Geotechnical Correlations for Soils and Rocks

Series Editor
Gilles Pijaudier-Cabot

Geotechnical Correlations for Soils and Rocks

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iSTE

WILEY

First published 2018 in Great Britain and the United States by ISTE Ltd and John Wiley & Sons, Inc.

ISTE Ltd
27-37 St George's Road
London SW19 4EU
UK

www.iste.co.uk

John Wiley & Sons, Inc.
111 River Street
Hoboken, NJ 07030
USA

www.wiley.com

Library of Congress Control Number: 2018936398

British Library Cataloguing-in-Publication Data
A CIP record for this book is available from the British Library
ISBN 978-1-78630-279-3

Contents

Acknowledgements xi

Preface xiii

Chapter 1. Physical Parameters 1

1.1. Unit weights and volumes 1
1.2. Soil behavior type index and soil classification index 3
1.3. Consistency or Atterberg limits 3
1.4. Consistency and liquidity indices 4
1.5. Rigidity index 5
1.6. Relative density of sands 5
1.7. Wave velocity 7
1.8. Cation exchange capacity 8

Chapter 2. Identification of Soil Types 11

2.1. From identification tests 11
2.2. From cone soil index $* Ic$ 12
2.3. From CPT 12
2.4. From PMT 15
2.5. From SPT 17
2.6. From DMT 18

Chapter 3. Hydraulic Parameters 21

3.1. Hydraulic conductivity 21
3.2. Water storage capacity 23
3.2.1. For a free water table 23
3.2.2. For a confined aquifer 24

Chapter 4. Strength Parameters of Saturated and Dry Soils 25

4.1. Undrained shear strength and cohesion 25
4.1.1. From identification tests 25
4.1.2. From laboratory tests 26
4.1.3. From CPT 27
4.1.4. From PMT 28
4.1.5. From SPT 29
4.1.6. From SCPT 29
4.1.7. From DMT 30
4.1.8. From VST 30
4.1.9. Overconsolidated soils 31
4.1.10. Miscellaneous: peats and remolded soils 32
4.2. Effective cohesion 33
4.3. Internal friction angle 33
4.3.1. From identification tests 33
4.3.2. From CPT and CPTu 35
4.3.3. From SCPT 36
4.3.4. From PMT 36
4.3.5. From SPT 37
4.3.6. From DMT 38
4.3.7. Peak, critical state and residual friction angles 39
4.3.8. Influence of intermediate stress 40
4.4. The angle of dilatancy 41
4.5. Sensitivity 42

Chapter 5. Soil Deformations 43

5.1. Compression and swelling 43
5.1.1. Compression index 43
5.1.2. Constants of compressibility 47
5.1.3. Swelling index 48
5.2. Soil moduli 48
5.2.1. From CPT 49
5.2.2. From DMT 51
5.2.3. From SPT 52
5.2.4. From CBR 53
5.2.5. Influence of loading rate 54
5.3. Small strain modulus 54
5.4. Poisson's ratio 58
5.5. Modulus of subgrade reaction 59
5.6. Resilient modulus 60
5.7. Collapse and expansion 60

Chapter 6. Soil State Parameters 63

6.1. Preconsolidation pressure 63
6.2. Overconsolidation ratio 66

Chapter 7. Consolidation 69

7.1. Primary consolidation coefficient 69
7.2. Secondary consolidation coefficient 70
7.3. Consolidation of peats 71
7.4. Degree of consolidation 72

Chapter 8. Coefficient of Earth Pressure at Rest 73

Chapter 9. Soil Compaction Tests 77

9.1. Proctor tests 77
9.1.1. Standard Proctor test 77
9.1.2. Modified Proctor test 78
9.2. CBR 79

Chapter 10. Unsaturated Soils 81

10.1. Suction 81
10.2. Bishop’s coefficient 83
10.3. Quasi-saturated domain 84
10.4. Stress dependency of suction 84
10.5. Drying path of quasi-saturated soils 85
10.6. Capillary or apparent cohesion 86
10.7. Estimation of porosity and degree of saturation from compression wave velocity 87

Chapter 11. Cross Relations between *In Situ* Test Parameters 89

11.1. CPT 89
11.1.1. Correction factors and correlations between different CPT tests or parameters 89
11.1.2. CPT and DPT 91
11.1.3. CPT and PMT 92
11.1.4. CPT and DMT 93
11.1.5. CPT and SPT 94
11.2. PMT 97
11.2.1. PMT and DPT 97
11.2.2. PMT and DMT 98
11.2.3. PMT and SPT 98

11.3. DMT 99
11.3.1. DMT and SPT 99
11.4. SPT 100
11.4.1. SPT and DPT 100
11.5. PANDA dynamic penetration test 100
11.5.1. PANDA and CPT 100
11.5.2. PANDA and DPT 101
11.5.3. PANDA and PMT 101
11.5.4. PANDA and VST 101

Chapter 12. Rocks 103

12.1. Introduction 103
12.2. Fundamental properties of intact minerals 106
12.3. Rock material (rm) 108
12.3.1. UCS 109
12.3.2. Abrasiveness 117
12.3.3. Attrition 119
12.3.4. Polished stone value (PSV) 122
12.4. Rock masses (RMs) 124
12.4.1. Shear strength of discontinuities 124
12.4.2. RM classification systems 126

Chapter 13. Usual Values of Soils and Rock Parameters 141

13.1. Physical parameters 141
13.1.1. Plasticity, unit weights and porosity 141
13.1.2. Consistency and related strength parameters 142
13.1.3. Soil indices 142
13.1.4. Soil and rock resistivity 143
13.1.5. Wave velocity 143
13.1.6. Clay minerals and CEC 144
13.2. Hydraulic parameters 144
13.2.1. Hydraulic conductivity 144
13.2.2. Water storage capacity 145
13.3. Strength parameters 145
13.4. Deformation parameters 145
13.4.1. Compression index 145
13.4.2. Soil modulus 146
13.4.3. Poisson's ratio 146
13.4.4. Small strain modulus 147
13.5. Consolidation parameters 147
13.5.1. Primary consolidation 147
13.6. *In situ* test parameters 148

13.6.1. CPT . . . 148
13.6.2. PMT . . . 148
13.6.3. DMT . . . 149
13.6.4. SPT . . . 149
13.7. Rock parameters . . . 149
13.7.1. Rock materials . . . 150
13.7.2. Rock masses . . . 153

List of Symbols . . . 159

List of Equations . . . 169

List of Abbreviations and Acronyms . . . 173

Bibliography . . . 177

Index . . . 205

Acknowledgements

The authors wish to express their special thanks to Jean G. Ronsmans, an old friend and a retired consulting engineer, for his detailed review of the manuscript and numerous helpful suggestions.

Many thanks also to Vincent Fiquet, CEO of OREX, and to Yves Rammer and Bernard Espion, our colleagues at the Université Libre de Bruxelles, for their comments and encouragement in this work.

The documentation received from Nicolas Charue, Director of GeoCoS, and the photographs from Pierre Van Miergroet, Chief of the In-situ Testing Department of OREX, are much appreciated and useful.

Last but not least, the interest expressed by some practitioners and former students has been a precious contribution to sustaining our motivation.

Well-directed criticism and suggestions from the reader are most welcome to help improve the next edition.

The authors of this publication have taken every possible care while preparing this book. However, they cannot guarantee that this book is complete and free of faults. The use of any data or equation from this book is entirely at the reader's own risk. It is assumed that the reader is a competent professional in the concerned domain or has acquired the assistance of an expert. The authors hereby exclude any and all liability for any and all damage which may result from the use of the equations and the data from this book.

The cover image depicts tests for the foundations of a parking lot at the junction of the rivers Sambre and Meuse at Namur. In the background is the citadel fortified by Vauban at the end of the 17th Century. Image copyright Pierre van Miergroet (OREX).

Preface

There is a vast difference between a structural engineer and a geotechnical engineer in terms of the material being used. A structural engineer defines the properties of the concerned material, such as concrete or steel, and carries out only a limited number of control tests according to the required standards. In contrast, the material used by a geotechnical engineer is natural and thus, by definition, it can be spatially heterogeneous and, in composition, it can be multiphasic, discontinuous or even anisotropic. This leaves the geotechnical engineer with no choice but to accept it as it is and to accommodate themselves to it. Moreover, while developments in numerical modeling depend increasingly on specific advanced test parameters, soil testing programs remain limited because of the competition between contractors to reduce costs and time.

Consequently, geotechnical survey programs are mainly based on commonly used tests that give basic parameters but not necessarily the required ones. To bridge this gap, designers ought to use correlations. The aim of this book is to help these designers in this critical operation; therefore, because the reader is considered a skilled geotechnician, no theoretical aspects are considered here. It must also be emphasized that correlations can never be a substitute for an adequate investigation or a field- or laboratory-testing program. This is particularly important for rocks whose characteristics depend significantly on their discontinuities. As a result, correlations between properties are few and not obvious and must be interpreted with caution. For rocks, the emphasis will be placed on "geological" aspects in order to be taken into account in the establishment of correlations. For both soils and rocks, correlations result from test programs on defined areas, either local or large, so they must be considered as site specific. It must also be emphasized that the tests on which they are based have been performed by skilled practitioners and in full compliance with the standards and the state of the art.

Given the above fact, it is well known that parameter measurements in the laboratory will differ from those obtained from field observations or *in situ* tests. Some of the common reasons for this are listed as follows:

– sampling technique;

– difference in sample orientation and anisotropy;

– effect of sample size due to some discontinuities;

– rate of testing;

– softening or decompression by the removal of load due to excavation;

– degree of saturation.

Measured or interpreted parameter value	Symbol	Coefficient of variation (V)
Unit weight	γ	3–7%
Buoyant unit weight	γ'	0–10%
Effective friction angle	φ'	2–13%
Undrained shear strength	s_u	13–40%
Undrained strength ratio	s_u/σ'_v	5–15%
Compression index	C_c	10–37%
Preconsolidation stress	σ'_p	10–35%
Hydraulic conductivity of saturated clay	k	68–90%
Hydraulic conductivity of partly saturated clay	k	130–240%
Coefficient of consolidation	c_v	33–68%
Standard penetration blow count	N	15–45%
Electric cone penetration test	q_c	5–15%
Mechanical cone penetration test	q_c	15–37%
Vane shear undrained strength	s_{uVST}	10–20%
Dilatometer tip resistance	q_0	5–15%

Table 1. *Coefficients of variation for geotechnical parameters according to [DUN 00]*

Correlations amplify some scatter that is largely inherent in experimental test results. Duncan [DUN 00] compiled a wide range of coefficients of variation (V) for geotechnical parameters proposed by different authors, which are presented in Table 1. The coefficients of variation for which sampling and testing conditions have not been specified must be considered as a rough estimation.

The reader also has to keep in mind the words of Rankine [RAN 62] in his *Manual of Civil Engineering*: "The properties of earth with respect to adhesion and friction are so variable, that the engineer should never trust to tables or to information obtained from books to guide him in designing earthworks, when he has it in his power to obtain the necessary data either by observation of existing earthworks in the same stratum or by experiment".

It should be noted that when several correlations link the same parameters, we have to be careful of their respective domains of validity. Moreover, if these are entirely the same, it means that none of them is perfect. If so, there would be only one correlation. Therefore, the best thing is to use all of them and compare their results with a critical perspective based on the engineer's experience.

This clearly shows that a combination of experience and judgment is absolutely necessary to select the appropriate design parameters deduced from the correlation out of this book.

It would be presumptuous of the authors to hope that almost all of the published correlations are presented in this book even though it took a large amount of time to complete it. However, it must be emphasized that it is only a very little part of the total time devoted to the research conducted by all those who published the correlations which are presented in this book. This book would not have been possible without all their works. Therefore, the work of the authors can be compared to gold washers who look for nuggets that are disseminated in geotechnical publications.

Jean-Claude VERBRUGGE
Christian SCHROEDER
March 2018

1

Physical Parameters

1.1. Unit weights and volumes

While performing CPTs, the unit weights of soils are generally not measured, leading to imprecision in derived parameters. Mayne [MAY 07] proposed a formula that relates the saturated unit weight to the CPT sleeve friction and specific gravity of grains (units: kN/m³ and kPa):

$$\gamma_{sat} = 2{,}6 \log f_s + 15\left({}^{\gamma_s}/_{\gamma_w}\right) - 26.5 \quad [1.1]$$

He also suggested two simple alternative expressions:

$$\gamma_{sat} = 26 - \frac{14}{1+[0.5log(f_s+1)]^2} \quad [1.2]$$

and

$$\gamma_{sat} = 12 + 1.5ln(f_s + 1) \quad [1.3]$$

Robertson *et al*. [ROB 15] proposed the following formula to estimate the total unit weight using CPT results:

$$\gamma/\gamma_w = 0.27\left[logR_f\right] + 0.36[log(q_t/p_a)] + 1.236 \quad [1.4]$$

with R_f expressed in %.

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For NC to low overconsolidated clays, Mayne and Peuchen [MAY 12] proposed the following method for a quick estimate of the total unit weight from the cone resistance–depth ratio:

$$m_q = {}^{\Delta q_t}/_{\Delta z} \approx {}^{q_t}/_{z} \tag{1.5}$$

As a rule-of-thumb estimate:

$$\gamma = \gamma_w + m_q/8 \tag{1.6}$$

or with a little more refinement:

$$\gamma = 0.636 q_t^{0.072}\left(10 + m_q/8\right) \tag{1.7}$$

In addition, for 30 < m_q < 70:

$$\gamma = \gamma_w + 0.056\left(m_q\right)^{1.21} \quad r^2 = 0.623 \tag{1.8}$$

If a seismic piezocone is used, γ_{sat} can be estimated from the correlation between shear wave and depth, as given by Mayne [MAY 07]:

$$\gamma_{sat} = 8{,}32 \log V_s - 1{,}61 log\ z \quad r^2 = 0{,}808 \tag{1.9}$$

or the mass density [MAY 99]:

$$\rho_t = 1 + \frac{1}{0.614 + 58.7(\log z + 1.095)/V_s} \tag{1.10}$$

The total unit weight can be estimated from the DMT as follows [MAY 02]:

$$\gamma = 1.12\gamma_w \left(\frac{E_D}{\sigma_{atm}}\right)^{0.1} . I_D^{-0.05} \tag{1.11}$$

From Vidalie's [VID 77] research on French muds, peats and soft clays with 30 < w_L < 180, 12 < γ < 20 (kN/m³), and all the soils being close to the A-line on the Casagrande chart, it is possible to derive a closed-form relationship between the total unit weight (kN/m³) and moisture content (%):

$$\gamma = 42.42 w^{-0.239} \quad R^2 = 0.9987 \tag{1.12}$$

1.2. Soil behavior type index and soil classification index

The soil behavior type index Ic is related to the boundaries of each SBT_n zone, which is defined from CPT results as follows:

$$I_C = [(3.47 - logQ_t)^2 + (logF_r + 1.22)^2]^{0.5} \qquad [1.13]$$

where

$$Q_t = (q_t - \sigma_{v0})/\sigma'_{v0} \qquad [1.14]$$

and

$$F_r(\%) = (f_s/(q_t - \sigma_{v0})).100 \qquad [1.15]$$

This index is widely used for correlations.

Based on CPTu results, Jefferies and Davies [JEF 93] introduced a cone soil classification index $*I_c$, which can be used for soil classification if $B_q < 1$:

$$*I_C = \left[\left\{3 - log\left[Q_t\left(1 - B_q\right)\right]\right\}^2 + \{1.5 + 1.3logF_r\}^2\right]^{0.5} \qquad [1.16]$$

1.3. Consistency or Atterberg limits

Skempton [SKE 53] developed the Casagrande plasticity chart, including the influence of soil activity (A), which provides some information on the minerals constituting the clay (Figure 1.1). In this chart, the equations for the A- and U-lines are, respectively:

A-line: $I_p = 0.73(w_L - 20)$ [1.17]

U-line: $I_p = 0.9(w_L - 8)$ [1.18]

Later, Biarez and Favre [BIA 76] proposed an alternative to the A- and U-lines:

$$I_p = 0.73(w_L - 13) \qquad [1.19]$$

Based on oedometric test results, it is possible to deduce the consistency index (CI) for remolded sands or clays from the consolidation stress σ_c [FAV 02]:

$$CI = 0.46(log\sigma_c - 0.54) \quad [1.20]$$

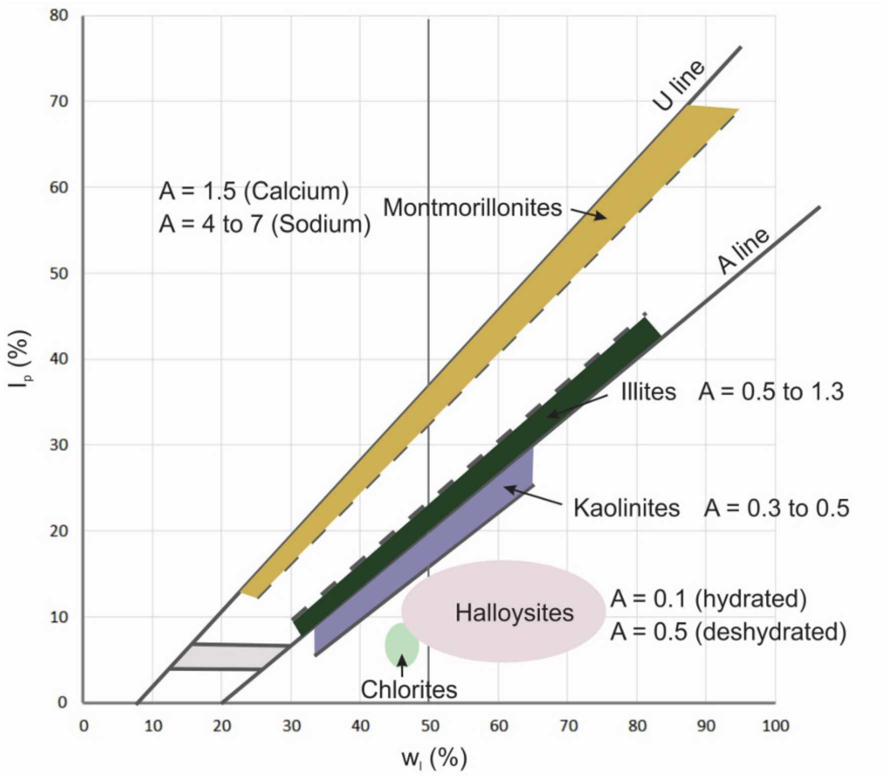

Figure 1.1. *Casagrande's plasticity chart (adapted from [SKE 53]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

1.4. Consistency and liquidity indices

For normally consolidated clays with $20 < w_L < 200$, consistency and liquidity indices can be deduced from the total overburden pressure, as described by Biarez and Favre [BIA 76]:

$$I_L = 0.46(1 - log\sigma_{v0}) \quad [1.21]$$

$$CI = 0.46(log\sigma_{v0} + 1.2) \quad [1.22]$$

where σ_{v0} is expressed in bars (1 bar ~ 100 kPa).

