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FIELD PROCEDURES MANUAL FOR COLLECTION OF GEOTECHNICAL DATA FROM DRILL CORE

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BR-23N

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1.0 INTRODUCTION

This document provides a description of procedures for collecting geotechnical data from drill core. The procedures described are designed to obtain the critical geotechnical characteristics of a rock mass which are likely to affect design of any potential surface or underground excavations.

2.0 ROCK MASS CLASSIFICATION

Geotechnical data collected from drill core is used to rate the rock mass according to its "quality". Rock mass classification schemes provide a basis for this assessment, and permit comparison with different rock mass conditions encountered at other mines. The classification systems have been refined over the years to include a large database of excavations and ground conditions.

Two rock mass classification schemes have been widely used in the mining industry. These are Barton's Q system and Bieniawski/Laubscher's RMR systems. Although the systems differ, they rely on similar data to be collected in order to classify rock mass strength. They are based on measurement of intact rock strength, fracture spacing and condition, groundwater pressure and orientation of structures with respect to mine excavations. The resulting quality rating can be used to design stable excavations and predict ground support requirements based on experience elsewhere.

Traditionally, geotechnical data collection is carried out at advanced stages of a mining project, when mine engineering studies have already been initiated. However, a large quantity of geotechnical data from exploration drill core is often lost because the core was not geotechnically logged. This is not cost-effective considering the substantial investment required to obtain this drill core.

Early collection of geotechnical data has the following advantages:

- Geotechnical data is recorded before the drill core is split for assay or can deteriorate due to such factors as weathering and handling.
- The cost of data collection during geological logging is lower than the costs associated with mobilizing another data collection program at a later date.

3.0 GEOTECHNICAL CORE LOGGING

The description of a rock mass for engineering purposes requires assessment of the characteristics of both the rock material and the fractures that intersect it (bedding planes, joints, foliation, cleavage). Rock material in its fresh, unweathered state can vary in strength from relatively soft (e.g. chalks, marls, claystones) to extremely hard (e.g. diabase, hornfels, quartzite). The strength along fractures can also vary significantly depending on the fracture condition.

The format for describing the geotechnical characteristics of a rock mass is based on international standard practice, and is structured to provide all data necessary for the previously mentioned rock mass classification schemes. It provides a simplified field guide which assumes a general working knowledge of geology, geologic terms and geologic processes.

The following basic parameters are required from geotechnical core logging:

- Depth;
- Description (Rock Type);
- Core recovery;
- RQD - rock quality designation;
- Strength of intact rock (Rock Hardness);
- Weathering/alteration;
- Fracture type;
- Fracture frequency;
- Dip angle of structure with respect to core axis;
- Fracture condition.

Example core logging forms for recording of geotechnical data are shown in Tables 1 and 2, and blank core logs are provided in Appendix A. It is recognized that alternative formats may be required to satisfy specific requirements of individual projects.

The core logging parameters are described in detail as follows:

3.1 Depth

Geologic and geotechnical core logging is usually carried out for each 5 to 10 ft. core run. The recorded depth is the length along core axis from collar of hole to the beginning (From) to the end (To) of the logged interval. Example: An interval from 15.7 feet to 20.7 feet would be recorded as:

From: 15.7 feet ... To: 20.7 feet.

In order to remove any sampling bias and to provide a consistent system of measurement, data should be collected over uniform intervals of drill core, such as core runs.

3.2 Rock Type

The rock type is recorded using either its proper name or a code. The name used to identify rock type must stay consistent throughout the project to avoid confusion with identification of geologic units. Uniform application of nomenclature is usually more critical than absolute accuracy of that nomenclature.

3.3 Core Recovery

Core Recovery records the total amount of core recovered over the measured length drilled for each core run. Measure and record core recovery to the nearest 1%.

Core losses are an important indication of potentially poor geotechnical conditions, since they most commonly occur in weak or highly fractured zones which may be important for determining rock mass properties. Rubble, redrill, or slough recovered at the top of a core lift that was not in place is not counted as recovered core and should be discarded or clearly labeled to avoid subsequent misclassification.

It is not uncommon for some core to slip through the core lifter and to be dropped out of its core tube. This problem frequently indicates a worn or unsuitable core lifter which should be replaced. Core should be represented on the log at the location it occupied in the ground. This requires some interpretation when rock cored during one run is dropped

and is recovered during a subsequent run. Core recoveries should not exceed 100 percent on any logged interval. Core which was drilled in a previous run can often be identified by marks from the drilling or the core lifter.

3.4 RQD - Rock Quality Designation

The RQD is a quantitative index of rock quality based on a core recovery procedure in which the core recovery is determined incorporating only those pieces of hard, sound core that are longer than twice the width of the core.. Shorter lengths are ignored, as shown in Figure 1. RQD is determined from the following expression:

$$RQD\,(\%) = 100\ x\ \frac{\textit{the sum of the lengths of core in pieces equal to or longer than } 10\ cm}{\textit{length of core run}}$$

For NQ core, the length index I is 4 inches or 10 cm.

It is important to distinguish between mechanical, drill induced breaks and natural breaks found in the core. A mechanical break caused by handling should not adversely affect the RQD index, which is a measure of the in-situ rock quality. The mechanical broken core segments should be approximated into a solid unit of core in order to arrive at a reliable RQD value. Naturally occurring fractures should be marked with a colored crayon to aid this process. RQD should be evaluated over an interval not less than 3 ft. (0.9 m).

It is sometimes difficult to distinguish natural fractures. If the origin of the fracture is uncertain, then it is considered to be a natural break in order to ensure conservative values for the RQD and fracture logs. As a guide, clean, fresh surfaces that are oriented at close to 90° to the core axis that can be rejoined with only a hair-line separation are typically drill-induced. Surfaces that are rounded, ground, weathered, contain infilling or coatings, occur at some angle other than perpendicular to the core axis, or cannot be rejoined cleanly should be counted as natural fractures. If in doubt, consider it to be a natural fracture.

One special case that may be encountered in measuring RQD is a single fracture parallel to the core axis. Sound core with a single fracture is counted as intact rock and assigned

an RQD of 100%. This method is used to avoid biasing the RQD measurement with a single fracture parallel to the drillhole.

RQD is valid only for sound core and should not be used for very poorly indurated materials such as clays and weak claystones. In these cases, the letters N/A (not applicable) should be entered into the RQD column.

3.5 Fracture Type

Record the types of fractures present in the core interval, using the definitions and abbreviations shown below:

Fracture (FR):	Any surface across which the rock is now, or has in the past, been broken.
Vein (V):	A fracture of finite thickness which is infilled by another mineral (e.g. quartz).
Joint (J):	A fracture with no infilling and no evidence of previous movement.
Fault (F):	A fracture across which there has been displacement of the rock.
Foliation (FO):	A preferential direction of structural weakness in the rock due to alignment of weak minerals, caused by heat and pressure during metamorphism.
Bedding (B):	A plan of structural weakness caused by sedimentary processes (e.g. mud seam in sandstone).

This data is recorded in the Fracture Count Column, see below.

3.6 Fracture Frequency

Record the number of natural fractures which occur over the length of core examined or a per unit length for a given fracture type. If the fracture count is recorded per run, then record run in the "per__" column. Otherwise record per metre or ft. The fracture frequency per metre/ft is determined by dividing the fracture count by the length of core

interval. Do not include drilling-induced fractures in the count. Also, do not include fractures with length of persistence less than the core diameter.

Fracture frequency is used to determine spacing between fractures within a rock mass. Fracture spacing has a direct influence on rock mass strength and behavior. Generally, the smaller the fracture spacing, the greater the difficulty in maintaining stable excavations. This parameter is therefore critically important for mine design studies. It is also required to complement RQD because measurements of RQD become increasingly less sensitive to changes in fracture spacing above 4 inches (10 cm).

3.7 Dip with Respect to Core Axis

Record the dip angle or range of dip angles for fractures observed in the core. If more than one joint set exists, record the average dip angle for each joint set. This angle is measured relative to the core axis, where 0° = parallel to core axis and 90° = perpendicular to core axis.

3.8 Fracture Condition

Two options are provided to estimate fracture condition. These options are based on different rock mass classification schemes, and are included to satisfy the requirements of a range of projects (open pit and underground). Values are obtained from Table 3.

Option 1 - Barton's Q System:

- Number of Fracture Sets (Jn)

  Observe the type and orientation of fractures in the drill core, and record a value from Table 3. Examples: One set corresponds to one distinct fracture orientation, such as a foliation. Two sets indicates that two distinct fracture orientations are present.

- Fracture Roughness (Jr)

  Observe the roughness of fractures in the logging interval, and record a value from Table 3 for the average roughness in that logging interval. Don't record the roughness for each individual fracture.

- Fracture Alteration (Ja)

  Observe the alteration of fractures in the logging interval, and record a value from Table 3 for the average alteration in the logging interval. This rating distinguishes between fractures which are filled with alteration minerals such as clay, and those which are not. Filled fractures lower the rock mass strength, thereby reducing stability of mining excavations.

Option 2 - Bieniawski's RMR System:

Observe the general condition of fractures in the core interval, and record a value from Table 3 which is representative of the fracture condition.

3.9 Strength of Intact Rock (Strength Index)

The strength of the pieces of intact core can be estimated using Table 4. A pocket knife and rock hammer is required for this task. Experience to date has shown that this simple observational approach provides a cost-effective solution for collection of data on intact rock strength, provided that strengths are calibrated with laboratory testing.

On projects where intact rock strength is a significant factor, point load tests should be performed in the field as part of the standard logging procedure. Point load tests provide a quantitative measure of rock strength which is necessary to bridge the gap between the qualitative description of rock characteristics (Strength Index) and the quantitative requirements of some engineering evaluations. Every effort should be made to test samples that are representative of the rock mass rather than concentrating testing on samples that are easiest to test. Both axial and diametral tests should be performed to provide a measure of anisotropy.

3.10 Weathering/Alteration Index

Record the degree of weathering using the system described in Table 5. This provides a qualitative measure of the degree of weathering for the original rock material. As the

hole depth increases the degree of weathering will decrease and alteration will begin to have an increasing influence on the rock strength. Record the type of alteration and its intensity.

3.11 Core Orientation

When logging oriented core from inclined boreholes, the orientations recorded are shown on Figure 1 and described below.

The core dip angle, α, is between the discontinuity plane and the core axis, and can be measured with a core goniometer.

Core dip direction, β, is the angle around core from a reference line, usually top of core, to the lowest intersection of the discontinuity plane with the core. All measurements are made clockwise around the core, when looking in a downhole direction, using a linear protactor, see Figure 2.

Where the dip direction cannot be determined due to loss of orientation for the run, the dip angle with respect to the core axis is recorded, and "999" is noted as the dip direction on the form.

Orientation is generally lost across a broken or lost core intersection in the run. However, if the core contains a unique constant foliation orientation, e.g. bedding or banding, this can be used to continue the orientation if its orientation has already been ascertained.

3.12 Comments

A brief summary of principal geological characteristics of the unit being described should be given. This will include any adjectives required to describe distinguishing features such as colour, grain size, fabric (schistose, slatey, gneissic, etc.), major minerals, descriptions of particularly weak zones and comments regarding overall competency of the core.

4.0 GENERAL DRILLHOLE DATA

The following information is required for each drillhole. This information must accompany the geotechnical core logging data.

Borehole No.:	As specified for the project.
Date:	Date record prepared.
Borehole Location:	Brief description of physical location of borehole. May be grid coordinates, chainage and off-set or geographical description.
Logged by:	First initial and surname of field personnel.
Collar Elevation:	Reference elevation below which all down-hole depths are measured.
Datum:	The survey datum to which the reference elevation is referred.
Drillrig:	Drillrig manufacturer and model number (e.g. Longyear 44).
Inclination:	Surveyed inclination from horizontal of borehole at collar (90° for vertical drill hole).
Azimuth:	Surveyed azimuth of inclined borehole at collar (use N/A for vertical drill holes).
Bit type:	Brief description of type of diamond core bit used (e.g. "face discharge, surface set").
Flush:	Medium used to cool the drill bit and flush cuttings out of the hole (e.g. Water, Compressed Air, Drilling Mud).
Feed:	The method by which pressure/force is applied to advance the bit in the hole (e.g. Hydraulic, Screw).
Core Diameter:	Record actual size in inches or mm.
Hole length:	Total length drilled.
Groundwater:	Note the presence (or absence) of groundwater in the drillhole, and volume of flows if present.

5.0 CORE PHOTOGRAPHY

Many mining projects have adopted core photography as a standard practice. This provides a permanent record which can be easily referred to, and a visual backup to the geological and geotechnical data collected during core logging. The cost of core photography is minimal relative to the cost of diamond drilling.

All diamond drill core should be photographed in color. The core boxes should be properly labeled as to project, hole number and depth using a felt pen. The core should be photographed wet to permit identification of geologic units. Photography is best carried out with lighting by electronic flash indoors to ensure uniform and consistent exposure conditions. Do not photograph core outdoors if it can be avoided.

