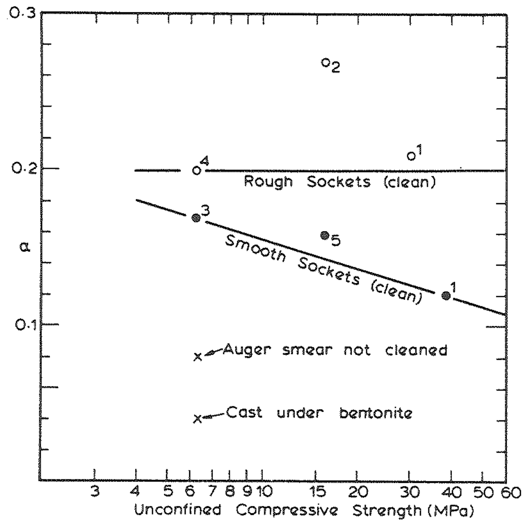

Figure 3 Side Shear Reduction Factor for Hawkesbury Sandstone

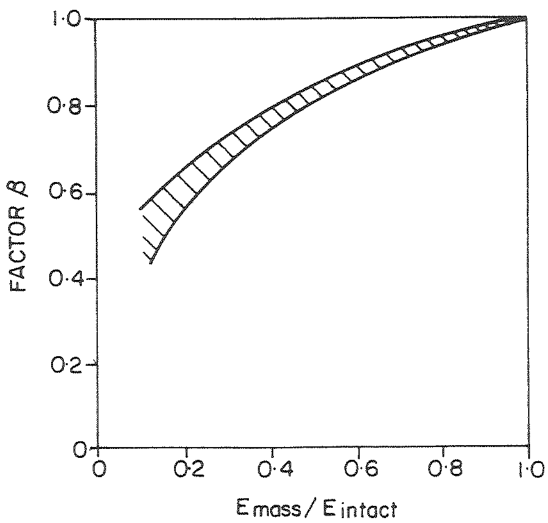

Figure 4 Reduction Factor for Rock Mass Stiffness

The data given in Figures 2 and 3 have been used as the basis for modifying the side shear strength recommendations given in the 1978 paper. At the same time the new guidelines, which are given in Table 5, are ultimate values so as to fit with limit state design methodology.

Sockets in uplift or ground anchors may be designed for the ultimate side shear values given in Table 5 but adopting a ϕ_g value of 0.5 (see Section 5). This presumes sidewalls free of smeared material and crushed rock and having a roughness at least equivalent to R2 or better. In small diameter anchor holes it is not usually feasible to assess the roughness. It may be assumed that if the holes are drilled using percussive equipment, and properly cleaned, the sidewall roughness will be appropriate. This is not true for holes drilled using diamond coring techniques and generalised design parameters cannot be given for this case. Sockets in uplift and anchors should, where appropriate, be checked for liftout of a mass of rock around the socket/anchor. The authors recognise that the shape of the liftout mass would be very complex. In Classes I to III shale and sandstone the mass is likely to comprise slab-like shapes whereas in Classes IV and V it is more likely to approach the cone shape commonly used for soil anchors. Based on limited laboratory and field testing, and precedent in soil anchors, it is suggested that the design check be made on the following basis:

- Assume a cone with an included angle of 90° measured from the distal end of the socket or anchor.
- Adopt a mobilised shear on the side of the cone (τ_c) of 10% of the ultimate values given in Table 5 for Classes I, II and III and 5% for Classes IV and V.
- Calculate the weight of the cone, using buoyant unit weight if appropriate (w).
- Calculate the vertical components of uplift resistance generated by shear on the side of the cone.
- Perform an ultimate strength limit state design check as per equations 7 and 8 in Section 6.

4.3 END BEARING

Detailed discussions of the bearing capacity of rock are given in Bishnoi (1968), Carter and Kulhawy (1987), Landanyi and Roy (1972), Pells and Turner (1980) and Wyllie (1992). It has been shown that:

1. For intact rock the ultimate bearing capacity is many times greater than the unconfined compressive strength, q_u of the rock (see Tables 3 and 4 for example of theoretical calculations and field measurements).