1.5. Rigidity index

This index is defined by the ratio of the shear modulus to the shear stress. For undrained and drained conditions, it is, respectively, given by:

$$I_r = \frac{G}{s_u} \; or \; I_r = \frac{G}{\sigma' tan\varphi'} \quad [1.23]$$

Keaveny and Mitchell [KEA 86] derived the rigidity index from the plasticity index and the OCR of the form:

$$I_r \approx \frac{exp\left(\frac{137-I_p}{23}\right)}{1+ln\left[1+\frac{(OCR+1)^{3.2}}{26}\right]^{0.8}} \quad [1.24]$$

From CPTu results, Mayne [MAY 01] proposed that:

$$I_r = exp\left[\left(\frac{1.5}{M} + 2.925\right)\left(\frac{q_t-\sigma_{v0}}{q_t-u_2}\right) - 2.925\right] \quad [1.25]$$

where

$$M = \frac{6\, sin\varphi'}{3-sin\varphi'} \quad [1.26]$$

Strictly speaking, the calculation of M thus needs CIU triaxial tests but can be approximated with the φ’ values presented in Chapter 4.

1.6. Relative density of sands

For clean sands with less than 15% fines and at medium compressibility, Jamiolkowski *et al*. [JAM 01] related the relative density to cone tip stress in the following way:

$$D_R(\%) = 100\left[0.268\ln\left(\frac{q_t/\sigma_{atm}}{\sqrt{\sigma'_{v0}/\sigma_{atm}}}\right) - 0.675\right] \quad [1.27]$$

For high or low compressibility of the sand, we have to add or subtract up to 15% of the value resulting from this formula.

For preconsolidated sands, Mayne [MAY 09a] suggested multiplying the value 0.675 by $OCR^{0.2}$ in the above formula. An alternative expression for quartz-silica sands [MAY 14] is:

$$D_R\ (\%) = 100\sqrt{\frac{1}{305.OCR^{0.2}}\left(\frac{q_t/\sigma_{atm}}{\sqrt{\sigma'_{v0}/\sigma_{atm}}}\right)} \qquad [1.28]$$

In addition, for carbonate sands, the author suggested that:

$$D_R(\%) = 0.87\left(\frac{q_t/\sigma_{atm}}{\sqrt{\sigma'_{v0}/\sigma_{atm}}}\right) \qquad [1.29]$$

Some refinements regarding the influence of compressibility and the OCR were derived by Kulhawy and Mayne [KUL 90] from tests performed in a calibration chamber:

$$D_R^2 = \frac{\left({q_c}/{\sigma_{atm}}\right)}{K\left({\sigma'_{v0}}/{\sigma_{atm}}\right)^{0.5}} \qquad [1.30]$$

where D_R in decimal form and K is given in Table 1.1.

Soil	K
NC high compressibility	280
NC medium compressibility	292
NC low compressibility	332
Average	350
Low OCR (<3)	390
Med. OCR (3–8)	403
High OCR (>8)	443

Table 1.1. *K values after [KUL 90]*

From DMT results of alluvial soils (clays, silts and sands), [TOG 15] proposed that:

$$D_R = 43ln(K_D)\ if\ I_D \geq 1.!\ and\ 4 \leq K_D \leq 7 \qquad [1.31]$$

$$D_R = 48ln(K_D) + 9\ if\ I_D \geq 1.!\ and\ K_D \leq 4 \qquad [1.32]$$

An expression for the relative density of sandy soils was derived from SPT results by Natarajan and Tolia [NAT 72]:

$$D_R = \left(\frac{2.8}{0.01\sigma'_{v0}+0.7}\right) N + 30 \qquad [1.33]$$

with D_R expressed in % and σ'_{v0} in kPa.

A simpler form was given by [KUL 90]:

$$D_R^2 = \frac{N_{60}}{60+25\ logD_{50}} \qquad [1.34]$$

with D_r in decimal form and D_{50} in mm.

Another expression was given by the same authors:

$$D_R(\%) = 12.2\ + 0.75[222N + 2311 - 711OCR - 779(\sigma'_{v0}/\sigma_{atm}) - 50C_u^2]^{0.5} \qquad [1.35]$$

with 1 < OCR < 3.

Although more parameters are required, the precision is not significant as $r^2 = 0.77$.

1.7. Wave velocity

Currently, the SCPT is uncommon. To estimate the shear wave velocity or to check the measured value, the correlations given below can be useful.

According to Baldi *et al.* [BAL 89], for uncemented sands (units: m/s and MPa):

$$V_s = 277q_t^{0.13}.(\sigma'_{v0})^{0.27} \qquad [1.36]$$

Moreover, for clays [MAY 95] (units: m/s and kPa):

$$V_s = 1.75q_t^{0,627}\ r^2 = 0.736 \qquad [1.37]$$

More generally, for all types of soils [HEG 95] (units: m/s and kPa):

$$V_s = [10.1 log q_t - 11.4]^{1.67}.\left[{}^{f_s}/_{q_t}.100\right]^{0.3} \qquad [1.38]$$

For clays, silts and sands, Mayne [MAY 06] directly relates V_s to the sleeve friction which is expressed in kPa:

$$V_s = 118.8\ log f_s + 18{,}5 \qquad [1.39]$$

For uncemented Holocene- and Pleistocene-age soils, Robertson and Cabal [ROB 15] suggested that (units: m/s and kPa):

$$V_s = [\alpha_{vs}\ (q_t - \sigma_v)/p_a]^{0.5} \qquad [1.40]$$

where

$$\alpha_{vs} = 10^{(0.55I_c+1.68)} \qquad [1.41]$$

For an alluvial site characterized by clay layers, which are sometimes weakly organic alternating with silt and sand [TOG 15], it is given by:

$$V_s = 277q_c^{0.13}(\sigma'_v)^a \qquad [1.42]$$

where a= 0.22 if $\sigma'_v \leq 100$ kPa, otherwise a=0.17.

1.8. Cation exchange capacity

Although widely used in soil chemistry and soil science, the cation exchange capacity (CEC) is quite unknown in soil mechanics. The cation exchange capacity of a soil is the number of moles of adsorbed cation charge that can be desorbed from the unit mass of soil under given conditions. This depends on the kind and amount of clay minerals present in the soil.

CEC is related to the swelling potential and the aptitude for lime stabilization of clayey soils. Table 1.2 gives the CEC values for most usual clay minerals [GRI 68].

Yilmaz [YIL 04] proposed the following relationship to yield the CEC from w_L:

$$CEC\ (meq/100g) = e^{(2.63+0.02w_L)}\ r = 0.97 \quad [1.43]$$

In addition, Vidalie [VID 77] proposed that:

$$CEC\ (meq/100g) = \alpha . I_P \quad [1.44]$$

with $0.25 < \alpha < 1$ and the accepted mean value being $\alpha = 0.5$.

Clay mineral	CEC (meq/100 gr)
Kaolinite	3–15
Smectite	80–150
Illite	10–40
Chlorite	10–40
Vermiculite	100–150

Table 1.2. *Clay minerals versus CEC values*

2

Identification of Soil Types

Soils are usually identified and classified using sieving tests and consistency limits. With the help of these tests, each country and relevant administrations have developed their own standards and regulations. As it would not be possible to present all of these details here, readers will have to refer to the one applicable to the country they are working in. Therefore, we consider only identifications and classifications made using *in situ* tests.

2.1. From identification tests

The Casagrande chart not only allows soil identification from consistency tests but also gives some idea about the associated clay minerals (Figures 1.1 and 2.1). This could be interesting when swelling phenomena is likely to occur. It is possible to associate some refinement with activity A_c:

– close under the U-line and $4 < A < 7$: sodic montmorillonite;

– close under the U-line and $A \sim 1.5$: calcic montmorillonite;

– close above the A-line and $0.5 < A < 1.3$: illite;

– close under the A-line and $0.3 < A < 0.5$: kaolinite;

– close under the A-line and $w_L > 150$: fibrous clays, attapulgites and halloysites.

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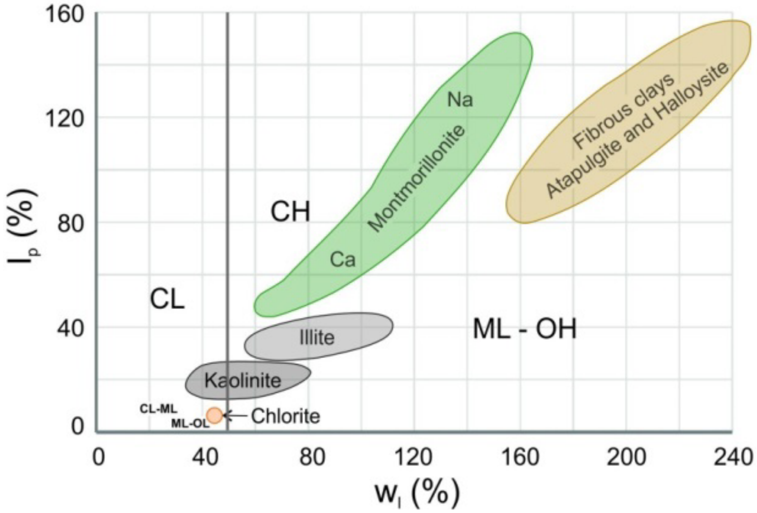

Figure 2.1. *Clay minerals related to the Casagrande chart. For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

2.2. From cone soil index *Ic

Soil type	*Ic
Organic clays	> 3.22
Clays	2.82–3.22
Silt mixtures	2.54–2.82
Sand mixtures	1.90–2.54
Sands	1.25–1.90
Gravelly sands	< 1.90

Table 2.1. *Cone soil classification index according to [JEF 93]*

2.3. From CPT

Since the first publication of Begemann in 1953, many refinements have been made in the graphs by linking soil type and CPT results. Searle [SEA 79] published a rather complete graph, allowing the estimation of soil type, relative density or consistency, undrained shear stress or internal friction angle from the cone tip resistance and the local friction.

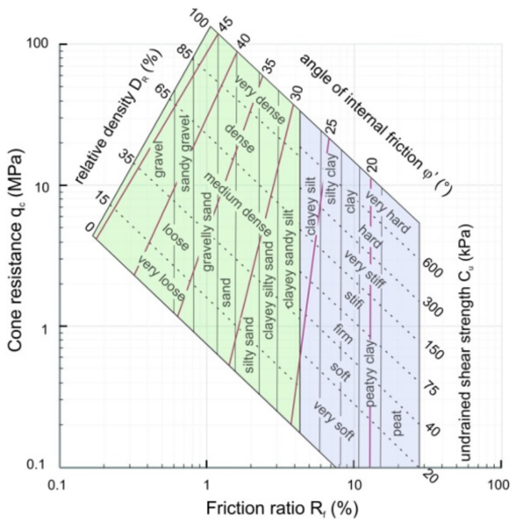

Figure 2.2. *Soil identification from CPT results (adapted from [SEA 79]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

Based on the normalized cone resistance friction ratio associated with the SBT Index Ic, Robertson published and updated a chart [ROB 90, ROB 15] in accordance with the classification of Table 2.2, as shown in Figures 2.3 and 2.4.

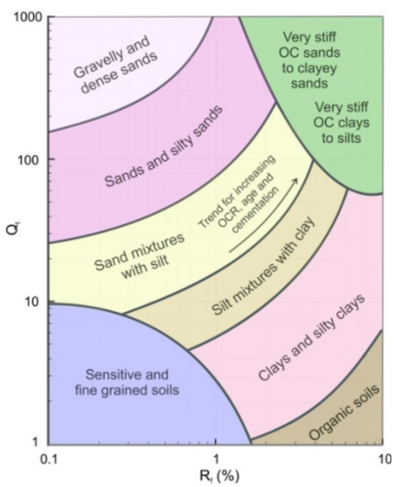

Figure 2.3. *Soil classification from CPT results (adapted from [ROB 15]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

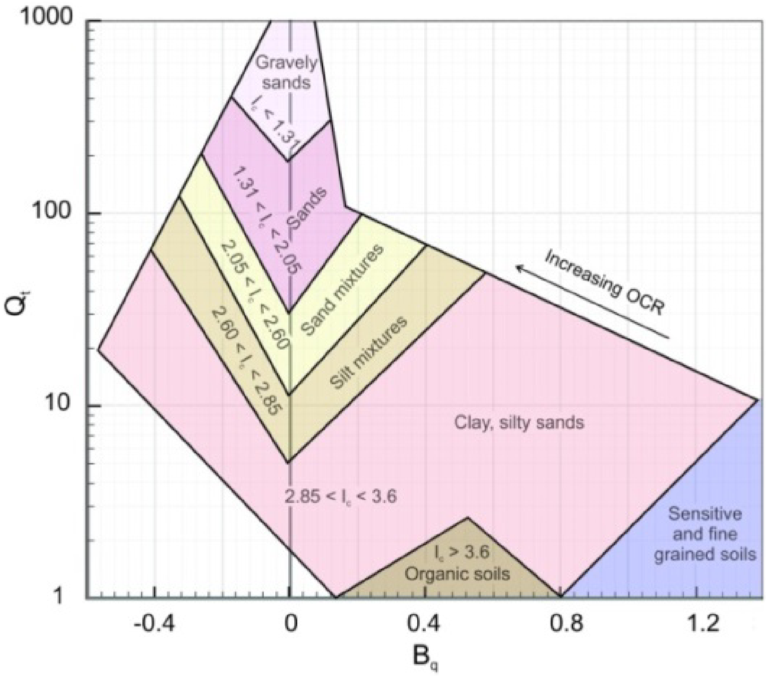

Figure 2.4. *Soil identification from CPTu results (adapted from [ROB 15]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

Zone	Soil behavior type (SBT)	Ic
1	Sensitive fine-grained	NA
2	Organic soil – clay	> 3.6
3	Clays – silty clay to clay	2.85–3.6
4	Silt mixtures – silty sand to silty clay	2.60–2.85
5	Sand mixtures – silty sand to sandy silt	2.05–2.60
6	Sands – clean sand to silty sand	1.31–2.05
7	Gravelly sand to dense sand	< 1.31
8	Very stiff sand to clayey sand, heavily OC or cemented	NA
9	Very stiff sand to clayey sand OC, heavily OC or cemented	NA

Table 2.2. *Robertson chart describing the SBT classification of zones [ROB 90, ROB 15]*

Based on the CPTu results, pore water pressure measurements give further information about the soil type (Figure 2.4). It should be noted that the use of these pore water values is only valid if the tests are conducted in a way that ensures perfect saturation and prevents the loss of saturation of filter elements.

An alternative chart to Figure 2.4 for classification of soils using normalized values of cone resistance and pore water pressure was published by Schneider *et al*. [SCH 08].

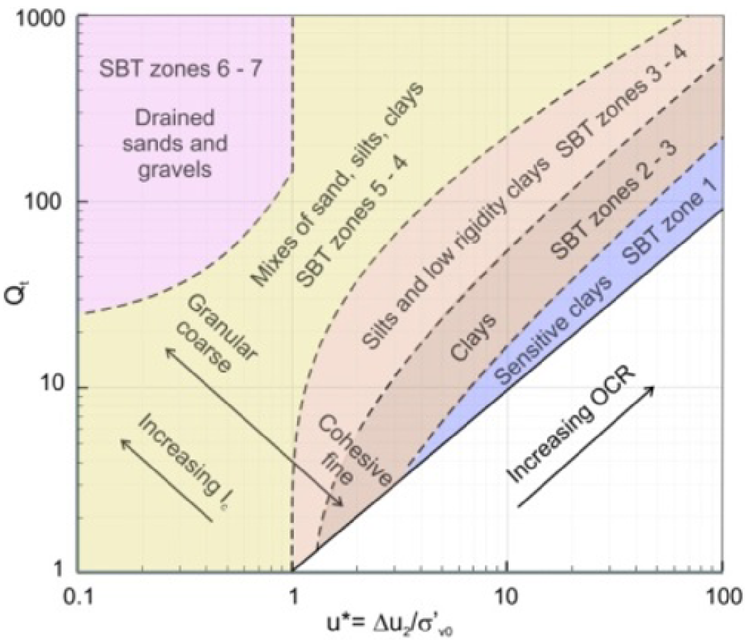

Figure 2.5. *Soil classification chart using CPTu results (adapted from [SCH 08]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

2.4. From PMT

The limit pressure p_{lm} and the ratio E_M/p_{lm} are required to classify the soils using PMT measurements. As shown in Table 2.3, the same value of the pressure limit can correspond to different soils. Therefore, a second criterion is required, which is given in Table 2.4.

Soil type	Consistency	p_{lm} (MPa)
Clay and silt	Very soft – soft	< 0.4
	Firm	0.4–1.2
	Stiff	1.2–2
	Very stiff	> 2
Sand and gravel	Very loose	< 0.2
	Mid loose	0.2–0.5
	Loose	0.5–1
	Dense	1–2
	Very dense	> 2
Chalk	Soft	< 0.7
	Weathered	0.7–3
	Sound	> 3
Marl and marly clay	Soft	< 1
	Stiff	1–4
	Very stiff	> 4
Rock	Weathered	2.5–4
	Fractured	> 4

Table 2.3. *Soil classification using p_{lm} according to [AFN 13]*

E_m/p_{LM}	Soil type
< 5	Remolded soil
5–8	Underconsolidated or slightly remolded clays
8–12	Normally consolidated clays
12–15	Slightly overconsolidated clays
> 15	Overconsolidated clays
6–8	Immerged sands and gravels
> 10	Sands, dry and compact sands, and gravels

Table 2.4. *Soil classification using E_M/p_{lm}*

This table may be summarized by the following rule of thumb:

– clays: $E_M/p_{lm} > 12$;

– sands: $7 < E_M/p_{lm} < 12$.

Baud and Gambin [BAU 13] replaced these two tables with the graph of Figure 2.6.

