REPORT BOOK 92/27

HOLDFASTS —
NARACOORTE CAVES

DEPARTMENT OF MINES AND ENERGY

GEOLOGICAL SURVEY

SOUTH AUSTRALIA

REPORT BOOK NO. 92/27

HOLDFASTS - NARACOORTE CAVES

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CLIENT

NATIONAL PARKS AND WILDLIFE SERVICE

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DEPARTMENT OF MINES AND ENERGY
SOUTH AUSTRALIA

DME 7/91

Holdfasts - Naracoorte Caves

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Holdfasts were installed adjacent to two entrances to *The Cathedral* cave at Naracoorte Caves to improve safety for rock climbers and emergency rescue personnel. Four rockbolts were grouted into holes drilled into rocks of the Gambier Limestone Formation. Load tests ensured adequacy of both design and installation. Regular inspection and maintenance is recommended.

INTRODUCTION

The National Parks and Wildlife Service (Naracoorte Region) contracted the Engineering Geology Section of the SA Department of Mines and Energy to install holdfasts for rock climbers adjacent to two entrances of The Cathedral cave. Rock-bolts were completed at four sites to depths of 1400 mm. Holdfast installation and quality control testing was carried out by the Engineering Geology Section.

GEOLOGY AND MAPPING

The bryozoal limestone is part of the Gambier Limestone formation of Miocene-Oligocene age. It is massive, horizontally bedded and contains few joints. The limestone is moderately strong to strong.

The caves have been formed by successive cycles of progressive roof collapse, phreatic surface fluctuation and dissolution of the limestone by groundwater.

DRILLING

Holes were drilled at each site into rock of suitable integrity using a Wacker portable petrol driven rock drill. Four holes of nominal diameter 32 mm were completed to depths of 1400 mm. The design required that each bolt be anchored in a minimum of 400mm of rock with an estimated strength classification of Moderately Strong or greater. However to avoid cutting the steel bolts (of standard 1500mm length) holes were drilled to 1400mm depth.

INSTALLATION

Galvanised, high strength steel rock-bolts were fully grouted into the drill holes in a two stage process. Rock-bolts were anchored with a minimum of 400mm of grout at the bottom and then pre-tensioned before final grouting. Grout used was Megapoxy H (rapid set). Pull-out tests were performed before affixing the belay rings. The belay rings were fabricated by Halls Holding Co. P/L of Glynde. Each ring is seated on a hemispherical washer which is bolted down onto a fitting domed plate to ensure maximum mobilisation of bolt strength. Table 1 shows details of the components used.

TESTING

Point Load Rock Strength Test

Three representative samples were taken and broken into suitable sized lumps for testing. Each test was carried out in accordance with the ISRM Suggested Method For Determining Point Load Strength. The test results are given in Appendix A.

The standardised point load strength index ($Is_{[50]}$) ranged from
1.0 MPa to 3.8 MPa. The rock is classed as Moderately Strong to Strong.

Direct Pull-out Tests

A direct pull-out test was conducted on each installation by applying a minimum axial upward force of 43.6 kN, for a period of 1 to 2 minutes, using an hydraulic ring jack. All stations performed satisfactorily under this loading (see Appendix B). Maximum deflections never exceeded 0.75mm.

The test results are taken to indicate that both design and installation are adequate for the proposed conditions of usage.

FUTURE INSPECTION AND MAINTENANCE

The designs of the rock-bolts and belay rings are such that they are expected to achieve a service life in excess of 20 years. However, as public safety is such an important issue, they should be carefully inspected and, where necessary, maintained on a regular basis. Table 2 sets out a suggested inspection schedule.

A 'minor' inspection comprises a visual check. All components should be checked for signs of accelerated corrosion, excessive wear and/or tampering.

A 'major' inspection, suggested at 5 yearly intervals, should be much more detailed. The nuts should be dismantled from the bolts and the belay rings removed. All components should be visually checked and, if suspect, they should be physically measured for signs of an unacceptable degree of wear or corrosion. Each bolt should be subjected to a pull-out test (minimum vertical load - 40 kN). Units should be reassembled to 30kN tension).

Should it come to the attention of NPWS that large loads are applied to any of the holdfasts, due to accident or any other means, they should be immediately inspected for damage, or taken out of commission until such inspection can be made and approval given for their continued use.

Tampering and Vandalism

Tampering or vandalism can not be completely ruled out. However the nuts have been fixed with 'Loctite' type 277 and will resist loosening. Although it is possible to cut the rock-bolts with a hacksaw, they are not readily accessible below the nut.

TABLE 1 : DETAILS OF COMPONENTS USED

COMPONENT	SPECIFICATION/DESCRIPTION	MANUFACTURER
ROCK BOLT (incl. nut plate and washer)	High strength chemical anchor rock bolts, 1500x24mm , hot dip galvanised, with nut & washer. Domed plate, 150x150x8mm.	ANI ARNALL Berkeley Road, Unanderra, NSW.
BELAY RING	Fabricated from 25mm diameter steel rod and 25mm steel plate.	HALLS HOLDING CO P/L 24 Clark Street, Stonyfell, SA.
GALVANIZING Belay ring	Hot dip galvanising.	KORVEST LTD 580 Prospect Road, Kilburn, SA.
EPOXY CEMENT	Megapoxy "H - Rapid Set" Hydrophilic Epoxy.	RESIMAX PTY LTD 97 East Street, Torrensville, SA.

TABLE 2 : RECOMMENDED SCHEDULE OF INSPECTIONS

YEARS FROM COMMISSIONING	APPROXIMATE DATE	TYPE OF INSPECTION
1	Early 1993	Minor
2	Early 1994	Minor
3	Early 1995	Minor
5	Early 1997	Major
8	Early 2000	Minor
10	Early 2002	Major
15	Early 2007	Major
20	Early 2012	Major

TYPE OF INSPECTION	SUGGESTED PROCEDURE
MINOR	Visual check of all components for signs of accelerated corrosion, excessive wear and/or tampering.
MAJOR	1. Dismantle nuts and belay rings 2. Visual inspection and replace -ment of components if necessary 3. Pull-out test 4. Reassemble to 30 kN tension

APPENDIX A

POINT LOAD TEST RESULTS

POINT LOAD STRENGTH TEST

JOB NO.: 92-51-2
SHEET: 1 of 2

DEPARTMENT OF MINES AND ENERGY – SOUTH AUSTRALIA

CLIENT: NPWS — TEST LOCALITY: GLENSIDE — DATE: 22/11/92
PROJECT: NARACOORTE — TEST MACHINE: ROB. RESEARCH — TESTED BY: TE/SW
LOCATION: THE CATHEDRAL — DATE CALIBRATED: — CHECKED BY: SW

