SUBSTANCE AND MASS PROPERTIES FOR THE DESIGN OF ENGINEERING STRUCTURES IN THE HAWKESBURY SANDSTONE

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1 INTRODUCTION

The Hawkesbury Sandstone dominates the Sydney region, both from the viewpoint of engineering structures and the natural topography. This Formation thickens from its western and southern outcrop margins in the Blue Mountains and Illawarra to about 290 m near the Hawkesbury River.

This article is an expansion of one published in 1985 in the volume "Engineering Geology of the Sydney Region". The original document concentrated on the engineering properties of the intact sandstone (i.e. the 'substance') whereas this article covers both substance and rock mass parameters. Much of the original information on substance parameters is reproduced here with additions from papers published in the volume "Sandstone City" published in 2000 by the Geological Society of Australia. The rock mass data are taken from papers published since 1985.

2 SUBSTANCE PROPERTIES

2.1 COMPOSITION

When viewed in vertical section the Hawkesbury Sandstone may be divided into three facies (Conaghan in Herbert and Helby, 1980):

- sheet facies } - 95% of Formation
- massive facies }
- mudstone facies - 5% of Formation

The **sheet facies** comprises sets of cross-bedded strata bounded by planar sub-horizontal surfaces. The cross-bedded units range in thickness from fractions of a metre to greater than 5 m, but are typically of the order of a metre. The horizontal surfaces (usually termed bedding planes by geotechnical engineers) give this facies a sheet-like appearance when viewed from a distance. The cross-beds typically dip to the north east, indicating that the Hawkesbury Sandstone was deposited by a fluvial system on a coastal plain with the source rocks being the Lachlan Fold Belt to the south west. The sandstone of the **sheet facies** tends to be well sorted.

The term **massive facies** was coined "*to convey the gross aspect of this lithosome when viewed from a distance and should not be taken to mean wholly structureless at closer inspection*" (Conaghan). The sandstone is poorly sorted and therefore is fairly homogenous in grain size, and is typically more friable than the sheet facies in weathered exposures. Frequently, sandstone bodies of this facies have a discordant erosional lower surface and a planar concordant upper surface. Mudstone (or shale) breccia commonly occurs within troughs at or above the basal surface but clasts, and in particular mudchips and mudflakes, can occur dispersed throughout. Petrographic analyses indicate that the massive facies sandstone contains significantly higher proportions of clay and less chemical cement and quartz overgrowth than the sheet facies, which is why there is the characteristic difference in weathering.

The **mudstone facies** comprise numerous thin mudstone (also termed laminite) units with characteristic thicknesses in the range 0.3 m to 3 m. Occasionally they are thicker than 10 m and there is one unit approximately 35 m thick near Terrey Hills in Sydney's northern suburbs. Most units of the **mudstone facies** exhibit a fairly uniform thickness and appear to be sheet-like, although laterally discontinuous and frequently terminated latterly by erosion surface overlain by **massive facies** sandstone. This sudden lateral termination can make borehole interpolation very difficult. The **mudstone facies** comprises largely dark grey to black, laminated mudstone/siltstone but ranging to closely bedded siltstone / sandstone, frequently termed laminite. The material does not swell significantly on exposure, but does slake. Quartz is usually the most abundant mineral, with illite clays up to 30% and variable amounts of kaolinite.

Petrographic analyses presented by Standard (1969) indicate that on average the Hawkesbury Sandstone has the following composition:

•	detrital quartz grains	68%
•	lithic fragments	2%
•	felspar	1%
•	mica	1%
•	clay matrix	20%
•	secondary quartz	6%
•	siderite (iron carbonate)	4%

Analysis by Robson (1978) of 42 samples taken from 16 sites gave:

•	quartz grains	mean 58.4%, SD 13.0%
•	rock fragments, felspar, mica	mean 3.5%, SD 2.8%
•	matrix clay	mean 24.2%, SD 7.1%
•	secondary silicates	mean 8.4%, SD 4.4%
•	dry unit weight (gm/cc)	mean 2.37%, SD 0.13%
•	porosity	mean 16.1%, SD 3.5%

Secondary silica occurs mostly as overgrowths around grains and the development thus of crystal faces imparts a glistening effect to the rock. The degree of overgrowth development is variable and has an important bearing on the strength and stiffness properties of the material.

Scanning electron microscope and electron probe studies reveal strange filament structures composed of potassium aluminium silicate which seem to act as a cementing agent in the sandstone. However, Dragovich (2000) suggests that these structures are the result of etching by organic acids produced by fungal hyphae!

The average composition of the matrix clay is 55% to 75% kaolinite, 20% to 30% illite and the balance mixed-layered clays. It appears that the proportion of kaolinite decreases and illite increases to the south (i.e. towards the source area of the sandstones).

2.2 STRENGTH AND DEFORMATION BEHAVIOUR UNDER UNIAXIAL AND TRIAXIAL COMPRESSIVE STRESS

2.2.1 Uniaxial Strength

Countless unconfined compressive strength (uniaxial) tests have been conducted for engineering developments in the Sydney area.

Figure 1 gives uniaxial stress strain curves for specimens from a borehole at North Head and from a quarry at Gosford. These curves show the typical stiffening that occurs up to about 50% of the peak stress. They also show the very significant difference that exists between the strength of the material in the dry and saturated states. Further data showing this strength difference are given in Table 1.

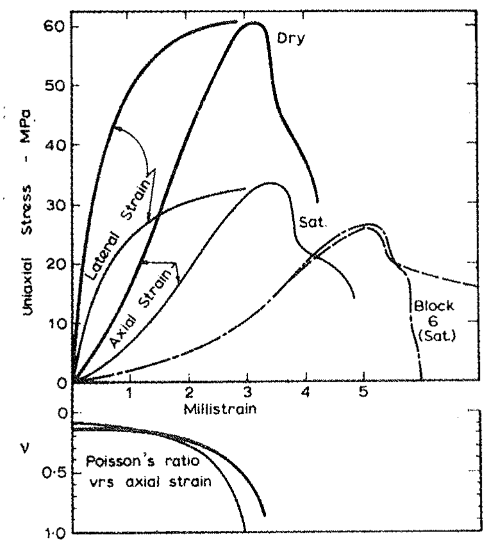

Figure 1: Typical stress-strain curves; samples from North Head and Gosford.

Table 1: Comparison of wet and dry uniaxial strengths.