Method	Bearing capacity as multiple of unconfined compression strength q_u	
	$\phi_p = 40°$	$\phi_p = 45°$
Ladanyi expanding sphere	11	13
Modified Bell (brittle)	9	12
Classical plasticity	34	56

Table 3 Theoretical bearing capacity of rock

Material	Test type	Substance unconfined strength, q_u MPa	Bearing capacity as multiple of q_u
Sandstone (1)	Laboratory	20-33	11 (average)
Sandstone (2)	Laboratory	103	> 10
Limestone	Laboratory	75	7 to 11
Class 2 Hawkesbury	Field	14	5.5
Class 4 Hawkesbury	Field	6	2 to 2.5
Melbourne Mudstone (Surface)	Field	3	6 (brittle failures)
Melbourne Mudstone (L/D >3)	Field	2	>12.5 (work hardening)

Table 4. Measured bearing capacities - Model and field tests

2. The load-displacement behaviour for a massive (intact) rock is nearly linear up to bearing pressures of between 2 and 4 times q_u.
3. The ultimate bearing capacity of a jointed rock mass beneath the toe of a socketed pile can be approximated by Ladanyi's spherical expansion theory.
4. Ultimate bearing capacities for intact and jointed rock are attained at large displacements, typically > 5% of the minimum footing dimension.
5. The load-deflection behaviour of a jointed rock mass is nearly linear up to pressures at which significant cracking propagates through inter-joint blocks. Based on the work of Bishnoi (1968) such cracking may be expected at between about 75% and 125% q_u.

The above points mean that for footing design in many rock masses the base behaviour can be modelled as linearly elastic up to Serviceability Limits.

Based on the research findings summarised above, values of end bearing pressure on the Sydney shales and sandstones to cause settlements of < 1% of a footing diameter (or minimum plan dimension) are given in Table 5. Load-displacement behaviour would be linear up to those values. Also given in Table 5 are ultimate end bearing values which may be used for calculating geotechnical strength limit states.

An important issue in regard to city buildings is the design of footings adjacent to property boundaries. In the past (eg NSW Ordinance 70) the practice was to reduce pressures by some nominal value. This is illogical as it may not be safe if there is an adjacent deep excavation and associated joints in the rock which would allow kinematic failure of the mass beneath the boundary footing. Equally the reduction may be unnecessary given that end bearing pressures are usually controlled by allowable settlements.

It is suggested that if it is required that all footings settle about the same amount then boundary footing pressures should be about 60% of the pressure on footings remote from the boundary; a boundary footing is in this regard defined as one where the distance from the centre of the footing to the boundary is less than the footing width normal to the boundary. Where there are existing adjacent excavations below footing level a careful check must be made for kinematic failure of a joint bounded block. In the Sydney CBD area such problems are particularly prevalent adjacent to N-S oriented faces. This is because the dominant joint set strikes NNE with dips of 65° to 90° either west or east.

Class	Ultimate end bearing[1] MPa	Serviceability end bearing pressure[2] MPa	Ultimate shaft adhesion[3] kPa	Typical E_{field} MPa
I	>120	12	3000	>2000
II	60 to 120	0.5 q_u Max. 12	1500 to 3000	900 to 2000
III	20 to 40	0.5 q_u Max. 6	800 to 1500	350 to 1200
IV	4 to 15	0.5 q_u Max. 3.5	250 to 800	100 to 700
V	> 3	1.0	150	50 to 100

[1]Ultimate values occur at large settlements (> 5% of minimum footing dimensions).
[2]End bearing pressure to cause settlement of <1% of minimum footing dimension.
[3]Clean socket of roughness category R2 or better.

Table 5a Design values for vertical loading on sandstone

Class	Ultimate end bearing[1] MPa	Serviceability end bearing pressure[2] MPa	Ultimate shaft adhesion[3] kPa	Typical E_{field} MPa
I	>120	Max. 8	1000	>2000
II	30 to 120	0.5 q_u Max. 6	600 to 1000	700 to 2000
III	6 to 30	0.5 q_u Max. 3.5	350 to 600	200 to 1200
IV	> 3	1.0	150	100 to 500
V	> 3	0.7	50 to 100	50 to 300

[1]Ultimate values occur at large settlements (> 5% of minimum footing dimensions).
[2]End bearing pressure to cause settlement of <1% of minimum footing dimension.
[3]Clean socket of roughness category R2 or better. Values may have to be reduced because of smear.