SAMPLE ID and LOCATION		SAMPLE NC1	SAMPLE NC1	SAMPLE NC1	SAMPLE NC2
SAMPLE DESCRIPTION (NOTE PLANES OF WEAKNESS)		LIMESTONE, OFF-WHITE TO LIGHT BROWN, MASSIVE, NODULAR CRYPTOCRYSTALLINE	see left	see left	SAME AS SAMPLE NC1
MOISTURE / STORAGE HISTORY		SURFACE LUMP	SURFACE LUMP	SURFACE LUMP	SURFACE LUMP
DIAMETER 1	W_1 (mm)	60	47	50	50
DIAMETER 2	W_2 (mm)	65	60	60	65
AVE. DIAMETER $W = \frac{W_1 + W_2}{2}$	W (mm)	62.5	53.5	55	57.5
PLATEN SEPARATION	D (mm)	31	34	42	42.5
LENGTH / DIAM. RATIO $0.3 < D/W < 1.0$	D/W	0.5	0.64	0.76	0.74
EQUIVALENT DIAM. $D_e = \sqrt{1.2732(W \times D)}$	D_e (mm)	49.7	48.1	54.2	55.8
FAILURE LOAD (or GAUGE PRESSURE)	kN (kPa)	5	5.2	7.7	3.0
CORRECTED LOAD	P (kN)	5	5.2	7.7	3.0
UN-CORRECTED P.L. STRENGTH INDEX $I_s = (P/D_e^2) \times 1000$	I_s (MPa)	2.03	2.25	2.62	0.96
SIZE CORR. FACTOR $F = (D_e/50)^{0.45}$	F	1.00	0.98	1.04	1.05
POINT LOAD STRENGTH (SIZE CORRECTED) $Is_{(50)} = Is \times F$	$Is_{(50)}$ (MPa)	2.02	2.21	2.72	1.01
APPROX. EQUIV. U.C.S. $Qu \simeq 24 \times Is_{(50)}$	Qu (MPa) (see Note 2)	48.5	53.0	65.2	24.3
TEST TYPE, SKETCH AND NOTES A = AXIAL SAMPLE TEST D = DIAMETRAL SAMPLE TEST B = BLOCK SAMPLE TEST I = IRREGULAR LUMP TEST		I	I	I	I

NOTES:
1. Testing in accordance with ISRM Point Load Test Method – see Explanatory Sheet 3.
2. Value for UCS is approximate only. Conversion from $Is_{(50)}$ to Qu is only accurate if extensively calibrated for specific site materials.

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POINT LOAD STRENGTH TEST

JOB NO. : 92-51-2
SHEET : 2 of 2

DEPARTMENT OF MINES AND ENERGY – SOUTH AUSTRALIA

CLIENT : NPWS — TEST LOCALITY : GLENSIDE — DATE : 22/1/92
PROJECT : NARACOORTE — TEST MACHINE : ROB. RESEARCH — TESTED BY : TE/SW
LOCATION : THE CATHEDRAL — DATE CALIBRATED : — CHECKED BY : SW

SAMPLE ID and LOCATION		SAMPLE NC3	SAMPLE NC3	SAMPLE NC3	SAMPLE NC3
SAMPLE DESCRIPTION (NOTE PLANES OF WEAKNESS)		LIMESTONE, OFF-WHITE, MASSIVE, NODULAR CRYPTOCRYSTALLINE	see left	see left	see left
MOISTURE / STORAGE HISTORY		SURFACE LUMP	SURFACE LUMP	SURFACE LUMP	SURFACE LUMP
DIAMETER 1	W_1 (mm)	65	55	50	45
DIAMETER 2	W_2 (mm)	70	70	55	60
AVE. DIAMETER $W = \frac{W_1 + W_2}{2}$	W (mm)	67.5	62.5	52.5	52.5
PLATEN SEPARATION	D(mm)	66	35	60.5	40
LENGTH / DIAM. RATIO $0.3 < D/W < 1.0$	D/W	0.98	0.56	1.15	0.76
EQUIVALENT DIAM. $D_e = \sqrt{1.2732(W \times D)}$	D_e (mm)	75.3	52.8	63.6	51.7
FAILURE LOAD (or GAUGE PRESSURE)	kN(kPa)	18	10	13	9
CORRECTED LOAD	P(kN)	18	10	13	9
UN-CORRECTED P.L. STRENGTH INDEX $I_s = (P/D_e^2) \times 1000$	I_s (MPa)	3.17	3.59	3.21	3.37
SIZE CORR. FACTOR $F = (D_e/50)^{0.45}$	F	1.20	1.02	1.11	1.02
POINT LOAD STRENGTH (SIZE CORRECTED) $I_{s(50)} = I_s \times F$	$I_{s(50)}$ (MPa)	3.81	3.68	3.58	3.42
APPROX. EQUIV. U.C.S. $Q_u \triangleq 24 \times I_{s(50)}$	Q_u (MPa) (see Note 2)	92	88.3	86.0	82.0
TEST TYPE, SKETCH AND NOTES A = AXIAL SAMPLE TEST D = DIAMETRAL SAMPLE TEST B = BLOCK SAMPLE TEST I = IRREGULAR LUMP TEST		I	I	I	I

NOTES:
1. Testing in accordance with ISRM Point Load Test Method – see Explanatory Sheet 3.
2. Value for UCS is approximate only. Conversion from $I_{s(50)}$ to Q_u is only accurate if extensively calibrated for specific site materials.

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APPENDIX B

PULL-OUT TEST RESULTS

ROCK BOLT TEST

Client: *National Parks and Wildlife Service*

Project: *Naracoorte Caves*

Location: The Cathedral (NW ENTRANCE)

Date tested: April 7, 1992

Tested by: SW/TE

Computed by: SW

Checked by: TE

Bolt Ident: 1					Bolt Ident: 2				
Time	Elapsed Time (mins)	Load (kN)	Dial Gauge	Deflection (mm)	Time	Elapsed Time (mins)	Load (kN)	Dial Gauge	Deflection (mm)
9:19	0	10.7	16.52	0.00	9:40	0	10.7	15.25	0.00
9:20	1	18.9	16.64	0.12	:41	1	18.9	15.25	0.00
9:21	2	27.1	16.73	0.21	:42	2	27.1	15.29	0.04
:22	3	35.4	16.87	0.35	:43	3	35.4	15.37	0.12
:23	4	43.6	17.14	0.62	:44	4	43.6	15.48	0.23
:24	5	35.4	17.10	0.58	:45	5	35.4	15.47	0.22
:25	6	27.1	17.00	0.48	:46	6	27.1	15.47	0.22
:26	7	18.9	16.85	0.33	:47	7	18.9	15.45	0.20
:27	8	10.7	16.66	0.14	:48	8	10.7	15.41	0.16
:28	9	43.6	17.22	0.70	:49	9	43.6	15.55	0.30
:29	10	10.7	16.69	0.17	:50	10	10.7	15.47	0.22