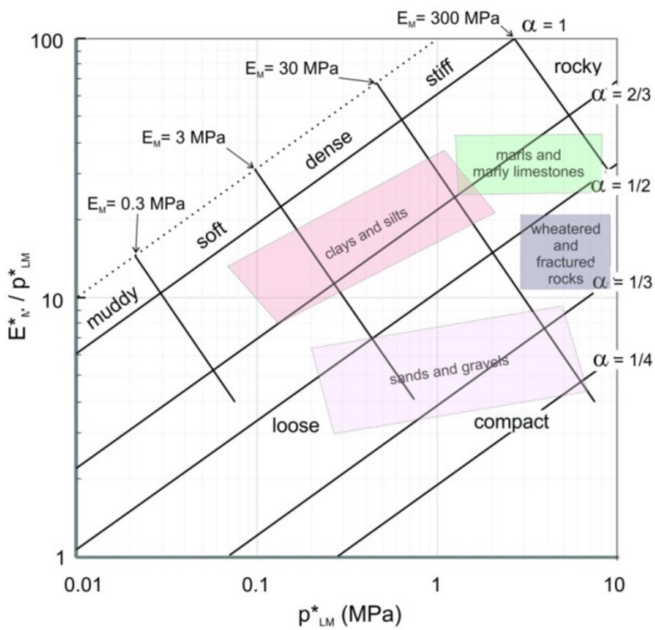

Figure 2.6. *Soil identification chart using MPT (adapted from [BAU 13]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

2.5. From SPT

The N values resulting from SPT test measurements are dependent on energy efficiency whose variation on a site is the main cause of scatter or even inconsistency and therefore corrected values are usually preferred.

The sole N informs about density or consistency but is not sufficient for a full identification. Fortunately, it may be coupled to the sample visual identification or to laboratory tests.

According to AASHTO 1988 [AAS 88], the following correlations may be used for soil characterization from SPT N values, respectively, for granular (Table 2.5) and cohesive soils (Table 2.6):

N	Relative density
0–4	Very loose
5–10	Loose
11–24	Medium dense
25–50	Dense
> 50	Very dense

Table 2.5. *Correlation between the N value and the relative density of sands [AAS 88]*

N	Consistency
0–1	Very soft
2–4	Soft
5–8	Medium stiff
9–15	Stiff
16–30	Very stiff
31–60	Hard
> 60	Very hard

Table 2.6. *Correlation between the N value and the consistency of clays [AAS 88]*

2.6. From DMT

A first general classification of soils using the I_D values was published by Marchetti [MAR 80]:

– clays: $I_D < 0.6$;

– silts: $0.6 < I_D < 1.8$;

– sands: $I_D > 1.8$.

This was extended by Guskov and Gayduck [GUS 15], as shown in Table 2.7.

[GUS 15] Soil type	Plasticity index	[GUS 15] ID	Marchetti ID	Marchetti Soil type
Fat clay – heavy	> 27	< 0.09	0.1–0.6	Clay
Fat clay – silty	17–27	0.09–0.37	0.1–0.6	Clay
Fat clay – sandy	17–27	0.37–0.43	0.1–0.6	Clay
Lean clay – heavy silty	12–17	0.43–0.74	0.1–1.8	Clay/silt
Lean clay – heavy sandy	12–17	0.74–0.84	0.6–1.8	Silt
Lean clay – light silty	7–12	0.84–1.46	0.6–1.8	Silt
Lean clay – light sandy	7–12	1.46–1.64	0.6–1.8	Silt
Silty clay	1–17	1.64–1.88	0.6–(10)	Silt/sand
Silty clay – sandy	1–17	1.88–2.00	1.8–(10)	Sand
Fine sand – silty	x	2.00–2.20	1.8–(10)	Sand
Fine sand	x	2.20–2.50	1.8–(10)	Sand
Medium sand	x	2.50 <	1.8–(10)	Sand

Table 2.7. *DMT soil classification according to Guskov and Gayduk [GUS 15]*

For the PMT, some refinements are possible when ID and ED are combined, as shown in Figure 2.7 [MAR 01].

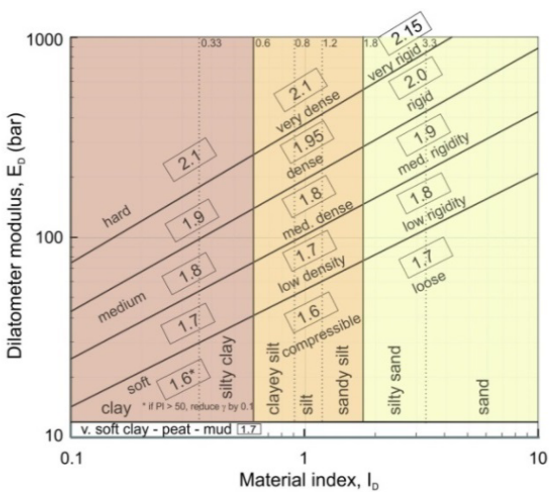

Figure 2.7. *Soil classification using DMT according to [MAR 01]. For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

The equation of the oblique lines shown in Figure 2.7 has the following general form:

$$E_D = 10^{(n + m log I_D)} \qquad [2.1]$$

The values of n and m are given in Table 2.8.

Line	n	M
A	1.737	0.585
B	2.013	0.621
C	2.289	0.657
D	2.564	0.694

Table 2.8. *n and m coefficients*

3

Hydraulic Parameters

3.1. Hydraulic conductivity

Hydraulic conductivity can be measured in two ways: either in the laboratory or in the field. Measurements conducted in the laboratory are accurate and refined but have limited applications compared to the field because they do not represent large discontinuities such as fractures, anisotropy and alteration. Therefore, correlations specified herein will be based only on the laboratory values of k.

The well-known Kozeny formula [KOZ 27] will not be considered here because it requires the specific surface of soil to be measured, which is never done in standard geotechnical tests.

For saturated clean sands with $0.1 < D_{10} < 3$ mm and the percentage passing a No. 200 sieve being less than 5%, the Hazen formula [HAZ 11] relating hydraulic conductivity to a specific grain size can be used:

$$k = C.D_{10}{}^2 \qquad [3.1]$$

with k expressed in m/s and D_{10} in mm. The value of C ranges between 0.4 and 1.5 depending on sand size and sorting, with a mean value of 1 being primarily used.

For saturated sands with less than 3% particles finer than 20 µm at 40% porosity, Van Ganse [VAN 65] proposed that

$$k = 0.25.D_{50}{}^2 \qquad [3.2]$$

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Jean-Claude Verbrugge and Christian Schroeder.

The influence of porosity on k at a void index e can be estimated using the Casagrande formula [TER 62]:

$$k = 1.4\, k_{0.85}.\, e^2 \tag{3.3}$$

or more completely from Van Ganse [VAN 65]:

$$\frac{k_{n,S}}{k_{0.4;1}} = \frac{45}{8} - \frac{n^3.S^3}{(1-nS)^2} \tag{3.4}$$

where:

$k_{0.85}$ is the conductivity corresponding to a void ratio of 0.85;

k_{nS} is the conductivity at porosity n and saturation ratio S ($0 < S < 1$);

$k_{0.4;1}$ is the conductivity at $n = 0.4$ and $S = 1$.

For only the saturation ratio, we can use

$$k_S = k_1 \left(\frac{S-0.2}{0{,}8}\right)^2 \tag{3.5}$$

where:

k_S is the conductivity at the saturation ratio S ($0 < S < 1$);

k_1 is the conductivity at saturation ($S = 1$).

Carrier and Beckman [CAR 84] presented a more general formula, which is also valid for cohesive soils and not limited to sands unlike the above cases:

$$k = 0.0174.\left\{\frac{e-0.027\left(w_L-0.242 I_p\right)}{I_p}\right\}^{4.29} / 1+e \tag{3.6}$$

For soils with $10^{-10} < k < 10^{-3}$ (m/s), ranging from gravelly sands to organic clays, Robertson and Cabal [ROB 15] approximated the permeability from CPT results using the SBTn index I_C:

$$k = 10^{(0.952-3.04 I_c)}\ for\ 1.0 < I_C < 3.27 \tag{3.7}$$

and

$$k = 10^{(-4.52-1.37 I_c)}\ for\ 3.27 < I_C < 4.0 \tag{3.8}$$

Clay soils are often anisotropic, different hydraulic conductivity with in the horizontal (k_h) and vertical (k_v) directions. The ratio of the former to the latter is called the ratio of anisotropy. Baligh and Levadoux [BAL 80] suggested the following values that relate to the nature of clay:

– no evidence of layering: $k_h/k_v = 1{,}2 \pm 0{,}2$;

– slight layering: $k_h/k_v = 2\ to\ 5$;

– varved clays: $k_h/k_v = 10 \pm 5$.

Mayne [MAY 07] proposed slightly different limits:

– Homogeneous clay: $k_h/k_v = 1.0\ to\ 1.5$;

– Clay with discontinuous lenses and layers: $k_h/k_v = 2\ to\ 4$;

– Varved clays and silts, continuous permeable layers: $k_h/k_v = 1.5\ to\ 15$.

In practice, a value of 3 is commonly used for the ratio of anisotropy.

Water pressure dissipation during a CPTu can be recorded using a u_2 piezocone. When t_{50} corresponding to U = 50% is calculated, horizontal conductivity can be estimated as [MAY 01]:

$$k_h = (251 t_{50})^{-1.25} \qquad [3.9]$$

where k_h is expressed in cm/s and t_{50} in seconds.

3.2. Water storage capacity

The water has a restricted movement when saturating the soil, and a part of it is retained by suction or water-holding capacity. Thus, the water that is free to move is not equal to the porosity except for very coarse materials. This is called the effective porosity n', which decreases with grain size. Some orders of magnitude of n' are given below.

3.2.1. *For a free water table*

– coarse alluvial deposits without clay: 30–40%;

– gravel: 20–25%;

– sand, sandy gravel: 15–20%;

– fine sand: 5–10%;

– clayey or cemented gravel: +/−5%;

– silt, loam: 2–5%;

– clay, sandy clay: 3%.

3.2.2. *For a confined aquifer*

$$n' = \frac{H\gamma_w}{E} \qquad [3.10]$$

where H is the thickness of the aquifer.

4

Strength Parameters of Saturated and Dry Soils

4.1. Undrained shear strength and cohesion

For short-term loadings on saturated cohesive soils, the undrained shear strength s_u and the undrained cohesion c_u are equal and equivalent. Therefore, the two symbols will be used here. The unsaturated case is examined in Chapter 10. It should also be kept in mind that the values of these parameters depend on some important factors such the testing procedures and devices used, the strain level, boundary conditions, the disturbance factor and the theoretical model used for interpreting measurements. Hence, the undrained shear strength obtained for the same site may be significantly different, depending on the *in situ* or laboratory tests used.

Many relationships have been proposed to derive the undrained shear strength of clays using physical or mechanical parameters.

4.1.1. *From identification tests*

The most common relationship is the one proposed by Skempton [SKE 57]:

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$$c_u / \sigma'_{v0} = 0.11 + 0.0037\, I_p \ (\pm 20\%) \tag{4.1}$$

This is similar to that of Wroth [WRO 85], which is valid for isotropic compression:

$$c_u / \sigma'_{v0} = 0.129 + 0.00435\, I_p \ (\pm 20\%) \tag{4.2}$$

Bjerrum and Simons [BJE 60] formulated

$$c_u / \sigma'_{v0} = 0.045 \sqrt{I_p} \ (\pm 25\%) \tag{4.3}$$

and

$$c_u / \sigma'_{v0} = 0.18 / \sqrt{I_L} \tag{4.4}$$

which is valid for $I_L > 0.5$, where I_L is the liquidity index deduced from $CI = 1 - I_L$.

As the liquid limit can be determined more accurately, Karlsson and Viberg [KAR 67] used this parameter:

$$c_u / \sigma'_{v0} = 0.005\, w_L \ (\pm 30\%) \tag{4.5}$$

4.1.2. *From laboratory tests*

This primarily involves the case of triaxial tests, where the correlations are made with the effective friction angle in the following general form [KUL 90]:

$$c_u / \sigma'_{v0} = \beta \varphi' \tag{4.6}$$

where $\beta = 0.0100$ for the common case, and $\beta = 0.0120$ and $\beta = 0.0117$ for isotropic and anisotropic consolidations, respectively.

4.1.3. *From CPT*

The correlations between the undrained cohesion or shear stress and the CPT tip resistance are usually represented in the following general form:

$$c_u = \frac{q_c}{A} \ or \ = \frac{q_n}{B} \ or = \frac{q_t}{C} \qquad [4.7]$$

From a practical point of view, the undrained shear cohesion is calculated by making the CPT tip resistance equal to the ultimate point resistance of a pile deduced from a formula for deep foundations ultimate stress calculation, and assuming that the friction angle is zero. Therefore, for the above A, B and C coefficients, we have:

$$A, B \ or \ C = N_c \qquad [4.8]$$

where N_c is the bearing capacity factor related to cohesion, which depends on the theoretical formula used as well as on the soil type (clay or silt), OCR, cone type used, the mode of laboratory testing, etc.

As a rule of thumb, for a mechanical M1 cone, the values of these parameters are approximately 12–16 for sensitive clays, 15–20 for NC clays and up to 20–30 for hard overconsolidated clays, respectively. These values tend to increase with plasticity and decrease with sensitivity and increasing B_q. About half of these values must be taken for an M4 cone.

For an electrical cone, the values of the parameters are lower: 10–13 for sensitive clays and 12–15 for NC clays, silts, loams and peats.

For stiff overconsolidated clays, sensitivity must be taken into account, according to Nuyens *et al*. [NUY 95a, NUY 95b]:

$$c_u = \frac{q_c}{10 \, S_t} \qquad [4.9]$$

For the different tests used, more refined correlations are given below.

According to Amar and Jezequel [AMA 72]:

$$c_u = {}^{q_n}/_{12} \ For \ q_n < \ 0.6 \text{ MPa} \qquad [4.10]$$

$$c_u = \left({}^{q_n}/_{30}\right) + 0{,}03 \ For \ q_n > \ 0.6 \text{ MPa} \qquad [4.11]$$

For Boom clay, De Beer [DEB 67] measured $(c_u)_{VST}$ by the vane shear test:

$$(c_u)_{VST} = {}^{q_c}/_{11} \quad For\ q_c < 3\ MPa \qquad [4.12]$$

According to Carpentier [CAR 70]:

$$(c_u)_{VST} = {}^{q_c}/_{15} \quad For\ 0.8 < q_c < 1.6\ MPa \qquad [4.13]$$

For soft clays, Low *et al.* [LOW 10] recommended the following approximation:

$$N_c = 13.6 \pm 1.9 \qquad [4.14]$$

For cohesive soils, Robertson [ROB 12] suggested that:

$$N_c = 10.5 + 7.\log F_R \qquad [4.15]$$

In addition, for excess pore water pressure measured during the CPTu, the following correlation is proposed:

$$c_u = (u_2 - u_0)/(6.8 \pm 2.2) \qquad [4.16]$$

Another correlation for the excess pressure is given by:

$$c_u = (u_2 - u_0)/B_q.N_c \qquad [4.17]$$

By comparing the studies of other authors, Kulhawy and Mayne [KUL 90] evaluated N_c from the rigidity index:

$$N_c = 2.57 + 1.33(lnI_r + 1) \qquad [4.18]$$

4.1.4. *From PMT*

Cassan [CAS 05] proposed relationships from the PMT using the limit pressure p_{LM}

For $p_{LM} - p_0 \leq 0.3$ MPa:

$$c_u = \frac{p_{LM} - p_0}{5.5} \qquad [4.19]$$

For $0.3 \leq p_{LM} - p_0 \leq 1$ MPa:

$$c_u = \frac{p_{LM} - p_0}{12} + 0.03 \quad [4.20]$$

Or also:

$$c_u = \frac{p_{LM} - p_0}{10} + 0.025 \quad [4.21]$$

For $1 \leq p_{LM} - p_0 \leq 2.5$ MPa:

$$c_u = \frac{p_{LM} - p_0}{35} + 0.085 \quad [4.22]$$

Or, using the creep pressure p_f, it is given by

$$c_u = (0.30 \pm 0.05)(\sigma'_{v0})^{0.2}\left(p_f - u_0\right)^{0.8} \quad [4.23]$$

4.1.5. *From SPT*

From SPT results on clays, Hara *et al.* [HAR 74] suggested that:

$$c_u = 0.29N^{0.72}.\sigma_{atm}\; r^2 = 0.865 \quad [4.24]$$

This was modified by Kulhawy and Mayne [KUL 90] as:

$$c_u = 0.29N_{60}^{0.72}.\sigma_{atm} \quad [4.25]$$

$$c_u = 0.06N_{60} \quad [4.26]$$

4.1.6. *From SCPT*

If V_s is measured in m/s, Levesque *et al.* [LEV 07] proposed the correlation for intact clays:

$$s_u(kPa) = (V_s/7.93)^{1.59} \quad [4.27]$$

4.1.7. *From DMT*

The original correlation was proposed by Marchetti [MAR 80] from tests performed on Italian clays using a dilatometer, which is valid for $I_D < 1.2$:

$$c_u / \sigma'_{v0} = 0.22(0.5K_D)^{1.25} \quad [4.28]$$

However, it seems that the coefficient 0.22 is not a constant value, but depends on the reference test type used for measuring shear stress which is generally less. Modified values are 0.14 for direct simple shear and 0.20 and 0.19 for triaxial compression and field vane tests, respectively [LAC 88].

Schmertmann [SCH 81] derived the undrained cohesion directly from the first corrected pressure reading and hydrostatic pore water pressure as follows:

$$c_u = (p_0 - u_0)/10 \quad [4.29]$$

Slightly different equations were proposed by Cao [CAO 15]:

$$c_u = 0.12(p_0 - \sigma_{v0}) = 0.09(p_1 - \sigma_{v0}) \quad [4.30]$$

According to Galas, cited in [MŁY 15],

$$c_u = 0.164\sigma'_{v0}K_D^{0.345}[(p_1 - u_0)/\sigma'_{v0}]^{0.544} \quad [4.31]$$

4.1.8. *From VST*

It is commonly reported that the VST measurements of s_u are affected by soil plasticity and thus must be corrected:

$$s_u(field) = \mu . s_u(VST) \quad [4.32]$$

and with I_p in %:

$$\mu = 2.5(I_p)^{-0.3} \leq 1.1 \quad [4.33]$$

Even after this correction, the scatter remains significant.

4.1.9. *Overconsolidated soils*

For overconsolidated soils, the following equation can be used:

$$c_u/\sigma'_{v0} = \beta\ OCR^{\Lambda} \quad [4.34]$$

or

$$c_u/\sigma'_p = \beta \quad [4.35]$$

where Λ is the plastic volumetric strain potential defined as:

$$\Lambda = 1 - C_s/C_c \quad [4.36]$$

Several authors have shown that both β and Λ vary with the test mode.

According to Ladd *et al.* [LAD 77], β = 0.25, and β = 0.22 following Mesri [MES 75] and Mayne [MAY 07]. For varved clays, β may decrease as low as 0.16 [SAB 02]. Most β values are in the range of 0.15–0.30, including the domain $\beta = 0.23\ \pm 0.04$ suggested by [JAM 85]. For direct shear tests on overconsolidated intact clays, the recommended value is 0.23, which decreases to 0.21 if OCR < 2.