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GEOTECHNICAL SUMMARY LOG

TABLE 1

PROJECT NAME: _____

BOREHOLE NUMBER: 95-82 **INCLINATION:** -90 **AZIMUTH** —

NORTHING: — **EASTING:** — **ELEV:** — **DATE:** 28 MAR 96

DEPTH FROM (metres)	DEPTH TO (metres)	ROCK TYPE	T.C.R. (%)	R.Q.D. (%)	STRENGTH INDEX	WEATH/ ALT. INDEX	FRACTURE ORIENTATION AND COUNT PER RUN LENGTH																Jn	BX/GOUGE	BROKEN CORE	DESCRIPTION AND REMARKS
							70-90				50-70				30-50				0-30							
							No.	TYPE	Jr	Ja	No.	TYPE	Jr	Ja	No.	TYPE	Jr	Ja	No.	TYPE	Jr	Ja				
0	5	CASING																								
5.0	9.0	IF	3.0/3.0 100%	2.62/3.00 87%	R4	W3					4	J	3	1	1	J	3	.75					2			-WEAK CHL ALTN -SERICITE -Fe STAINING ON FRAC SURFACES.
											1	J	3	.75												
											1	J	2	.75												
											1	CN	3	.75												
9.0	11.0	FV DYKE	3.0/3.0 100%	2.66/3.0 89%	R3	W2	1	J	3	.75	5	J	3	1	1	J	3	1	3	J	3	1	3			-MOD SER ALTN -Fe STN ON FRAC SURFACES.
											3	J	3	.75					1	J	3	.75				
11.0	14.0	FV DYKE	2.83/3.0 94%	2.83/3.0 94%	R3	W1									2	J	3	.75					0.5			MOD SER ALTN
14.0	17.0	FV DYKE	3.0/3.0 100%	3.0/3.0 100%	R3	W1													1	J	3	1	0.5			-MOD SER ALTN -Fe STAINING ON FRAC SURF
17.0	20.0	FV DYKE + I.F.	2.88/3.0 94%	2.79/3.0 93%	FVD R3 IF R4	W1	1	J	3	.75	2	J	3	.75	1	V	3	.75	1	V	3	.75	3			MOD SER ALTN. FV/IF CONT @ 18.16m @ 35° TO C.A.
											1	V	3	.75	1	J	2	.75								
20.0	23.0	IF	3.0/3.0 100%	3.0/3.0 100%	R4	W1	1	J	3	.75	1	J	1.5	.75	1	J	2	.75					4			WK. CHL ALTN; SER. -CARB VEINS -CONJUGATE FRACTURE.
											1	V	1	.75												
23.0	26.0	I.F.	2.82/3.0 94%	2.79/3.0 93%	R4	W1																	0.5			WK CHL ALTN; SER.

PROJECT NUMBER: 617e

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LOGGED BY: CJR **SHEET** 1 **OF** 1

TABLE 2

RECORD OF ROCK CORE DRILLING AND TESTING – BOREHOLE NO. 93-22

Project No. 6170 Name: LUCKY STRIKE Date: 20 MARCH 93 Sheet 2 of 8

Borehole Location N3100 E 4265 Logged By

Elevation 1393 Ref. Point Datum Drillrig LONGYEAR 38

Inclination 90 Azimuth N/A. Bit Type FACE DISCHARGE Flush H_2O Feed Hydraulic

Core Dia.: Depth	WATER NOTES	ROCK TYPE: DEPTH / ELEV	ROCK TYPE: DESCRIPTION	DEPTH SCALE 1:20	CORE RECOVERY DATA: Run No. (from – to)	T.C.R. (Length – %)	R.Q.D. (Length – %)	Fractures Per 300mm	DISCONTINUITY DATA: DIP DIRECTION	ANGLE w.r.t. CORE AXIS	SYMBOLIC LOG	TYPE AND SURFACE DESCRIPTION	STRENGTH DATA: Weathering Index	Strength Index	Point Load Index	ADDITIONAL TESTING
NQ		120.1			Run #37 continued	100%	63%	2	0°	60°		120,23 FO, PL, SM	W1	R4		
									0°	58°		120,25 FO, PL, SM				
									0°	60°		120,58 FO, PL, SM				
									0°	58°		120,44 FO, PL, SM		R3		
		120,5						3	0°	60°		120,50 FO, PL, SM				
									0°	60°		120,56 FO, PL, SM				
									0°	60°		120,72 FO, PL, SM		R4		
									0°	60°		120,80 FO, PL, SM				
									0°	60°		120,85 FO, PL, SM				
									0°	60°		120,93 FO, PL, SM				
									0°	56°		121,07 FO, PL, SM				
		121,0		121,0		2.66	2.55	2	8°	50°		121,25 FO, PL, SM-R				
						3.1	3.1									
				121,5				3	50°	58°		121,46 J, PL, SM-R				
						86%	81%		0°	65°		121,60 FO, PL, SM				
									310°	46°		R1,60 J, ST, VR				
								1	210	40°		121,64 J, ST, VR				
				122,0												
								1								
		122,2	Fresh, very strong quartz - sericite - alteration phase intermixed with pyrite						0°	49°		122,18 FO, PL, SM-R				
				122,5				0								
								B								
				123,0												
								0								
				123,5				0								
				124,0												
						3.45	0.90									
						3.0	30					Lost Core				Lost Core
		124,4	Fresh, very strong sulphide rock / alteration phase rock	124,5	Run #39 124,1 – 127,1	8%	30%						W3	R5	124,1	
												124,65 0.475 gouge				
										40°		124,913, PL, K	W1	R5		
				125,0												
									0°	62°						
									0°	50°						
				125,5												
									0°			125,21 FO, PL, SM				
									130°			125,66 J, PL, R				

ABBREVIATIONS:

T.C.R. – Total Core Recovery

R.Q.D. – Rock Quality Designation

J – Joint
F – Fault
S – Shear
B – Bedding
FO – Foliation

PL – Planar
C – Curved
U – Undulating
ST – Stepped

P – Polished
K – Slickensided
SM – Smooth
R – Rough

TABLE 3
RATING GUIDE FOR FRACTURE CONDITION

1) Barton's Q System:

PARAMETER	ITEM AND DESCRIPTION	VALUE
Number of Fracture Sets (Jn)	Massive One set One set plus random Two sets Two sets plus random Three sets Three sets plus random Four or more sets Crushed rock	0.5 2.0 3.0 4.0 6.0 9.0 12.0 15.0 20.0
Fracture Roughness (Jr)	Rough and wavy Smooth and wavy Rough and planar Smooth and planar, or filled Slickensided and planar	3.0 2.0 1.5 1.0 0.5
Fracture Alteration (Ja)	**Unfilled Fractures:** Healed fractures Staining only, no alteration Slightly altered fracture walls Silty or sandy coatings Clay coatings **Filled Fractures:** Sand or crushed rock filling Stiff clay filling less than 5 mm thick Soft clay filling less than 5 mm thick Swelling clay filling less than 5 mm thick Stiff clay filling more than 5 mm thick Soft clay filling more than 5 mm thick Swelling clay filling more than 5 mm thick	 0.75 1.0 2.0 3.0 4.0 4.0 6.0 8.0 12.0 10.0 15.0 20.0

Reference: Adapted from Hoek, 1980, "Underground Excavations in Rock", Institute of Mining and Metallurgy

2) Bieniawski's RMR System:

Fracture Condition	Value
Very rough surfaces, fractures not continuous, no separation, unweathered.	30
Slightly rough surfaces, separation < 1 mm, slightly weathered.	25
Slightly rough surfaces, separation < 1 mm, highly weathered.	20
Slickensided surfaces OR Gouge < 5 mm thick OR Separation = 1-5 mm Continuous fractures	10
Soft gouge > 5 mm thick OR Separation > 5 mm. Continuous fractures.	0

Reference: Bieniawski, 1989, "Engineering Rock Mass Classifications", John Wiley & Sons.

TABLE 4
INTACT ROCK STRENGTH CLASSIFICATION

GRADE	DESCRIPTION	FIELD IDENTIFICATION	APPROX. RANGE OF UNIAXIAL COMPRESSIVE STRENGTH (Mpa)
R0	Extremely weak rock	Indented by thumbnail.	0.25 - 1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	1.0 - 5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	5.0 - 25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer.	25 - 50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it.	50 - 100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100 - 250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	>250

NOTE: Materials having a uniaxial compressive strength of less than about 0.5 Mpa and cohesionless materials should be classified using soil classification systems. 1 Mpa = 145 psi

REFERENCE: Brown, 1981, "Suggested Methods for Rock Characterization Testing and Monitoring", International Society for Rock Mechanics.

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TABLE 5
WEATHERING CLASSIFICATION

Term	Symbol	Description	Discoloration Extent	Fracture Condition	Surface Characteristics
Fresh	W1	No visible sign of rock material weatherings.	None	Closed or Discolored	Unchanged
Slightly Weathered	W2	Discoloration indicates weathering of rock material on discontinuity surfaces. Less than 5% of rock mass altered.	<20% of fracture spacing on both sides of fracture	Discolored, may contain thin filling	Partial discoloration
Moderately Weathered	W3	Less than 50% of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.	>20% of fracture spacing on both sides of fracture	Discolored, may contain thick filling	Partial to complete discoloration, not friable except poorly cemented rocks
Highly Weathered	W4	More than 50% of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.	Throughout	Filled with alteration minerals	Friable and possibly pitted
Completely Weathered	W5	100% of rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	Throughout	Filled with alteration minerals	Resembles soil
Residual Soil	W6	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.	Throughout	N/A	Resembles soil

Reference: Brown, 1981, "Suggested Methods for Rock Characterization Testing and Monitoring", International Society for Rock Mechanics.

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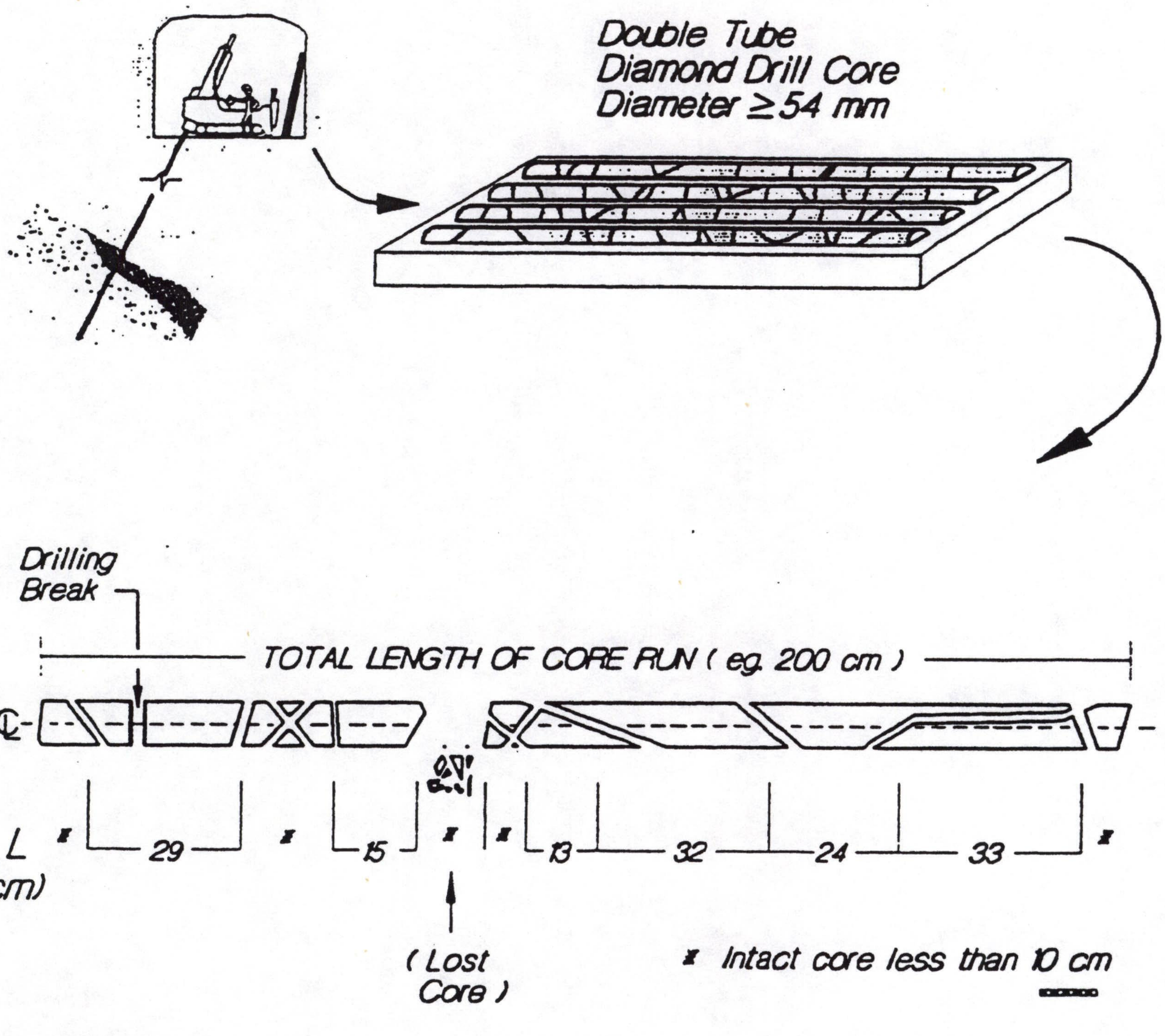