ROCK BOLT TEST

Client: *National Parks and Wildlife Service*

Project: *Naracoorte Caves*

Location: *The Cathedral* (SE ENTRANCE)

Date tested: *April 7, 1992*

Tested by: *SW/TE*

Computed by: *SW*

Checked by: *TE*

Bolt Ident: 3					**Bolt Ident:** 4				
Time	**Elapsed Time (mins)**	**Load (kN)**	**Dial Gauge**	**Deflection (mm)**	**Time**	**Elapsed Time (mins)**	**Load (kN)**	**Dial Gauge**	**Deflection (mm)**
8:36	0	10.7	14.60	0.00	10:00	0	10.7	0.42	0.00
8:38	2	18.9	14.67	0.07	10:01	1	18.9	0.46	0.04
8:40	4	27.1	14.71	0.11	:02	2	27.1	0.54	0.12
8:42	6	35.4	15.08	0.48	:03	3	35.4	0.64	0.22
8:44	8	43.6	15.24	0.64	:04	4	43.6	0.81	0.39
8:45	9	35.4	15.24	0.64	:05	5	35.4	0.83	0.41
8:46	10	27.1	15.22	0.62	:06	6	27.1	0.84	0.42
8:47	11	18.9	15.18	0.58	:07	7	18.9	1.04	***
8:48	12	10.7	15.12	0.52	:08	8	10.7	1.03	***
8:49	13	43.6	15.32	0.72	:09	9	43.6	1.32	***
8:50	14	10.7	15.15	0.55	:10	10	10.7	1.10	***
					***	PLATE SUPPORTING DIAL			
						GAUGE MOVED. INITIAL			
						READING NO LONGER VALID			

APPENDIX C

EXPLANATORY SHEETS

Suggested Method for Determining Point Load Strength ______ [in part]

PROCEDURE

Specimen selection and preparation

6.(a) A test sample is defined as a set of rock specimens of similar strength for which a single Point Load Strength value is to be determined.

(b) The test sample of rock core or fragments is to contain sufficient specimens conforming with the size and shape requirements for diametral, axial, block or irregular lump testing as specified below.[7]

(c) For routine testing and classification, specimens should be tested either fully water-saturated or at their natural water content.[8]

Calibration

7. The test equipment should be periodically calibrated using an independently certified load cell and set of displacement blocks, checking the P and D readings over the full range of loads and displacements pertinent to testing.

The diametral test[2]

8.(a) Core specimens with length/diameter ratio greater than 1.0 are suitable for diametral testing.

(b) There should preferably be at least 10 tests per sample, more if the sample is heterogeneous or anisotropic.[7]

(c) The specimen is inserted in the test machine and the platens closed to make contact along a core diameter, ensuring that the distance L between the contact points and the nearest free end is at least 0.5 times the core diameter (Fig. 3a).

(d) The distance D is recorded $\pm 2\%$.[6]

(e) The load is steadily increased such that failure occurs within 10–60 sec, and the failure load P is recorded. The test should be rejected as invalid if the fracture surface passes through only one loading point (Fig. 4d).

(f) The procedure (c) through (e) above is repeated for the remaining specimens in the sample.

The axial test[2]

9.(a) Core specimens with length/diameter ratio of 0.3–1.0 are suitable for axial testing (Fig. 3b). Long pieces of core can be tested diametrally to produce suitable lengths for subsequent axial testing (provided that they are not weakend by this initial testing); alternatively, suitable specimens can be obtained by saw-cutting or chisel-splitting.

(b) There should preferably be at least 10 tests per sample, more if the sample is heterogeneous or anisotropic.[7]

(c) The specimen is inserted in the test machine and the platens closed to make contact along a line perpendicular to the core end faces (in the case of isotropic rock, the core axis, but see paragraph 11 and Fig. 5).

(d) The distance D between platen contact points is recorded $\pm 2\%$.[6] The specimen width W perpendicular to the loading direction is recorded $\pm 5\%$.[9]

(e) The load is steadily increased such that failure occurs within 10–60 sec, and the failure load P is recorded. The test should be rejected as invalid if the fracture surface passes through only one loading point (Fig. 4c).

(f) The procedures (c) through (e) above are repeated for the remaining tests in the sample.

The block and irregular lump tests

10.(a) Rock blocks or lumps of size 50 ± 35 mm and of the shape shown in Fig. 3(c) and (d) are suitable for the block and the irregular lump tests. The ratio D/W should be between 0.3 and 1.0, preferably close to 1.0. The distance L (Fig. 3c and d) should be at least $0.5W$. Specimens of this size and shape may be selected if available or may be prepared by trimming larger pieces by saw-or chisel-cutting.

(b) There should preferably be at least 10 tests per sample, more if the rock is heterogeneous or anisotropic.[7]

(c) The specimen is inserted in the testing machine and the platens closed to make contact with the smallest dimension of the lump or block, away from edges and corners (Fig. 3c and d).

(d) The distance D between platen contact points is recorded $\pm 2\%$. The smallest specimen width W perpendicular to the loading direction is recorded $\pm 5\%$. If the sides are not parallel, then W is calculated as $(W_1 + W_2)/2$ as shown in Fig. 3d.* This smallest width W is used irrespective of the actual mode of failure (Figs 3 and 4)

(e) The load is steadily increased such that failure occurs within 10–60 sec, and the failure load P is recorded. The test should be rejected as invalid if the fracture surface passes through only one loading point (see examples for other shapes in Fig. 4d or e).

(f) The procedure (c) through (e) above is repeated for the remaining tests in the sample.

Anisotropic rock

11. (a) When a rock sample is shaly, bedded, schistose or otherwise observably anisotropic it should be tested in directions which give the greatest and least strength values, which are in general parallel and normal to the planes of anisotropy.

(b) If the sample consists of core drilled through the weakness planes, a set of diametral tests may be completed first, spaced at intervals which will yield pieces which can then be tested axially.

(c) Best results are obtained when the core axis is perpendicular to the planes of weakness, so that when possible the core should be drilled in this direction. The angle between the core axis and the normal to the weakness planes should preferably not exceed 30°.

(d) For measurement of the I_s value in the directions of least strength, care should be taken to ensure that load is applied along a single weakness plane. Similarly when testing for the I_s value in the direction of greatest strength, care should be taken to ensure that the load is applied perpendicularly to the weakness planes (Fig. 5).

(e) If the sample consists of blocks or irregular lumps, it should be tested as two sub-samples, with load applied firstly perpendicular to, then along the observable planes of weakness.[10] Again, the required minimum strength value is obtained when the platens make contact along a single plane of weakness.