Reference	Source	Uniaxial Compressive Strength MPa		Ratio Wet/Dry
		Saturated	Dry	
Pells, 1977	Gosford Quarries	11.7 (4)	39.2 (2)	0.30
Pells, 1977	Gosford Quarries	25.0 (7)	56.0 (3)	0.45
Robson, 1978	Waterloo, Sydney	23.1 (3)	42.3 (3)	0.55
Robson, 1978	Milsons Pt, Sydney	33.0 (2)	51.7 (2)	0.64
Robson, 1978	Frenchs Forest, Sydney	27.6 (3)	41.4 (4)	0.67
Robson, 1978	Mortdale, Sydney *	39.7 (6)	99.8 (6)	0.40
Robson, 1978	Elizabeth St, Sydney	30.7 (2)	47.2 (3)	0.65
Coffey's, 1970	Little Bay Wave Platform	9.4 (3)	23.2 (3)	0.40
Coffey's, 1970	Little Bay BH6, Sydney	10.0 (2)	29.2 (2)	0.34

Note: (i) Bracketed figure is number of specimens tested
(ii) * a typical clayey sandstone

Further points which should be noted with respect to the question of dry versus saturated conditions:

(i) The sandstone does not have to be 100% saturated in order to attain the minimum strength associated with full saturation. A degree of saturation of greater than about 90% is sufficient.

(ii) In most cases, resaturation of specimens that have been allowed to dry out returns the strength to the initial saturated condition (assuming the specimens came from below the water table).

Further UCS data are given by McNally and McQueen (2000) and are summarised in Table 2.

Table 2: Strength data from McNally and McQueen (2000)

Mean UCS (MPa)	**UCS range ± 1 SD** (No of tests)	**Description**
BLUE MOUNTAINS SEWER TUNNELS		
23.8	15.3-32.3 (27)	Winmalee Tunnel. Sandstone, wet
19.8	13.6-26.0 (69)	Hazelbrook Tunnel. Sandstone, wet excludes 123 MPa iron-cemented Sandstone
34.0	24.3-43.7 (26)	Sandstone, as above, dry
13.5	8.2-18.8 (14)	Katoomba Tunnel. Sandstone, wet
MALABAR OUTFALL		
29.2	- (13)	Sandstone, fine to medium, 63% quartz, mainly clay cement, wet (Massive Facies)
46.2	- (10)	Sandstone, as above, dry
43.9	- (9)	Sandstone, fine to coarse, 78% quartz, clay and silica cement, wet (Sheet Facies)
42.3	- (8)	Sandstone, as above, dry
51.6	- (6)	Sandstone, conglomeratic, 76% quartz; clay silica and carbonate cement; wet (Sheet Facies)
53.5	- (6)	Sandstone, fine to coarse, 57% quartz, high carbonate, dry
OCEAN OUTFALL TUNNELS		
37.5	25-50 (13)	North Head Outfall
30.7	20.1-41.3	Bondi Outfall
EASTERN DISTRIBUTOR		
22.9	15.2-30.6 (34)	Sandstone, wet

2.2.2 Modulus in Uniaxial Compression

Figures 2a and 2b give two sets of data relating Young's Modulus values to uniaxial compressive strength. The modulus values in Figure 2a (from Pells, 1977) are secant Young's moduli in the stress range 0 MPa to 6 MPa. This low stress range was chosen as being generally more applicable to stresses imposed by foundations than the tangent moduli at 50% of uniaxial strength. Such tangent moduli are given in Figure 2b (from Robson, 1978). Because the sandstone stiffens with increasing stress, the tangent moduli are generally higher than the 0-6 MPa secant moduli, for a given uniaxial strength. Table 3 gives some quantitative data in this regard.

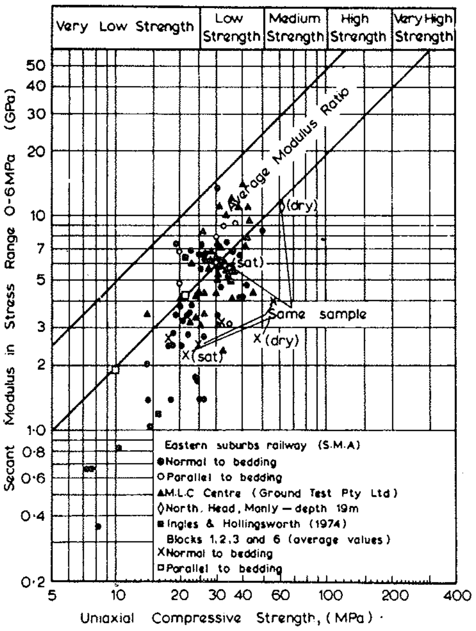

Figure 2a: data from Pells (1977)

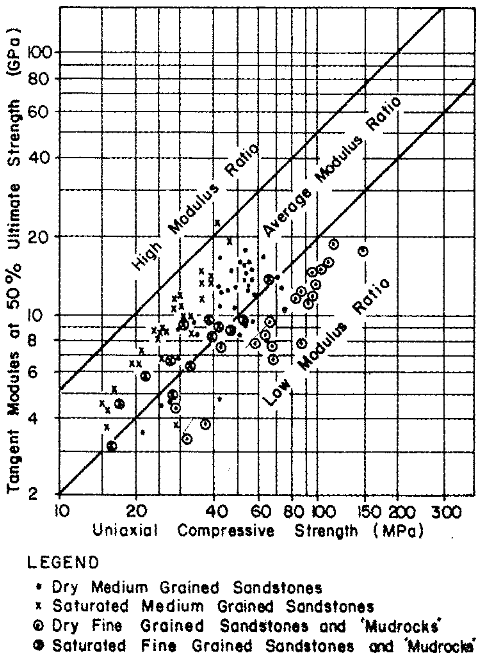

Figure 2b: data from Robson (1978).

Table 3: Comparison of dry and saturated moduli of Hawkesbury Sandstone.

Location	Material (all medium grained)	Tangent Modulus @ $0.5\sigma_c$ Dry Ed (MPa)	Tangent Modulus @ $0.5\sigma_c$ Saturated Es (MPa)	Ratio Es/Ed	Ratio Dry Strength / Wet Strength
Bondi	Massive, fresh	13.8	8.1	0.59	0.45
Waterloo	Laminated, fresh	10.3	11.0	1.07	0.69
Waterloo	Bedded, fresh	11.6	8.7	0.75	0.59
Kirribilli	Laminated, SW	4.8	5.0	1.04	0.61
Kirribilli	Thin bedded, SW	12.0	8.4	0.70	0.55
Fr Forest	Thin bedded, MW	12.7	8.6	0.68	0.57
Elizabeth St	Thin Bedded, fresh	11.7	13.9	1.19	0.68

Poisson's Ratio values are not frequently measured mainly because it has been well established that, in uniaxial compression, the values always fall in the range of 0.10 to 0.25 with a mean of about 0.2. Figure 1 shows that the Poisson's Ratio value stays reasonably constant to a stress level of 60% to 70% of the uniaxial strength. Dilatancy then initiates and increases rapidly immediately prior to failure.

2.2.3 Triaxial Strength

Pells (1977) discusses in some detail the behaviour of Hawkesbury sandstone under triaxial compression. Particular attention is given in that paper to pore pressure behaviour and it is shown that the Principle of Effective Stress holds fully with regard to the shear strength properties. The material behaves essentially as a highly over-consolidated, very dense sand.