$$RQD = \frac{\sum \text{Length of core pieces} > 10\ cm}{\text{Total Length of Core Run}} \times 100 \qquad (\%)$$

$$eg: \quad \frac{29 + 15 + 13 + 32 + 24 + 33}{200} \times 100 = 73\ \%$$

SCHEMATIC ONLY
Not to Scale

REFERENCE: *Hutchinson and Diederichs, 1995*

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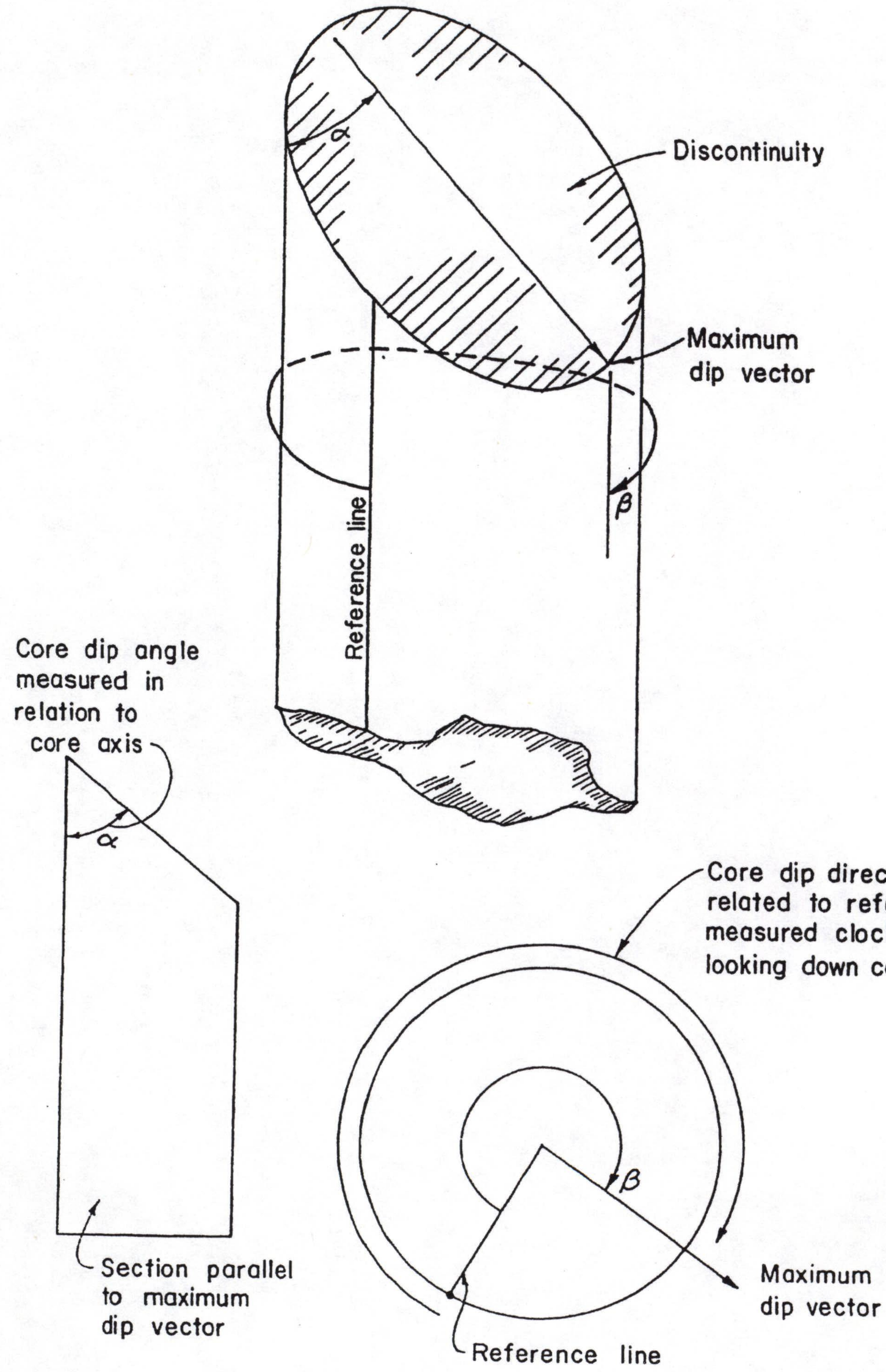
ROCK QUALITY DESIGNATION

Figure **1**

Project No. 962-1448 Drawn ~ Reviewed C.C. Date Apr '96



SCHEMATIC ONLY
Not to Scale

Project No. 962-1448 Drawn ~ Reviewed C.C. Date Apr '96

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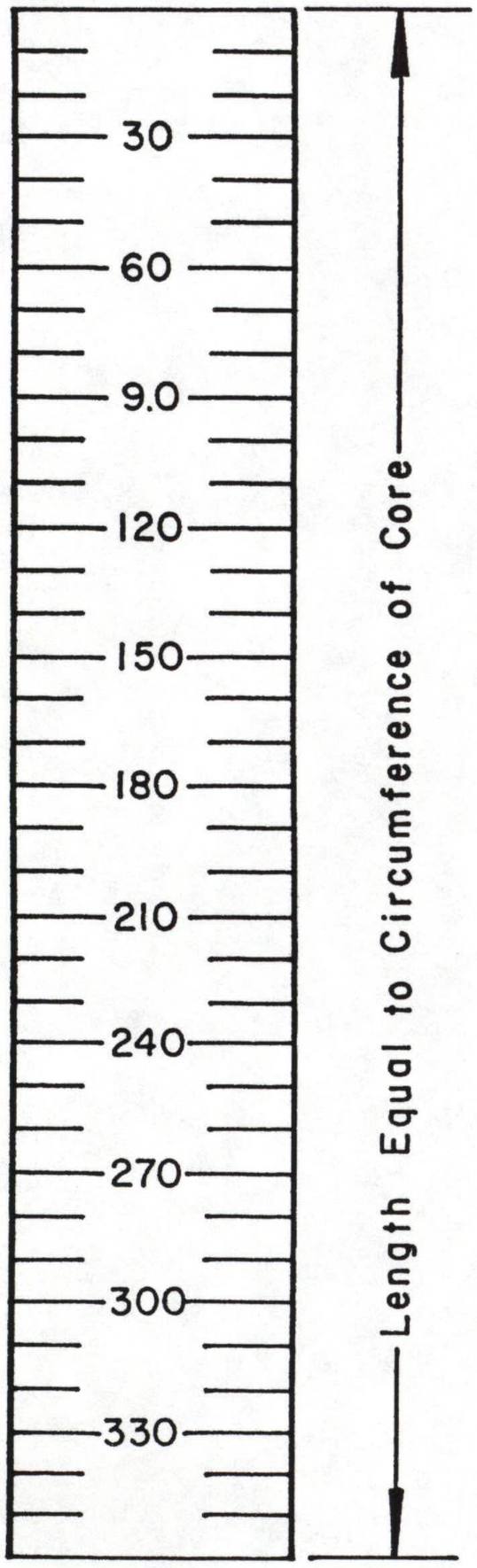
CORE DIP AND DIP DIRECTION ANGLE

Figure 2



Project No. 962-1448 Drawn ~ Reviewed C.C. Date Apr '96

SCHEMATIC ONLY
Not to Scale

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LINEAR PROTRACTOR

Figure 3

APPENDIX A

SAMPLE CORE LOGS

Q SYSTEM - RECORD OF ROCK CORE DRILLING AND TESTING - BOREHOLE NO. _____

Project No. _____ Project Name _____ Date _____ Sheet ____ of ____

Borehole Location _____ Logged by _____

Elevation _____ Ref. Point _____ Datum _____ Drill Rig _____

Inclination _____ Azimuth _____ Bit Type/Size _____ Flush _____ Feed _____

Core Dia: Depth	ROCK TYPE	CORE RECOVERY DATA						STRENGTH DATA		FRACTURE ORIENTATION AND COUNT PER RUN LENGTH														Jn			ADDITIONAL TESTING OR COMMENTS
										70 - 90				50 - 69				30 - 49				0 - 29						
		RUN	From	To	TCR	RQD	Fractures per	Strength	Weath.	Count	TYPE	Jr	Ja	Count	TYPE	Jr	Ja	Count	TYPE	Jr	Ja	Count	TYPE	Jr	Ja		Fault Breccia/Gouge	Broken Core	

LOGGED BY: _____ PROJECT NAME: _____ HOLE NO: _____

PROJECT NO. _____ PAGE: ____ OF ____

RMR SYSTEM - RECORD OF ROCK CORE DRILLING AND TESTING - BOREHOLE NO. _____

Project No. _____ Project Name _____ Date _____ Sheet ____ of ____

Borehole Location _____ Logged by _____

Elevation _____ Ref. Point _____ Datum _____ Drill Rig _____

Inclination _____ Azimuth _____ Bit Type/Size _____ Flush _____ Feed _____

Core Dia: Depth	ROCK TYPE	RUN	CORE RECOVERY DATA					STRENGTH DATA		FRACTURE ORIENTATION AND COUNT PER RUN LENGTH																						ADDITIONAL TESTING OR COMMMENTS
										70 - 90					50 - 69					30 - 49					0 - 29							
			From	To	TCR	RQD	Fractures per	Strength	Weath.	Count	TYPE	Shape	Rough.	Cond.	Count	TYPE	Shape	Rough.	Cond.	Count	TYPE	Shape	Rough.	Cond.	Count	TYPE	Shape	Rough.	Cond.	Fault Breccia/Gouge	Broken Core	

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PROJECT NO.

HOLE NO:

PAGE: OF

APPENDIX B

SAMPLE POINT LOAD TEST RECORDS

POINT LOAD TEST RECORD

PROJECT NAME: _____ **PAGE** _____ **OF** _____

DATE: _____

Project No. _____

BOREHOLE NO. _____ **RAM AREA: 12.6 X 10(-4) m^2**

DEPTH	TYPE	PARALLEL/PERP.	W (mm)	D (mm)	PRESSURE (kPa/PSi)	ROCK TYPE/COMMENTS

PRELIMINARY POINT LOAD TEST RESULTS

DRILLHOLE:

DEPTH (m)	TEST	D (mm)	De^2 (mm^2)	De (mm)	P (kN)	Is (MPa)	F	Is(50) (MPa)	UCS (MPa)	ISRM CLASS.	FIELD STRENGTH ESTIMATE	ROCK TYPE
5.40	D	40	1600	40	4.9	3.1	0.9	2.7	58	R4	R3	Epiclastics
6.10	D	48	2304	48	22.3	9.7	1.0	9.5	212	R5	R3	Epiclastics
8.40	D	49	2401	49	2.5	1.0	1.0	1.0	23	R2	R3	Epiclastics
10.20	D	48	2304	48	1.5	0.7	1.0	0.6	14	R2	R3	Epiclastics
11.70	D	50	2500	50	13.0	5.2	1.0	5.2	118	R5	R3	Epiclastics
13.10	D	50	2500	50	9.0	3.6	1.0	3.6	82	R4	R4	Epiclastics
14.70	D	51	2601	51	12.5	4.8	1.0	4.9	111	R5	R4	Epiclastics
15.40	D	51	2601	51	11.5	4.4	1.0	4.5	102	R5	R3	Epiclastics
16.75	D	50	2500	50	9.5	3.8	1.0	3.8	86	R4	R3	Epiclastics
18.50	D	45	2025	45	12.9	6.4	0.9	6.0	132	R5	R4/R5	Epiclastics
20.30	D	49	2401	49	15.2	6.3	1.0	6.3	141	R5	R4/R5	Epiclastics
21.60	D	46	2116	46	18.0	8.5	1.0	8.2	180	R5	R4/R5	Epiclastics
23.60	D	45	2025	45	15.0	7.4	0.9	7.0	154	R5	R4/R5	Epiclastics
25.65	D	44	1936	44	13.5	7.0	0.9	6.5	142	R5	R4/R5	Epiclastics
27.65	D	49	2401	49	17.5	7.3	1.0	7.2	163	R5	R4/R5	Epiclastics
30.50	D	48	2304	48	15.9	6.9	1.0	6.8	151	R5	R4/R5	Epiclastics

COMPRESSIVE STRENGTH BASED ON UCS = (14 + 0.175De)xIs(50); HOEK AND BROWN

UCS SAMPLE INTERVALS

Sa#1	11.80m - 12.00m
Sa#2	13.30m - 13.55m
Sa#3	21.70m - 22.00m
Sa#4	27.75m - 28.00m

APPENDIX C

POINT LOAD TESTING PROCEDURES

Suggested Method for Determining Point Load Strength

SCOPE

1.(a) The Point Load Strength test is intended as an index test for the strength classification of rock materials. It may also be used to predict other strength parameters with which it is correlated, for example uniaxial tensile and compressive strength.[1]*

(b) The test measures the Point Load Strength Index ($I_{s(50)}$) of rock specimens, and their Strength Anisotropy Index ($I_{a(50)}$) which is the ratio of Point Load Strengths in directions which give the greatest and least values.