CALCULATIONS

Uncorrected point load strength

12. The Uncorrected Point Load Strength I_s is calculated as P/D_e^2 where D_e, the "equivalent core diameter", is given by:

$D_e^2 = D^2$ for diametral tests;

$= 4A/\pi$ for axial, block and lump tests;

and

$A = WD =$ minimum cross sectional area of a plane through the platen contact points.[6]

Size correction

13.(a) I_s varies as a function of D in the diametral test, and as a function of D_e in axial, block and irregular lump tests, so that a size correction must be applied to obtain a unique Point Load Strength value for the rock sample, and one that can be used for purposes of rock strength classification.

(b) The size-corrected Point Load Strength Index $I_{s(50)}$ of a rock specimen or sample is defined as the value of I_s that would have been measured by a diametral test with $D = 50$ mm.

(c) The most reliable method of obtaining $I_{s(50)}$, preferred when a precise rock classification is essential, is to conduct diametral tests at or close to $D = 50$ mm. Size correction is then either unnecessary ($D = 50$ mm) or introduces a minimum of error. The latter is the case, for example, for diametral tests on NX core, $D = 54$ mm. This procedure is not mandatory. Most point load strength testing is in fact done using other sizes or shapes of specimen. In such cases, the size correction (d) or (e) below must be applied.

(d) The most reliable method of size correction is to test the sample over a range of D or D_e values and to plot graphically the relation between P and D_e^2. If a log–log plot is used the relation is generally a straight line (Fig. 6). Points that deviate substantially from the straight line may be disregarded (although they should not be deleted). The value of P_{50} corresponding to $D_e^2 = 2500$ mm² ($D_e = 50$ mm) can then be obtained by interpolation, if necessary by extrapolation, and the size-corrected Point Load Strength Index calculated as $P_{50}/50^2$.

(e) When neither (c) nor (d) is practical, for example when testing single sized core at a diameter other than 50 mm or if only a few small pieces are available, size correction may be accomplished by using the formula:

$$I_{s(50)} = F \times I_s$$

The "Size Correction Factor F" can be obtained from the chart in Fig. 7,[11] or from the expression:

$$F = (D_e/50)^{0.45}$$

For tests near the standard 50 mm size, very little error is introduced by using the approximate expression:

$$F = \sqrt{(D_e/50)}$$

(f) The size correction procedures specified in this paragraph have been found to be applicable irrespective of the degree of anisotropy I_a and the direction of loading with respect to planes of weakness, a result that greatly enhances the usefulness of this test.

Mean value calculation

14.(a) Mean values of $I_{s(50)}$ as defined in (b) below are to be used when classifying samples with regard to their Point Load Strength and Point Load Strength Anisotropy Indices.

(b) The mean value of $I_{s(50)}$ is to be calculated by deleting the two highest and lowest values from the 10 or more valid tests, and calculating the mean of the remaining values. If significantly fewer specimens are tested, only the highest and lowest values are to be deleted and the mean calculated from those remaining.[12]

Point load strength anisotropy index

15. The Strength Anistropy Index $I_{a(50)}$ is defined as the ratio of mean $I_{s(50)}$ values measured perpendicular and parallel to planes of weakness, i.e. the ratio of greatest to least Point Load Strength Indices. $I_{a(50)}$ assumes values close to 1.0 for quasi-isotropic rocks and higher values when the rock is anisotropic. On average, uniaxial compressive strength is 20–25 times point load strength, as shown in Fig. 9. However, in tests on many different rock types the ratio can vary between 15 and 50 especially for anisotropic rocks, so that errors of up to 100% are possible in using an arbitrary ratio value to predict compressive strength from point load strength.

$I_{s(50)}$ is approximately 0.80 times the uniaxial tensile or Brazilian tensile strength.

7. Because this test is intended primarily as a simple and practical one for field classification of rock materials, the requirements relating to sample size, shape, numbers of tests etc, can when necessary be relaxed to overcome practical limitations. Such modifications to procedure should however be clearly stated in the report.

11. The size correction factor chart (Fig. 7) is derived from data on cores tested diametrally and axially and from tests on blocks and irregular lumps, for rocks of various strengths, and gives an averaged factor. Some rocks do not conform to this behaviour, and size correction should therefore be considered an approximate method, although sufficient for most practical rock classification applications. When a large number of tests are to be run on the same type of rock it may be advantageous to first perform a series of tests at different sizes to obtain a graph of load vs D_e^2 as in Fig. 6. If the slope of such a log–log graph is determined as "n", the size correction factor is then $(D_e/50)^m$ where $m = 2(1 - n)$. This can either be calculated directly or a chart constructed.

*Paragraph and figure numbers are those of the original reference: ISRM Commission on Testing Methods 1985. Suggested method for determining Point Load Strength (revised version) Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. 22, 51-60.

Important references for further reading:-

GUIFU, X., Hong, L. On the statistical analysis of data and strength expression in the rock point load tests. Proc. 5th Int. Cong. Int. Ass. of Engng. Geology, Beunos Aires 1986.

READ, J.R.L., THORNTON, P.N., REGAN, W.M. A rational approach to the Point Load Test. Third Australia New Zealand Conf. on Geomechanics, Wellington 1980. Vol. 2, pp2-35 to 2-39.

TURK, N. and DEARMAN, W.R. Improvements in the determination of point load strength. Bull. Int. Ass. Engng. Geol. No. 31. paris 1985 pp137-142.

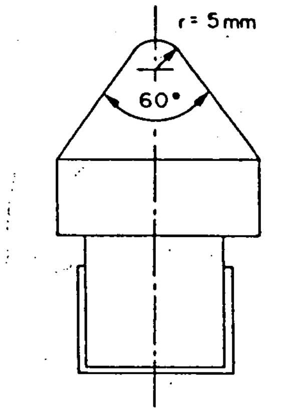


Fig. 2. Platen shape and tip radius.

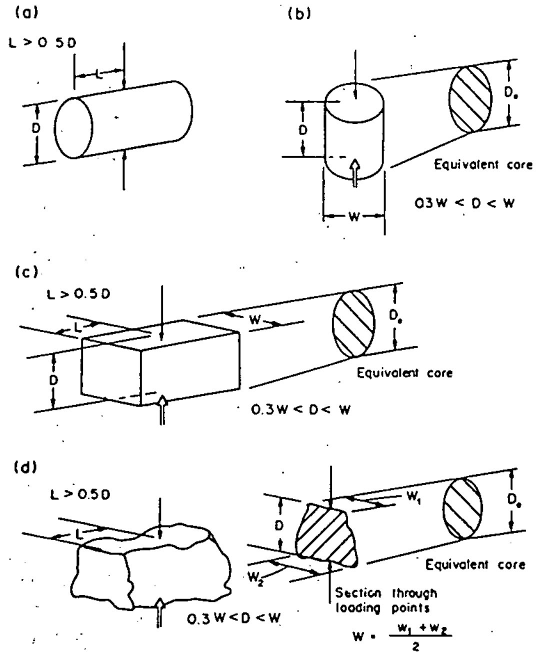


Fig. 3. Specimen shape requirements for (a) the diametral test, (b) the axial test, (c) the block test, and (d) the irregular lump test.