Table 4 gives effective stress cohesion and friction parameters for 5 quarried blocks of sandstone (the blocks came from old buildings on Sydney University Campus). Up to six near identical specimens were obtained from each block and thus these c' and ϕ' parameters are considered to be valid measures of the typical triaxial parameters of the sandstone. This is an important point because this author has come across triaxial data on these sandstones, conducted on core specimens which of necessity are not identical, and which have given friction angles both very much lower and very much higher than those in Table 4.

Table 4: Triaxial strength parameters

Block No	Uniaxial Strength	Peak Shear Strength		Post-Peak Shear Strength		Predicted* Peak
	MPa	c' MPa	ϕ' deg	c'_r MPa	ϕ'_r deg	ϕ' deg
1	18.0	3.7	48	1.2	39	43
2	11.7	2.4	45	-	-	41
3 & 5	21.5	3.9	49	-	-	44
4	32.5	5.0	53	-	-	46
6	25.0	6.0	41	2.0	36	45

* From Bieniawski (1974)

$$\frac{\tau_m}{\sigma_c} = 0.75\left[\frac{\sigma_m}{\sigma_c}\right]^{0.9} + 0.1$$

As shown in Table 4, the measured friction angles are in reasonable agreement with those predicted by the empirical equation given by Bieniawski (1974).

With regard to modulus values, the triaxial testing conducted by this author indicates that there is a significant stiffening in the initial portion of the stress-strain curve (see Figure 3). However, the tangent moduli at 40% and 80% of peak strength are not significantly different. For example, Table 5 shows the 80% tangent moduli at different confining pressures on samples from a single block.

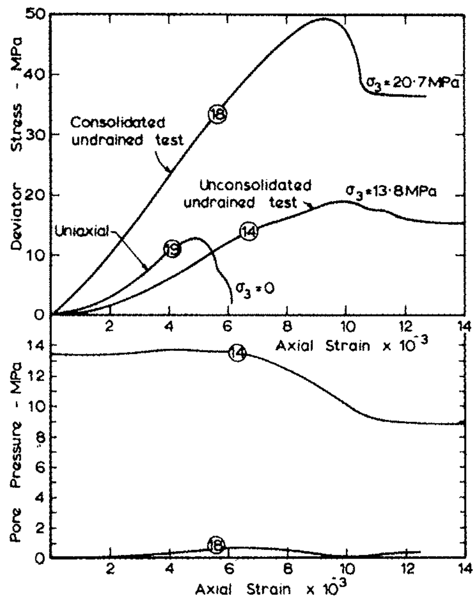

Figure 3: Some typical complete stress-strain curve for core specimens of Hawkesbury Sandstone.

Table 5: Effect of confining pressure on tangent moduli at 80% σ_C

Confining Pressure (MPa)	Modulus (GPa)
0	4.0 to 9.7
10.3	5.0
20.6	5.1

2.3 POINT LOAD STRENGTH AND BRAZILIAN STRENGTH

2.3.1 Point Load Testing

Robson (1978) presents and discusses detailed Point Load strength results on 60 samples of rocks from the Sydney Basin. Forty-three of these samples were of Hawkesbury Sandstone.

The conclusions from his tests were:

(i) For axial tests on saturated samples – best fit $\sigma_c = 20\ I_{s50}$ but with range from 15 to 29.
(ii) For diametral tests on saturated specimens – best fit $\sigma_c = 24\ I_{s50}$ but with range from 14 to 35.
(iii) For saturated specimens, the anisotropy index was close to unity (1.13).
(iv) For oven dried specimens, the anisotropy index was 1.3.

2.3.2 Brazilian Tensile Testing

This can be a very useful test inasmuch as it requires even shorter lengths of core than Point Load testing. The correlation between unconfined compressive strength and Brazilian Tensile Strength is excellent.

A very careful study concluded by Ferry (1983) indicates that:

$$\sigma_c \cong (12 \text{ to } 15)\sigma_t$$

where σ_c = Unconfined compressive strength

σ_t = Brazilian tensile strength

The similarity between this relationship and that for the Point Load test indicates that the Point Load test comes close to being a pure measure of tensile strength.

Figure 4 shows unconfined compressive tensile results from moderately to highly weathered sandstone in the Pymble quarry (Pells, Rowe and Turner, 1980). Using modal values these data indicate

$$\sigma_c = 13\ \sigma_t$$

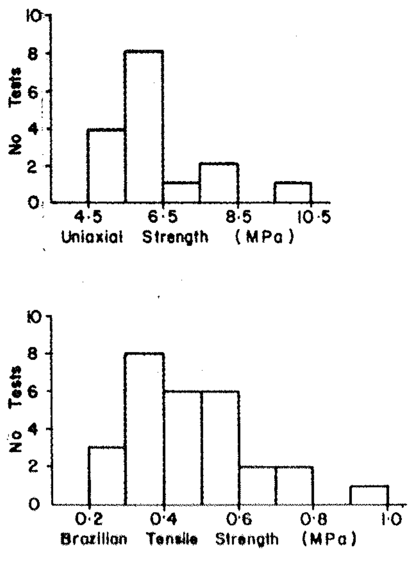

Figure 4: Unconfined compressive strength versus Brazilian tensile strength.

Data given by Roxborough (1982) on core samples from boreholes drilled for the Ocean Outfall Project, gave the results summarised in Table 6.

Table 6: Compressive and tensile strength data from ocean outfall project.

LOCATION	Unconfined Compressive Strength		Brazilian Tensile Strength		Ratio on Mean
	Mean (MPa)	Std Dev (MPa)	Mean (MPa)	Std Dev (MPa)	
North Head Borehole NH2	31.1	12.9	3.73	1.44	8
Bondi Borehole B6	30.7	10.6	2.67	0.69	12
Malabar Borehole M4	31.5	13.7	4.24	0.98	7

2.4 SONIC VELOCITY

Studies have been conducted by Rodwell (1976), Robson (1978) and Ferry (1983) into the relationship between the P-wave velocity of intact Hawkesbury Sandstone and mechanical properties such as unconfined compressive strength. These studies are summarised in Figures 5a, 5b and 5c.

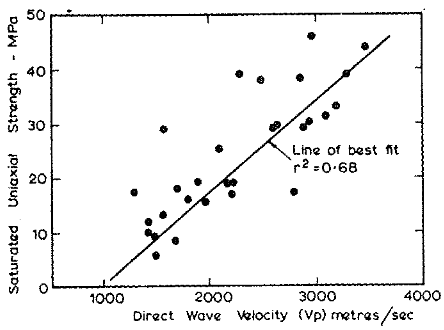

Figure 5a: After Rodwell (1976).