(c) Rock specimens in the form of either core (the *diametral* and *axial* tests), cut blocks (the *block* test), or irregular lumps (the *irregular lump* test) are broken by application of concentrated load through a pair of spherically truncated, conical platens.[2] Little or no specimen preparation is needed.

(d) The test can be performed with portable equipment or using a laboratory testing machine, and so may be conducted either in the field or the laboratory.

APPARATUS

2. The testing machine (Fig. 1) consists of a loading system (for the portable version typically comprising a loading frame, pump, ram and platens), a system for measuring the load P required to break the specimen, and a system for measuring the distance D between the two platen contact points (but see 5(e) below).

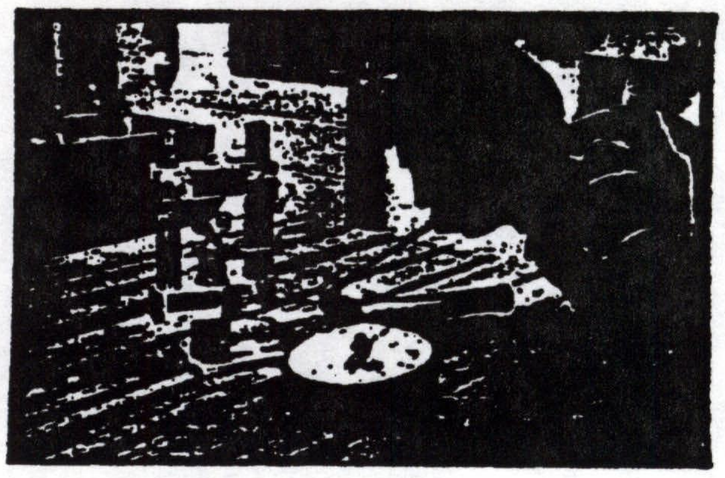

Fig. 1. Photograph of portable point load test machine.

Loading system

3.(a) The loading system should have a platen-to-platen clearance that allows testing of rock specimens in the required size range. Typically this range is 15–100 mm so that an adjustable clearance is needed to accommodate both small and large specimens.

(b) The loading capacity should be sufficient to break the largest and strongest specimens to be tested.[3]

(c) The test machine should be designed and constructed so that it does not permanently distort during repeated applications of the maximum test load, and so that the platens remain co-axial within $\pm$ 0.2 mm throughout the testing. No spherical seat or other non-rigid component is permitted in the loading system. Loading system rigidity is essential to avoid problems of slippage when specimens of irregular geometry are tested.

(d) Spherically-truncated, conical platens of the standard geometry shown in Fig. 2 are to be used. The 60° cone and 5 mm radius spherical platen tip should meet tangentially.[4] The platens should be of hard material such as tungsten carbide or hardened steel so that they remain undamaged during testing.

Load measuring system

4.(a) The load measuring system, for example a load cell or a hydraulic pressure gauge or transducer connected to the ram, should permit determination of the failure load P required to break the specimen and should conform to the requirements (b) through (d) below.

(b) Measurements of P should be to an accuracy of $\pm$ 5%P or better, irrespective of the size and strength of specimen that is tested.[3,5]

(c) The system is to be resistant to hydraulic shock and vibration so that the accuracy of readings is not adversely affected by repeated testing.

(d) Failure is often sudden and a maximum load indicating device is essential so that the failure load is retained and can be recorded after each test.

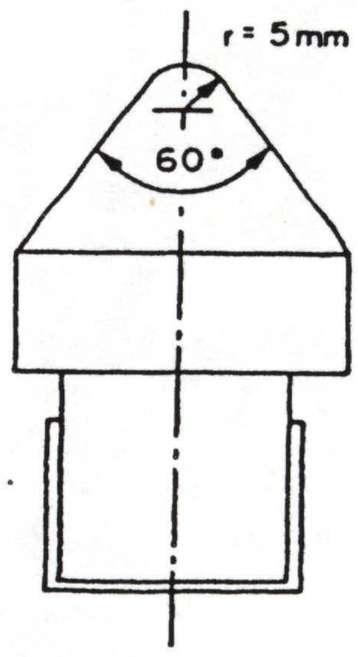

Fig. 2. Platen shape and tip radius.

* Superscript numbers refer to Notes at the end of the text.

Distance measuring system

5.(a) The distance measuring system, for example a direct reading scale or displacement transducer, is to permit measurement of the distance D between specimen-platen contact points and should conform with requirements (b) through (d) below.[6]

(b) Measurements of D should be to an accuracy of $\pm 2\%D$ or better irrespective of the size of specimen tested.

(c) The system is to be resistant to hydraulic shock and vibration so that the accuracy of readings is not adversely affected by repeated testing.

(d) The measuring system should allow a check of the "zero displacement" value when the two platens are in contact, and should preferably include a zero adjustment.

(e) An instrument such as calipers or a steel rule is required, to measure the width W of specimens for all but the diametral test.

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

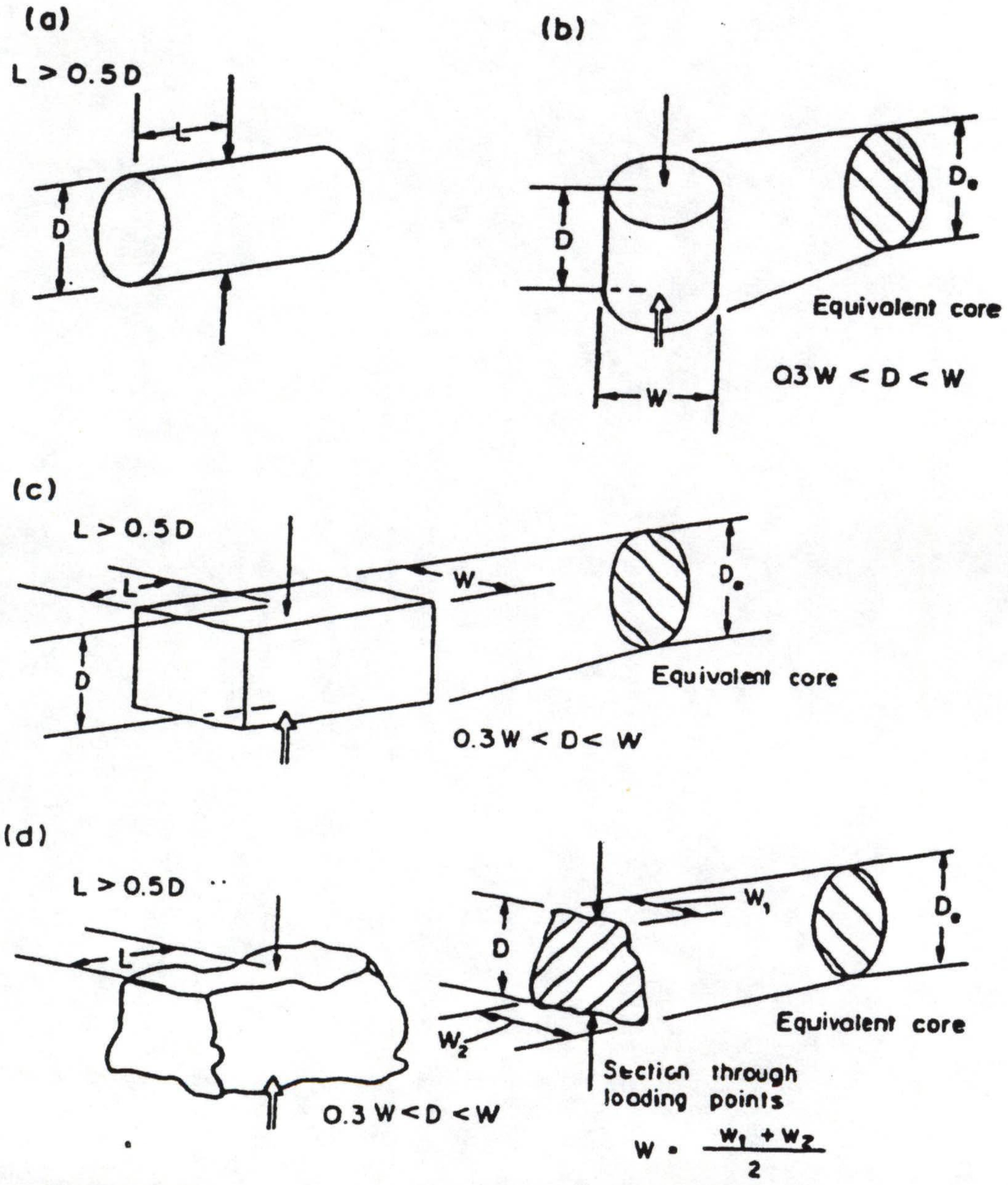

Fig. 3. Specimen shape requirements for (a) the diametral test, (b) the axial test, (c) the block test, and (d) the irregular lump test.

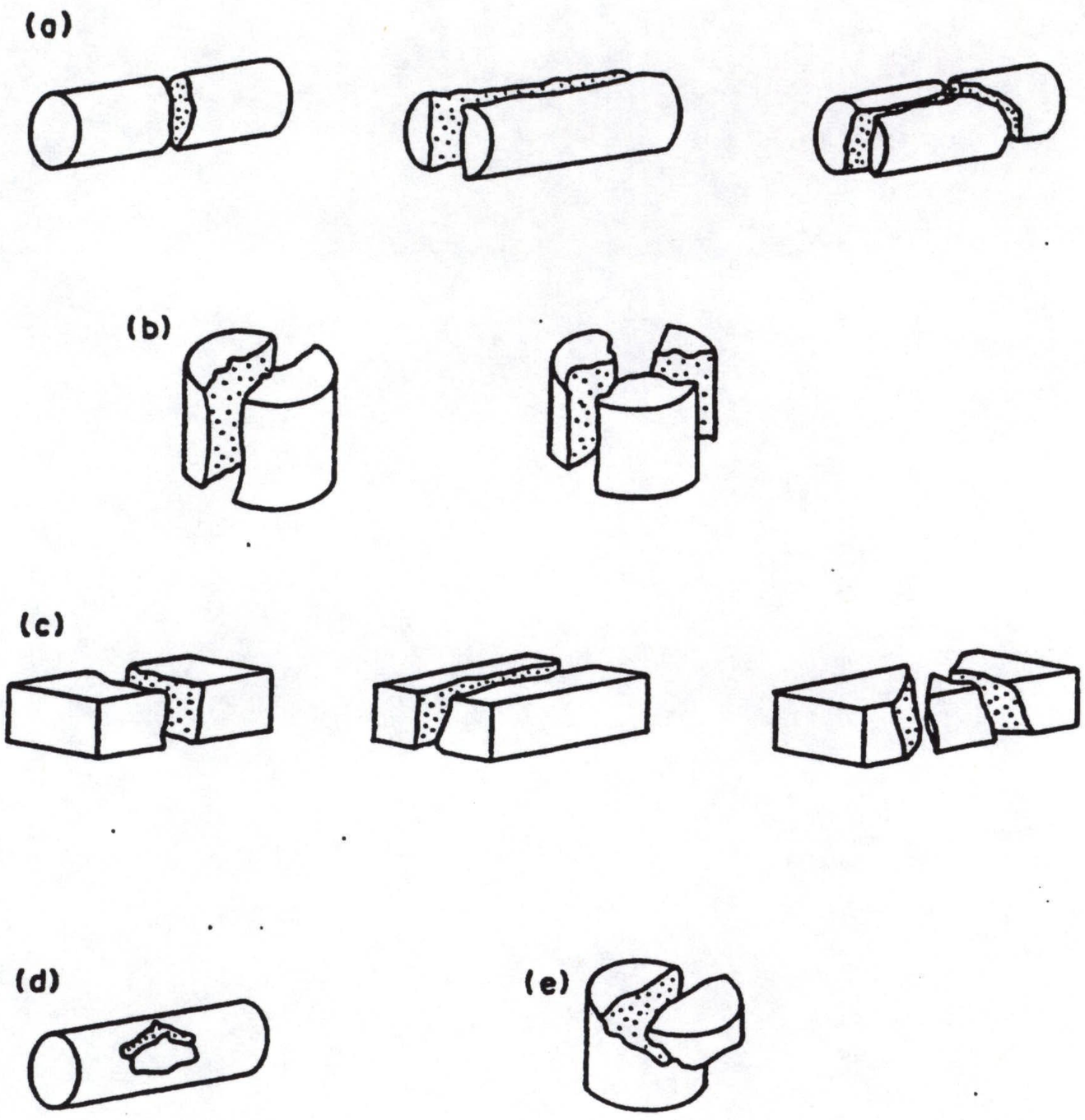

Fig. 4. Typical modes of failure for valid and invalid tests. (a) Valid diametral tests; (b) valid axial tests; (c) valid block tests; (d) invalid core test; (e) invalid axial test.