For clays with medium to low sensitivity, Λ = 0.7 – 0.8, while for sensitive and structured clays, Λ = 0.9 – 1. A typical value that is commonly used is 0.8.

Skempton's equation [4.1] is also valid for OC soils if σ'_{v0} is substituted by σ'_p [CHA 88].

Mayne [MAY 09a, MAY 09b] suggested the following relationship between the undrained shear resistance for uncemented soils (c′ = 0) measured in direct shear tests and the effective friction angle:

$$c_u/\sigma'_{v0} = 0.5(sin\varphi')\ OCR^{\Lambda} \quad [4.37]$$

with Λ = 0.8.

4.1.10. *Miscellaneous: peats and remolded soils*

Peats and muds generally contain organic matter which has an influence on their mechanical behavior. The amount of organic matter is usually measured by the percentage of humus carbon Ch present. According to Vidalie [VID 77]:

$$\log c_u = -0.63 \log(Ch) + 1.47 \quad [4.38]$$

where c_u is expressed in kPa and Ch in %.

According to Carrier [CAR 85], for remolded clays, the undrained shear strength c_{ur} (kPa) can be estimated from:

$$c_{ur} = 166\, I_p / \left\{ 0.163 + \frac{37.1\, e + w_p}{4.14 + \frac{1}{Ac}} \right\} \quad [4.39]$$

where e is the void ratio and Ac is the activity.

For remolded clays, [LUN 97] suggested that:

$$s_u = f_s \quad [4.40]$$

The stress path history also has an influence on the unsaturated shear strength. Laboratory tests have related the cases of isotropic consolidation (subscript Iso) to K_0, and shown that anisotropic consolidation (subscript Aniso) for NC clays in the domain $\left(\frac{c_u}{\sigma'_{v0}}\right)_{Iso}$ comprises between 0.25 and 0.7 [KUL 90]:

$$\left(\frac{c_u}{\sigma'_{v0}}\right)_{Aniso} = 0.15 + 0.49 \left(\frac{c_u}{\sigma'_{v0}}\right)_{Iso} \quad r^2 = 0.761 \quad [4.41]$$

It is well known from triaxial testing that the strain rate influences the measured value of shear strength [LAD 74, GRA 83]. A corrected value can be estimated using:

$$\frac{c_u}{c_u^{1\%}} = 1.00 + 0.10 log\dot{\varepsilon} \quad [4.42]$$

where $\dot{\varepsilon}$ is the strain rate in % per hour and $c_u^{1\%}$ is the undrained cohesion measured at a rate of 1% per hour.

4.2. Effective cohesion

The effective cohesion c′ depends on the stress history and essentially on the preconsolidation stress. According to Mesri and Abdel-Ghaffar [MES 93], it can be estimated by:

$$c' = 0.10\sigma_p'\,\sigma_n'\;in\;the\;range\;2 < {\sigma_p'}/{\sigma_n'} < 5 \qquad [4.43]$$

$$c' = 0.024\sigma_p'\,\sigma_n'\;in\;the\;range\;10 < {\sigma_p'}/{\sigma_n'} < 20 \qquad [4.44]$$

where σ_n' is the effective stress normal to the shear plane. It seems conservative to recommend the latter correlation in any case. Both correlations are valid for short-term analyses. For long-term analyses involving uncemented sands, silts and insensitive clays, the value $c' = 0$ can be taken to be on the safe side.

4.3. Internal friction angle

4.3.1. *From identification tests*

Most of the proposed relationships relate φ or φ′ to physical parameters, especially to consistency limits.

Caquot and Kerisel [CAQ 66] related φ′ to e_f, the void ratio at a peak value of σ_1'/σ_3':

$$e_f.tan\varphi' = a \qquad [4.45]$$

with a = 0.3 for clays, 0.45 for silts and 0.5 for sands. For cohesionless soils, Graux [GRA 67] suggested a = 0.55 as a first estimation.

For single-mineral soils, which is unusual, Koerner [KOE 70] proposed a basic value φ′ = 36° adapted by correction values depending on particle shape, size and gradation, relative density and mineral type:

$$\varphi' = 36° + \varphi_1 + \varphi_2 + \varphi_3 + \varphi_4 + \varphi_5 \qquad [4.46]$$

Shape correction φ_1

= −6° for high sphericity and rounded shape;

= +2° for low sphericity and angular shape.

Particle size correction φ_2

= −11° for D_{10}> 2.0 mm (gravel);

= −9° for 2.0 $>$ D_{10}> 0.6 mm (coarse sand);

= −4° for 0.6 $>$ D_{10}> 0.2 mm (medium sand);

= 0 for 0.2 $>$ D_{10}> 0.06 mm (fine sand).

Gradation correction φ_3 (C_u = uniformity coefficient)

= −2° for C_u > 2.0 (well graded);

= −1° for C_u = 2.0 (medium graded);

= 0 for C_u > 2.0 (poorly graded).

Relative density correction φ_4

= −1° for 0 < D_r< 0.5 (loose);

= 0 for 0.5 < D_r< 0.75;

= +4° for 0.75 < D_r< 1 (dense).

Correction for mineral type φ_5

= 0 for quarts;

= +4° for feldspar, calcite, chlorite;

= + 6° for muscovite, mica.

Owers and Khera [OWE 90] proposed three relationships

$$\varphi' = 36° - 0.25 I_p \text{ for } I_p < 20 \quad [4.47]$$

$$\varphi' = 31° - 0.20\,(I_P - 20) \text{ for } 20 < I_P < 50 \quad [4.48]$$

$$\varphi' = 25° - 0.06\,(I_P - 50) \text{ for } 50 < I_P \quad [4.49]$$

For normally consolidated clays, Kenney [KEN 59] suggested that

$$\sin\varphi' = 0.82 - 0.24 \log I_P \qquad [4.50]$$

Mitchell [MIT 93] proposed a very close equation:

$$\sin\varphi' = 0.806 - 0.228 \log I_P \qquad [4.51]$$

This differs from that proposed by Mayne [MAY 80]:

$$\sin\varphi' = 0.656 - 0.409 \left(I_p/w_L\right) \qquad [4.52]$$

Ghembaza [GHE 04] made a summary of the numerous proposed relationships that relate φ′ to I_p (%). The scatter is important and covers a large domain that can be described by the following three equations:

$$\text{Lower limit: } \varphi' = 37 - 12 \log I_P \qquad [4.53]$$

$$\text{Mean value: } \varphi' = 45 - 13 \log I_P \qquad [4.54]$$

$$\text{Upper limit: } \varphi' = 53 - 14 \log I_P \qquad [4.55]$$

Based on 109 tests on Belgian soils, Van Wambeke [VAN 75] proposed the following equation with a precision of +/–5°:

$$tan\varphi = 0.100 + \frac{16.8}{I_P + 19.0} \qquad [4.56]$$

4.3.2. *From CPT and CPTu*

From CPT tests performed on clean sands in a calibration chamber, Robertson and Campanella [ROB 83] suggested the following equation:

$$\varphi' = arctan[0.1 + 0{,}38 log(q_t/\sigma'_{v0})] \qquad [4.57]$$

Kulhawy and Mayne [KUL 90] gave an alternative and more appropriate value, which is as follows:

$$\varphi' = 17.6 + 11.log\left[\frac{(q_t/\sigma_{atm})}{\sqrt{\sigma'_{v0}/\sigma_{atm}}}\right] \qquad [4.58]$$

From CPTu results, Mayne and Campanella [MAY 06] suggested the following equation for soils other than clean sands:

$$\varphi' = 29.5 B_q^{0.121}\left[0.256 + 0.336 B_q + log\left(\frac{q_t - \sigma_{v0}}{\sigma'_{v0}}\right)\right] \qquad [4.59]$$

This equation is applicable in the range of 20° < φ < 45° and for $0.1 < B_q < 1$. If $B_q < 0.1$, correlations are preferable for clean sands.

The results of the above correlation are very close to the following one given by Uzielli *et al.* [UZI 13]:

$$\varphi' = 25\left[\frac{(q_t/\sigma_{atm})}{\sqrt{\sigma'_{v0}/\sigma_{atm}}}\right]^{0.10} \quad r^2 = 0.92 \qquad [4.60]$$

4.3.3. *From SCPT*

Uzielli *et al.* [UZI 13] related the friction angle to the shear wave velocity from SCPTU tests by:

$$\varphi' = 3.9\left[\frac{V_S}{(\sigma'_{v0}/\sigma_{atm})^{0.25}}\right]^{0.44} \quad r^2 = 0.67 \qquad [4.61]$$

4.3.4. *From PMT*

For sands, the original correlation proposed by Menard [MEN 57] is:

$$\varphi' = 24 + 13.3 log\left(\frac{p_l}{100.b}\right) \qquad [4.62]$$

where p_l is expressed in kPa, and b = 1.8 for moist sands and 3.5 for dry sands, with the mean recommended value being 2.5.

This was slightly modified by Van Wambeke [VAN 78]:

$$\varphi' = 18.7 + 13.3 log(p_l - p_0) + b' \qquad [4.63]$$

where $b' = 2$ for humid soils and $b' = -2$ for dry soils.

For non-dilatant soils, Combarieu [COM 96a] suggested that:

$$sin\varphi' = \frac{log[p_{LM}/p_0]}{log[\frac{\pi}{2}(E_M/p_{LM})]} \quad [4.64]$$

where p_{LM} is the conventional limit pressure, as defined by Menard, and not the abscissa of the asymptote of the pressiometer curve.

For dilatant soils, Combarieu [COM 96b] presented the following hypothesis on the value of the angle of dilatancy in accordance with the most practical cases:

$$K_0 < {}^{1}\!/_{(1+sin\varphi)} \quad [4.65]$$

$$sin\varphi' = \frac{9}{8}\left[\frac{1}{8} + \frac{ln\frac{p_{LM}-u}{q_0-u}}{ln\frac{p_{LM}-u}{q_0-u}+\left(\frac{3}{2}\right)^3}\right] \quad [4.66]$$

4.3.5. *From SPT*

From SPT results, Natarajan and Tolia [NAT 72] proposed the following empirical equation, which is valid for fine sands to gravel:

$$\varphi = \left(\frac{7}{0.1\sigma_{v0}+7}\right)N + 28 \quad [4.67]$$

From data published by other authors, Sabatini *et al.* [SAB 02] derived the following two correlations:

$$\varphi' = (15.4N_{60})^{0.5} + 20 \quad [4.68]$$

$$\varphi' \approx atan\left[\frac{N_{60}}{\left(12.2+20.3^{\sigma'_{v0}}/\sigma_{atm}\right)}\right]^{0.34} \quad [4.69]$$

The former is similar to that given by Ohsaki *et al.* reported by [FIG 15]:

$$\varphi' = (20.\mathrm{N})^{0.5} + 15 \quad [4.70]$$

4.3.6. *From DMT*

Marchetti [MAR 97] proposed a lower limit correlation for DMT results:

$$\varphi' = 28° + 14.6 \, log \, K_D - 2.1 log^2 K_D \qquad [4.71]$$

Schmertman [SCH 82] proposed a method for determining the friction angle of sands if both blade resistance measurements during DMT q_D and CPT q_c are available:

$$\varphi' = 25\left(2.3 - {}^{q_D}/_{q_c}\right) \qquad [4.72]$$

For sands, an alternative correlation proposed by Campanella and Robertson [CAM 91] is:

$$\varphi' = 37.3° \left(\frac{K_D - 0.8}{K_D + 0.8}\right)^{0.47} \qquad [4.73]$$

This relationship was later modified by Mayne [MAY 15] as:

$$\varphi' \approx 37.3° \left(\frac{K_D - 0.8}{K_D + 0.8}\right)^{0.082} \qquad [4.74]$$

Considering different lateral stress states, [MAY 15] suggested that:

$$\varphi' = 28.2 + \frac{(K_D - 0.5)}{0.074 + 0.063(K_D - 0.5)^{0.92}} \qquad [4.75]$$

$$\varphi' = 27.5 + \frac{(K_D - 0.5)}{0.080 + 0.063(K_D - 0.5)^{0.94}} \qquad [4.76]$$

$$\varphi' = 26.8 + \frac{(K_D - 0.5)}{0.10 + 0.062(K_D - 0.5)^{0.95}} \qquad [4.77]$$

These equations give overpredictions compared with the values derived by [MAR 97].

From a test performed on an experimental site in Italy, Togliani *et al.* [TOG 15] proposed that:

$$\varphi' = 17 + 11(I_D K_D)^{0.32}\ if\ I_D \geq 1.2\ and\ K_D \leq 7 \qquad [4.78]$$

From the critical state angle and K_D, [ROB 12] suggested that:

$$\varphi' = \varphi'_{cs} + 15.84.\log(25K_D) - 26.88 \qquad [4.79]$$

4.3.7. *Peak, critical state and residual friction angles*

For sands, the relationship between the peak friction angle ($\boldsymbol{\varphi'_p}$), the critical state angle $\boldsymbol{\varphi'_{cs}}$ and the dilatancy angle $\boldsymbol{\psi}$ is given by:

$$\boldsymbol{\varphi'} = \boldsymbol{\varphi'_p} = \boldsymbol{\varphi'_{cs}} + \beta\boldsymbol{\psi} \qquad [4.80]$$

with $0.8 < \beta < 1$. For sands, Bolton [BOL 86] recommended that $\beta = 0.8$.

The critical state angle of sands can be estimated from particle roundness R [CHO 06]:

$$\varphi'_{cs} = 42 - 17R\ r^2 = 0.823 \qquad [4.81]$$

with R = 1 for very rounded sands and R = 0 for very angular sands.

Biarez and Favre [BIA 76] deduced the residual effective friction angle φ'_r from w_L:

$$tan\varphi'_r = 1.64 - 0.8log(w_L)\ for\ w_L < 50 \qquad [4.82]$$

$$tan\varphi'_r = 0.78 - 0.3log(w_L)\ for\ w_L > 50 \qquad [4.83]$$

For clays, Kanji [ORT 04, p. 123] presented the following correlation, with I_p expressed in %:

$$\varphi'_r = {46.6}\Big/{(I_p)^{0.446}} \qquad [4.84]$$

The values of [4.84] are represented by the trend lines of Figure 4.1, which is adapted from Abramson *et al*. [ABR 96].

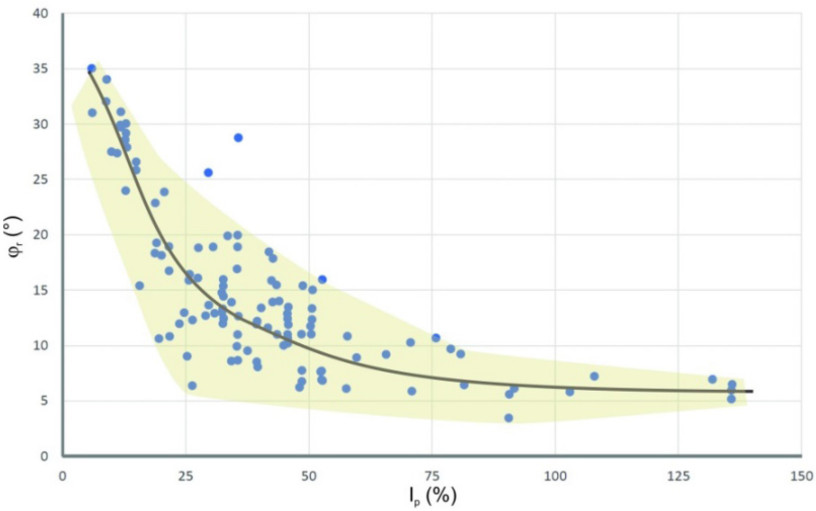

Figure 4.1. *Residual friction angle versus plasticity index (adapted from [ABR 96])*

4.3.8. *Influence of intermediate stress*

It is well known that the plane strain friction angle φ'_{pl} is greater than φ'_{tr}, resulting from classical triaxial tests. Empirical relationships have been proposed by different authors. Of these relationships, four are given below with the respective authors:

Bishop [BIS 66]:

$$\sin\varphi'_{pl} + 3\left(\frac{1}{\sin\varphi'_{tr}} - \frac{1}{\sin\varphi'_{pl}}\right) = 1 \quad [4.85]$$

Green [GRE 72]:

$$3\sin\varphi'_{pl} - \sin\varphi'_{tr}\,(\sin\varphi'_{tr} + \sin\varphi'_{pl}) = 2\sin\varphi'_{tr} \quad [4.86]$$

Lade and Lee [LAD 76]:

$$\varphi'_{pl} = 1.5\,\varphi'_{tr} - 17° \text{ if } \varphi'_{tr} > 34° \text{ and } \varphi'_{pl} = \varphi'_{tr} \text{ if } \varphi'_{tr} < \text{or} = 34° \quad [4.87]$$

Hansen [HAN 79] for dense and very dense sands:

$$\varphi'_{pl} = 1.1\varphi'_{tr} \quad [4.88]$$

As φ'_{tr} values are usually less than 45°, φ'_{pl} values, obtained from the equations given above, differ by less than 7°. Green provides the highest values for $30° < \varphi'_{tr} < 48°$. The lowest result is given by Bishop for $\varphi'_{tr} > 41°$ and by Lade and Lee for $30° < \varphi'_{tr} < 41°$. The latter is thus a useful equation for obtaining a conservative value for φ'_{pl} when $\varphi'_{tr} > 30°$ for silts and sands.

Kulhawy and Mayne [KUL 90] proposed more ready-to-use relationships, taking into account the soil type. If $\beta = \varphi'_{stress\ state}/\varphi'_{triaxial}$, for different stress states (the first value corresponding to cohesionless soils and the second to NC cohesive soils), we have:

– triaxial extension:	$\beta = 1.12$	$\beta = 1.22$;
– plane strain compression:	$\beta = 1.12$	$\beta = 1.10$;
– plane strain extension:	$\beta = 1.25$	$\beta = 1.34$.

The β values are influenced by the intermediate principal stress σ'_2, so a principal stress factor b is introduced:

$$b = \frac{\sigma'_2 - \sigma'_3}{\sigma'_1 - \sigma'_3} \qquad [4.89]$$

Here, b = 0 for triaxial compression, b = 1 for triaxial extension and b = 0.3–0.4 for plane compression. Although the data show a large dispersion, the friction angle increases with b until the value of b reaches 0.5. Then, the evolution becomes more variable, depending on the soil type [KUL 90, BIA 94]. Thus, it is obvious that the use of the triaxial compression friction angle is conservative.