Table 5b Design values for vertical loading on shale

5 DESIGN SAFETY FACTORS - LIMIT STATE DESIGN

To date most design methods for footings on rock have been based on working loads coupled with conventional geotechnical engineering safety factors. Thus, for example, Williams & Pells (1981) propose a working load Safety Factor of 2.5 for side shear only sockets.

Unfortunately geotechnical engineers are being dragged, kicking and screaming, into the structural engineer's world of Limit State Design. The current Australian Piling Code (AS2159-1995) is a Limit State document and therefore, reluctantly, the writers accept that design of footings on rock must follow the same path. An interpretation of what this means is as follows.

5.1 LOADS AND LOAD COMBINATIONS

According to AS1170.1-1989 (Australian Loading Code) there are 6 basic combinations (plus 3 optional extras) of dead load, live load, wind load and earthquake load which have to be considered for assessment of the strength limit state. There are a further 5 different combinations for assuming short term serviceability limit states and a further 3 for long term serviceability limit states.

These multiple combinations of load can make the design process quite tedious. However, in many cases in the Sydney area the following combinations of load govern design of pad footings and rock socketed piles:

Strength Limit State

$$\text{Load } (S^*) = 1.25G + 1.5Q \tag{3}$$

$$\text{Load } (S^*) = 1.25G + W_u + \psi_c Q \tag{4}$$

Long Term Serviceability

$$\text{Load} = G + \psi_1 Q \tag{5}$$

where

G = dead load
Q = live load
W_u = wind load
ψ_c = 0.4 (except for storage facilities where $\psi_c = 0.6$)
ψ_1 = ψ_c

Earth pressure and liquid loads are considered as dead loads (part of G) for long term serviceability. For strength limit state, earth pressures are equivalent to live loads and liquid loads are equivalent to dead loads. Note that in the jargon of Limit State design, the term 'Design Action Effect' rather than 'Load' should have been used in Equations 3 to 5.

5.2 STRENGTH AND SERVICEABILITY

According to AS2159-1995 (Piling Code) the key definitions are:

Ultimate Strength

The limit state at which static equilibrium is lost or at which there is failure of the supporting ground or structural elements.

This covers four calculated values, namely:

R_g^* = Design geotechnical strength of pile
R_{ug} = Ultimate geotechnical strength of pile
R_s^* = Design structural strength of pile
R_{us} = Ultimate structural strength of pile

Serviceability

The limit state at which deformation of the piles will cause loss of serviceability of the structure.

In ordinary English this means the pile loading which is constrained by allowable settlements, or lateral movements. For some unknown reason this loading is not given a sub and superscripted symbol.

5.2.1 STRENGTH

The design geotechnical strength R_g^* is simply a factored down version of the calculated ultimate geotechnical strength, ie:

$$R_g^* = \phi g R ug$$

The design requirement is $R_g^* > S^*$.

Various values of ϕ_g are given in Table 4.1 of AS2159 but none cover the methods used for calculating the ultimate capacity of rock sockets. In the writer's opinion the following values are appropriate for calculations of complete sockets (side shear and end bearing) using any of the methods given in Section 6 of this Paper.

(i) Geological environments where there are substantial field testing data (in Australia this would at least include Melbourne mudstone and Hawkesbury sandstone) $\phi_g = 0.75$

(ii) Geological environments similar to (i) but where no specific field testing data are available $\phi_g = 0.65$

(iii) Geological environments not covered by the world wide data base on side shear and end bearing properties $\phi_g = 0.5$

For side shear only sockets the recommended values for the categories listed above are:

(i) ϕ_g = 0.6
(ii) ϕ_g = 0.5
(iii) ϕ_g = 0.35

The above recommendations presume that construction quality control of appropriate sidewall and base cleanliness is assured. If this is not the case then the Designer should be very conservative and it is not appropriate to give prescriptive design criteria.

Calculation of R_{ug} is discussed in Section 6.

5.2.2 SERVICEABILITY

Deflection limits (settlements and lateral movements) are constraints imposed by the structure and are generally provided by the Structural Engineer. In the absence of specific requirements it is usually reasonable to design for settlements of between 5mm and 15mm.

Interestingly AS2159 provides for no "safety factor" in design for serviceability. In fact Clause 4.4.4 states *"calculations of settlement, differential settlement .... shall be carried out using geotechnical parameters which are appropriately selected and to which no reduction factor is applied"*. Apart from this clause being tautological the writers consider that it is wrong. This is because, as is discussed in Section 6, design of footings on or in rock are often governed by serviceability. Yet no allowance is made for the substantial uncertainty in assessing ground deformation parameters.