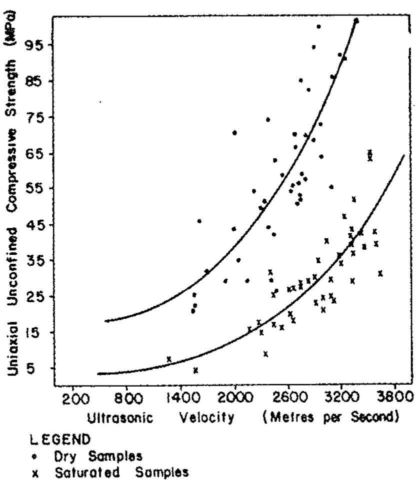

Figure 5b: After Robson (1978).

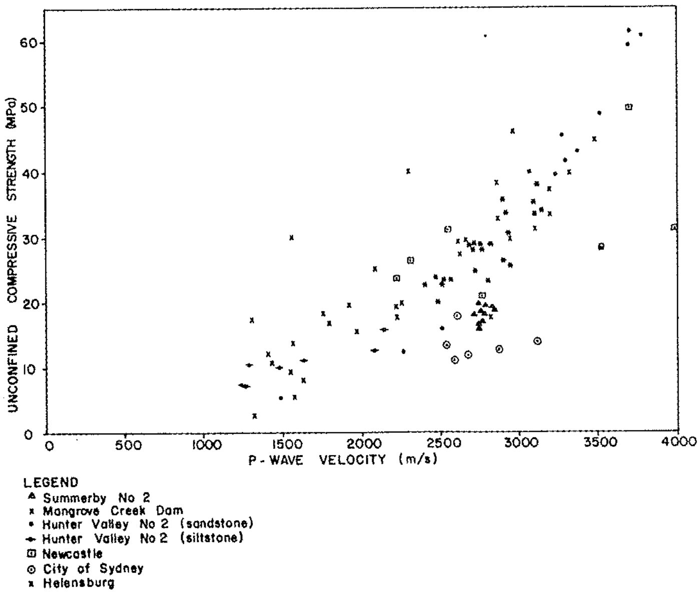

Figure 5c: After Ferry (1983).

Figure 5: Relationship between unconfined compressive strength and P-wave velocity.

The correlation between the P-wave velocity and unconfined strength is reasonably good. However, since specimen preparation for velocity testing must be to the same standard as for compressive testing, the predictive aspects are of little practical value.

The test is of more value in the sense of comparing laboratory seismic velocity values with field values in the same material. The ratio of field to laboratory velocities can provide a measure of the degree of *in situ* jointing.

2.5 DURABILITY TESTING

2.5.1 Sodium Sulphate Soundness

In a study undertaken for the Maritime Services Board (Coffey & Hollingsworth, 1970), sandstones from several sources were evaluated for possible use in breakwater construction. This study has been described by MacGregor (1982) and only the main points are summarised here.

Samples of sandstone were taken from existing breakwaters at Bumborah Point, Kurnell Groyne and Jervis Bay (this last site is not Hawkesbury Sandstone but Conjola Sandstone). It was concluded that the sodium sulphate test was the most useful for evaluating durability against physico-chemical breakdown. Table 7 relates the field performance of the different sandstones against measured laboratory data.

Table 7: Breakwater block performance.

Location	Actual Performance	Unconfined Compressive Strength		Sodium Sulphate Loss (%)
		Saturated MPa	Ratio Wet/Dry Strength	
Little Bay	Satisfactory	10.5	.45	33.0
Botany Bay – Protected	Satisfactory	13.5	.25	31.0
Kurnell Groyne	Satisfactory	15.7	.45	13.5
Jervis Bay	Satisfactory	-	-	17.2
Botany Bay – Exposed	Unsatisfactory	6.1	.25	77.0

A good relationship was observed between the proportion of expansive clay mineral present and the methylene blue test (see Figure 6). The wet to dry strength ratio also correlated well with the Sodium Sulphate soundness test (see Figure 7).

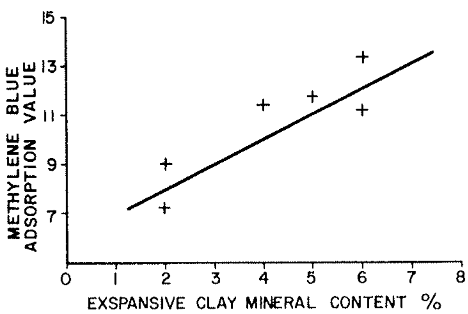

Figure 6: Relationship between Methylene Blue Absorption and Expansive Clay Mineral Content for different sandstone samples

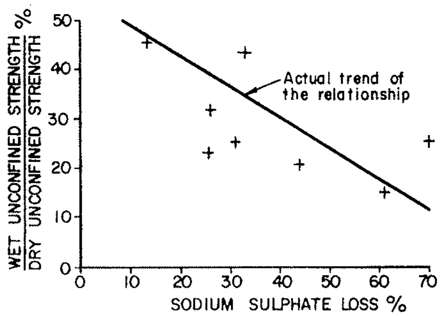

Figure 7: Wet/dry strength ratio versus sodium sulphate loss

Spry (2000) provides a detailed discussion of the petrographic nature of the Hawkesbury Sandstone, with a particular view to the use of this material as dimension stone. He provides good data on porosity because he notes that it is generally recognised that the performance as dimension stone is strongly influenced by porosity. He points out there are several ways of measuring porosity that give different results and recommends the standard conditions of ASTMC97.

The porosity of so-called 'yellow block' sandstone used for Sydney CBD buildings ranges from about 8% to 11%. In regard to durability testing, he recommends the 15 cycles of soaking in a 14% sodium sulphate solution as per the test method AS NZS 4456.10. He notes a good correlation between the % by weight loss in this test with the observed long term behaviour in buildings. His recommendations are:

A Grade, (best quality stone) <1% loss. Suitable for restoration or new thin cladding and application in aggressive environments.

B Grade, 1 to 4% loss. Suitable for high-grade construction, commercial building.

C Grade, 5 to 9% loss. Suitable for domestic and similar construction.

D Grade >10% loss. Suitable for less aggressive environments, less-important or less-expensive construction (domestic, paving).

2.5.2 Slake Durability

Very little testing using the ISRM Slake Durability test has been conducted on these sandstones. Robson (1978) tested two samples (from Liverpool and North Head) and measured 3% to 7% losses after four cycles. He concluded that the test was not severe enough for distinguishing between the different sandstones.

2.6 CREEP BEHAVIOUR

The author is aware of two studies that included creep testing of the Hawkesbury Sandstone.

The first of these was by the Snowy Mountains Authority (1969) undertaken for the Eastern Suburbs Railway Line. The results are summarised in Table 8. These tests were conducted at low axial stress levels (1.9 MPa to 1.4 MPa) and in most cases the long term modulus was 80% to 90% of the short term value. However, with three of the samples, the long term value was only 50% to 60% of the short term modulus. The reason for this range of behaviour is not known. However, note should be taken of the fact that at such low stress levels (typically <5% of unconfined strength) pores and microfissures in the rock are still closing and thus relatively greater creep may occur than at higher stresses when much of the stress is transmitted directly through the quartz skeleton.