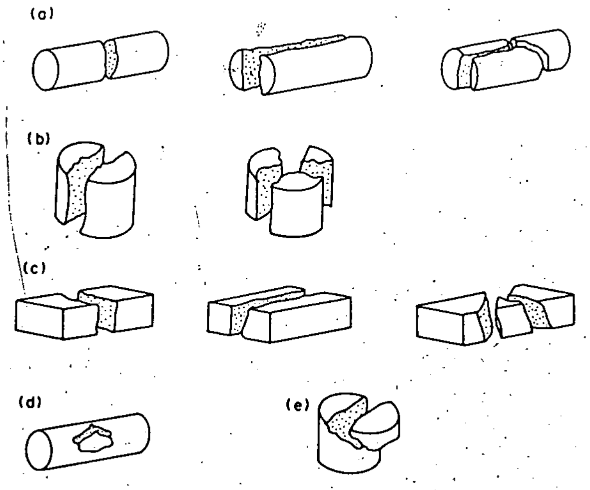


Fig. 4. Typical modes of failure for valid and invalid tests. (a) Valid diametral tests; (b) valid axial tests; (c) valid block tests; (d) invalid core test; (e) invalid axial test.

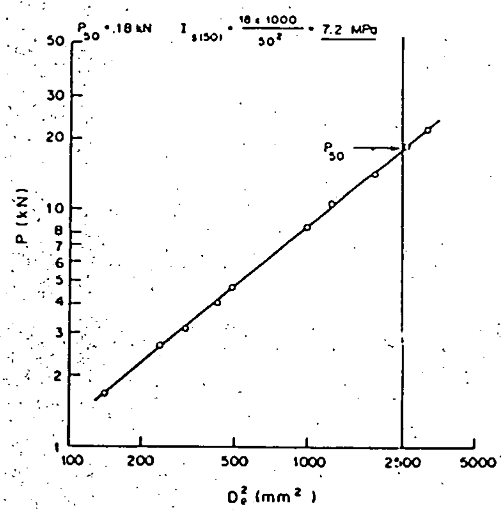


Fig. 6. Procedure for graphical determination of $I_{s(50)}$ from a set of results at D_e values other than 50 mm.

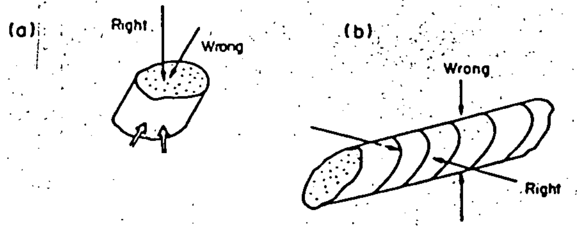


Fig. 5. Loading directions for tests on anisotropic rock.

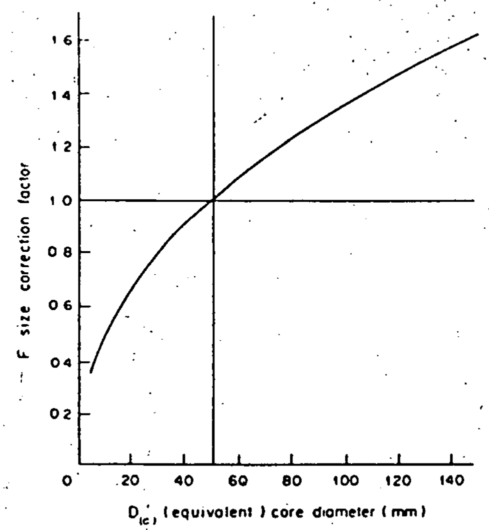


Fig. 7. Size correction factor chart.

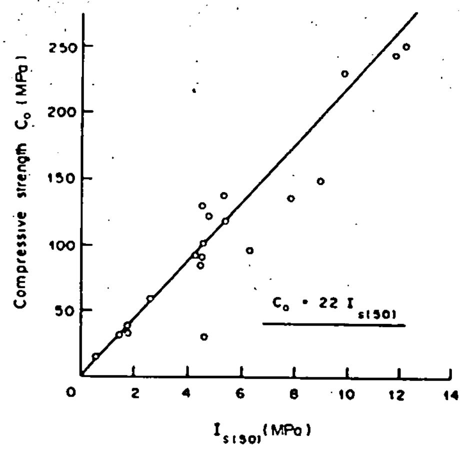


Fig. 9. Example of correlation between point load and uniaxial compressive strength results.

NOTE Ⓐ SOIL TYPE

42. Classification of soils

42.1 Use of the system. Use of the British Soil Classification System for Engineering Purposes (BSCS) described in 42.3 is discretionary; for many purposes, a full description of soils in accordance with clause 41 will suffice. The BSCS is recommended primarily for soils to be used as construction materials, when it is particularly useful. When the symbols for the BSCS are used, a full written description, including both the soil group name and supplementary descriptive terms as discussed in clause 41, is also required.

Not included in these notes

42.2 Nature and purpose of soil classification

42.2.1 *Distinction between soil description and soil classification.* A full description gives detailed information on the grading, plasticity, colour, and particle characteristics of a soil, as well as on the fabric, the state of bedding, nature of discontinuities and strength condition in which it occurs in a sample, borehole or exposure. Few, if any, soils will have identical descriptions. On the other hand, a soil classification places a soil in a limited number of groups on the basis of grading and plasticity of a disturbed sample. These characteristics are independent of the particular condition in which a soil occurs, and disregard the influence of the structure, including fabric, of the soil mass; but they can give a good guide to how the disturbed soil will behave when used as a construction material, under various conditions of moisture content.

41.3 Material characteristics of soils

41.3.1 *Range of application.* Material characteristics refer to those characteristics that can be described from visual and manual examination of either disturbed or undisturbed samples, and include soil name, colour, particle shape and particle composition.

In a soil description, the main characteristics should preferably be given in approximately the following order.

(a) *Mass characteristics* (see 41.2)
 (1) Field strength or compactness (see table 6), and indication of moisture condition.
 (2) Bedding.
 (3) Discontinuities.
 (4) State of weathering.
(b) *Material characteristics* (see 41.3)
 (1) Colour.
 (2) Particle shape and composition.
 (3) Soil name (in capitals, e.g. SAND), grading and plasticity.
(c) *Geological formation, age and type of deposit* (see 41.4).
(d) *Classification* (optional) (see clause 42).
 Soil group symbol.

Examples:
Firm closely-fissured yellowish-brown CLAY of high plasticity. London Clay.