It is recommended that modulus reduction factor should be applied for calculations relating to long term serviceability. Suggested values are:

(a) Mean in situ deformation properties assessed from pressuremeter testing or other large scale in situ measurements $\phi_m = 0.75$

(b) Mean in situ deformation properties assessed by correlation with rock mechanics indices such as RQD or RMR $\phi_m = 0.5$

Table 5 gives typical in situ modulus values for the different classes of shale and sandstone. These values are essentially the same as given in the 1978 paper. They have been checked against a number of field records over the past two decades. Choice of the appropriate value from the ranges given in the table must be made on the basis of whether the rock is at the upper level or lower level of the particular class. On the assumption that this choice is made appropriately the authors consider that a ϕ_m value of 0.75 may be adopted for serviceability state calculations.

6 DESIGN METHODS

6.1 FOUNDATIONS IN COMPRESSION

Given the design parameters in Table 5 it is a trivial mater to perform the geotechnical design for pad or strip footings.

For socketed piles the writers considered that there are three design methods which may be used with confidence. These are:

- Elastic design (Rowe & Pells, 1980; Pells et al, 1978)
- Side slip design (Rowe & Armitage, 1984; Carter and Kulhawy, 1987)
- Non-linear design (Williams et al, 1980; Seidel & Haberfield, 1999)

The use of these methods in conjunction with limit state design procedures is presented in detail in a review paper at the 8th ANZ Conference (Pells, 1999) and there is no need to repeat the information here.

The writers strongly support the use of the method of Rowe and Armitage (1984). Its use requires no more than the parameters given in Table 5, and the simple graphical procedures allow a designer to readily appreciate the relative influence of geometrical and geotechnical parameters on the design.

The method developed by Williams et al (1980) allows for non-linear behaviour of a complete rock socket. The method could, in principle, be used for any geological environment but requires good field test data to provide the empirical parameters. The data given in Table 5 are not suited to this method. The Williams method is a little laborious and it is understood (Seidel, personal communications) that the advent of the computer program ROCKET has largely lead its demise.

As set out in Seidel & Haberfield (1999, Pt3) ROCKET essentially addresses vertical slip displacements between concrete shaft and rock. The computed side shear resistance is coupled with an assumption of linear load-displacement behaviour of the base in order to predict load displacement behaviour of a complete socket. To calculate total displacements of a complete socket additional calculations are made of:

- vertical elastic deformation of the rock mass, and
- elastic shortening of the shaft.

The writers, as with all other users, have no knowledge of the inner workings of ROCKET. It is a reasonably user-friendly program which clearly, in principle, allows a designer to explore the sensitivity of a design to the 8 input parameters which affect sidewall behaviour. The program is not suited to be used with the parameters given in Table 5.

6.2 FOUNDATIONS IN UPLIFT

As discussed in Section 4.2 it is recommended that designs be checked for 'piston-pullout' and liftout of a cone of rock. Both are ultimate strength limit state calculations and the following equations are appropriate.

'Piston-pullout'

$$R_{ug} = \phi_g \pi d L \tau_{ave\ peak} \tag{6}$$

where

d = shaft diameter
L = shaft length
$\tau_{ave\ peak}$ = ultimate side shear value from Table 5
$\phi_g = 0.5$

Cone-liftout

$$R_{ug} = \phi_g \pi L^2 \tau_c \tag{7}$$

where

τ_c = mobilised side shear on 90° cone (see Section 4.2)
$\phi_g = 0.75$
$S^*_{mod} = S^* - w$
w = effective weight of cone

7 CONCLUSIONS

1. The classification system for shales and sandstones in the Sydney region given in 1978 has stood the test of time and warrants no significant revision. The only changes proposed in this paper remove some ambiguities in relation to the definitions of defect spacings.
2. The design parameters for side shear and end bearing given in the 1978 paper warrant modifications in light of the substantial research work completed in the past two decades and in accordance with current requirements for limit state design. New recommendations are given for these design parameters.
3. The design parameters for side shear and end bearing must be used in conjunction with design methods which appropriately model the interaction between side shear and bearing load-displacement characteristics. The paper summarises alternative methods which may be used with confidence.
4. Designs of surface footings and socketed piles should be done in accordance with limit state requirements. The existing Australian Piling Code does not adequately cover rock sockets and therefore this paper makes recommendations to fill gaps in the Code.