Table 8: Creep results

Uniaxial Strength MPa	Stress Range MPa	Short term secant modulus GPa	Number of days creep	Ratio Long term modulus / Short term modulus
8	1.4	0.2	21	0.9
9	1.0	1.4	10	0.8
13	1.4	1.4	13	0.8
19	1.0	1.4	26*	0.5
22	1.0	3.5	20	0.8
24	1.0	0.7	41*	0.5
26	1.0	2.4	42*	0.6
28	1.4	10.7	19	0.9
28	1.0	7.2	10	0.9
35	1.0	6.9	7	0.9
45	1.4	6.2	8	0.9
* Creep still continuing				

In an undergraduate thesis (Dewberry, 1978) four long term creep tests of model footings were conducted on weathered intact Hawkesbury Sandstone. Circular steel footings, 25 mm diameter, were used and were initially loaded to between 85% and 90% of the static ultimate bearing capacity. The results are summarised in Table 9.

Unconfined compression tests were conducted at different loading rates and the results are shown in Figure 8. The results show a decrease in stiffness and strength with increasing time to failure, although the strain to failure was almost constant. Figure 9 shows that, within the range tested, the peak strength decreased linearly with the log of the time to failure.

Table 9: Creep tests on model footings (25 mm diameter).

Test No	Initial Load as % of Ultimate Capacity	Initial Settlement	Additional Settlement in mm after		Test Result
		mm	10 min	2 hrs	
1	≅ 85% (= 15 tonne)	≅ 4.5	0.41	-	Equipment failure
2	≅ 85%	≅ 4.5	**0.27**	2.2	Failure after 2.2 hours
3	≅ 85%	≅ 4.5	**1.50**	2.1	No failure after 443 hours
4	≅ 90% (= 16 tonne)	≅ 4.3	**0.65**	1.3	No failure after 2543 hours

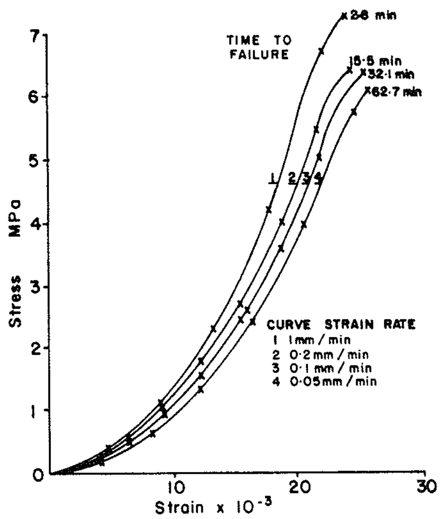

Figure 8: Effect of loading rate on the stress-strain behaviour under unconfined compression.

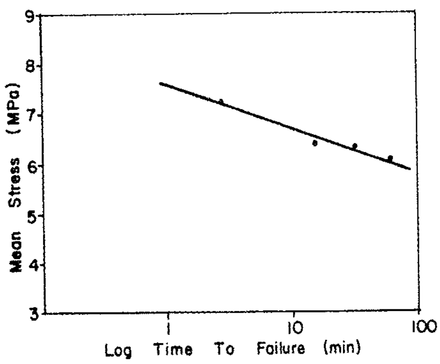

Figure 9: Relationship between applied stress and time to failure.

3 ROCK MASS PARAMETERS

3.1 GENERAL

The classification system for Sydney sandstone and shales, produced through the Australian Geomechanics Society (Pells et al., 1978; Pells, Mostyn and Walker, 1998), is intended to assist in the design of foundations on rock in the Sydney area. The five class system has proved to be a good tool for communicating rock mass quality for other geotechnical projects such as tunnels and deep basement excavations. However, the classification system is not a design tool for works other than foundations on rock. Tunnels, slopes, deep basements and retaining walls have to be designed using normal methods of applied mechanics. Such methods, whether hand stability calculations or complex numerical analyses, require engineering parameters covering strength and deformation characteristics. In some instances, such as rock substance strength and modulus, the parameters may be measured by laboratory testing. However, when it comes to rock mass parameters use has to be made of parameters back figured from monitoring of actual excavations and retaining structures; published correlations from other geological environments, such as mass modulus versus RMR; or semi-theoretical approaches such as Hoek's approach of estimating mass modulus from Hoek-Brown parameters.

Rock mass parameters proposed by Bertuzzi & Pells (2002) are presented here.

3.2 SOURCES OF INFORMATION FOR MASS PARAMETERS

Rock mass modulus values for mainly Class II and Class III sandstone have been backfigured from lateral measurements in deep basements (Pells, 1990), from tunnel convergence measurements (Hole, 2000) and from settlement monitoring of pad and socket footings (Rowe & Pells, 1980).

A large scale cable jacking modulus test at Lucas Heights (Clarke and Pells, 2004) gave accurate mass modulus measurements of borderline Class II/III sandstone in the range 800 to 1100 MPa. Similarly, load testing of an instrumented 1.5 m diameter bored pile for the Glebe Island Bridge (Poulos et al., 1993) gave a modulus value for Class II sandstone of 1200 MPa.

These field measurements provide a good database and therefore there is a reasonably high level of confidence in regard to the sandstone mass modulus values. Mass modulus values for the shales are largely taken from the estimates made by members of the Australian Geomechanics Society obtained in preparing the 1978 paper by Pells, Douglas, Rodway, Thorne and McMahon.

Permeability values have been obtained from site investigations for numerous tunnelling projects including the Ocean Outfalls, Sydney Harbour Tunnel, Eastern Distributor, M5 East, Cross City, Energy Australia and TransGrid cables tunnels and Parramatta Rail Link. Overall the permeability database represents approximately 5 km of tested borehole.

There is very little direct data for defect normal and shear stiffness. These are very difficult parameters to measure in the laboratory for real defects. Some field data on k_n values were obtained by Clarke and Pells, 2004. Normal stiffness can be estimated using the relationship between the defect's normal stiffness (k_n) and the modulus of its infill material (E), so that:

$$k_n = \frac{E}{t}$$

where k_n = normal stiffness GPa/m
E = infill modulus MPa
t = infill thickness mm

The elastic relationship between shear (k_s) and normal (k_n) stiffness is k_s:

$$k_s = \frac{k_n}{2(1+\nu)}$$

which suggests that k_s should be 0.33 to 0.5 times k_n. However, the authors note that the ratio k_s/k_n is actually dependent on the normal stress. Kulhawy (1975) showed limited experimental data with $k_s/k_n = 0.04$ to 1.20. Bandis et al. (1983) carried out further testing which suggested that for normal stresses greater than about 1 MPa, a ratio of about 0.10 could be used. This is the ratio the authors currently use in the absence of specific data.

Tables 10 and 11 present the rock mass design parameters given by Bertuzzi & Pells (2002). They represented the authors' views and it is quite likely that other practitioners have different views as to certain of these parameters, particularly k_n and k_s values for defects, where there is a paucity of data.