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. 4e).

(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.

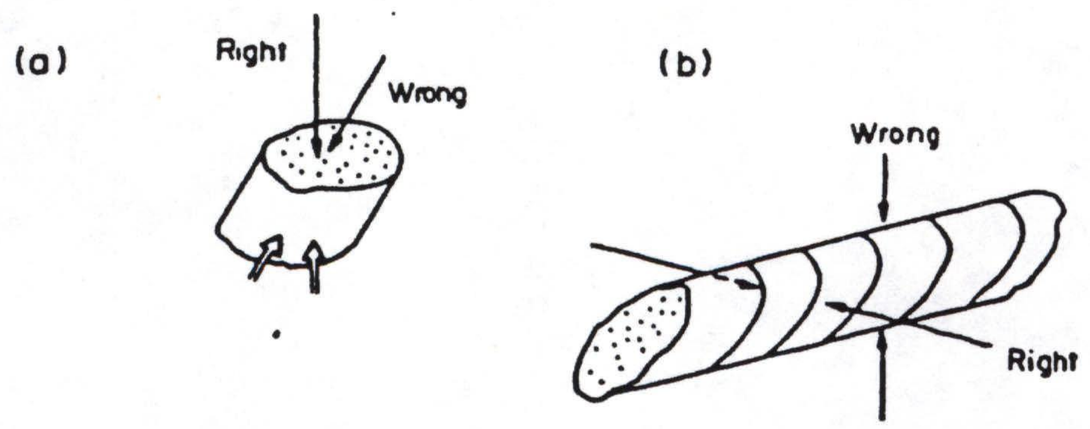

Fig. 5. Loading directions for tests on anisotropic rock.

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.[6] 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 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 $D_e^2 = 2500$ mm^2 ($D_e = 50$ mm) can then be obtained by interpolation, if

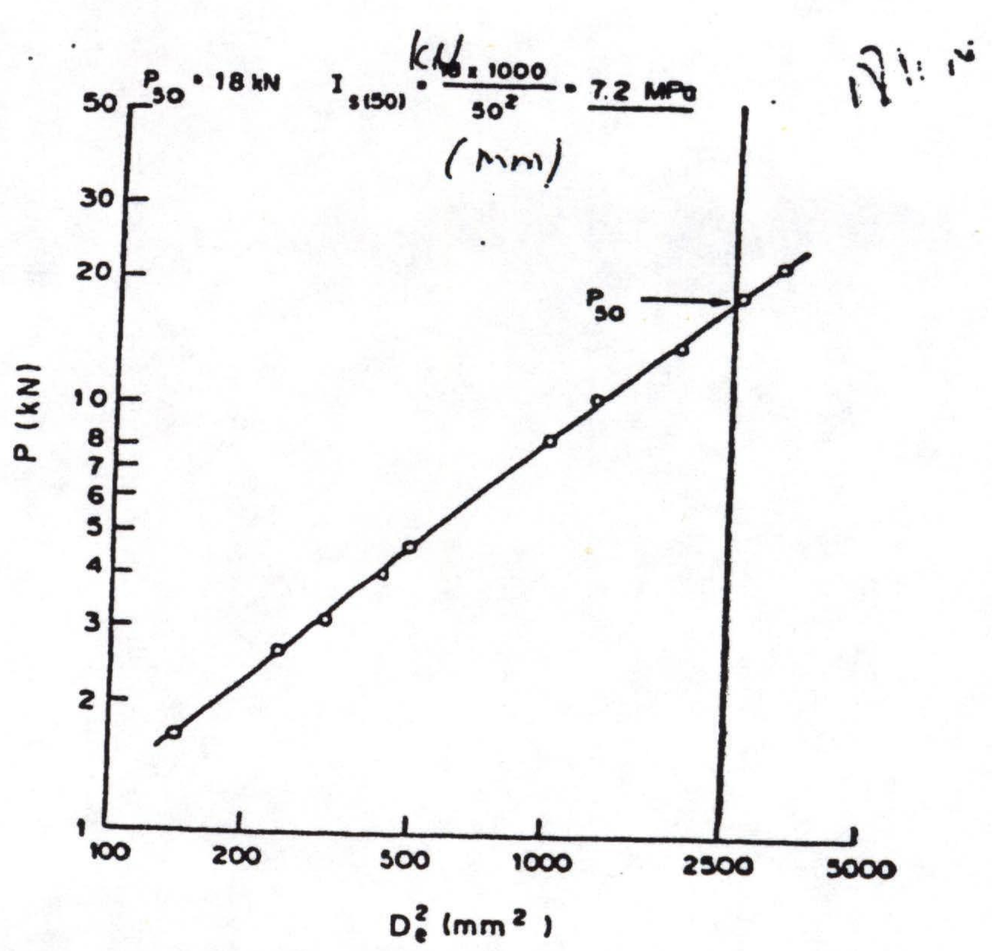

Fig. 6. Procedure for graphical determination of $I_{s(50)}$ from a set of results at D_e values other than 50 mm.

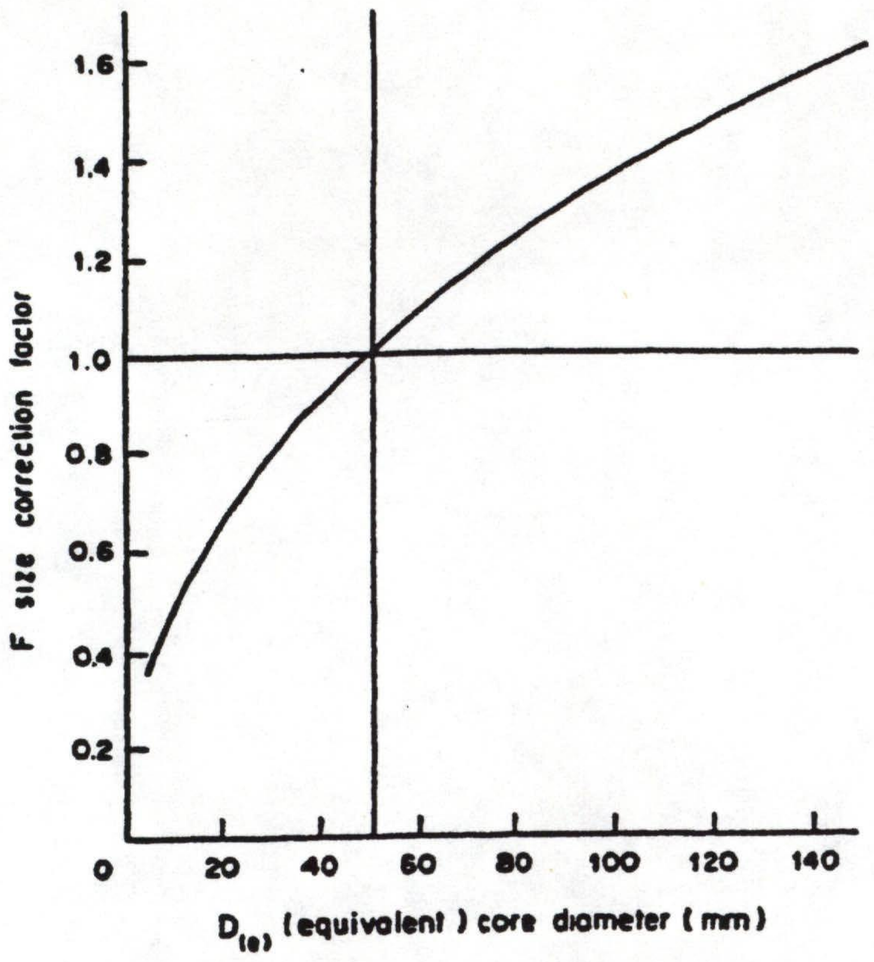

Fig. 7. Size correction factor chart.

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.

REPORTING OF RESULTS

16. Results for diametral tests, axial tests, block tests and irregular lump tests, and for tests perpendicular and parallel to planes of weakness should be tabulated separately (see typical results form, Fig. 8). The report should contain calibration data for the test machine and at least the following information for each sample tested:

(a) The sample number, source location and rock type, and the nature and *in situ* orientation of any planes of anistropy or weakness.

(b) Information on the water content of the rock at the time of testing.

(c) Information on which specimens were loaded parallel (//), perpendicular (⊥), or at unknown or random directions with respect to planes of weakness.

(d) A tabulation of the values of P, D, (W, D_e^2 and D_e if required), I_s, (F if required) and $I_{s(50)}$ for each specimen in the sample.

(e) For all isotropic samples, a summary tabulation of mean $I_{s(50)}$ values.

(f) For all anisotropic samples, a summary tabulation of mean $I_{s(50)}$ values for sub-samples tested perpendicular and parallel to the planes of weakness, and of the corresponding $I_{a(50)}$ values.

NOTES

1. When first introduced, the point load strength test was used mainly to predict uniaxial compressive strength which was then the established test for general-purpose rock strength classification. Point load strength now often replaces uniaxial compressive strength in this role since when properly conducted it is as reliable and much quicker to measure. $I_{s(50)}$ should be used directly for rock classification, since correlations with uniaxial compressive strength are only approximate. 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. The point load strength test is a form of "indirect tensile" test, but this is largely irrelevant to its primary role in rock classification and strength characterization. $I_{s(50)}$ is approximately 0.80 times the uniaxial tensile or Brazilian tensile strength.

2. Of the four alternative forms of this test, the diametral test and the axial test with saw-cut faces are the most accurate if performed near the standard 50 mm size, and are preferred for strength classification when core is available. Axial test specimens with saw-cut faces can easily be obtained from large block samples by coring in the laboratory. Specimens in this form are

Sample Details

Point Load Test

Date 17/11/83

1 block sample from Gamblethorpe Opencast site.

Fine grained pale grey Coal Measures sandstone with numerous coaly streaks along horizontal bedding planes.

Specimens 1-6 chisel cut blocks, air-dried 2 weeks;
7-10 sawn blocks, air-dried 2 weeks;
11-15 cores, air-dried 2 weeks;
16-20 cores, air-dried 2 weeks;
- tested in laboratory.

No.	Type	W (mm)	D (mm)	P (kN)	D_e^2 (mm^2)	D_e (mm)	I_s	F	I_s (50)
1	i ⊥	30.4	17.2	2.687	666	25.8	4.03	0.75	~~3.03~~
2	i ⊥	16	8	0.977	163	12.8	5.99	0.54	3.24
3	i ⊥	19.7	15.6	1.962	391	19.8	5.02	0.66	3.31
4	i ⊥	35.8	18.1	3.641	825	28.7	4.41	0.765	3.46
5	i ⊥	42.5	29	6.119	1569	39.6	3.90	0.875	3.49
6	i ⊥	42	35	7.391	1872	43.3	3.95	0.935	~~3.69~~
7	b ⊥	44	21	4.600	1176	34.3	3.91	0.84	3.29
8	b ⊥	40	30	5.940	1528	39.1	3.88	0.89	3.46
9	b ⊥	19.5	15	2.040	372	19.3	5.48	0.655	~~3.59~~
10	b ⊥	33	16	2.87	672	25.9	4.27	0.75	~~3.20~~
11	d //	-	49.93	5.107	-	-	-	-	2.05
12	d //	-	49.88	4.615	-	-	-	-	1.85
13	d //	-	49.82	5.682	-	-	-	-	~~2.29~~
14	d //	-	49.82	4.139	-	-	-	-	~~1.67~~
15	d //	-	49.86	4.546	-	-	-	-	1.83
16	d //	-	25.23	1.837	-	-	2.89	0.74	2.14
17	d //	-	25.00	1.891	-	-	3.02	0.735	2.22
18	d //	-	25.07	2.118	-	-	3.37	0.735	~~2.48~~
19	d //	-	25.06	1.454	-	-	2.32	0.735	~~1.70~~
20	d //	-	25.04	1.540			2.46	0.735	1.81

d = diametral;
a = axial;
b = block;
i = irregular lump test;
⊥ = perpendicular;
// = parallel to planes of weakness.

Mean I_s (50) ⊥	3.38
Mean I_s (50) //	1.98
I_a (50)	1.71

Fig. 8. Typical results form.

particularly suitable when the rock is anisotropic and the direction of weakness planes must be noted.

3. Loads of up to 50 kN are commonly required for the larger hard rock specimens. The maximum specimen size that can be tested by a given machine is determined by the machine's load capacity, and the smallest by the machine's load and distance measuring sensitivity. Tests on specimens smaller than $D = 25$ mm require particular precautions to ensure that the measuring sensitivity is sufficient. The range of required test loads should be estimated before testing, from approximate assumed strength values, to ensure that the load capacity and sensitivity of the equipment are adequate. It may be necessary to change the load measuring gauge or load cell, or to test smaller or larger specimens to conform with the capacity of available equipment or with the accuracy specifications for this test.