4.4. The angle of dilatancy

For sandy soils, with $I_D > 1.2$, Robertson [ROB 12] suggested that:

$$\psi = 0.56 - 0.33 log(25K_D) \qquad [4.90]$$

Cox and Mayne [COX 15] took into account the stress path for granular soils and the state of consolidation for cohesive soils.

Plane strain conditions:

$$\psi = 6.25(5D_r - 1) \qquad [4.91]$$

Triaxial conditions:

$$\psi = 3.75(5D_r - 1) \qquad [4.92]$$

NC or LOC:

$$\psi = 0 \qquad [4.93]$$

OC:

$$\psi = \varphi'/3 \qquad [4.94]$$

HOC:

$$\psi = \varphi'/6 \qquad [4.95]$$

4.5. Sensitivity

Sensitivity is generally considered for soft clays or silts and accurate measurements require laboratory tests. However, sensitivity can be estimated from the CPT using the relationship suggested by Mayne [MAY 07]:

$$S_t = 0.073\,(q_t - \sigma_{v0})/f_s \qquad [4.96]$$

As a guide value only, Robertson and Cabal [ROB 15] approximated sensitivity and the remolded strength by:

$$S_t = 7/F_r \qquad [4.97]$$

As a rule of thumb, soils with $I_L > 1$ can be considered to be sensitive.

5

Soil Deformations

5.1. Compression and swelling

5.1.1. *Compression index*

Many correlations between the primary compression index C_c and physical parameters such as liquid limit, void ratio or moisture content have been published (units: w, w_L, I_P in %, e, e_0 decimal). Some of them are general, while others refer to the state (remolded, NC or OC), the soil type (silt, clay or peat) and the origin (alluvial, marine, glacial or organic).

Some of these correlations generally present a large data scatter up to 30%. Therefore, their validity must be limited to the soil type or localization, whereon they are established. The best results are generally obtained with the liquid limit w_L, as it integrates moisture, mineralogical aspects and grain size distribution.

According to Terzaghi and Peck [TER 67], for NC clays:

$$C_c = 0.009\, w_L - 0.090 \quad [5.1]$$

This is close to that of Mayne [MAY 80] and Biarrez [BIA 94]:

$$C_c = 0.0092\, w_L - 0.119 \quad [5.2]$$

$$C_c = 0.009(w_L - 13) \quad [5.3]$$

Geotechnical Correlations for Soils and Rocks, First Edition.
Jean-Claude Verbrugge and Christian Schroeder.

Also, according to Mayne [MAY 80]:

$$C_c = (I_p + 26)/138 \qquad [5.4]$$

For clays from Greece and the USA, Azzouz *et al*. [AZZ 76] established three equations:

$$C_c = 0.006\, w_L - 0.054 \text{ for } w_L < 100\% \qquad [5.5]$$

$$C_c = 0.01\, w - 0.05 \qquad [5.6]$$

$$C_c = 0.4\, e_0 - 0.1 \qquad [5.7]$$

For American clays, Rendon-Herrero [REN 80] obtained a different relationship versus e_0:

$$C_c = 0.30(e_0 - 0.27) \qquad [5.8]$$

Both [5.7] and [5.8] are similar to the first equation proposed by Nishida [NIS 56]:

$$C_c = 0.54\, e_0 - 0.19 \qquad [5.9]$$

$$C_c = 0.014\, w - 0.189 \qquad [5.10]$$

The correlations, respectively, by Koppula and Morgenstern [KOP 81] and Herrero [HER 83] are very similar:

$$C_c = 0.01\, w \qquad [5.11]$$

$$C_c = 0.01\, w - 0.075 \qquad [5.12]$$

For OC clays and, more precisely, for pornic clay, Moulin [MOU 89] proposed that:

$$C_c = 0{,}15 e_0^2 + 1.25\, e_0 - 0.025 \qquad [5.13]$$

On the one hand, for remolded clays, respectively, from Skempton [SKE 44], Terzaghi and Peck [TER 67], Holtz and Kovacs [HOL 91], and Biarez and Favre [BIA 75], the first three correlations are very close:

$$C_c = 0.007\, w_L - 0.049 \qquad [5.14]$$

$$C_c = 0.007\,(w_L - 10) \qquad [5.15]$$

$$C_c = 0.007\,(w_L - 7) \qquad [5.16]$$

$$C_c = I_p/0{,}81 \qquad [5.17]$$

On the other hand, for remolded loamy Belgian soils, according to Van Wambeke cited in [VER 68, p. 368]:

$$C_c = 0.0085\,w_L - 0.105 \text{ for } 0 < I_p \leq 5 \qquad [5.18]$$

$$C_c = 0.0077\,w_L - 0.085 \text{ for } 5 < I_p \leq 15 \qquad [5.19]$$

Hereafter, the five correlations are specific for silts, low plastic soils, silty clays and inorganic soils:

Dzwilewski and Richards [DZW 74]:

$$C_c = 0.34 + 0.02 \qquad [5.20]$$

Hough [HOU 57]:

$$C_c = 0.29\,e_0 - 0.08 \qquad [5.21]$$

$$C_c = 0.4049(e_0 - 0.3216) \qquad [5.22]$$

$$C_c = 0.0102(\,w - 9.15) \qquad [5.23]$$

Sowers [SOW 70]:

$$C_c = 0.75\,e_0 - 0.38 \qquad [5.24]$$

Ortiago [ORT 95] presented the following correlation as valid for soils of different geological origins:

$$C_c = 0.5(\gamma_w/\gamma_d)^{2.4} \qquad [5.25]$$

For alluvial clays, Rivard and Goodwin [RIV 78] proposed that:

$$C_c = 0.0047\,w_L - 0.003 \qquad [5.26]$$

$$C_c = 0.0102\,w - 0.004 \qquad [5.27]$$

Moreover, for alluvial clays and silts from Bangladesh, Serajuddin [SER 87] proposed that:

$$C_c = 0.01(w - 7.548) \quad [5.28]$$

Dascal and Laroque [DAS 73] presented correlations, respectively, for lacustrine and marine clays:

$$C_c = 1.34e_0 - 1.11 \; R = 0.86 \quad [5.29]$$

$$C_c = 0.42w - 1.314 \; R = 0.85 \quad [5.30]$$

$$C_c = 0.92e_0 - 0.557 \; R = 0.84 \quad [5.31]$$

For tropical lateritic and saprolitic soils with a non-negligible scatter [ORT 95]:

$$C_c = 1.04 \log e_0 + 0.357 \quad [5.32]$$

Although the use of the compression index is not strictly applicable for peats because of the importance of the secondary consolidation, some authors have also published correlations for these soils associated with others.

For French muds, peats and soft clays with $30 < w_L < 180$, $12 < \gamma < 20$ (kN/m³), Vidalie [VID 77] proposed the following relationships, with all the soils being close to the A-line on the Casagrande chart (Figure 1.1):

$$C_C = 0.575e - 0.241 \; R = 0.966 \quad [5.33]$$

$$C_C = 0.0147w - 0.213 \; R = 0.963 \quad [5.34]$$

For peats and varved clays, Kougure *et al*. [KOG 77] proposed that:

$$C_c = 0.370e_0^{1.17} \; R = 0.917 \quad [5.35]$$

$$C_c = 0.62e_0 - 0.35 \; R = 0.918 \quad [5.36]$$

$$C_c = 0.00722w^{1.07} \; R = 0.916 \quad [5.37]$$

$$C_c = 0.013(w - 7) \; R = 0.918 \quad [5.38]$$

According to Biarez and Hicher [BIA 94], it is possible to draw the linear part of the oedometric curve if the liquid and plastic limits are known. As shown in Figure 5.1, the compressibility path corresponds to the reference line between the two points of coordinates:

$$\sigma' = 7\ kPa\ and\ w = w_L\ or\ e_L = (\gamma_s/\gamma_w)w_L \quad [5.39]$$

$$\sigma' = 1\ MPa\ and\ w = w_p\ or\ e_p = (\gamma_s/\gamma_w)w_p \quad [5.40]$$

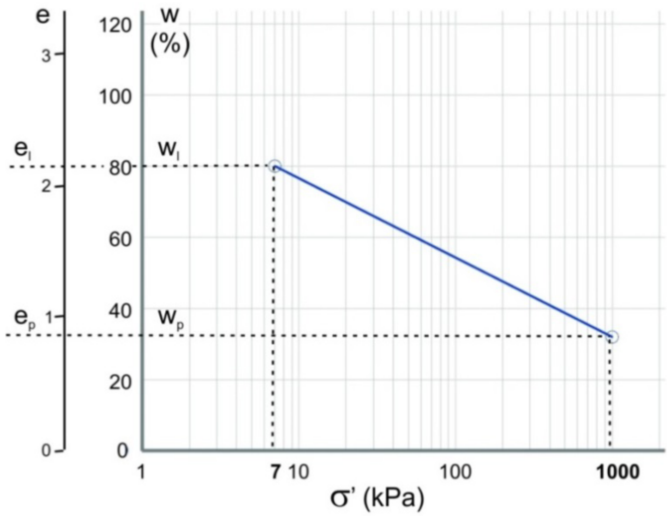

Figure 5.1. *Compressibility path according to [BIA 94]*

5.1.2. *Constants of compressibility*

In some countries, the settlements are calculated using the constant of compressibility C. The correlations given above remain applicable thanks to the equation:

$$\frac{2.3}{C} = \frac{C_c}{(1+e_0)} \quad [5.41]$$

For French muds, peats and soft clays already presented above, Vidalie related C to w or γ_d [VID 77]:

$$C = 2.3/(0.0039w + 0.013) \; R = 0.816 \quad [5.42]$$

Valid for w < 100%:

$$C = 2.3/(-0.0300\gamma_d + 0.554) \; R = 0.896 \quad [5.43]$$

$$C = 2.3/(0.403 log(w) - 0.478) \; R = 0.862 \quad [5.44]$$

For a large scope of soils [ORT 95]:

$$C = 7/\left[1 + \frac{0.0133 I_p(1.192 + A_c^{-1}) - 0.027 w_L - 1}{1 + 0.027w}\right] \quad [5.45]$$

5.1.3. *Swelling index*

When unloading a soil, the swelling index C_s must be used instead of C_c. For Japan soils, Nakase *et al.* [NAK 88] found that:

$$C_s = 0.00084(I_p - 4.6) \; R = 0.94 \quad [5.46]$$

Both C_s and C_c are related to soil plasticity following the ratio:

$C_s/C_c = 8$ for $I_p = 0$ and $= 3$ for $I_p > 25$.

Intermediate values of I_p are obtained by linear interpolation.

This is in accordance with the values commonly used; $C_s/C_c = A$ with:

– A = 3 for clays;

– A = 4–6 for loams, silts, silty and clayed sands;

– A = 7–8 for sands.

5.2. Soil moduli

Although soils are not linear elastic materials, the soil modulus is commonly used in deformation or settlement calculations. Therefore, if a modulus E_M can be directly obtained from PMTs, this is not the case for the other tests.

5.2.1. *From CPT*

When CPT results are only available, it is assumed that E or M = αq_c, where the value of α is related to the soil type, as shown in Tables I [BAC 65] and II [GIE 69] and [SAN 72]. Moreover, α depends on the stress history of the soil, its mineralogy and the level of strain. When the latter increases, the modulus decreases.

Soil type	α
Peat	0.75
Sand	0–2
Silty sand	1–2.5
Clayey sand	3–6
Soft clay	3–8

Table 5.1. *α from Bachelier and Parez [BAC 65]*

Soil classification and type	qc (MPa) or w values (%)	α CPT – M	α CPT – E
CL: Low plastic clay	<0.7	4–6.5	3.7–10
	0.7–2	3–4	2.5–6.3
	>2	1.3–2.2	1.25–3
ML: Low plastic silt	<2	3–6	3.5–7.5
	>2	1–3	1.25–3.7
CH – MH: High plastic clay and silt	<2	2.5–5	2.5–7.5
OL: Organic silt	<1.2	2–8	2.5–10
	>1.2	0.5–4	
T – OH: Peat and very organic clay	<0.7 and		
	50 < w < 100	1.5–4	
	100 < w < 200	1–1.5	
	w > 200	0.4–1	
Chalk	<3	2–4	
	>3	1.5–3	
Sand	<10	2	
Compact sandy gravel	>10	2–3	

Table 5.2. *α from Gielly [GIE 69] and Sanglerat [SAN 72]*

Verbrugge [VER 81] collected a large number of published relationships between E and q_c, which are given in Table 5.3.

Relationship E vs qc (kN/m^2)	Soil	Country
E = 2qc	Cohesionless	France
E < 2.2qc	Cohesionless	France
E = 1.9qc	Cohesionless	Netherland
E = 1.9qc	Cohesive	
E = 1.5qc	Cohesive	Netherland, UK
E = 1.5qc	Cohesionless	Belgium
E = 1.5qc, qc >3,000	Cohesive	Greece
E = 3qc, qc <1,500	Cohesive	Greece
E = 6qc, qc <7,500	Cohesionless	Portugal
E = 2qc + 30,000, qc >7,500	Cohesionless	Portugal
E = 3qc + 1,000	Cohesionless	
E = (2.8 ± 0.3)qc + (26,500 ± 3,700), qc >3,000	Cohesionless	Germany
E = 2.5(qc + 3,000)	Middle sand submerged	South Africa
E = 1.67(qc + 1,500), Ip <15	Clay sand	South Africa
E = 2(qc + 2,500)	Clay sand	South Africa
E <2.5qc	Cohesionless	
E = 5qc + 1,000	Cohesionless	

Table 5.3. *Relationships between E and q_c [VER 81]*

From a close analysis of all these values, [VER 81] proposed the following relationship as valid for values of qc greater than 400 kN/m^2:

$$E = 2.2q_c + 3600\ (kN/m^2) \qquad [5.47]$$

This relationship has given good results for pile settlement calculations.

For Belgian soils, Van Wambeke [VAN 75] suggested the following practical values:

– $\alpha = 1.5$ for sands;

– α = 2.3 for loams and silts;

– α = 3 for clays.

Robertson [ROB 09b] linked α to Q_t and to the SBT index I_c.

For Ic > 2.2 corresponding to fine-grained soils:

$$\alpha = Q_t \, for \; Q_t < 14 \tag{5.48}$$

$$\alpha = 14 \, for \; Q_t < 14 \tag{5.49}$$

In addition, for Ic < 2.2 corresponding to coarse-grained soils:

$$\alpha = 0.0188\left[10^{(0.55 I_c + 1.68)}\right] \tag{5.50}$$

The values resulting from correlations [5.48]–[5.50] are higher than those from the previous ones because they are related to stress levels of approximately 0.1%, lower than those usually encountered for settlements.

We also have to take into account the facts put forward by Sanglerat [SAN 77] that, for a given soil, E grows with a depth up to 30% and, for overconsolidated soils, we have to take twice the above-mentioned values and three times in the case of reloading the soil, which is in accordance with Gambin [GAM 63] and Cassan [CAS 66].

5.2.2. *From DMT*

The basic equation of Marchetti that relates the constraint modulus M or E to the dilatometer modulus E_D is given by:

$$E = M_{DMT} = R_M E_D \tag{5.51}$$

The value of the proportionality factor is not constant according to Monaco *et al*. [MON 99]:

$$R_M = 0.14 + 2.36 \log K_D \;\; if \; I_D \leq 0.6 \tag{5.52}$$

$$R_M = 0.5 + 2\log K_D \;\; if \; I_D \geq 3 \qquad [5.53]$$

$$R_M = R_{M0} + (2.5 - R_{M0})\log K_D \; if \; 0.6 < I_D < 3 \qquad [5.54]$$

with

$$R_{M0} = 0.14 + 0.15(I_D - 0.6) \qquad [5.55]$$

and

$$R_M = 0.32 + 2.18\log K_D \;\; if \; K_D > 10 \qquad [5.56]$$

$$R_M = 0.85 \text{ if } R_M \leq 0.85 \qquad [5.57]$$

5.2.3. *From SPT*

The scatter in the correlations between E and the SPT-N values has drawn considerable attention. The following values must be taken as rough estimations [KUL 90]:

$$E/\sigma_{atm} = \beta N_{60} \qquad [5.58]$$

where $\beta = 5$ for sands with fines, $\beta = 10$ for clean NC sands and $\beta = 15$ for clean OC sands.

By replacing N_{60} with $(N_1)_{60}$ in the previous equation, [SAB 02] suggested that:

– for silts, sandy silts, slightly cohesive mixtures, $\beta = 4$;

– for clean fine to medium sands, slightly silty sands, $\beta = 7$;

– for coarse sands and sands with little gravel, $\beta = 10$;

– for sandy gravels, $\beta = 12$.

Poulos and Small [POU 00] suggested a correlation between the short-term Young modulus E_S, the SPT-N value and the plasticity index (Figure 5.2).

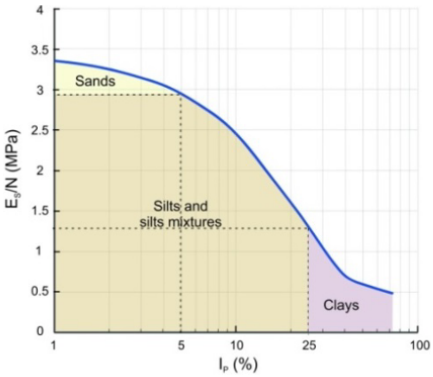

Figure 5.2. *Correlation between N, I_p and "short-term modulus" (adapted from Poulos and Small [POU 00]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

5.2.4. *From CBR*

In a pavement design, the proportionality between the E and CBR values is commonly established:

$$E\ (MPa) = K \cdot CBR(\%) \qquad [5.59]$$

where K=10 if CBR<10% and decreases further. In many counties, a mean value of 5 is used for compacted soils [MAR 83]. Figure 5.3 shows equation [5.59] and the approximate correlation between the CBR and the "long-term Young modulus" according to [POU 00]. The difference between the two curves clearly shows the influence of the duration of loading.

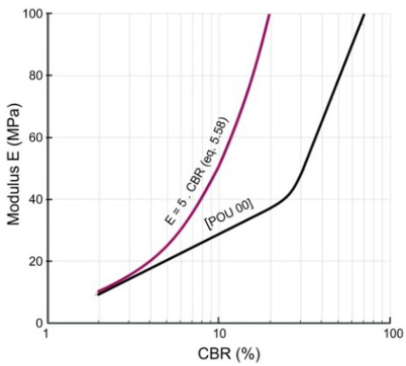

Figure 5.3. *Correlation between the CBR and the modulus*

5.2.5. *Influence of loading rate*

For a very fast loading, the soil generally shows a stiffer behavior. This means that a higher value of E has to be used in calculation. Therefore, Poulos and Small [POU 00] suggested multiplying previous values of α by a mean factor of 5. More detailed adapted values of α are given in Table 5.4.