3.3 NATURAL STRESS FIELD

Several papers have been published giving detailed analyses of the natural stress field in the Traissic rocks of the Sydney Basin and particular note should be taken of those by Enever et al. (1980), Enever (1999) and McQueen (2000).

With the purpose of reducing the world to simple equations that make design easier, the writer has proposed Pells (2002) that, away from topographic effects, the stress field may be approximated by the equations:

$$\sigma_1 = \sigma_{NS} = 1.5 + 1.2\sigma_v \ to \ 2.0\sigma_v \quad \text{MPa} \qquad ......①$$

$$\sigma_2 = \sigma_{WE} = 0.5\sigma_1 \ to \ 0.7\sigma_1 \quad \text{MPa} \qquad ......②$$

$$\sigma_3 = \sigma_v = 0.024H \quad \text{MPa} \qquad ......③$$

Table 10: Rock mass parameters.

CLASS	SUBSTANCE			MASS STRENGTH			MASS					
	STRENGTH		ELASTICITY				UNIT WEIGHT (kN/m^3)	MODULUS (MPa)	PERMEABILITY (uL)		GSI (b)
	UCS (MPa)	σ_t (MPa)	E (GPa)	UCS (MPa)	c' (kPa)	ϕ' (°)			LOG MEAN	RANGE	
Sandstone I / II	12-50	2-6	8-14	15-25	(a)	(a)	24	900-2500	0.2	<0.01 to 2	65-75
Sandstone III	7-25	0.5-3	6-10	5-20	(a)	(a)	24	350-1200	1	0.1 to 50	45-65
Sandstone IV / V	1-7	0.1-0.5		<1-4	(a)	(a)	24	50-700	5-10	1 to 100	30-45
Shale I / II	7-40	1-4	7-15 (c)	10-20	(a)	(a)	24	700-2500	0.2	<0.01 to 25	64-73
Shale III	2-15	0.1-2	5-10 (c)	1-7	(a)	(a)	24	200-1200	1	0.1 to 50	40-64
Shale IV / V	1-2	<0.2		<1	(a)	(a)	24	50-500	1	<1 to 25	30-40

(a) The value of GSI which is included in the table can be used to obtain c' and ϕ' which are dependent on the in situ stress.
(b) Geological Strength Index defined by Hoek et al (1995).
(c) Substance modulus for shale is dependent on moisture content (m%). For design use the relationship based on Won (1985), ie $E = 3.6e^{-0.415m}$ GPa.

Table 11: Parameters of discontinuities.

DESCRIPTION	THICKNESS (A) (mm)	FRICTION ANGLE (°)	STIFFNESS (GPa/m)	
			NORMAL (K_N)	SHEAR (K_S)
Major bedding plane	Tight	35-45	4000	400
	1-5	30-35	200	20
	5-10	20-25	10	1
Erosional Plane	5-30	20-35	5	0.5
Cross Bed Partings	Tight	25-35	4000	400
	1-3	20-28	1500	150
Joint	Tight	35-40	4000	400
	1	22-28	1500	150
	3	18-22	500	50

(a) The infill is typically a sandy clay when present in sandstone, and a clay when present in shale.

However, several experiences within the last few years have indicated that questions should be raised in regard to the published interpretation of the stress field. These experiences are from two sources, namely:

1. Conflicting data from different stress measurement techniques at the same location, and
2. Unexpected, or inexplicable, failures in certain tunnels and caverns, such as those at relatively shallow depths in the Northside Storage Tunnels, a City cable tunnel and a City road tunnel; and the quite large scale collapse in the Elgas Cavern.

3.3.1 Accuracy of stress measurement techniques

In regard to conflicting data it should first be noted that the vast majority of measurements of natural stresses in the Sydney rocks have involved hydrofracture testing. The reasons for this are threefold, namely:

- hydrofracturing can be done in deep boreholes well below the water table,
- the method of interpretation of the field data has been promulgated as being robust and the results have appeared to be sensible and
- practical field equipment was developed by the CSIRO and has been made available in an effective commercial manner.

However, in 1990, measurements were made for the Opera House Carpark cavern using the Rock Slotter technique and hydrofracture. The results were very different and at that time the Rock Slotter results were accepted for design purposes. The fact that there were substantial differences was put in the "too hard basket" and forgotten. However, in 2003, questions were again raised in regard to some very high horizontal stresses measured by hydrofracture methods for the Epping-Chatswood rail tunnel project (see Figure 10).