4. The conical platen design is intended to give standardized penetration of softer specimens. When testing is confined to hard rocks and small (less than 2 mm) penetrations the conical design is unimportant provided that the tip radius remains at the standard 5 mm. For such testing the platen can be manufactured by embedding a hard steel or tungsten carbide ball in a

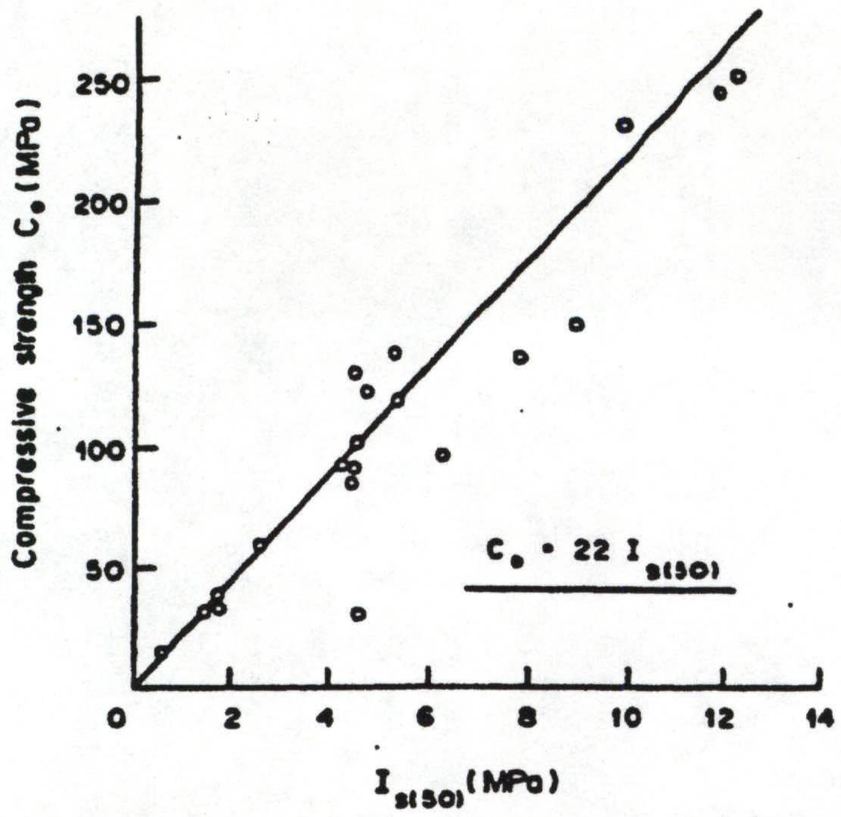

Fig. 9. Example of correlation between point load and uniaxial compressive strength results.

softer metal base of any geometry that will ensure that only the platen tip is in contact with the rock.

5. If a quick-retracting ram is used to reduce the delay between tests, either the ram return spring force and ram friction should together be less than about 5% of the smallest load to be measured during testing, or an independent load cell rather than an oil pressure gauge should be used for load determination. These forces can be significant when testing weaker and smaller specimens.

6. If significant platen penetration occurs, the dimension D to be used in calculating point load strength should be the value D' measured at the instant of failure, which will be smaller than the initial value suggested in paragraphs 8(d), 9(d) and 10(d). The error in assuming D to be its initial value is negligible when the specimen is large or strong. The failure value may always be used as an alternative to the initial value and is preferred if the equipment allows it to be measured (for example by electrical maximum-indicating load and displacement measurement). When testing specimens that are smaller than 25 mm, such as rock aggregate particles, equipment with electrical readout is usually necessary to obtain the required measuring accuracy, and should be designed to record D' at failure. Measurements of W or D made perpendicular to the line joining the platens are not affected and are retained at their original values. The value of D_e for strength calculation can then be found from:

$$D_e^2 = D \times D' \quad \text{for cores}$$

$$D_e^2 = \frac{4}{\pi}(W \times D') \quad \text{for other shapes}$$

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. It is often better to obtain strength values of limited reliability than none at all. For example, rock is often too broken or slabby to provide specimens of the ideal sizes and shapes, or may be available in limited quantities such as when the test is used to log the strength of drill core. In core logging applications, the concept of a "sample" has little meaning and tests are often conducted at an arbitrary depth interval, say one test every 1 m or 3 m depending on the apparent variability or uniformity of strength in the core and on the total length of core to be strength-logged.

8. As for all strength tests on rocks, point load strength varies with the water content of the specimens. The variations are particularly pronounced for water saturations below 25%. Oven dried specimens, for example, are usually very much stronger than moist ones. At water saturations above 50% the strength is less influenced by small changes in water content, so that tests in this water content range are recommended unless tests on dry rock are specifically required.

All specimens in a sample should be tested at a similar and well-defined water content, and one that is appropriate to the project for which the test data are required. Field testing of chisel-cut samples, not affected by drilling fluids, offers a method for testing at the *in situ* water content. If possible, numerical values should be given for both water content and degree of saturation at the time of testing. The ISRM Suggested Method for Water Content Determination should be employed. Whether or not water content measurements can be made, the sample storage conditions and delay between sampling and testing should be reported.

9. Some researchers argue in favour of measuring W as the minimum dimension of the failure surface after testing rather than of the specimen before failure (the German standard for this test is an example). Point load strengths computed using the two alternative W definitions may differ slightly. The minimum specimen dimension alternative has been adopted in this Suggested Method mainly because it is quicker and easier to measure, particularly in the field when fragments of broken specimens are easily lost.

10. Commonly the shortest dimension of naturally occurring anisotropic rock lumps is perpendicular to the weakness planes.

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.

12. Mean results for small populations are generally better measures when the extreme values are not included in the calculation.

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APPENDIX D

UCS AND ELASTIC MODULI SAMPLING AND TESTING PROCEDURES

Suggested Methods for Determining the Uniaxial Compressive Strength and Deformability of Rock Materials

PART 1. SUGGESTED METHOD FOR DETERMINATION OF THE UNIAXIAL COMPRESSIVE STRENGTH OF ROCK MATERIALS

1. SCOPE

This method of test is intended to measure the uniaxial compressive strength of a rock sample in the form of specimens of regular geometry. The test is mainly intended for strength classification and characterization of intact rock.

2. APPARATUS

(a) A suitable machine shall be used for applying and measuring axial load to the specimen. It shall be of sufficient capacity and capable of applying load at a rate conforming to the requirements set in Section 3. It shall be verified at suitable time intervals and shall comply with accepted national requirements such as prescribed in either ASTM Methods E4: Verification of Testing Machines or British Standard 1610, Grade A or Deutsche Normen DIN 51 220, DIN 51 223, Klasse 1 and DIN 51 300.

(b) A spherical seat, if any, of the testing machine, if not complying with specification 2(d) below, shall be removed or placed in a locked position, the two loading faces of the machine being parallel to each other.

(c) Steel platens in the form of discs and having a Rockwell hardness of not less than HRC58 shall be placed at the specimen ends. The diameter of the platens shall be between D and D + 2 mm where D is the diameter of the specimen. The thickness of the platens shall be at least 15 mm or D/3. Surfaces of the discs should be ground and their flatness should be better than 0.005 mm.

(d) One of the two platens shall incorporate a spherical seat. The spherical seat should be placed on the upper end of the specimen. It should be lightly lubricated with mineral oil so that it locks after the deadweight of the cross-head has been picked up. The specimen, the platens and spherical seat shall be accurately centred with respect to one another and to the loading machine. The curvature centre of the seat surface should coincide with the centre of the top end of the specimen.

3. PROCEDURE

(a) Test specimens shall be right circular cylinders having a height to diameter ratio of 2.5–3.0 and a diameter preferably of not less than NX core size, approximately 54 mm. The diameter of the specimen should be related to the size of the largest grain in the rock by the ratio of at least 10:1.

(b) The ends of the specimen shall be flat to 0.02 mm and shall not depart from perpendicularity to the axis of the specimen by more than 0.001 radian (about 3.5 min) or 0.05 mm in 50 mm.

(c) The sides of the specimen shall be smooth and free of abrupt irregularities and straight to within 0.3 mm over the full length of the specimen.

(d) The use of capping materials or end surface treatments other than machining is not permitted.

(e) The diameter of the test specimen shall be measured to the nearest 0.1 mm by averaging two diameters measured at right angles to each other at about the upper-height, the mid-height and the lower height of the specimen. The average diameter shall be used for calculating the cross-sectional area. The height of the specimen shall be determined to the nearest 1.0 mm.

(f) Samples shall be stored, for no longer than 30 days, in such a way as to preserve the natural water content, as far as possible, and tested in that condition.* This moisture condition shall be reported in accordance with "Suggested method for determination of the water content of a rock sample", Method 1, ISRM Committee on Laboratory Tests, Document No. 2, First Revision, December 1977 (this volume, p. 83).

(g) Load on the specimen shall be applied continuously at a constant stress rate such that failure will occur within 5–10 min of loading, alternatively the stress rate shall be within the limits of 0.5–1.0 MPa/s.

* It is recognized that in some cases for some materials it may be desired to test specimens in other moisture conditions, for example, saturated or oven dry at 105°C. Such conditions shall be

recorded in newtons (or kilonewtons and meganewtons where appropriate) to within 1%.

(j) The number of specimens tested should be determined from practical considerations but at least five are preferred.

4. CALCULATIONS

(a) The uniaxial compressive strength of the specimen shall be calculated by dividing the maximum load carried by the specimen during the test, by the original cross-sectional area.

5. REPORTING OF RESULTS

(a) Lithologic description of the rock.

(b) Orientation of the axis of loading with respect to specimen anisotropy, e.g. bedding planes, foliation, etc.

(c) Source of sample, including: geographic location, depth and orientations, dates and method of sampling and storage history and environment.

(d) Number of specimens tested.

(e) Specimen diameter and height.

(f) Water content and degree of saturation at time of test.

(g) Test duration and stress rate.

(h) Date of testing and type of testing machine.

(i) Mode of failure, e.g. shear, axial cleavage, etc.

(j) Any other observations or available physical data such as specific gravity, porosity and permeability citing the method of determination for each.

(k) Uniaxial compressive strength for each specimen in the sample, expressed to three significant figures, together with the average result for the sample. The pascal (Pa) or its multiples shall be used as the unit of stress and strength.

(l) Should it be necessary in some instances to test specimens that do not comply with specifications as stated above these facts shall be noted in the test report.

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PART 2. SUGGESTED METHOD FOR DETERMINING DEFORMABILITY OF ROCK MATERIALS IN UNIAXIAL COMPRESSION

1. SCOPE

This method of test is intended to determine stress-strain curves and Young's modulus and Poisson's ratio in uniaxial compression of a rock specimen of regular geometry. The test is mainly intended for classification and characterization of intact rock.

2. APPARATUS

(a) to (d)—See Part 1.

(e) Electrical resistance strain gauges, linear variable differential transformers, compressometers, optical devices or other suitable measuring devices. Their design shall be such that the average of two circumferential and two axial strain measurements, equally spaced, can be determined for each increment of load. The devices should be robust and stable, with strain sensitivity of the order of 5×10^{-6}.

Both axial and circumferential strains shall be determined within an accuracy of 2% of the reading and a precision of 0.2 percent of full scale.

If electrical resistance strain gauges are used, the length of the gauges over which axial and circumferential strains are determined shall be at least ten grain diameters in magnitude and the gauges should not encroach within D/2 of the specimen ends, where D is the diameter of the specimen.

If dial micrometers of LVDT's are used for measuring axial deformation due to loading, these devices should be graduated to read in 0.002 mm units and accurate within 0.002 mm in any 0.02 mm range and within 0.005 mm in any 0.25 mm range. The dial micrometer or LVDT's should not encroach within D/2 of the specimen ends.

(f) An apparatus for recording the loads and deformations; preferably an X–Y recorder capable of direct plotting of load-deformation curves.

3. PROCEDURE

(a) to (e)—See Part 1.

(f) Moisture can have a significant effect on the deformability of the test specimen. When possible, *in situ* moisture conditions should be preserved until the time of the test. When the characteristic of the rock material under conditions varying from saturation to dry is required, proper note shall be made of moisture conditions so that correlation between deformability and moisture content can be made. Excess moisture can create a problem of adhesion of strain gauges which may require making a change in moisture content of the sample. The moisture condition shall be reported

in accordance with "Suggested method for determination of the water content of a rock sample", Method 1, ISRM Committee on Laboratory Tests, Document No. 2, December 1977 (this volume, p. 83).

(g) Load on the specimen shall be applied continuously at a constant stress rate such that failure will occur within 5–10 min of loading, alternatively the stress rate shall be within the limits of 0.5–1.0 MPa/s.

(h) Load and axial and circumferential strains or deformations shall be recorded at evenly spaced load intervals during the test, if not continually recorded. At least ten readings should be taken over the load range to define the axial and diametric stress-strain curves.

(i) It is sometimes advisable for a few cycles of loading and unloading to be performed.

(j) The number of specimens instrumented and tested under a specified set of conditions shall be governed by practical considerations but at least five are preferred.