41.3.2.5 *Colour.* Details are given in the extreme right hand column of table 6. For more detailed descriptions, colour charts based on the system of Munsell may be used [156, 157] †.

41.3.2.6 *Particle shape and composition.* Where appropriate, particle shape may be described by reference to the general form of the particles, their angularity which indicates the degree of rounding at edges and corners, and their surface characteristics. Some recommended terms are as follows.

Angularity	angular subangular subrounded rounded
Form	equidimensional flat elongated flat and elongated irregular
Surface texture	rough smooth

41.3.3 *Made ground.* It is rarely possible to carry out significant soil tests on made ground, and descriptions of the material are all that remains after the samples have been discarded or pits filled in. Good descriptions are, therefore, of even greater importance with this type of material and should include information on the following as well as on the soil constituents.

(a) Mode of origin of the material.

(b) Presence of large objects such as concrete, masonry or old motor cars.

(c) Presence of voids or collapsible hollow objects.

(d) Chemical waste, and dangerous or poisonous substances.

(e) Organic matter, with a note on the degree of decomposition.

(f) Odourous smell.

(g) Striking colour tints.

(h) Any dates readable on buried newspapers.

(i) Signs of heat or internal combustion under ground, i.e. steam emerging from borehole.

41.3.2 *Soil name*

41.3.2.1 *Introduction.* The soil name is based on particle size distribution and plastic properties. These characteristics are used because they can be measured readily with reasonable precision, and estimated with sufficient accuracy for descriptive purposes.

Table 7. Names and descriptive letters for grading and plasticity characteristics

		Descriptive name	Letter
Coarse components	Main terms	GRAVEL SAND	G S
	Qualifying terms	Well graded Poorly graded Uniform Gap graded	W P Pu Pg
Fine components	Main terms	FINE SOIL, FINES may be differentiated into M or C SILT (M-SOIL)* plots below A-line of plasticity chart of figure 31 (of restricted plastic range) CLAY plots above A-line (fully plastic)	F M C
	Qualifying terms	Of low plasticity Of intermediate plasticity Of high plasticity Of very high plasticity Of extremely high plasticity Of upper plasticity range† incorporating groups I, H, V and E	L I H V E U
Organic components	Main term	PEAT	Pt
	Qualifying term	Organic may be suffixed to any group	O

*See note 5 following table 8.
†This term is a useful guide when it is not possible or not required to designate the range of liquid limit more closely, e.g. during the rapid assessment of soils.

41.3.2.3 *Plasticity.* Clay and silt, both alone and in mixtures with coarser material, may be classified as follows:

Term	Range of liquid limit
of low plasticity	under 35 %
of intermediate plasticity	35 % to 50 %
of high plasticity	50 % to 70 %
of very high plasticity	70 % to 90 %
of extremely high plasticity	over 90 %

Term	Composition of the coarse fraction
Slightly sandy GRAVEL	up to 5 % sand
Sandy GRAVEL	5 % to 20 % sand
Very sandy GRAVEL	over 20 % sand
GRAVEL/SAND	about equal proportions of gravel and sand
Very gravelly SAND	over 20 % gravel
Gravelly SAND	20 to 5 % gravel
Slightly gravelly SAND	up to 5 % gravel

41.3.2.4 *Deposits containing boulder-sized and cobble-sized particles.* Usually, very coarse deposits can be described only in excavations or exposures. They are described as follows:

	Main name	Estimated boulder or cobble content of very coarse fraction
Over 50 % of material is very coarse (over 60 mm)	BOULDERS or BOULDER GRAVEL	Over 50 % is of boulder size (over 200 mm)
	COBBLES or COBBLE GRAVEL	Over 50 % is of cobble size (200 mm to 60 mm)

BOULDERS			> 200mm
COBBLES			> 60 < 200mm
GRAVEL	Coarse		20 - 60mm
	Medium		6 - 20mm
	Fine		2 - 6mm
SAND	Coarse	0.6 - 2mm	
	Medium		0.2 - 0.6mm
	Fine		0.06 - 0.2
SILT	Coarse		0.02 - 2mm
	Medium		0.006 - 0.02
	Fine		0.002 - 0.006
CLAY			< 0.002

NOTE Ⓑ GRAPHIC LOG

Made ground

GRAVEL (GW,GP)

SAND (SW,SP)

SILT (ML,MH)

CLAY (CL,CH)

ORGANIC SOILS (OL,OH,Pt)
THIN LAYER

COMPOSITE/INTERBEDDED - EXAMPLES

SILTY SAND (SM)

SANDY CLAY (CL)

SP CLAY (CL) WITH THIN SAND (SP) INTERBEDS

CLAY (CH) WITH SAND (SC) DYKE,AND FISSURE (BOTH VERTICAL)

NOTE: In all the above cases bedding would have been either observed as, or assumed to be horizontal, unless otherwise stated.

To be used for Peat exclusively

NOTE Ⓒ MOISTURE CONTENT

2. DEGREE OF SATURATION OF SANDS — DESCRIPTIVE TERMS

Condition of sand	Criteria	Degree of saturation (%)
Dry	Oven dried. Not usually met in field.	0
Humid	Feels dry, grains "run" freely in bands.	1–25
Damp	Feels cool, slight darkening of colour, grains have slight tendency to adhere to one another.	25–50
Moist	Feels cool, darker colour, grains tend to adhere to one another.	50–75
Wet	Feels cold, makes hands wet; should be close to water table.	75–99
Saturated	Below water table, or static water level in excavation or drill holes.	100

7.8.4. MOISTURE CONTENT OF CLAY SOILS[1]

Abbreviations	Meaning
MC ≃ LL	Moisture content near liquid limit
MC < LL	Moisture content less than liquid limit
MC > PL	Moisture content greater than plastic limit
MC ≃ PL	Moisture content near plastic limit
MC < PL	Moisture content less or equal to plastic limit
MC < PL	Moisture content less than plastic limit
MC ≪ PL	Moisture content much less than plastic limit

(Taken from AIMM Field Geologists Manual. 2nd edition)

NOTE Ⓓ DENSITY

The relative density of sands and gravels may be determined by the standard penetration test. A scale in terms of *N*-values (see BS 1377) is as follows.

Term		SPT *N*-values: blows/300 mm penetration	Relative density %
Very loose	(VL)	0 to 4	< 15
Loose	(L)	4 to 10	15 to 35
Medium dense	(MD)	10 to 30	35 to 65
Dense	(D)	30 to 50	65 to 85
Very dense	(VD)	over 50	> 85

Correct for effect of overburden pressure

Rocks					
Sedimentary		Metamorphic		Igneous	
	Chalk				
	Limestone		Coarse-grained		Coarse-grained
Rudaceous	Conglomerate				
Rudaceous	Breccia		Medium-grained		Medium-grained
Arenaceous	Sandstone		Fine-grained		Fine-grained
Argillaceous	Siltstone				
Argillaceous	Mudstone				
Argillaceous	Shale				
	Coal				
	Pyroclastic (volcanic ash)				
	Gypsum, Rocksalt etc				

Note that symbols for organic and composite/interbedded examples are modified BS5930 1981 by D. Stapledon. All other symbols are BS5930 1981.