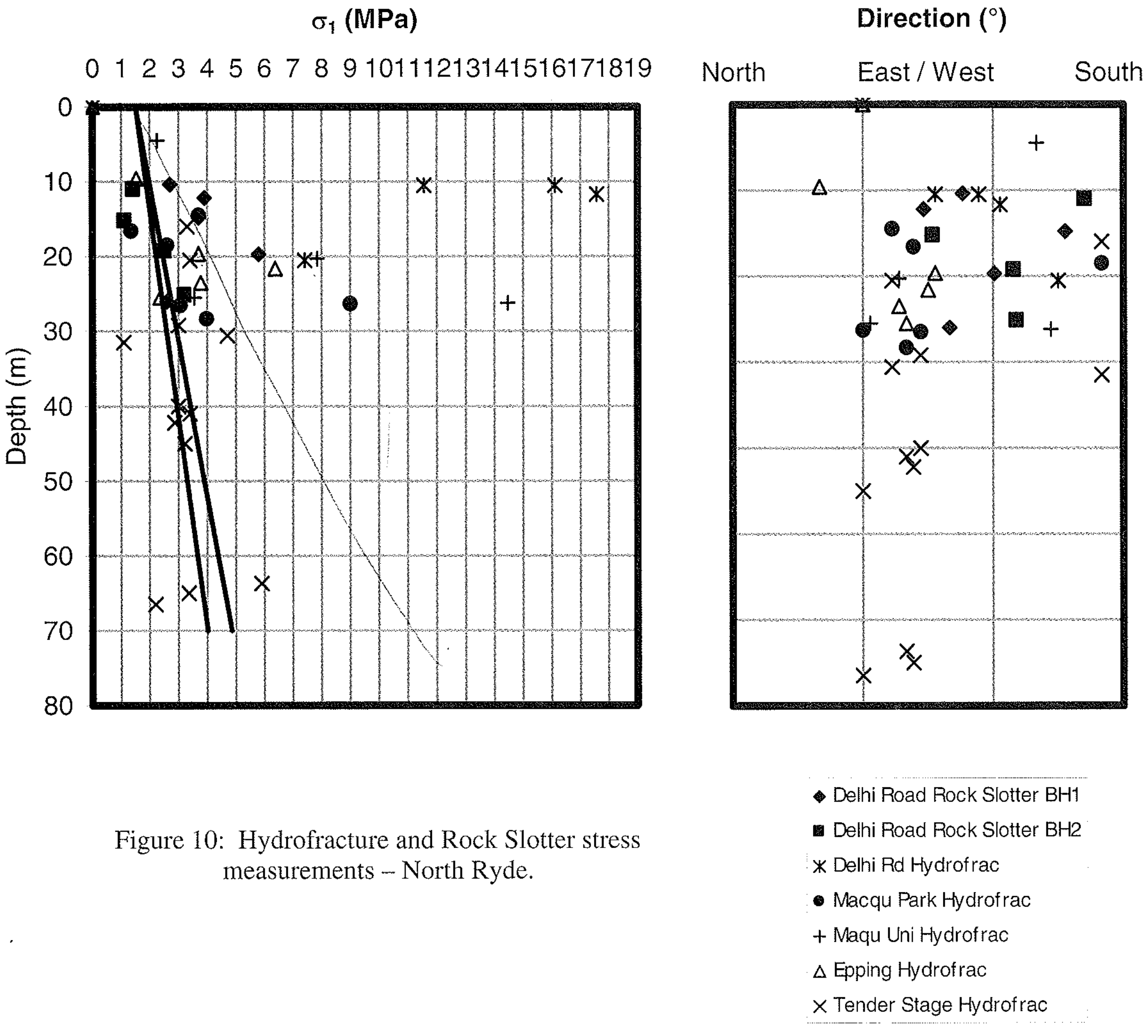

Figure 10: Hydrofracture and Rock Slotter stress measurements – North Ryde.

These caused the writer to examine some of the basics in relation to hydrofracture measurements and, in particular, to read a paper by Charles Fairhurst (Fairhurst, 1986) who originally proposed the use of hydrofracturing for stress measurement. The following comments by Fairhurst are very illuminating:

> *"Uncertainty as to interpretation of the fluid pressure-flow behaviour during crack initiation and propagation ...result in an associated uncertainty in the calculation of the maximum and minimum* in situ *stresses by hydraulic fracturing. Thus while the technique is valuable because it allows estimates to be made of* in situ *stresses at considerable depth, hydraulic fracturing is probably most reliable as an indicator of the directions of maximum and minimum stress".*

However, probably the most important point made by Fairhurst is that:

> *"Difficulties of interpreting the* in situ *measurements, especially in the practically important situations where discontinuities and inhomogeneities in the rock mass have a significant, but uncertain, influence make the focus on stress-determination unrewarding.*
>
> *A more effective design strategy is to give greater emphasis to the overall effects of interaction between stress states, rock mass properties and excavation geometry. ... Convergence measurements is the primary example of such an integrated effect ..."*

What Fairhurst is referring to in the second paragraph is that the best measure of the natural stress field in a rock mass is to analyse displacement measurements around full scale excavations made in that rock mass.

It must be remembered that analyses of hydrofracture test records rely on the theory of elasticity and there are many possible errors in the analyses. In the textbook by Armadei and Stephansson there are 55 pages of discussion, mathematics and laboratory data in relation to variations in interpretation of the test. It is not appropriate to discuss all that material here. However, there are three points worth noting:

(i) In a relatively porous and permeable rock such as Hawkesbury Sandstone, *"the coupled diffusion-deformation phenomena which exist in fluid-saturated porous rocks are not taken into account"* (Armadei & Stephansson, p 143). The extent to which this causes errors in the interpretation of test data in Hawkesbury Sandstone is not known to the writer.

(ii) The classical tensile strength criterion cannot be used to explain the creation of horizontal fractures in the borehole wall, a situation that occurs when the *in situ* principal horizontal stresses are significantly larger than the vertical stress (Amadei & Stephansson, p 155 & 156). In fact 3D numerical simulation by Yong Sun of hydraulic fracture testing in very high horizontal stresses measured at Delhi Road, showed that failure around the boreholes should not have commenced by tensile fracture in the vertical plane, but by tensile fracture on a horizontal plane.

(iii) The tensile strength used to calculate the maximum principal stress is difficult to measure and is size dependant. If the alternative approach is used of the difference between Fracture Initiation Pressure and Fracture Reopening Pressure, then there are the implicit simplifying assumptions of elastic theory.

What this all means is that the results of hydrofracture tests are not without error, and the test has particlar limitations in porous and permeable rock (e.g. Hawkesbury Sandstone as compared with, say, granite), and where there are relatively high horizontal stresses with low overburden stresses. It can be seen from Figure 10 that a series of very successful Rock Slotter tests at North Ryde did not support the very high horizontal stresses interpreted from the hydrofracture tests.

Other test methods such as the Rock Slotter and strain gauge overcoring techniques also have limitations. Fundamentally this is because they require the theory of elasticity for interpretation. There is nothing wrong with the theory, it's just that rocks are not linearly elastic (see for example the stress-strain curves in Figure 11 used for Rock Slotter test interpretations on the Epping-Chatswood project). In terms of accuracy it is generally accepted that the CSIRO HI cell, and similar overcoring strain gauge devices, give the most accurate measure of rock mass stress at a point. These are also the most difficult tests to perform.

There is also the fact that stress is a mathematical concept, not a physical entity. The physical entity is displacement (or strain). This must be considered in order to sort out the unexpected, as discussed in Section 3.3.2 below.

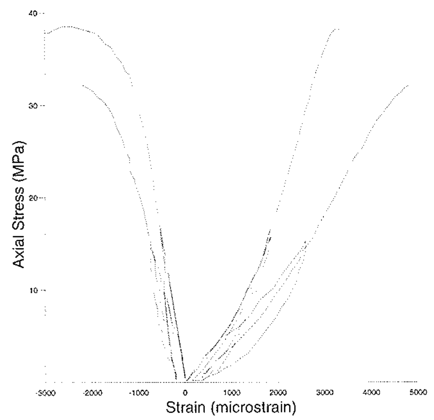

Figure 11: Samples from North Ryde.

3.3.2 Non-uniformity of the stress field

In considering data such as those from the Epping-Chatswood project reproduced in Figure 10, care must be taken to discriminate between measurements which are probably false and measurements which correctly represent the non-uniformity of the stress field.

While it is reasonable to expect consistency in the natural stress field on a scale of 100's of metres, it is clearly nonsense to expect this on the scale of bedding thickness or block size.

To explore the causes of non-uniformity it must be noted that high horizontal stresses in sedimentary rocks are the result of two phenomena:

1. "Overconsolidation" in soil mechanics terminology, a phenomenon which explains the high horizontal stresses in diverse materials such as London Clay, compacted fill and sedimentary rocks, where erosion of overlying material creates high overconsolidation ratios and
2. Tectonic strains generated by volcanic activity and crustal plate movements.