Soil type	α
Loose sand	5
Medium sand	8
Dense sand	10
Loam or silt	12
Loamy clay	15
Very plastic clay	20

Table 5.4. *α values for fast loading according to [POU 00]*

Wheel loadings are often applied on rafts and slabs during a short time, and the E value to be used is not the same as the "long-term modulus" given before for foundation. Poulos and Small [POU 00] proposed the following equation with β given in Table 5.5:

$$E \text{ (long term)} = \beta \, E \text{ (short term)} \qquad [5.60]$$

Soil type	βo
Gravel	0.9
Sand	0.8
Silt and silty clay	0.7
Stiff clay	0.6
Plastic clay	0.4

Table 5.5. *β values according to [POU 00]*

5.3. Small strain modulus

It is well known that the behavior of soil is truly elastic only for very small strains ($\varepsilon < \pm 10$ μstrains $= 10^{-5}$). In this domain, its modulus, E_0, generally derived from seismic velocity measurements, is quite higher than the usual values used for design calculations (Figure 5.4).

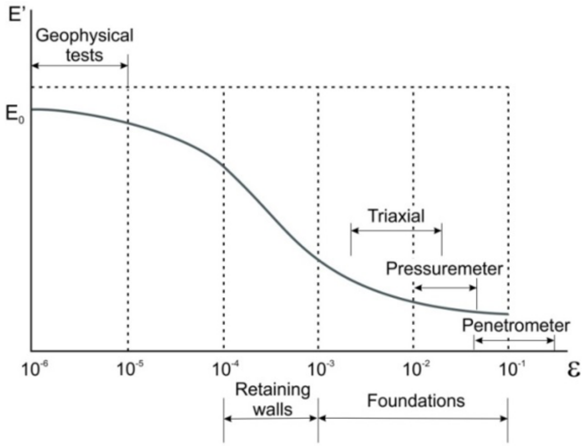

Figure 5.4. *Evolution of the soil modulus with strain*

The ratio of E and E_0 depends on many factors but, as a first approximation, Simons *et al.* [SIM 02] suggested adopting:

For soft clays:

$$E \approx 0.50E_0 \tag{5.61}$$

and for stiff clays or weak rocks:

$$E \approx 0.85E_0 \tag{5.62}$$

These values seem to be too high. For the foundation design, Sabatini *et al.* [SAB 02] refined the correlations given above by a closed-form equation:

$$E/E_0 = 1 - (q/q_{ult})^{0.3} = 1 - (1/FOS)^{0.3} \tag{5.63}$$

where q and q_{ult} are, respectively, the working and ultimate stresses, and FOS is the factor of safety.

This ratio is in closer agreement with the values commonly established between 0.05 and 0.3 depending on the loading case.

Although the range of strains for the CPT largely exceeds 10^{-4}, the results have been correlated with the small strain shear modulus. Robertson and Campanella [ROB 83] proposed the following relationship:

$$\frac{G_0}{q_c} = \beta_1 \left(\frac{\sigma_{atm}}{q_c}\right)^{0.389} \quad [5.64]$$

with $\beta_1 = 50$.

Later, this was modified by Rix and Stokoe [RIX 91]:

$$\frac{G_0}{q_c} = \beta_2 \left(\frac{q_c}{\sigma_{atm}} \sqrt{\frac{\sigma_{atm}}{\sigma'_{v0}}}\right)^{-0.75} \quad [5.65]$$

Here, $\beta_2 = 290$

Lee *et al.* [LEE 09] showed that these two relationships give reasonably good predictions for clean sands, but overestimate G_0 for silty sands. They suggested taking into account the silt content and thus:

$$\frac{G_0}{q_c} = \beta_3 \left(\frac{q_c}{\sigma_{atm}} \sqrt{\frac{\sigma_{atm}}{\sigma'_{m0}}}\right)^{-0.75} \quad [5.66]$$

$$\beta_1 = 25. e^{-0.24 s_{co}} + 25 \quad [5.67]$$

$$\beta_2 = 150. e^{-0.23 s_{co}} + 140 \quad [5.68]$$

$$\beta_3 = 110. e^{-0.23 s_{co}} + 160 \quad [5.69]$$

where s_{co} is the silt content in % and σ'_{m0} the *in situ* mean effective stress.

From Sabatini *et al.* [SAB 02], with kPa units:

$$G_0 = 1.634(q_c)^{0.25}(\sigma'_{v0})^{0.375} \quad [5.70]$$

For Holocene-age uncemented coarse-grained soils [ROB 15]:

$$\left({G_0}/{q_t}\right) Q_{tn}^{0.75} = K_G \quad [5.71]$$

where $215 < K_G < 330$ and K_G increases with age, cementation and bonding.

For Japanese clays, with some scatter [SHI 04]:

$$G_0 = 50(q_t - \sigma_{v0}) \quad [5.72]$$

The same author suggested the following from DMT results:

$$G_0 = 7.5E_D \quad [5.73]$$

According to Rocha *et al*. [ROC 15], with an important scatter:

$$\frac{G_0}{M_{DMT}} = 6.5K_D^{-0.691} \quad [5.74]$$

A refined form of this equation was proposed by Marchetti [MAR 15]:

For clays, $I_D < 0.6$:

$$\frac{G_0}{M_{DMT}} = 26.177K_D^{-1.0066}\ r^2 = 0.61 \quad [5.75]$$

For silts, $0.6 < I_D < 1.8$:

$$\frac{G_0}{M_{DMT}} = 15.686K_D^{-0.921}\ r^2 = 0.81 \quad [5.76]$$

For sands, $I_D > 1.8$:

$$\frac{G_0}{M_{DMT}} = 4.5613K_D^{-0.7967}\ r^2 = 0.65 \quad [5.77]$$

Wroth *et al*. [WRO 79] correlated the small strain shear modulus with the SPT-N value in the domain:

$$60N^{0.71} \leq G_0/\sigma_{atm} \leq 300N^{0.8} \quad [5.78]$$

with a suggested correlation:

$$G_0/\sigma_{atm} = 120N^{0.77} \quad [5.79]$$

For the SPT N_{60} value [SAB 02]:

$$G_0 = 15{,}560(N_{60})^{0.68} \quad [5.80]$$

with G_0 expressed in kPa.

From tests with a resonant column, Hardin and Drnevich [HAR 72] established the following equation:

$$G_0/\sigma_{atm} = 321\,\frac{(2.97-e)^2}{1+e}\,OCR^a\left(\frac{\sigma_m}{\sigma_{atm}}\right)^{0.5} \quad [5.81]$$

where a is approximated by

$$a = 0.0041 I_p\,(\%) + 0.128\; r^2 = 0.940 \quad [5.82]$$

and

$$\sigma_m = (\sigma_1 + \sigma_2 + \sigma_3)/3 \quad [5.83]$$

Massarch reported a later version of the equation proposed by Hardin [MAS 04, p. 137], giving a reasonable agreement for soft clays and silts:

$$G_0/\sigma_{atm} = \frac{625}{0.3+0.7e^2}\,OCR^a\left(\frac{\sigma'_m}{\sigma_{atm}}\right)^{0.5} \quad [5.84]$$

$$a = 0.006 I_p\,(\%) + 0.045 \quad [5.85]$$

$$\sigma'_m = (1 + 2K_0)\sigma'_v/3 \quad [5.86]$$

5.4. Poisson's ratio

In numerical modeling, the Poisson ratio ν is often required. It is possible to deduce the value from the theoretical equation between K_0 and ν, on the one side, and the well-known equation of Jäky [JAK 44] relating K_0 to φ, on the other side.

$$K_0 = \upsilon/(1-\upsilon) = 1 - sin\varphi' \quad [5.87]$$

For drained loading, Traurmann and Kulhawy [TRA 87] approximated ν by:

$$\nu = 0.1 + 0.3\,(\varphi' - 25)/20 \;\; 25° \leq \varphi' \leq 45° \quad [5.88]$$

As a rule of thumb, ν = 0.3 or 1/3 for the general case and 0.47–0.49 for saturated clays can also be considered. More refined values are given in Table 5.6.

The behavior of dilatant soils is inelastic and hence ν may exceed 0.5.

Soil type	Quick loading	Slow loading
Gravel	0.30	0.30
Sand	0.35	0.30
Silt and silty clay	0.45	0.35
Stiff clay	0.45	0.25
Plastic clay	0.50	0.40
Compacted clay	0.45	0.30

Table 5.6. *Poisson's ratio according to [POU 00]*

5.5. Modulus of subgrade reaction

The modulus of subgrade reaction is widely used in the Winkler beam method on an elastic foundation. This simple method has the advantage of giving a rather good evaluation of the ground pressure and of shear and bending moments. However, inversely, the deflection shape differs from reality. Fortunately, the results of calculations are not very sensitive to the value of subgrade modulus because the bending moment is a function of only its fourth roots.

We must keep in mind that the modulus of subgrade reaction is not an intrinsic property of soil. It depends on the experimental setup and mainly on the plate diameter. Therefore, its value is not unique for a given type of soil and we have to be careful when using this parameter if no information is available on how it was measured. For the design, we also have to take into account the differences between the plate diameter and the foundation width. Hence, no correlation for the plate test modulus is considered here.

The most reliable estimation is given by Bowles [BOW 96] as it takes into account the dimensions of the foundation:

$$k\ (kN/m^3) = 120\ \times Allowable\ bearing\ pressure\ (kPa) \qquad [5.89]$$

or

$$k\ (kN/m^3) = 40\ \times Ultimate\ bearing\ pressure\ (kPa) \qquad [5.90]$$

Some refinement of this method is to know the allowable bearing pressure to calculate the related settlement. The ratio of the former to the later may be taken as an approximation of the subgrade modulus, reflecting the actual geometry of the foundation.

5.6. Resilient modulus

Resilient modulus is related to the recoverable deformation behavior of soils or granular materials under repetitive triaxial loading (>10^5 cycles). It is the secant modulus at unloading, which is calculated as the ratio of deviator stress to axial recoverable strain. This modulus is largely used in pavement design.

Arm [ARM 96] tested soils with 10–100% contents of <60 µm fines (clays, silts, sands) and obtained the following:

For silts with a coefficient of uniformity C_U between 3 and 9:

$$M_R = 43.74 - 1.98c_u \; r^2 = 0.56 \quad [5.91]$$

For clays:

$$M_{R-dev} = 2724.39\sigma_{dev}^{-1.07} \; r^2 = 0.88 \quad [5.92]$$

where the subscript "dev" refers to the deviatoric stress state.

5.7. Collapse and expansion

Many equations have been published for estimating soil volume changes mainly from identification tests (where S is the swell in %):

Seed *et al*. [SEE 62]:

$$S = 0.00216 I_P^{2.44} \quad [5.93]$$

[RAN 65]:

$$S = 0.00413(w_L - w_S)^{2.67} \quad [5.94]$$

Nayak and Christensen [NAY 71]:

$$S = 0.0229 I_P^{1.45}(cc/w) + 6.38 \qquad [5.95]$$

where cc is the clay content and w is the initial moisture content.

[CHE 75]:

$$S = 0.2558 e^{0.08381 I_P} \qquad [5.96]$$

Weston [WES 80]:

$$S = 0.000195 w_L^{4.17} w^{-2.33} \qquad [5.97]$$

According to Elarabi [ELA 05], large differences may appear between measured values and those predicted by the correlations. Information about the mineralogy seems to be of paramount importance. From Figure 2.1, it is possible to estimate the eventual presence of swelling clay minerals.

Combining the dry density and the liquid limit, Sabatini *et al.* [SAB 02] plotted the graph shown in Figure 5.5, which is a guide to evaluating the susceptibility of collapse or expansion for cohesive soils.

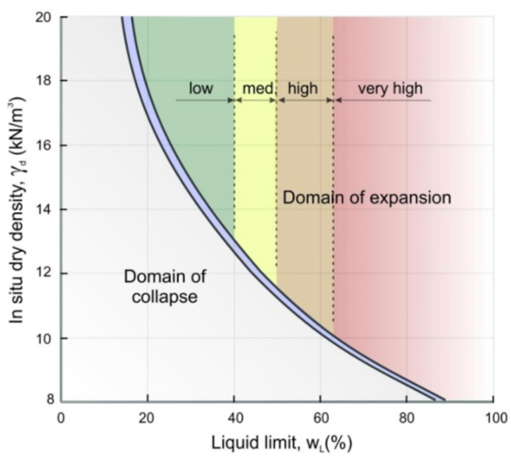

Figure 5.5. *Collapsibility and expandability of cohesive soils (adapted from [SAB 02]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

Figure 5.6 shows the swelling potential of remolded soils related to the clay fraction and the activity, as described by Seed *et al*. [SEE 62].

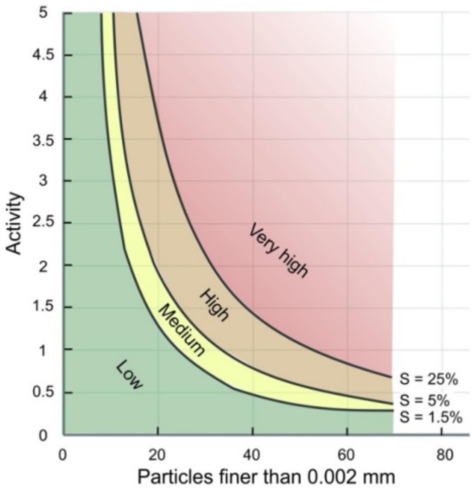

Figure 5.6. *Chart for evaluating the swelling potential (adapted from [SEE 62]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

The swelling potential of clayey soils can also be estimated from the cation exchange capacity (CEC), as suggested by Yilmaz [YIL 04].

CEC (meq/100 gr)	Swelling classification
<27	Low
27–37	Medium
37–55	High
>55	Very high

Table 5.7. *Swelling classification from CEC [YIL 04]*

6

Soil State Parameters

6.1. Preconsolidation pressure

The effective preconsolidation pressure, σ'_p, is very important for an accurate calculation of settlements which requires oedometer tests. Owing to lack of information, the overburden pressure is often used instead although this is not recommended.

For peat and varved clays, Kogure and Oshira [KOG 77] developed two equations:

$$\sigma'_p = 165 e_0^{-0.988} \; R = 0.810 \quad [6.1]$$

$$\sigma'_p = 43.9 w^{-0.913} \; R = 0.821 \quad [6.2]$$

In addition, for disturbed clays, Peters and Lamb [PET 79] proposed the following equation:

$$\sigma'_p = 107.10^{-(I_L - 0{,}68)/0{,}88} \quad [6.3]$$

In these equations, σ'_p is expressed in kPa and *w* in %.

Other correlations with the liquidity index are given by Stas and Kulhaway [STA 84]:

$$\sigma'_p / \sigma_{atm} = 10^{(1.11 - 1.62 I_L)} \quad [6.4]$$

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as well as by Wood [WOO 83]:

$$\sigma'_p/\sigma_{atm} = 0.063.10^{2(1-I_L)} \quad [6.5]$$

For intact and fissured clays, Kulhawy and Mayne [KUL 90] suggested a relationship based on the rigidity index:

$$\sigma'_p = 0.76\, s_u\, .\, lnI_r\; r^2 = 0.895 \quad [6.6]$$

Many authors correlated the preconsolidation pressure with the CPT point resistance, q_c, as done by Mayne [MAY 86] for intact and fissured clays:

$$\sigma'_p = 0.29 q_c\; R = 0.858 \quad [6.7]$$

as well as by Mesri [MES 01]:

$$\sigma'_p = \frac{q_t - \sigma_{v0}}{A} \quad [6.8]$$

where A = 3.57 for inorganic clays and silts and A = 4.24 for organic clays and silts. This is in close agreement with the results of Mayne [MAY 95] and Demers and Leroueil [DEM 02], which proposed the first order of estimate for intact clays:

$$\sigma'_p = 0.33(q_t - \sigma_{v0})\; R = 0.904 \quad [6.9]$$

This was later refined by Mayne [MAY 14] as follows:

$$\sigma'_p = 0.33(q_t - \sigma_{v0})^{m'}(\sigma_{atm}/100)^{1-m'} \quad [6.10]$$

where m' is related to the CPT material index I_c :

$$m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}} \quad [6.11]$$

For $1.5 \leq I_c \leq 3.5$, this corresponds to $m' \cong 0.72$ for sands, 0.8 for silty sands, 0.85 for silts, 0.9 for organic and sensitive fine-grained soils and 1 for intact clays. In this case, the two equations for σ'_p are the same.

Following Mayne [MAY 05] for piezocones, depending on the type of cone used:

– For a type 1 piezocone with a midface filter element:

$$\sigma'_p = 0.75(q_t - u_1) \tag{6.12}$$

– For a type 2 piezocone with a shoulder filter element:

$$\sigma'_p = 0.60(q_t - u_2) \tag{6.13}$$

For soft to stiff intact clays, Chen and Mayne [CHE 96] proposed relationships between the preconsolidation pressure and the water pressure measured by the piezocone independently of tip resistance.

– For type 1 piezocone:

$$\sigma'_p = 0.47(u_1 - u_0)\ R = 0.838 \tag{6.14}$$

– For type 2 piezocone:

$$\sigma'_p = 0.54(u_2 - u_0)\ R = 0.827 \tag{6.15}$$

From the SCPT, small strain shear modulus G_0 is obtained, and according to Mayne [MAY 07], for all types of soils, preconsolidation stress can be evaluated as:

$$\sigma'_p = 0.161 G_0^{0.478}.(\sigma'_{v0})^{0.42}\ r^2 = 0.919 \tag{6.16}$$

From the SPT and DMT results, respectively, we have [KUL 90]:

$$\sigma'_p = 0.47N.\sigma_{atm}\ r^2 = 0.699 \tag{6.17}$$

$$\sigma'_p = 0.51(p_0 - u_0)\ r^2 = 0.896 \tag{6.18}$$

A ratio of the preconsolidation pressure to the limit pressure of the self-boring pressure meter obtained as a result of compilation by different authors is given by Kulhawy and Mayne [KUL 90]:

$$\sigma'_p = 0.45\ p_l\ r^2 = 0.908 \tag{6.19}$$

For Swedish clays, Hanbo [HAN 57] suggested a correlation with the shear strength measured in the vane test:

$$\sigma'_p = 222 s_u / w_L \quad [6.20]$$

and according to Mayne and Mitchell, cited in [KUL 90],

$$\sigma'_p = 22 I_p^{-0.49}\, s_u\; r^2 = 0.569 \quad [6.21]$$

with w_L and I_P expressed in %.