Table 12. Special symbols for borehole record

Fault

Slip surface

Organic layer+

Marine band

Shell band

(+ Not within BS 5930 1981)

Examples:

Medium-grained igneous faulted against coarse-grained metamorphic rock

Fault in sandstone

Slip surface in sandstone

Slip surface in shale

NOTE Ⓔ CONSISTENCY

Consistency*: For materials which have cohesive properties the following terms are used to describe consistency.

		Term		qu unconfined compressive strength (KPa)	Undrained shear strength (KN/m²)
SILT		Soft or loose	Easily moulded or crushed in the fingers.		
SILT		Firm or dense	Can be moulded or crushed by strong pressure in the fingers.		
CLAY	(VS)	Very soft	Exudes between fingers when squeezed in hand.	< 25	less than 20
CLAY	(S)	Soft	Moulded by light finger pressure.	25 to 50	20 to 40
CLAY	(F)	Firm	Can be moulded by strong finger pressure.	50 to 100	40 to 75
CLAY	(St)	Stiff	Cannot be moulded by fingers. Can be indented by thumb.	100 to 200	75 to 150
CLAY	(VSt)	Very stiff	Can be indented by thumb nail.	200 to 400	greater than 150
		Hard+		> 400	

+ From Australian Standard 1726-1981 (ref: D.3.2.1)

* Legend colum may be used to indicate pocket penetrometer values (p.p.).

NOTE Ⓕ IN-SITU TESTS

2.5 In situ tests

S Standard penetration test (SPT). A 50 mm diameter split spoon sampler is driven 450 mm into the soil using a 65 kg hammer with a 760 mm drop, and the penetration resistance is expressed as the number of blows required to obtain 300 mm penetration below an initial penetration of 150 mm through any disturbed ground at the bottom of the borehole.

In the borehole record, the depth of the test is that at the start of the normal 450 mm penetration. The number of blows to achieve the standard penetration of 300 mm (the '*N*' value) is shown after the test index letter, but the seating blows through the initial 150 mm penetration are not reported unless the full penetration of 450 mm is not achieved. In the latter case, the symbols below are added to the test index letter:

S+ Seating blows only.

S‡ Blow count includes seating blows.

S* No penetration.

S + Split spoon sampler sank under its own weight.

The test is usually completed when the number of blows reaches 50. For tests achieving the full penetration of 450 mm, the depth at which the test procedure is commenced is given in the depth column on the borehole record, whilst for those tests not achieving full penetration, the depths of both the top and the bottom of the test drive are shown. If a sample is not recovered in the split spoon sampler, a disturbed sample is taken on completion of the test drive. Both are given the same depth as the top of the SP test drive.

C Dynamic Cone Penetration Test (CPT). A test conducted usually in coarse granular soils using the same procedure as for the SPT but with a 50 mm diameter, 60° apex solid cone fitted to the split spoon sampler. Variations in test results are indicated by the same symbols as for the SPT. The bulk disturbed sample taken, is given the same depth as the top of the CP test drive.

V Vane test.

J Borehole jack test. See text of report for full description.

K Permeability test. See text of report for full description.

Legend column may be used to indicate drill water loss % if applicable.

NOTE (G) ROCK TYPE

44.1.2 *General description.* Rocks seen in natural outcrops, cores and excavations should generally be described in the following sequence:

colour;
grain size;
texture and structure; (Fabric).
state of weathering;
rock name (in capitals, e.g. GRANITE);
strength;
other characteristics and properties.

Term	Description
Fresh	No visible sign of weathering of the rock material.
Discoloured	The colour of the original fresh rock material is changed and is evidence of weathering. The degree of change from the original colour should be indicated. If the colour change is confined to particular mineral constituents this should be mentioned.
Decomposed	The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed.
Disintegrated	The rock is weathered to the condition of a soil in which the original material fabric is still intact. The rock is friable, but the mineral grains are not decomposed.

44.2.5 *Rock name.* An aid to the identification of rocks for engineering purposes is given in table 9. The table follows general geological practice, but is intended as a guide only; geological training is required for the satisfactory identification of rocks. Engineering properties cannot be inferred from the rock names in the table.

44.3 Description of rock masses

44.3.1 *Introduction.* The description of rock masses requires information additional to the description of the rock material. A rock mass should be described first as a rock material, followed by additional information about discontinuities and other features of engineering significance. Such information includes:

(a) the description of rock types in the mass, with reference to major geological structures;
(b) the dip magnitude and direction, nature, spacing and persistence, width of opening of discontinuities;
(c) details of the weathering profile.

44.3.2 *Structure.* The structure of the rock mass is concerned with the larger-scale inter-relationship of textural features. Common terms should be used where possible. Terms frequently used to describe sedimentary rocks include bedded, laminated; metamorphic rocks may be foliated, banded, cleaved; igneous rocks may be massive, flow banded.

Descriptive terms used for the spacing of these planar structures are as follows.

Term	Spacing
Very thick	greater than 2 m
Thick	600 mm to 2 m
Medium	200 mm to 600 mm
Thin	60 mm to 200 mm
Very thin	20 mm to 60 mm
Thickly laminated (Sedimentary) Narrow (Metamorphic and igneous)	6 mm to 20 mm
Thinly laminated (Sedimentary) Very narrow (Metamorphic and igneous)	less than 6 mm

Spacings can be shown graphically adjacent to "core" column if desired.

44.2.2 *Grain size.* A descriptive classification scheme is given in table 9. Grain size refers to the average dimension of the mineral or rock fragments comprising the rock. It is usually sufficient to estimate the size by eye, which may be aided by a hand lens in the assessment of fine-grained or amorphous rocks. The limit of unaided vision is approximately 0.06 mm.

44.2.3 *Texture and fabric.* The texture of a rock refers to individual grains. The arrangement of grains, referred to as the rock fabric, may show a preferred orientation. Terms frequently used include: porphyritic, crystalline, cryptocrystalline, granular, amorphous and glassy.

Table 10. Scale of weathering grades of rock mass

Term	Description	Grade	Legend +
Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.	I	(Fr)
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.	II	(SW)
Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.	III	(MW)
Highly weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV	(HW)
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V	(CW)
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI	(RS)

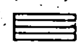
(+Choice of symbol at discretion of user and is not within BS 5930 1981)

44.2.8 *Examples.* The following descriptions are given for guidance in the use of appropriate descriptive terms.