4. CALCULATIONS

(a) Axial strain, ϵ_a, and diametric strain, ϵ_d, may be recorded directly from strain indicating equipment or may be calculated from deformation readings depending upon the type of instrumentation such as discussed in paragraph 2(e).

(b) Axial strain is calculated from the equation

$$\epsilon_a = \frac{\Delta l}{l_0}$$

where

l_0 = original measured axial length

Δl = change in measured axial length (defined to be positive for a decrease in length)

(c) Diametric strain may be determined either by measuring the changes in specimen diameter or by measuring the circumferential strain. In the case of measuring the changes in diameter, the diametric strain is calculated from the equation

$$\epsilon_d = \frac{\Delta d}{d_0}$$

where

d_0 = original undeformed diameter of the specimen

Δd = change in diameter (defined to be negative for an increase in diameter)

In the case of measuring the circumferential strain ϵ_d, the circumference is $C = \pi d$, thus the change in circumference is $\Delta C = \pi \Delta d$. Consequently, the circumferential strain, ϵ_c, is related to diametric strain, ϵ_d, by

$$\epsilon_c = \frac{\Delta C}{C_0} = \frac{\Delta d}{d_0},$$

so that

$$\epsilon_c = \epsilon_d$$

where C_0 and d_0 are original specimen circumference and diameter, respectively.

(d) The compressive stress in the test specimen, σ, is calculated by dividing the compressive load P on the specimen by the initial cross-sectional area, A_0. Thus

$$\sigma = \frac{P}{A_0}$$

where in this test procedure, compressive stresses and strains are considered positive.

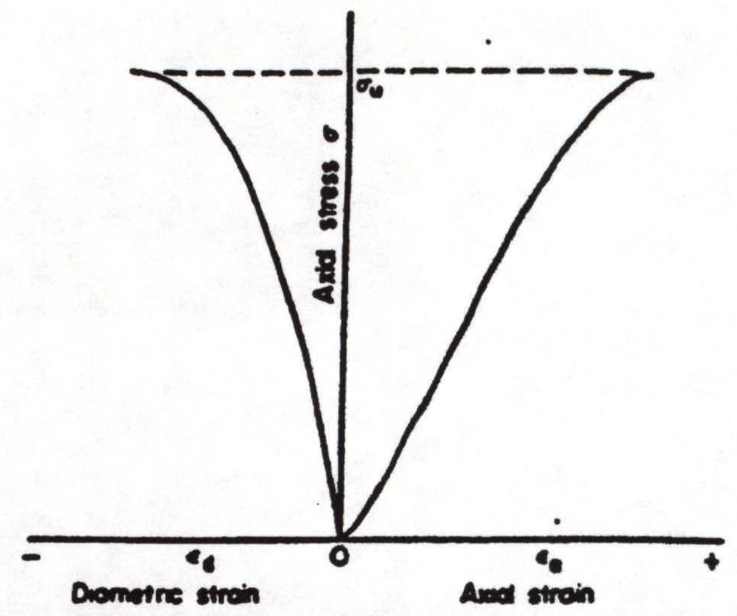

Fig. 1. Format for graphical presentation of axial and diametric stress-strain curves.

(e) Fig. 1 illustrates typical plot of axial stress versus axial and diametric strains. These curves show typical behaviour of rock materials from zero stress up to ultimate strength, σ_u. The complete curves give the best description of the deformation behaviour of rocks having non-linear stress-strain behaviour at low and high stress levels.

(f) Axial Young's modulus, E (defined as the ratio of the axial stress change to axial strain produced by the stress change) of the specimen may be calculated using any one of several methods employed in accepted engineering practice. The most common methods, listed in Fig. 2, are as follows:

(1) Tangent Young's modulus, E_t, is measured at a stress level which is some fixed percentage of the ulti-

Fig. 2. Methods for calculating Young's modulus from axial stress-strain curve.

mate strength (Fig. 2a). It is generally taken at a stress level equal to 50% of the ultimate uniaxial compressive strength.

(2) Average Young's modulus, E_{av}, is determined from the average slopes of the more-or-less straight line portion of the axial stress-axial strain curve (Fig. 2b).

(3) Secant Young's modulus, E_s, is usually measured from zero stress to some fixed percentage of the ultimate strength (Fig. 2c), generally at 50%.

Axial Young's modulus E is expressed in units of stress i.e. pascal (Pa) but the most appropriate multiple is the gigapascal (GPa = 10^9 Pa).

(g) Poisson's ratio, ν, may be calculated from the equation

$$\nu = -\frac{\text{slope of axial stress–strain curve}}{\text{slope of diametric stress–strain curve}}$$

$$= -\frac{E}{\text{slope of diametric curve}}.$$

where the slope of the diametric curve is calculated in the same manner for either of the three ways discussed for Young's modulus in paragraph 4(f). Note that Poisson's ratio in this equation has a positive value, since the slope of the diametric curve is negative by the conventions used in this procedure.

(h) The volumetric strain, ϵ_v, for a given stress level is calculated from the equation

$$\epsilon_v = \epsilon_a + 2\epsilon_d.$$

5. REPORTING OF RESULTS

The report should include the following:

(a) to (j)—See Part 1.

(k) Values of applied load, stress and strain as tabulated results or as recorded on a chart.

(l) Young's modulus and Poisson's ratio for each specimen in the sample, expressed to three significant figures, together with the average result for the sample.

(m) Method of determination of Young's modulus and at what axial stress level or levels determined.

(n) Sould it be necessary in some instances to test specimens that do not comply with the above specifications, these facts shall be noted in the test report.

REFERENCE

Standard method of test for elastic moduli of rock core specimens in uniaxial compression. *American Society for Testing and Materials*, ASTM Designation D 3148-72.

Designation: D 2938 – 86[ε1]

Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens[1]

This standard is issued under the fixed designation D 2938; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

[ε1] NOTE—Section 9 was added editorially in December 1991.

1. Scope

1.1 This test method covers the determination of the unconfined compressive strength of intact cylindrical rock specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]

E 4 Practices for Load Verification of Testing Machines[3]

E 122 Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device,* to apply and measure axial load on the specimens, of sufficient capacity to apply load at a rate conforming to the requirements set forth in 5.3. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

3.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than HRC 58 (Note 1). One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.013 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide with the bearing face of the specimen. The movabl portion of the bearing block shall be held closely in th spherical seat, but the design shall be such that the bearin face can be rotated and tilted through small angles in an direction.

NOTE 1—False platens, with plane bearing faces conforming to th requirements of this method may be used. These shall consist of disk about ½ to ¾ in. (12.7 to 19.05 mm) thick, oil-hardened to more tha HRC 58, and surface ground. With abrasive rocks, these platens tend t roughen after a number of specimens have been tested, and hence nee to be resurfaced from time to time.

4. Test Specimens

4.1 Prepare test specimens in accordance with Practic D 4543.

4.2 The moisture condition of the specimen at time of tes can have a significant effect upon the indicated strength o the rock. Good practice generally dictates that laborator tests be made upon specimens representative of field condi tions. Thus it follows that the field moisture condition of th specimen should be preserved until time of test. On the othe hand, there may be reasons for testing specimens at othe moisture contents including zero. In any case, tailor th moisture content of the test specimen to the problem at han and report it in accordance with 7.1.7.

5. Procedure

5.1 Check the ability of the spherical seat to rotate freel in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lowe bearing blocks and of the test specimen and place the tes specimen on the lower bearing block. As the load is graduall brought to bear on the specimen, adjust the movable portio of the spherically seated block so that uniform seating i obtained.

5.3 Many rock types fail in a violent manner when loade to failure in compression. A protective shield should b placed around the test specimen to prevent injury fro flying rock fragments.

5.4 Apply the load continuously and without shock t produce an approximately constant rate of load or deforma tion such that failure will occur within 5 to 15 min o loading.

NOTE 2—Results of tests by several investigators have shown tha strain rates within this range will provide strength values that ar reasonably free of rapid loading effects and reproducible within accept able tolerances.

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[1] This method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

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[2] *Annual Book of ASTM Standards,* Vol 04.08.

[3] *Annual Book of ASTM Standards,* Vols 03.01, 04.02, and 08.03.

[4] *Annual Book of ASTM Standards,* Vol 14.02.

6. Calculation

6.1 Calculate the unconfined compressive strength of the specimen by dividing the maximum load carried by the specimen during the test by the cross-sectional area calculated and express the result to the nearest 10 psi (68.9 kPa).

7. Report

7.1 The report should include the following:

7.1.1 Source of sample including project name and location, and, if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

7.1.2 Physical description of sample including rock type; location and orientation of apparent weakness planes, bedding planes, and schisotosity; large inclusions or inhomogeneities, if any.

7.1.3 Date of sampling and testing.

7.1.4 Specimen diameter and length, conformance with dimensional requirements (see Note 4).

7.1.5 Rate of loading or deformation rate.

7.1.6 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

7.1.7 Unconfined compressive strength for each specimen as calculated average unconfined compressive strength of all specimens tested, standard deviation or coefficient of variation.

7.1.8 Type and location of failure. A sketch of the fractured sample is recommended.

7.1.9 Other available physical data.

NOTE 3—If only cores with a length-to-diameter ratio (L/D) of less than 2 are available, these may be used for the test and this fact noted in the report. However, a length-to-diameter ratio (L/D) as close to 2 as possible should be used. This apparent compressive strength may then be corrected in accordance with the following equation:

$$C = C_a/(0.88 + (0.24b/h))$$

where:

C = computed compressive strength of an equivalent L/D = 2 specimen,
C_a = measured compressive strength of the specimen tested,
b = test core diameter, and
h = test core height.

NOTE 4—The number of specimens tested may depend upon availability of specimens, but normally a minimum of ten is preferred. The number of specimens tested should be indicated. The statistical basis of relating the required number of specimens to the variability of measurements is given in Recommended Practice E 122.

8. Precision and Bias

8.1 The variability of rock and resultant inability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

9. Keywords

9.1 compression testing; compressive strength; loading tests; rock

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This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

Designation: D 3148 – 86[ε2]

Standard Test Method for Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression[1]

This standard is issued under the fixed designation D 3148; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

[ε1] NOTE—Figure 2 was corrected editorially in December 1987.
[ε2] NOTE—Section 10 was added editorially in December 1991.

INTRODUCTION

The deformation and strength properties of rock cores measured in the laboratory usually do not accurately reflect large scale *in situ* properties because the latter are strongly influenced by joints, faults, inhomogeneities, weakness planes, and other factors. Therefore, laboratory values for intact specimens must be employed with proper judgment in engineering applications.

1. Scope

1.1 This test method covers the determination of elastic moduli of intact rock core specimens in uniaxial compression. Procedure A specifies the apparatus, instrumentation, and procedures for determining the axial stress - strain curve and Young's modulus, E, of cylindrical rock specimens loaded in uniaxial compression. Method B specifies the additional apparatus, instrumentation, and procedures which are necessary also to determine the lateral stress -strain curve and Poisson's ratio, ν.

NOTE 1—Some applications require the value of Young's modulus, but not Poisson's ratio. Thus, the decision to use Procedure A or Procedure A and B shall be determined by the engineer in charge of the project.

NOTE 2—This test method does not include the procedures necessary to obtain a stress - strain curve beyond the ultimate strength.

1.2 Test methods are not normally specified for rock moduli in tension because its low tensile strength does not permit sufficient data points to be obtained to be significant. However, the basic principles given here may be applied to tension testing.

1.3 The relation between the three elastic constants,

$$G = E/2(1 + \nu)$$

where:
G = modulus of rigidity,
E = Young's modulus, and
ν = Poisson's ratio,
and most elastic design equations are based on the assumption of isotropy. The engineering applicability of these equations is therefore decreased if the rock is anisotropic. When possible, it is desirable to conduct tests in the plane of foliation, bedding, etc., and at right angles to it to determine the degree of anisotropy. It is noted that equations developed for isotropic materials may give only approximate calculated results if the difference in elastic moduli in any two directions is greater than 10 % for a given stress level.

NOTE 3—Elastic moduli measured by sonic methods may often be employed as preliminary measures of anisotropy.

1.4 The values stated in inch-pound units are to be regarded as the standard.

1.5 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*
D 2938 Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens[2]
D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]
E 4 Practices for Load Verification of Testing Machines[3]
E 122 Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device* (Procedures A and B), for applying and measuring axial load to the specimen and of sufficient capacity to apply load at a rate in accordance with the requirements prescribed in 5.4. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

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[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

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[2] *Annual Book of ASTM Standards*, Vol 04.08.
[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, and 08.03.
[4] *Annual Book of ASTM Standards*, Vol 14.02.

NOTE 4—The loading apparatus employed for this test is the same as that required for Test Method D 2938.

3.2 *Bearing Surfaces* (Procedures A and B)—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than 58 HRC. One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0006 in. (15 μm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (25 μm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide approximately with the center of the bearing face of the specimen. The movable portion of the bearing block shall fit closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction. Accomplish seating by rotating the movable bearing block while the specimen is held in contact with the fixed block.