For medium and soft clays, Mayne [MAY 88] established that

$$\sigma'_p = 7.04\, s_u^{0.83}\; R = 0.89 \quad [6.22]$$

$$\sigma'_p = 3.45\, s_u\; R = 0.88 \quad [6.23]$$

Correlation [6.23] is close to that proposed by Kulhawy and Mayne [KUL 90]:

$$\sigma'_p = 3.54\, s_u\; r^2 = 0.832 \quad [6.24]$$

We may also use the definition that links the preconsolidation and overburden stresses:

$$\sigma'_p = OCR.\sigma'_{v0} \quad [6.25]$$

where OCR is deduced from one of the correlations presented below.

6.2. Overconsolidation ratio

For insensitive soils at the critical state, Wood [WOO 83] developed the following equation:

$$logOCR = [2 - 2I_L - log(15.87\sigma'_{v0}/\sigma_{atm})]/0.8 \quad [6.26]$$

This can be used as an approximation for uncemented soils with low-sensitivity.

The results of chamber tests on sands reported by Mayne [MAY 05] allow the evaluation of the OCR using the following expression:

$$OCR = \left[\frac{0{,}192(q_t/\sigma_{atm})^{0{,}22}}{(1-sin\varphi').(\sigma'_{v0}/\sigma_{atm})^{0{,}31}}\right]^{\left(\frac{1}{sin\varphi'-0{,}27}\right)} \qquad [6.27]$$

Balachowski [BAŁ 06] proposed a less sophisticated relationship in the following general form:

$$OCR = \frac{a(q_t-\sigma_{v0})}{\sigma'_{v0}} \qquad [6.28]$$

where *a* ranges from 0.2 to 0.5 and decreases with the plasticity index and void ratio. An average value of 0.3 is generally assumed. Similar equations were proposed by Robertson [ROB 09a, ROB 09b] and Rabarijoely *et al.* [RAB 13]:

$$OCR = 0{,}24\left[\frac{q_t-\sigma_{v0}}{\sigma'_{v0}}\right]^{1.25} \qquad [6.29]$$

$$OCR = 0{,}28\left[\frac{q_c-\sigma_{v0}}{\sigma'_{v0}}\right]^{0.82} \qquad [6.30]$$

According to Karlsud *et al.* [KAR 05], and taking into account the sensitivity, with a large scatter:

$$OCR = (Q_t/3)^{1.20}\, for\, S_t < 15 \qquad [6.31]$$

and

$$OCR = (Q_t/2)^{1.11}\, for\, S_t > 15 \qquad [6.32]$$

For SPT results for intact and fissured clays, the OCR is correlated with *N* by [KUL 90]:

$$OCR = 0.58.N.\sigma_{atm}/\sigma'_{v0}\;\; r^2 = 0.661 \qquad [6.33]$$

From vane test results, for medium and soft clays we have [MAY 88]:

$$OCR = 3.55(s_u/\sigma'_{v0})^{0.66}\; R = 0.80 \qquad [6.34]$$

$$OCR = 4.31(s_u/\sigma'_{v0})\ R = 0.81 \qquad [6.35]$$

Similar to [KUL 90]:

$$OCR = 3.22(s_u/\sigma'_{v0})\ R = 0.806 \qquad [6.36]$$

Marchetti [MAR 80] related the OCR to the horizontal stress index K_D from the DMT as:

$$OCR = (0.5K_D)^{1.56}\ for\ 0.2 \leq I_D \leq 2 \qquad [6.37]$$

However, the coefficient 0.5 seems to depend on the soil type, such that [KUL 90]:

$$OCR = (\beta K_D)^{1.56} \qquad [6.38]$$

with β = 0.27 for glacial tills, β = 0.35 for sensitive clays and β = 0.75 for fissured clays.

From tests on Polish soils, Rabarijoely and Garbulewski [RAB 13] proposed a modified form:

$$OCR = 0.48K_D^{1.2} \qquad [6.39]$$

This is an evolution of an earlier equation developed by Lechowicz and Rabarijoely [BAL 06] for organic Polish soils:

$$OCR = (0.45K_D)^{1.40} \qquad [6.40]$$

For the DMT, Cao *et al*. [CAO 15] proposed alternative correlations with the measured pressures:

$$OCR = 2\left[\frac{p_0 - \sigma_{v0}}{4.13\sigma'_{v0}}\right]^{1.18} \qquad [6.41]$$

and

$$OCR = 2\left[\frac{p_1 - \sigma_{v0}}{4.77\sigma'_{v0}}\right]^{1.18} \qquad [6.42]$$

7

Consolidation

7.1. Primary consolidation coefficient

Carrier [CAR 85] suggested the following relationship between the primary consolidation coefficient c_v and some physical parameters:

$$c_v = \left(\frac{28.67}{I_p}\right).\{(1.192 + A_c^{-1})^{6.993}.\frac{(4.135 I_L + 1)^{4.29}}{(2.03 I_L + 1.192 + A_c^{-1})^{7.993}}\} \qquad [7.1]$$

where c_v is expressed in m^2/year and A_c is the activity.

This equation is valid for remolded clays. As remolding reduces c_v, the calculated values must be seen as a lower bound for intact clays.

Sabatini *et al.* [SAB 02] correlated the consolidation coefficient with the liquid limit of the soil, as shown in Figure 7.1, taking into account the state of the soil.

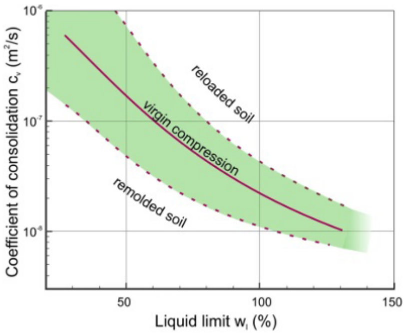

Figure 7.1. *c_V versus w_L (adapted from [SAB 02]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

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For clays exhibiting some anisotropy, the horizontal coefficient of consolidation c_h differs from the vertical one. As the latter is more currently and easily measured, it is possible to derive the former using the following equation:

$$c_h = c_v \frac{k_h}{k_v} \quad [7.2]$$

Some values for the ratio $\frac{k_h}{k_v}$ are given in section 3.1 of Chapter 3.

An approximation for c_h was presented by Robertson *et al.* [ROB 92], where t_{50} can be obtained from a CPTu dissipation test:

$$c_h = (1.67.10^{-6})10^{(1-\log t_{50})} \quad [7.3]$$

7.2. Secondary consolidation coefficient

The secondary compression index C_α is related to the long-term evolution for very plastic soils ($w_L > 100$) and organic soils such as peats, after the hydraulic primary consolidation is completed. Only a few relationships are available, which are mostly related to C_c as a ratio:

$$C_\alpha / C_c = B \quad [7.4]$$

where:

– for inorganic silts and clays, $B = 0.04$;

– for organic silts and clays, $B = 0.05 \pm 0.01$;

– for peats, $B = 0.06 \pm 0.01$.

For peats, B may increase up to 0.08 or even up to 0.1. More refined relationships are given, respectively, by Mesri [MES 73], Mesri and Godlewski [MES 77], Mesri and Castro [MES 87], Mesri [MES 94], which was reported by [ROB 15], and Mesri *et al.* [MES 97]:

$$\frac{C_\alpha}{C_c} = 0.01\, w\, (1 + e_p) \quad [7.5]$$

$$\frac{C_\alpha}{C_c} = \frac{0.04}{1 + e_0} \quad [7.6]$$

$$\frac{C_\alpha}{C_c} = 0.04 \pm 0.01 \qquad [7.7]$$

$$C_\alpha \approx 0.1(\sigma'_v/E) \qquad [7.8]$$

$$C_\alpha = 0.0001w \qquad [7.9]$$

where e_p is the void index at the end of the primary consolidation and w is given in %.

Monnet [MON 15] reported that the applied vertical stress influences the value of C_α, and suggested that:

$$C_\alpha = 0.44(\sigma'_z/E_{oed})(1 + e) \qquad [7.10]$$

Secondary consolidation is negligible if the applied stress $q < 0.8\sigma'_p$.

7.3. Consolidation of peats

For peats, the primary settlement is almost negligible compared to the secondary one, and the equations given above are valid. However, peats are characterized by a high water content associated with an important percentage of organic matter. The major part of their deformational behavior results from the evolution under loading of these two elements. Based on laboratory tests on Netherlands peats, Den Haan [DEN 89] suggested a set of two correlations to estimate the ultimate settlement by taking this specificity into account:

$$\frac{w}{I_{Loss}} = 27.7\left(\frac{\sigma'}{\sigma_{atm}}\right)^{-0.437} \qquad [7.11]$$

and

$$\frac{\Delta h}{h} = \frac{(w_i - w)}{w_i + 37.1 + 0.362.I_{Loss}} \qquad [7.12]$$

where:

– w is the water content after loading (%);

– w_i is the initial water content;

– I_{Loss} is the ignition loss after five hours at 550°C (%);

– σ' is the effective stress applied to the peat;

– h is the thickness of the peat layer;

– Δh is the settlement of the peat layer.

7.4. Degree of consolidation

In Terzaghi's theory, the degree of consolidation U for a constant initial excess pore pressure distribution with depth is calculated from a serial development depending on the time factor T_v. Some correlations between these two parameters may be useful.

If the time factor is less than 0.2, corresponding to the degrees of consolidation under 0.50 (50%), the correlation of Terzaghi is very simple:

$$U = \sqrt{\frac{4T_v}{\pi}} = 1.128\sqrt{T_v} \qquad [7.13]$$

Other authors [ATK 78] have proposed equations valid in larger domains:

$$U = 1.155\sqrt{T_v} \qquad [7.14]$$

This is close to the previous one and valid for $U < 0.33$ (33%). For $U > 0.33$:

$$U = 1 - 0.67.exp(0.25 - 3T_v) \qquad [7.15]$$

The following correlation is valid for $0 < U < 0.95$, and the divergence from the theoretical solution does not exceed 1%:

$$U = \left[\frac{T_v^3}{T_v^3 + 0.5}\right]^{1/6} \qquad [7.16]$$

8

Coefficient of Earth Pressure at Rest

To evaluate the coefficient of earth pressure at rest, Jaky [JAK 44] published the first relationship, which is still largely used today:

$$K_0 = 1 - sin\varphi' \quad [8.1]$$

However, slightly different values were later published by Fraser in 1957 and Kezdi in 1962, reported by [VER 71] and [BRO 65]:

$$K_0 = 0{,}9(1 - sin\varphi') \quad [8.2]$$

$$K_0 = \left(1 + \tfrac{2}{3}sin\varphi'\right)\frac{1-sin\varphi'}{1+sin\varphi'} \quad [8.3]$$

$$K_0 = 0{,}95 - sin\varphi' \quad [8.4]$$

According to Kenney [KEN 59], cited in [VER 71], K_0 depends on the plasticity index of soil:

$$K_0 = 0{,}19 + 0{,}233\ logI_p \quad [8.5]$$

All of these equations are only valid for normally consolidated soils and mainly for sands or sandy soils. For NC clays, Massarck [MAS 79] proposed that:

$$K_0 = 0.0042I_P + 0.44 \quad [8.6]$$

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The above correlations have been extended for cases of overconsolidation in a general form:

$$K_0 = (1 - sin\varphi').(OCR)^{\alpha} \quad [8.7]$$

where $\alpha = 0.5$ according to Schmidt [SCH 66] or $\alpha = 0.46 \pm 0.05$ according to Jamiolkowski [JAM 79]. According to Mayne and Kulhawy [MAY 82],

$$K_0 = (1 - sin\varphi').(OCR)^{sin\varphi'} \quad [8.8]$$

Higher values are sometimes found for cemented soils, which are also influenced by sensitivity.

For practical applications, the last two equations often reduce to:

$$K_0 = 0.5.(OCR)^{0.5} \quad [8.9]$$

Overcompaction of backfills or embankments partly induces overconsolidation, and horizontal stress can be two or three times the NC value of K_0.

From regression analyses of the results obtained from the laboratory and *in situ* tests, Kulhawy and Mayne [KUL 90] deduced the following three equations for CPT, CPTu and SPT, respectively:

$$K_0 = 0.10\,(q_t - \sigma_{v0})/\sigma'_{v0} \quad r^2 = 0.816 \quad [8.10]$$

$$K_0 = 0.24\,\Delta u_2/\sigma'_{v0} \quad r^2 = 0.827 \quad [8.11]$$

$$K_0 = 0.073N.\sigma_{atm}/\sigma'_{v0} \quad r^2 = 0.771 \quad [8.12]$$

From the equation of Kulhawy *et al.* [KUL 90] that yields horizontal stress obtained from tests performed on sands in a calibration chamber, by combining CPT result and relative density, it is possible to deduce K_0 as follows:

$$K_0 = \frac{(q_c/\sigma_{atm})^{1.25}}{35(\sigma'_{v0}/\sigma_{atm}).exp(D_r/20)} \quad [8.13]$$

Mayne [MAY 07] suggested another expression for K_0 derived from CPT results of chamber tests on quartz sands:

$$K_0 = 0.192\left({}^{q_t}/_{\sigma_{atm}}\right)^{0.22} \cdot \left({}^{\sigma_{atm}}/_{\sigma'_{v0}}\right)^{0.31} \cdot (OCR)^{0.27} \quad [8.14]$$

Based on DMT results, according to Marchetti [MAR 80]:

$$K_0 = (K_D/1.5)^{0.47} - 0.6 \quad [8.15]$$

Kulhawy and Mayne [KUL 90] suggested the substitution of 1.5 in the above equation with β to obtain:

$$K_0 = (K_D/\beta)^{0.47} - 0.6 \quad [8.16]$$

with β = 0.9 for fissured clays, β = 2.0 for sensitive clays and β = 3.0 for glacial tills.

For organic soils, Lechowicz and Rabarijoely, cited in [BAŁ 06], suggested the following correlation:

$$K_0 = 0.32(K_D)^{0.48} \quad [8.17]$$

Based on both DMT and CPT results, Baldi *et al.* [BAL 86] suggested the following correlation:

$$K_0 = 0.376 + 0.095K_D - b.\frac{q_c}{\sigma'_{v0}} \quad [8.18]$$

where b = 0.00093 for artificial sands, b = 0.005 for seasoned sands and b = 0.002 for freshly deposited sands.

For sands, an alternative correlation was suggested by [KUL 90]:

$$K_0 = 0.359 + 0.071K_D - 0.00093\frac{q_c}{\sigma'_{v0}} \quad [8.19]$$

as well as by Tuna *et al.* [TUN 08].

$$K_0 = 0.376 + 0.024K_D - 0.00172\frac{q_c}{\sigma'_{v0}} \quad [8.20]$$

9

Soil Compaction Tests

9.1. Proctor tests

Proctor tests are widely used to define compacting criteria in soil constructions such as roads, embankments and earth dams. Despite this, correlations giving the optimal values of water content and density have seldom been published. For the standard and modified Proctor tests, the published correlations link the optimal dry unit weight or dry density and water content to the liquid limit of the soil. Their validity of soil types is defined by a domain of w_L values. Some authors have also mentioned correlations for the capillary pressure u_c at optimum. In the equations presented in this chapter, the units are expressed as follows: % for w, t/m³ for ρ, kN/m³ for γ and kPa for u_c.

9.1.1. *Standard Proctor test*

As order of magnitude for soils with $20 < w_L < 100$ is not recommended for design, Biarez and Favre [BIA 76] proposed that:

$$\gamma_{d-opt} = 22.00 - 0.01 w_L \quad [9.1]$$

$$w_{opt} = 3.00 + 0.35 w_L \quad [9.2]$$

The former equation was later modified by Biarez and Hicher [BIA 94] for non-plastic soils with $D_{60}/D_{10} \geq 10$.

$$\gamma_{d-opt} = 22.00 - 0.06 F \quad [9.3]$$

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where F is the percentage of particles finer than 0.08 mm.

The correlations of Popovic and Sarac [POP 80] are valid for soils with $25 < w_L < 70$:

$$\rho_{d-opt} = \frac{2.7}{1.283 + 0.00818 w_L} \quad [9.4]$$

$$w_{opt} = 8.14 + 0.257 w_L \quad [9.5]$$

According to Gress and Autret [GRE 02], for soils with w_L values in the range of 20 and 60:

$$\rho_{d-opt} = 2.09 - 0.00927 w_L \quad [9.6]$$

$$w_{opt} = 7.92 + 0.268 w_L \quad [9.7]$$

The largest validity domain is given by Fleureau *et al.* [FLE 02] for soils with $17 < w_L < 170$:

$$\gamma_{d-opt} = 21.00 - 0.113 w_L + 0.00024 w_L^2 \quad r^2 = 0.86 \quad [9.8]$$

$$w_{opt} = 1.99 + 0.46 w_L - 0.0012 w_L^2 \quad r^2 = 0.94 \quad [9.9]$$

$$u_{c-opt} = 0.118 w_L^{1.98} \quad r^2 = 0.88 \quad [9.10]$$

9.1.2. *Modified Proctor test*

Here, the only correlations are those from Fleureau *et al.* [FLE 02] for the same soil domain as for the SPO:

$$\gamma_{d-opt} = 20.56 - 0.086 w_L + 0.00037 w_L^2 \quad r^2 = 0.77 \quad [9.11]$$

$$w_{opt} = 4.55 + 0.32 w_L - 0.0013 w_L^2 \quad r^2 = 0.88 \quad [9.12]$$

$$u_{c-opt} = 1.72 w_L^{1.64} \quad r^2 = 0.88 \quad [9.13]$$

For mixtures of sands, kaolinite and montmorillonite, Acar and Nyeretse [ACA 92] related the capillary pressure (in kPa) at the optimum of the modified Proctor tests (OPM) to the plasticity index:

$$u_{c-opt} = 69.7 I_p^{0,58} \quad [9.14]$$

9.2. CBR

As shown in section 5.2.4, the CBR and the soil modulus are related:

$$CBR\ (\%) = E\ (MPa)/K \quad [9.15]$$

where K = 10 if CBR <10%, decreases beyond and a mean value of five is commonly used for compacted soils. A similar equation is given for E_D based on the DMT:

$$CBR\ (\%) = \ 0.058\ E_D^{-0.475} \quad [9.16]$$

with E_D expressed in bars (1 bar = 100 kPa).

If the compaction and the bearing capacity of an embankment are tested with a PANDA light dynamic penetrometer, the CBR is correlated with the dynamic tip resistance, as established by Gourves [in WEL 17]:

$$log(CBR) = 0.35 + 1.057 log(q_d) \quad [9.17]$$

with q_d in MPa.

10

Unsaturated Soils

Suction and moisture content significantly influence the properties, parameters and behavior of unsaturated soils.