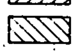
For example, a metamorphic rock material might be described as a 'dark grey, medium grained, thinly foliated, fresh GNEISS, very strong'; a typical description of a sedimentary rock might be a 'yellowish brown, coarse grained, wholly discoloured, micaceous SANDSTONE, moderately weak'; an igneous rock might be described as 'dark greyish green, medium grained, partly discoloured, quartz DOLERITE, extremely strong'.

For strength term see note (H)

In CONDENSED REPORT LOG use the following symbols. Where rock mass is converted to residual soil use appropriate soil symbol.

Fresh to slightly weathered 

Moderately weathered 

Highly weathered 

Completly weathered 

NOTE Ⓗ ROCK STRENGTH

44.2.6 *Strength of rock material.* A scale of strength, based on uniaxial compressive testing, is as follows.

Term	Symbol	Compressive strength, MN/m^2	Approx. Is 50 (MPa) *	Geol. Society (Ref.3) +	Approx kg/cm^2
Very weak	(VW)	less than 1.25	< 0·05	Very weak: Broken by hand with difficulty.	<15
Weak	(W)	1.25 to 5	0·05 – 0·2	Weak: Material crumbles under blows with the sharp end of a geological pick.	15 – 50
Moderately weak	(MW)	5 to 12.5	0·2 – 0·5	Moderately weak: Too hard to cut by hand into a triaxial specimen.	50 – 130
Moderately strong	(MS)	12.5 to 50	0·5 – 2·0	Moderately strong: 5 mm indentations with sharp end of pick.	130 – 500
Strong	(S)	50 to 100	2·0 – 4·0	Strong: Hand held specimen can be broken with single blow of geological hammer.	500 – 1000
Very strong	(VS)	100 to 200	4·0 – 8·0	Very strong: More than one blow of geological hammer required to break specimen.	1000 – 2050
Extremely strong	(ES)	greater than 200	> 8·0		

(note : 1MPa ~ 145psi ~ $10 kg/cm^2$)

The strength of a rock material determined in the uniaxial compression test is dependent on the moisture content of the specimen, anisotropy and the test procedure adopted.

* Legend column may be used to indicate point load strength index, Is 50 (MN/m^2).(Correcting for any fabric anisotropy).

\+ Geological Society. Engineering Group Working Party (1977) The Description of Rock Masses for Engineering Purposes. Q. Jnl. Eng Geol. V-10. pp355 - 388.

Australian Standard AS 1726-1981

D5.2 Strength. The following terms are used to describe rock strength:

Rock strength class	*Abbreviation*	*Point load strength index, I, (50) (MPa)*
Extremely low	EL	< 0.03
Very low	VL	0.03 to 0.1
Low	L	0.1 to 0.3
Medium	M	0.3 to 1
High	H	1 to 3
Very high	VH	3 to 10
Extremely high	EH	> 10

NOTE Ⓘ CORE QUALITY

44.2.7 *Fracture state.*

Elsewhere [161], a determination of Rock Quality Description (RQD) has been proposed as a quantitative measure of the fracture state of rock. RQD is the percentage of rock recovered as sound lengths which are 100 mm or more in length. Only core lengths determined by geological fractures should be measured. Descriptive terms are as follows.

RQD	Term	Suggested symbol +
0 % to 25 %	Very poor	(VP)
25 % to 50 %	Poor	(P)
50 % to 75 %	Fair	(F)
75 % to 90 %	Good	(G)
90 % to 100 %	Excellent	(E)

(+ Not within BS 5930 1981)

2.3 Rock core descriptions (Values may be shown graphically in "core" column)

TCR Total core recovery. The length of the total amount of core sample recovered expressed as a percentage of the length of core run.

SCR Solid core recovery. The length of core recovered as solid cylinders, expressed as a percentage of the length of core run.

RQD Rock quality designation. The sum length of all core pieces that are 10 cm or longer, measured along the centre line of the core, expressed as a percentage of the core drilled.

NOTE Ⓙ HYDROGEOLOGY

Water cut, standpipe and piezometer installation indicate if sample taken, date and by whom. Show base of standpipe and centre of piezometer in legend column. Show time and dates of water level observations.

NOTE Ⓚ DISCONTINUITY DESCRIPTION

44.3.3.2 ***Discontinuity spacing in one dimension.***
The following descriptive scheme should be used.

Term	Spacing	Suggested Symbol +
Very widely spaced	greater than 2 m	(VWS)
Widely spaced	600 mm to 2 m	(WS)
Medium spaced	200 mm to 600 mm	(MS)
Closely spaced	60 mm to 200 mm	(CS)
Very closely spaced	20 mm to 60 mm	(VCS)
Extremely closely spaced	less than 20 mm	(ECS)

(+ Not within BS 5930 1981)

44.3.3.3 ***Discontinuity spacing in three dimensions.***
The spacing of discontinuities may be described with reference to the size and shape of rock blocks bounded by the discontinuities. Rock blocks may be approximately equidimensional, tabular or columnar in shape. Descriptive terms may be used in accordance with the following.

First term	Maximum dimension
Very large	greater than 2 m
Large	600 mm to 2 m
Medium	200 mm to 600 mm
Small	60 mm to 200 mm
Very small	less than 60 mm

Second term	Nature of block
Blocky	Equidimensional
Tabular	Thickness much less than length or width
Columnar	Height much greater than cross section

The use of these terms requires an understanding of the distribution of discontinuities in three-dimensions; in consequence they cannot be used in the description of drill core.

NOTE Ⓛ

41.4 Geological formation, age and type of deposit. The geological formation should be named where this can be done with confidence, but it may not be easy to tell to which formation a sample belongs, or to locate formation boundaries in a borehole or exposure; conjecture should be avoided.

The characteristic lithology is sometimes indicated in the formation name, e.g. London Clay, but it should be remembered that at a particular location or horizon the lithology may be completely different from that indicated in the formation name.

A term indicating the geological origin or type of the deposit may be given on the map legend, e.g. Made ground, Peat, Head, Alluvium, River terrace, Brickearth, Blown (aeclian) sand, Till. The term can indicate to the engineer some of the characteristics that the deposit may be expected to show.

NOTE Ⓜ

41.5 Additional information. Any additional information on the composition, structure, behaviour or other characteristics of the soil that would be of value in assessing its nature and properties should be recorded. Special note should be made if the properties of the material are considered to be unusual in relation to the rest of its description. Note should also be made if there is doubt whether the sample described is representative of the material at the level from which it was sampled, due, for instance, to fracture of particles or loss of fines during sampling, or to the sample size or borehole diameter being too small in relation to the grading or structure of the material being sampled. Where relevant, it should be made clear whether the sample on which the description is based was disturbed or undisturbed. Where the strength of the soil is likely to vary because of seasonal variations in moisture content, this should be noted.