NOTE 5—False platens with plane bearing faces conforming to the requirements of this method may be used. These shall consist of disks about 0.6 to 0.8 in. (15 to 20 mm) thick, oil-hardened preferably through the disks to more than 58 HRC and surface ground. With abrasive rocks these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

3.3 *Axial Strain Determination* (Procedure A)—The axial deformations or strains may be determined from data obtained by electrical resistance strain gages, compressometers, optical devices, or other suitable means. The design of the measuring device shall be such that the average of at least two axial strain measurements can be determined for each increment of load. Measuring positions shall be equally spaced around the circumference of the specimen close to midheight. The gage length over which the axial strains are determined shall be at least 10 grain diameters in magnitude. The axial strains shall be determined with an accuracy of 2 % of the reading and a precision of 0.2 % of full-scale.

NOTE 6—Accuracy should be within 2 % of value of readings above 250 μm/m strain and within 5 μm/m strain for readings lower than 250 μm/m strain.

3.4 *Lateral Strain Determination* (Procedure B)—The lateral deformations or strains may be measured by any of the methods mentioned in 3.3. Either circumferential or diametric deformations (or strains) may be measured. At least two lateral deformation sensors shall be used. These shall be equally spaced around the circumference of the specimen close to midheight. The average deformation (or strain) from the two sensors shall be recorded at each load increment. The use of a single transducer that wraps all the way around the specimen to measure the total change in circumference is also permitted. The gage length and the accuracy and precision of the lateral strain measurement system shall be the same as those specified in 3.3 for the axial direction.

4. Test Specimens

4.1 Test specimens shall be prepared in accordance with Practice D 4543.

4.2 The moisture condition of the sample shall be noted and reported in 8.1.9.

NOTE 7—The moisture condition of the specimen at time of test can have a significant effect upon the strength of the rock and, hence, upon the shape of the deformation curves. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus, it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand there may be reasons for testing specimens at other moisture contents from saturation to dry. In any case the moisture content of the test specimen should be tailored to the problem at hand. Excess moisture will affect the adhesion of strain gages, if used, and the accuracy of their performance.

5. Procedure (Procedures A and B)

5.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lower bearing blocks and of the test specimen and place the test specimen with the strain-measuring device attached on the lower bearing block. Carefully align the axis of the specimen with the center of thrust of the spherically seated block. Make electrical connections or adjustments to the strain- or deformation-measuring device. As the load is gradually brought to bear on the specimen, adjust the movable portion of the spherically seated block so that uniform seating is obtained.

5.3 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments.

5.4 Apply the load continuously and without shock to produce an approximately constant rate of load or deformation such that failure would occur within 5 to 15 min from initiation of loading, if carried to failure (Note 8). Record the load and the axial strain or deformation frequently at evenly spaced load intervals during Procedure A of Practice D 4543. Take at least ten readings over the load range to define the axial stress - strain curve. Also record the circumferential or diametric strains (or deformations) at the same increments of load for Procedure B of Practice D 4543. Continuous recording of data with strip chart or X-Y recorders is permitted as long as the precision and accuracy of the recording system meets the requirements in 3.3.

NOTE 8—Results of tests by several investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances.

6. Calculation (Procedure A)

6.1 The axial strain, ϵ_a, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon type of apparatus or instrumentation employed. Calculate the axial strain, ϵ_a, as follows:

$$\epsilon_a = \Delta l / l$$

where:

l = original undeformed axial gage length, in. (mm), and
Δl = change in measured axial length (negative for a decrease in length) in. (mm).

6.2 Calculate the compressive stress in the test specimen from the compressive load on the specimen and the initial

computed cross-sectional area as follows:

$$\sigma = -P/A$$

where:

σ = stress, psi (MPa),
P = load, lbf (N), and
A = area, in.^{2} (mm^2).

NOTE 9—Tensile stresses and strains are used as being positive herein. A consistent application of a compression-positive sign convention may be employed if desired.

6.3 Plot the stress versus strain curve for the axial direction (Fig. 1). The complete curve gives the best description of the deformation behavior of rocks having nonlinear stress - strain relationships at low and high stress levels.

6.4 The axial Young's modulus, E, may be calculated using any one of several methods employed in engineering practice. The most common methods, described in Fig. 2, are as follows:

6.4.1 Tangent modulus at a stress level which is some fixed percentage of the maximum strength.

6.4.2 Average slope of the more-or-less straight line portion of the stress - strain curve.

6.4.3 Secant modulus, usually from zero stress to some fixed percentage of maximum strength.

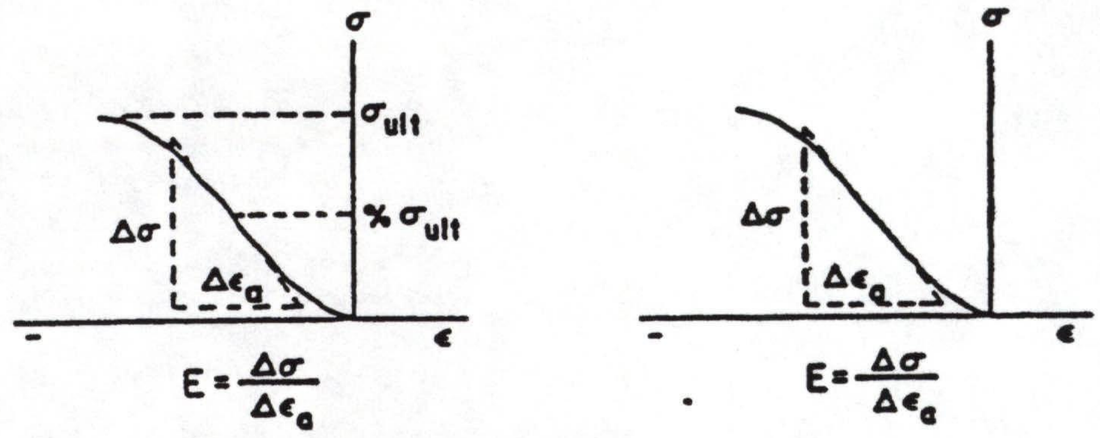

FIG. 2 Methods for Calculating Young's Modulus, E

7. Calculation (Procedure B)

7.1 The circumferential or diametric strain, ϵ_d, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon the type of apparatus or instrumentation employed.

7.1.1 Calculate the diametric strain, ϵ_d, from the following equation:

$$\epsilon_d = \Delta d/d$$

where:

d = original undeformed diameter, in. (mm), and

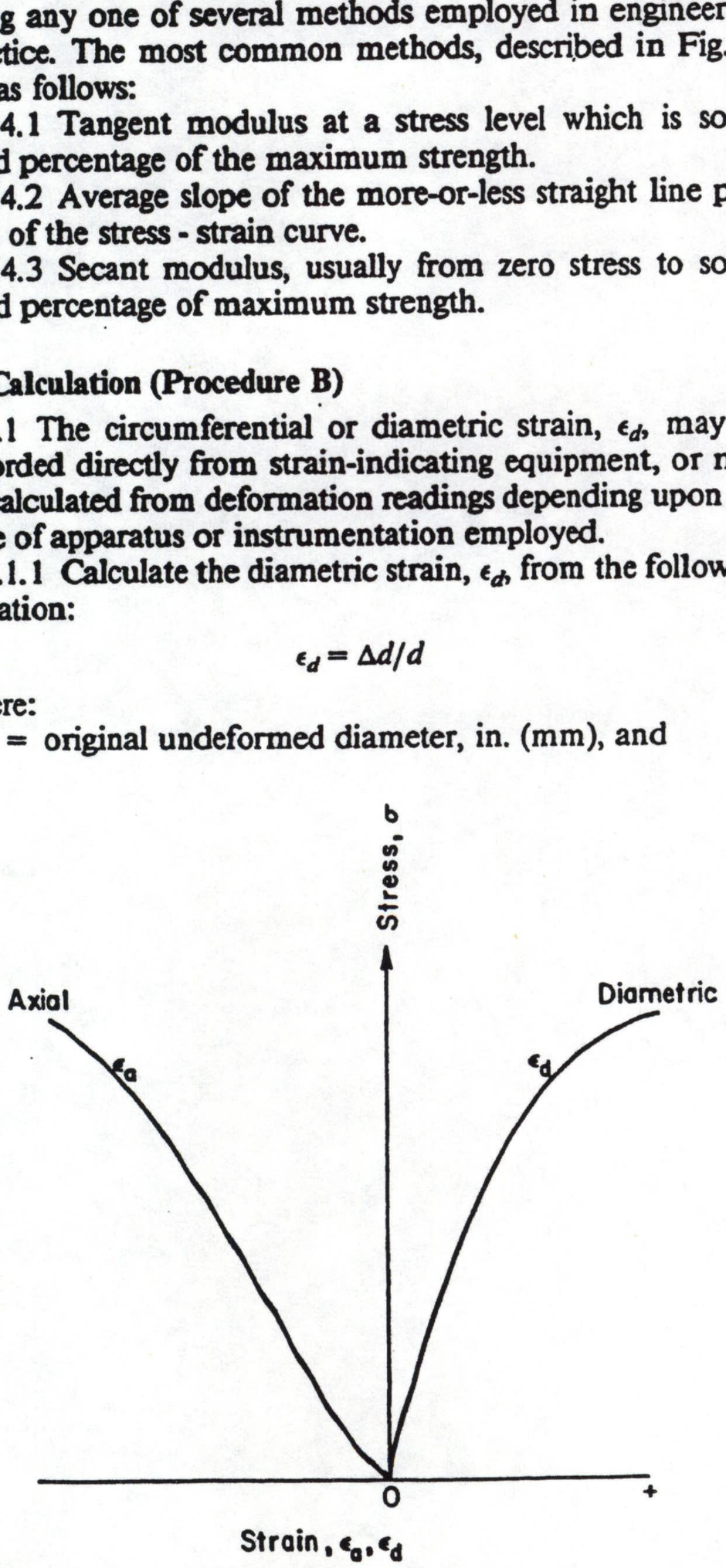

FIG. 1 Format for Graphical Presentation of Data

Δd = change in diameter (positive for an increase in diameter), in. (mm).

NOTE 10—It should be noted that the circumferentially applied electrical resistance strain gages reflect diametric strain, the value necessary in computing Poisson's ratio, ν. Since $C = \pi d$; and $\Delta C = \pi \Delta d$, the circumferential strain, ϵ_c, is related to the diametric strain, ϵ_d, through the relation:

$$\epsilon_c = \Delta C/C = \pi \Delta d/\pi d = \Delta d/d$$

so that $\epsilon_c = \epsilon_d$ where C and d are the specimen circumference and diameter, respectively.

7.2 Plot the stresses calculated in 6.2 versus the corresponding diametric strains determined in 7.1.

7.3 The value of Poisson's ratio, ν, is greatly affected by nonlinearities at low stress levels in the axial and lateral stress - strain curves. It is suggested that Poisson's ratio be calculated from the equation:

$$\nu = -\text{slope of axial curve/slope of lateral curve}$$
$$= -E/\text{slope of lateral curve}$$

where the slope of the lateral curve is determined in the same manner as was done in 6.4 for Young's modulus, E.

NOTE 11—The denominator in the equation in 7.3 will have a negative value if the sign convention is applied properly. The negative sign in the equation thereby assures a positive value for ν.

8. Report

8.1 *Procedure A*—The report shall include the following:

8.1.1 Source of sample including project name and location. Often the location is specified in terms of the drill hole number and depth of specimen from collar of hole.

8.1.2 Date test is performed.

8.1.3 Specimen diameter and height, conformance with dimensional requirements.

8.1.4 Rate of loading or deformation rate.

8.1.5 Values of applied load, stress and axial strain as tabulated results or as recorded on a chart.

8.1.6 Plot of stress versus axial strain as shown in Fig. 1, if data are tabulated in 8.1.5. If data are recorded directly on a chart, the load and deformation axes may be scaled to give stress and strain without replotting the curve.

8.1.7 Young's modulus, *E*, method of determination as given in Fig. 2, and at what stress level or levels determined.

8.1.8 Physical description of sample including rock type such as sandstone, limestone, granite, etc.; location and orientation of apparent weakness planes, bedding planes, and schistosity; and large inclusions or inhomogeneities, if any.

8.1.9 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air-dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

8.2 *Procedure B*—Procedure B of the test method must always accompany Procedure A in order to determin Poisson's ratio. The report for Procedure B, which is i addition to that for Procedure A, shall include the following

8.2.1 Values of applied load, stress, and diametric strai as tabulated results or as recorded on a chart.

8.2.2 Plot of stress versus diametric strain as shown in Fig 1 if data is tabulated in 8.2.1. If data is recorded directly on chart, the load and deformation axes may be scaled to giv stress and strain without replotting the curve.

8.2.3 Poisson's ratio, ν, method of determination in 7.3 and at what stress level or levels determined.

9. Precision and Bias

9.1 The variability of rock and resultant inability t determine a true reference value prevent development of meaningful statement of bias. Data are being evaluated t determine the precision of this test method. In addition, th subcommittee is seeking pertinent data from users of th method.

10. Keywords

10.1 compression testing; loading tests; modulus of elasticity; modulus-Young's; rock

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