REFERENCES

American Society of Civil Engineers (1996). Rock Foundations. Technical Engineering and Design Guides As Adapted from US. Corps of Engineers No 16, ASCE Press, New York.

Bishnoi, B.L. (1968). Bearing Capacity of Closely Jointed Rock. PhD Thesis Georgia Institute of Technology, Atlanta.

Carter, J.P. and Kulhawy, F.H. (1987). Analysis and Design of Drilled Shaft Foundations Socketed into Rock. Report 1493-4 for Electric Power Research Institute, Palo Alto, California.

Gill, S.A. (1970). Load Transfer Mechanism for Caissons Socketed into Rock. PhD Thesis, Northwestern University.

Holden, J.C. (1984) Construction of Bored Piles in Weathered Rocks. Road Construction Authority of Victoria, Technical Report No 69.

Horvath, R.G. (1982). Behaviour of Rock-Socketed Drilled Pier Foundations. PhD Thesis, University of Toronto.

Kulhawy, F and Carter, J.P. (1988). Design of Drilled Shaft Foundations. Univ. Sydney, Short Course Lecture Notes, Sydney.

Ladanyi, B. and Roy, A. (1972). Some Aspects of Bearing Capacity of Rock Masses. Proc. 7th Canadian Symposium on Rock Mechanics, Edmonton.

McMahon, B.K., Douglas D.J. and Burgess, P.J. (1975). Engineering Classification of Sedimentary Rocks in the Sydney Area. Australian Geomechanics Journal Vol G5, No 1, p51-53.

Pells, P.J.N., Rowe, R.K. and Turner, R.M. (1980). An Experimental Investigation into Side Shear for Socketed Piles in Sandstone. Int. Conf. Structural Foundations on Rock, Sydney, Balkema.

Pells, P.J.N. and Turner, R.M. (1980) End bearing on Rock with Particular Reference to Sandstone. Int. Conf. Structural Foundations on Rock, Sydney, Balkema.

Pells, P.J.N., Douglas, D.J., Rodway, B., Thorne, C.P. and McMahon, B.R. (1978). Design loadings for Shales and Sandstones in the Sydney Region. Australian Geomechanics Journal, G8.

Pells, P.J.N. (1999). State of Practice for the Design of Socketed Piles in Rock. 8th ANZ Geomechanics Conference, Hobart.

Rowe, R.K. and Armitage, H.H. (1984). The Design of Piles Socketed Into Weak Rock, Geotechnical Research Report GEDT-11-84, University of Western Ontario.

Rowe, R.K. and Pells, P.J.N. (1980). A theoretical study of pile-rock socket behaviour. Int. Conf. Structural Foundations on Rock, Sydney, Balkema.

Seidel, J.P. and Haberfield, C.M. (1995). The Axial Capacity of Pile Sockets in Rocks and Hard Soils. Ground Engineering, March 1995.

Seidel, J. and Haberfield, C.M. (1999). The Shear Behaviour of Concrete - Soft Rock Joints. Part 1: Experimental Investigations; Part 2: Theoretical Analysis; Part 3: Performance of Drilled Shafts. ASCE Journal of Geotechnical Engineering. (In press)

Walker, B.F. and Pells, P.J.N. (1998). The construction of bored piles socketed into sandstone and shale. Australian Geomechanics, v33, Pt3.

Webb, D.L. and Davies, P. (1980). Ultimate Tensile Loads of Bored Piles Socketed into Sandstone Rock. Inst. Conf of Structural Foundations on rock, Sydney, Balkema.

Williams, A.F. (1980). The Design and Performance of Piles Socketed into Weak Rock. PhD Thesis, Monash university.

Williams, A.F, Johnston, I.W. and Donald, I.B. (1980). The Design of Socketed Piles in Weak Rock. Int. Conf. Structural Foundations on Rock, Sydney Balkema.

Williams, A.F and Pells, P.J.N. (1981). Side Resistance Rock Sockets in Sandstone, Mudstone and Shale. Canadian Geotechnical Journal, Vol 18, p 502-513.

Wyllie, D.C. (1992). Foundations on Rock. Spon.