In the Sydney Basin, overconsolidation is the cause of both major (σ_1) and intermediate (σ_2) principal stresses being horizontal, and their magnitudes increasing, relative to vertical stress (σ_3), with decreasing depth. However, the fact that σ_1 is typically north-south and almost double σ_2, is tectonic in origin. These tectonic effects must be non-uniform because the near horizontal beds of sandstone and shale are of varying stiffness. As a matter of simple mechanics, in such a system uniform lateral strain must result in higher stresses in the stiffer beds. This is partly what has led to stress-induced failures ("rockbursts") in stiff beds of sandstone at relatively shallow depths; such as:

- sandstone overlying a layer of laminite in the floor of the 30 m deep basement of the Quay West site in The Rocks,
- beds in the crown of the TBM Northside Storage tunnels near Tunks Park at depths of about 60 m and
- sandstone sandwiched between two thin micaceous laminite beds in a road tunnel in the City.

Non-uniformity of the stress field is further exacerbated by the presence of dykes. This was evidenced by stress measurements conducted for the Elgas cavern under Botany Bay, where CSIRO HI measurements showed significant differences in the magnitudes and orientations of σ_1 on opposite sides of a dyke.

3.3.3 Discussion

Clearly the real world is more complicated than the simple equations given at the beginning of Section 3.3. How do we deal with this real world? In the writer's view it is appropriate to:

(i) use equations, such as those presented herein, for initial design work, and then

(ii) conduct site specific stress measurements if initial designs indicate stress concentrations to be an issue, but particularly if the project is at a depth of 50 m or greater, and

(iii) make many measurements using preferably two different techniques and

(iv) check the measurements against the geological profile and the presence of macro features such as dykes, to help discriminate between meaningful and nonsensical data.

3.4 GEOLOGICAL STRUCTURES

The significant geological structures in Hawkesbury Sandstone comprise:

- near horizontal and undulating bedding planes, often comprising sandy clay seams,
- undulating erosional bedding surfaces often containing shale breccia,
- cross (current) bedding,
- high angle faults and low angle thrust faults which typically step along bedding,
- near vertical joints and
- volcanic dykes

Discussion of these geological structures is beyond the scope of this paper but their importance in relation to engineering works in the Hawkesbury Sandstone cannot be too strongly emphasized. For further details reference should be made to Herbert & Helby (1980), Standard (1969), Pells (1993), Branagan (1985) and Branagan, Mills and Norman (1988).

A map has recently been published by Pells, Braybrooke and Och (2004) updating information given in the Notes to the Sydney 100 000 Sheet in regard to major near vertical structural features in the Sydney CBD.

4 FOUNDATION DESIGN PARAMETERS

This section is abbreviated from papers by Pells, Mostyn and Walker (1998) and a review paper by Pells (1999). Designers of foundations on the Hawkesbury Sandstone, particularly heavily loaded socketed foundations, should take cognisance of the full papers.

4.1 CLASSIFICATION SYSTEM

The classification system used in Sydney is given in Tables 12a and 12b and is based on rock strength, defect spacing and allowable seams as discussed below. All three factors must be satisfied.

Table 12a: Classification for sandstone.

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 12b: Classification for shale.

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

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 1998 it was therefore changed (see Table 13) to the scale in the draft International Standard for Identification and Descriptions of Rock (ISO/DIS 14689).

Table 13: Defect spacing

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

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 Table 12 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.

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 ($\tau_{\text{ave peak}}$) and the rock substance unconfined compressive strength (q_u), see Rowe & Armitage (1984). The relationship is simply:

$$\tau_{\text{ave peak}} = \alpha\, q_u \qquad \ldots\ldots ④$$

Figure 12 gives the results of field and laboratory tests on mudstones and sandstones as evaluated by Williams & Pells (1981) who 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:

$$\tau_{\text{ave peak}} = \alpha\beta q_u \qquad \ldots\ldots ⑤$$

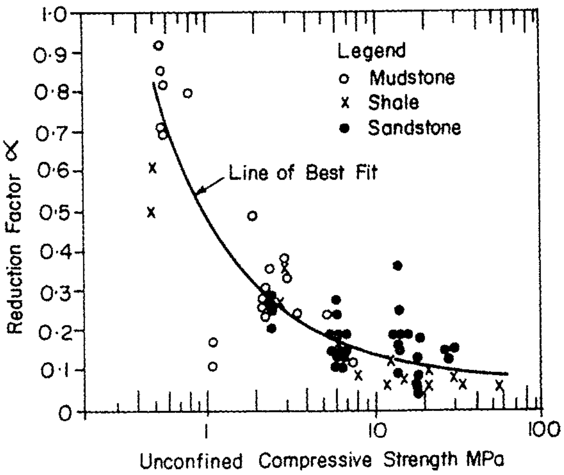

Figure 12: Side Shear Reduction Factor.

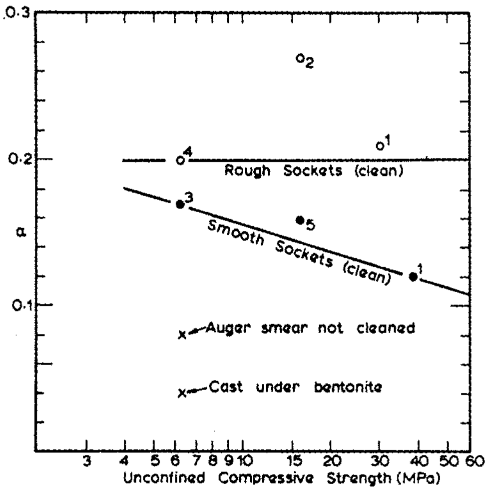

Figure 13: Side Shear Reduction Factor for Hawkesbury Sandstone.

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. Figure 13 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, $\alpha \geq 0.2$.

The data given in Figures 12 and 13 were 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 16, 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 16 but adopting a ϕ_g value of 0.5. 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 shape of the liftout mass would be very complex, in Classes I to III shale and sandstone the shape is likely to comprise slabs failing in 'bending', whereas in Classes IV and V it may 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 mobilized shear on the side of the cone of 10% of the ultimate values given in Table 16 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.
- Calculate the vertical components of uplift resistance generated by shear on the side of the cone.
- Perform an ultimate strength limit state design check.

4.3 END BEARING

Substantial laboratory and field testing in many countries has 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 14 and 15 for example of theoretical calculations and field measurements).
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.

Table 14: Theoretical bearing capacity of rock.

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 15: Measured bearing capacities – model and field tests.

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)

The above points mean that for footing design in the Sydney sandstones and shales, the base behaviour can be modeled as linearly elastic up to Serviceability Limits.

Based on the research findings summarised above, values of end bearing pressure to cause settlements of < 1% of a footing diameter (or minimum plan dimension) are given in Table 16. Load-displacement behaviour would be approximately linear up to those values. Also given in Table 16 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 (e.g. 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 center 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.

Table 16a: 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	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 16b: Design values for foundations on shale.

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.

4.4 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 into the structural engineer's world of Limit State Design. The current Australian Piling Code (AS2159-1995) is a Limit State document and therefore Pells, Mostyn & Walker (1982) provided guidelines as to how the design parameters given in Table 16 could be used to Limit State Design. This level of detail is outside the scope of the present paper.

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