10.1. Suction

Matrix suction, also simply called the suction, is a very important concept and parameter when dealing with unsaturated soils. It is defined as the difference between air and water pressures, with the latter being negatively related to the atmospheric pressure. Thus:

$$u_c = u_a - u_w \tag{10.1}$$

In most practical cases, u_a is close to σ_{atm} and is thus generally neglected.

The suction varies with moisture content and is represented by the soil–water characteristic curve. Many equations have been published to describe this curve [FRE 94], and the most commonly used one is that of van Genuchten [VAN 80], which links soil volumetric water content θ and the suction:

$$\theta = \theta_R + (\theta_S - \theta_R)[1 + (\alpha u_c)^n]^{-m} \tag{10.2}$$

Subscripts R and S, respectively, indicate residual and saturated moisture contents in % and the suction is expressed in cm of water column (1 kPa = 10 cm of water column).

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α, n and m are parameters, n and m are dimensionless and α is expressed in cm^{-1}. Vereecken [VER 89] derived correlations from simple test results of intact samples to obtain the following parameters:

$$\theta_R = 0.015 + 0.005CI + 0.014C \; R^2 = 0.703 \qquad [10.3]$$

$$\theta_S = 0.81 - 0.283\rho + 0.001C \; R^2 = 0.848 \qquad [10.4]$$

$$ln(\alpha) = -2.486 + 0.025Sa - 0.351C - 2.617\rho - 0.023CI$$

$$R^2 = 0.68 \qquad [10.5]$$

$$ln(n) = 0.053 - 0.009Sa - 0.013CI + 0.00015Sa^2 \; R^2 = 0.56 \qquad [10.6]$$

$$m = 1 \qquad [10.7]$$

where:

– Sa is the sand content (50–2,000 μm) in %;

– CI is the clay content (< 2 μm) in %;

– C is the carbon content in %;

– ρ is the bulk density in g/cm^3.

Currently, *in situ* suction measurements are performed at low depths (< ±2 m) in agronomy, but very seldom deeper for construction purposes. In this domain, Terzaghi and Peck [TER 62] were the first to propose a correlation:

$$u_c\;(MPa) = 1{,}5(100 - H_r) \qquad [10.8]$$

where H_r is the relative humidity of the surrounding air in equilibrium with the soil (in %). The validity domain is within a temperature range of 10–30°C and a relative humidity range of 70–100%. According to Verbrugge [VER 74], at 25°C and for H_r > 95%:

$$u_c\;(MPa) = 1{,}4(100 - H_r) \qquad [10.9]$$

The error is less than 3%.

10.2. Bishop's coefficient

The coefficient χ was introduced by Bishop [BIS 59] to derive effective stress from total stress and the suction:

$$\sigma' = \sigma - u_a + \chi u_c \qquad [10.10]$$

Although largely discussed, this formula remains widely used for practical purposes even though other equations, sometimes very close to this one, have been published [CRO 61, RUS 67, FRE 77].

The coefficient χ is not an intrinsic parameter but depends, for a given soil, on the moisture content or suction.

Two correlations are mostly used because of their simplicity. The first proposed by Donald [DON 61] and the second by Aitchison [AIT 61]:

$$\chi = S_r \qquad [10.11]$$

$$\chi = 0.22 + 0.78S_r \qquad [10.12]$$

The first overestimates σ', and the error increases with the percentage of particles finer than 2 μm. The second is convenient for sands and silts, but leads to gross errors for clays. Moreover, as χ and S_r must be simultaneously equal to zero, it is not valid close to saturation.

We have to keep in mind the large scatter observed between the values derived from these correlations and the experimental results. Based on tests on a loam, Verbrugge [VER 78] related χ to the suction by the relation:

$$\chi = 2.333 - 0.473 log(u_c) \qquad [10.13]$$

with u_c expressed in cm of water column, and χ with a deviation less than 0.05 if the suction is lower than 6 MPa.

It is also often related to the formula proposed by Fredlund [FRE 78], but different from that of Bishop, by relating the parameter ϕ^b to χ by the relation:

$$tan\Phi^b = \chi tan\varphi' \qquad [10.14]$$

Near saturation, χ is very close to 1, which, for practical purposes, may be convenient to adopt this value. Therefore, two domains must be considered for χ, as suggested by Loret and Kalili [LOR 00]:

$$\chi = 1 \text{ for } u_c < u_{cae} \quad [10.15]$$

and

$$\chi = \left(\frac{u_{cae}}{u_c}\right)^{0;55} \text{ for } u_c > u_{cae} \quad [10.16]$$

where u_{cae} is the suction at the air entry of the soil. This value depends on many factors, but corresponds to the limit of the quasi-saturated domain.

10.3. Quasi-saturated domain

In the quasi-saturated domain, saturated equations remain valid, but the pore pressure is negative. The limits were defined by Zerhouni [ZER 91] and related to the suction for soils containing at least 60% of particles smaller than 80 µm by the following relation (u_c in kPa; w_L and I_p in %):

$$u_c \leq 33w_L - 522.4\, with\, R^2 = 0.836 \quad [10.17]$$

or

$$u_c \leq 48.9I_P - 27.9\, with\, R^2 = 0.805 \quad [10.18]$$

10.4. Stress dependency of suction

As discussed previously, the suction influences stress in the soil, but this is reciprocal and their interactions are complex. However, some authors have suggested a simple approximation for the isotropic stress increase [CRO 61]:

$$\Delta u_c = \beta\Delta(\sigma - u_a) \approx \beta\Delta\sigma \quad [10.19]$$

According to various authors, β is related to the plasticity index:

– Croney and Coleman [CRO 61]:

$$\beta = 0.0231I_p + 0.007 \qquad [10.20]$$

– Aitchison *et al.* [AIT 66]:

$$\beta = 0.03I_p \qquad [10.21]$$

– and Russam [RUS 67]:

$$\beta = 0.027I_p - 0.12 \; if \; 5 < I_P < 40 \qquad [10.22]$$

$$\beta = 0 \; if \; I_P < 5 \qquad [10.23]$$

$$\beta = 1 \; if \; 40 < I_P \qquad [10.24]$$

For deviatoric stress states, Bishop [BIS 61] extended the above equation as follows:

$$\Delta u_c = \beta[\Delta(\sigma_3 - u_a) - \beta_1 \Delta(\sigma_1 - \sigma_3)] \qquad [10.25]$$

where σ_1 and σ_3 are the major and minor principal stresses, respectively.

This is similar to the empirical equation proposed by Skempton [SKE 54]. As a result of this similarity, $\beta_1 = 0.33$ for unconfined axisymmetric compression, but β depends on the degree of saturation and drops very significantly for a small decrease in it. For instance, β = 0.5, 0.3, 0.2 and 0.15 for S_r = 95, 90, 80 and 50%, respectively [BLA 73].

10.5. Drying path of quasi-saturated soils

As established by Fleureau *et al.* [FLE 02], it is possible to obtain the drying path by using the reference lines derived from those proposed by Biarez and Favre [BIA 75] for the compressibility of saturated soils (Figure 5.1):

$$w = w_L \; or \; e = (\gamma_s/\gamma_w)w_L \; for \; u_c = 7kPa \qquad [10.26]$$

$$w = w_P \; or \; e = (\gamma_s/\gamma_w)w_P \; for \; u_c = 1000 \; kPa \qquad [10.27]$$

After performing tests on 24 soils with $6 < I_P < 110$, [FLE 02] suggested the following for the wetting path:

– From the SPO:

$$C_{ms} = \frac{-\Delta e}{\Delta[log(u_c)]} = 0.029 - 0.0018 w_L + 5.10^{-6} w_L^2 \; r^2 = 0.97 \quad [10.28]$$

$$D_{ms} = \frac{-\Delta w}{\Delta[log(u_c)]} = -0.54 - 0.030 w_L + 3.3.10^{-6} w_L^2 \; r^2 = 0.85 \quad [10.29]$$

– From the MPO:

$$C_{ms} = \frac{-\Delta e}{\Delta[log(u_c)]} = 0.0040 - 0.0019 w_L \; r^2 = 0.74 \quad [10.30]$$

$$D_{ms} = \frac{-\Delta w}{\Delta[log(u_c)]} = -1.46 - 0.051 w_L \; r^2 = 0.40 \quad [10.31]$$

In equation [10.31], the low value of r^2 indicates that there is no correlation.

10.6. Capillary or apparent cohesion

Unsaturated soils exhibit an additional cohesion c_a due to capillary forces induced by the suction. A good estimation of it is given by [VER 83]:

$$c_a = \chi . u_c . tan\varphi' \quad [10.32]$$

In agronomy, field capacity (FC) is a widely used concept for moisture profiles of *in situ* top soils, where no groundwater complicates the profile. Although a true equilibrium is rarely or never reached, for practical purposes, FC is defined as the soil moisture content after infiltration when drainage has ceased. This induces the suction, and thus it is possible to approximate an *in situ* value of c_a using the above equation and the u_c value at field capacity. However, we have to keep in mind that rainfall reduces the suction and consequently the apparent cohesion. The common values of the suction at field capacity for Belgian and surrounding soils are given in Table 10.1.

Soil	Suction at FC (kPa)
Sandy soil	10
Sandy loam	30
Plastic loam	50
Clay soil	100

Table 10.1. *Suction at FC [VER 08]*

10.7. Estimation of porosity and degree of saturation from compression wave velocity

If undisturbed samples are not available, then the porosity and the degree of saturation of an unsaturated soil can be reasonably estimated from wave velocity measurements, as suggested by [WAT 72] and [CON 09], respectively:

$$n = 0.175 ln V_p + 1.56 \quad [10.33]$$

where V_p is expressed in m/s

$$S_r = 2\frac{0.5V_p^2 - V_s^2}{V_p^2 - V_s^2} \quad [10.34]$$

11

Cross Relations between *In Situ* Test Parameters

11.1. CPT

11.1.1. *Correction factors and correlations between different CPT tests or parameters*

Mechanical or electrical cones may be used for performing CPT tests, and the results differ slightly for q_c. According to [KUL 90], the following correlation is valid for Delft, Begemann and Gouda mechanical cones:

$$(q_c/\sigma_{atm})_{Elec.} = 0.47(q_c/\sigma_{atm})^{1.19}_{mech.}\ r^2 = 0.965 \qquad [11.1]$$

The difference is narrower for CPTu tests if the corrected tip resistance q_t is used.

The major difference between M1 and M4 mechanical cones is that the former has a sleeve behind the tip. The force measured for M1 cones is thus higher than for M4, which are in the ratio [NUY 95a]:

$$^{q_{c\,M1}}/_{q_{c\,M4}} = [9c_u + (0\ to\ 10)c_u]/9c_u = 1\ to\ 1.9 \qquad [11.2]$$

More refined values of M1, M2 and M4 cone resistances versus electrical cone resistances for Belgian soils, according to Whenham *et al.* [WHE 04], are listed in Table 11.1 (V is the coefficient of variation).

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Soil	M point	CPTM/CPTE	*V* (%)
Clay	M1	1.23	8
	M2	1.27	20
	M4	1.08	13
Sand	M1	0.97	12
	M2	0.9	11
	M4	1.07	12
Others	M1	0.99	19
	M2	1.01	19
	M4	1.01	18

Table 11.1. *Ratios of mechanical to electrical cone resistances [WHE 04]*

It must be noted that f_s is more influenced by the type of the cone, and the differences are sometimes affected by a factor up to 3. This may have a great influence on soil identification.

Because of boundary effects, the field and calibration chamber values of q_c often differ. Jamiolsky *et al.* [JAM 85] suggested the following relationship for sands:

$$q_c(chamber) = q_c(field)/K_q \quad [11.3]$$

with

$$K_q = 1 + (D_r - 30)/300 \quad [11.4]$$

When the local sleeve friction is not measured, correlations can be derived from the tip resistance.

For cohesionless quartz sands (Carpentier in [NUY 95b, p. 9]):

$$f_s = {}^{q_c}/_{200} \text{ for } q_c \geq 20MPa \quad [11.5]$$

$$f_s = {}^{q_c}/_{150} \text{ for } q_c \leq 10MPa \quad [11.6]$$

Intermediate values can be obtained by a linear interpolation.

For cohesive soils with a small rigidity index [NUY 95b]:

$$f_s = {}^{q_c}/_{15} \tag{11.7}$$

and for stiff clays:

$$f_s = {}^{q_c}/_{36.6} \tag{11.8}$$

For alluvial clays, expressed in kPa [TOG 15]:

$$f_s = 0.4\sigma_v'[0.106(V_s^{1.47})/\sigma_v']^{0.8} \tag{11.9}$$

$$f_s = q_c^{0.5} \tag{11.10}$$

11.1.2. *CPT and DPT*

Dynamic probe testing (DPT) is a kind of penetrometer that is commonly used in some European countries and elsewhere in the world. For the CPT, the number of different cones is rather limited and testing processes vary little, but their variability is large for the DPT. Therefore, it is of paramount importance to know the device and the process used for the tests before attempting any interpretation of results. Therefore, only if this information is available should the correlation given here be regarded as evaluations. As a first rule of thumb:

$$\mathbf{0.3 \leq \frac{q_c}{q_d} \leq 1}. \tag{11.11}$$

According to Pilot [PIL 83], the most frequent value of the ratio is 1, except for sandy clays and clayey sands, where $0.5 < a < 0.9$ indicates above the water table and $0.1 < a < 0.4$ indicates below the water table:

$$\frac{q_c}{q_d} = a \tag{11.12}$$

This differs from the values recommended by Waschkowski [WAS 82]: $a = 1$ for NC clays, silts and muds, loose and low dense sands, $0.5 < a < 1$ for OC clays and silts and $1 < a < 2$ for dense to very dense sands, silty sands, clayey sands and gravels.

The values recommended by Cassan [CAS 88] are in close agreement with those of Pilot, which are given by:

$$q_d = a\, q_c + b \quad [11.13]$$

where a = 0.93 for clayey sands, 0.79 for clayey silts, 0.4 for saturated sands and gravels, and 0.3 for silty sands and sandy silts; b is zero except for clayey sands, where b = 1.88 MPa.

11.1.3. *CPT and PMT*

Many authors have suggested correlations between the CPT tip resistance and the limit pressure of PMT. All of these present the same general form:

$$q_c/p_l \;\; or \;\; (q_c - \sigma'_{v0})/(p_l - \sigma_{v0}) = \beta \quad [11.14]$$

As demonstrated by Van Wambeke [VAN 75], whether σ'_{vo} and σ_{vo} are taken into account or not, no significant difference is observed because of the scatter of measurements. Therefore, they are generally neglected.

Vaillant and Aubrion [VAI 14] analyzed the values recommended by different authors for the ratios q_c/p_l and E_M/q_c. They synthesized the domain that corresponded to each type of soil and concluded with a proposal of mean values (Tables 11.2 and 11.3).

Soil type	q_c/p_l	E_M/q_c
Clay	1–5.3	1–8
Loam	1–7	0.5–4.9
Sand and gravel	5–12	0.3–2
Chalk	2.8–3.5	1–2

Table 11.2. *Dispersion of values from various authors according to [VAI 14]*

Soil type	q_c/p_l		E_M/q_c	
	Mean	St. Dev.	Mean	St. Dev.
Clay	3.1	0.7	4.3	0.6
Loam	5.4	0.8	2.5	0.7
Sand and gravel	9	1.1	1.1	0.3
Chalk	3.2		1.5	

Table 11.3. *Mean values according to [VAI 14]*

As a rule of thumb, [VAN 75] suggested that q_c/p_l is equal to 3 for clays, 6 for loams and 9 for sands. Vaillant and Aubrion [VAI 14] proposed to include E_M/q_c = 1.5–3 for sands and 4.5–6 for clays.

Results on alluvial sandy gravels near to the river Meuse in Belgium show two linear correlations [NUY 77]:

$$E_{M\,max} = 0.585\,q_c + 4.9\,for\ 10 \leq q_c \leq 60\ MPa \quad [11.15]$$

$$E_{M\,min} = 0.583\,q_c - 5.833\,for\ 10 \leq q_c \leq 70\ MPa \quad [11.16]$$

11.1.4. *CPT and DMT*

Robertson [ROB 09a, ROB 09b] established that the horizontal earth pressure index K_D is correlated with q_c and thus proposed that:

$$K_D = 0.8\left(\frac{q_c - \sigma_{v0}}{\sigma'_{v0}}\right)^{0.8} \quad [11.17]$$

or

$$q_c = 1.25\sigma'_{v0}.K_D^{1.25} + \sigma_{v0} \quad [11.18]$$

and different equations:

$$K_D = 2.1\left(\frac{q_c - \sigma_{v0}}{\sigma'_{v0}}\right)^{0.4} \quad [11.19]$$

or

$$q_c = 0.45\sigma'_{v0}.K_D^{2.0} + \sigma_{v0} \quad [11.20]$$

As a first evaluation, Togliani *et al*. [TOG 15] proposed, for q_c and Q_t (units: kPa):

$$q_c = 6.6(p_1 - p_0)K_D^{0.35} \; if \; OCR < 4 \quad [11.21]$$

$$q_c = 3.3(p_1 - p_0)K_D^{0.2} \; if \; OCR \geq 4 \quad [11.22]$$

$$Q_t = K_D^{1.7} \; if \; I_D \leq 1.8 \quad [11.23]$$

$$Q_t = 9I_D K_D \; if \; I_D > 1.8 \quad [11.24]$$

also:

$$f_s = (p_1 - p_0)^{0.68} \quad [11.25]$$

11.1.5. *CPT and SPT*

Since 1951, when Huizinga [HUI 51] suggested a value of 4 for the ratio of q_c to the N blow count from the SPT, numerous values have been proposed, which largely vary depending on the soil type. Verbrugge [VER 76] compiled these values and they are listed in Table 11.4. From this and based on some theoretical considerations, he established the following equation by expressing the ratio q_c/N as a function of the depth and the soil type that is taken into account via the local friction coefficient f_s:

$$q_c/N = 9350 + 225.7z/[(10.7 + 825F_R)(70.5 + 6.3z)] \quad [11.26]$$

The units are expressed as bar (= 100 kPa) for q_c and m for z. The values of F_R related to the soil type are given in Table 11.5. The ratios q_c/N calculated from equation [11.26] are in good accordance with those given in Table 11.4.

qc/N	Soil type	Reference
2	Clay	[SCH 57]
	Silt, sandy silt	[SCH 70b]
	Clayey silt, silty clay	[DES 74]
3	Clay, silt, silty sand	[BEL 67]
	Dense sand, clay, shale	[CAQ 66]
	Silty clay	[SAN 65]
4	Silty sand, sand	[MEY 56]
	Middle sand, silt	[CAQ 66]
	Fine sand, silty sand	[MEI 61]
	Sandy clay	[SAN 65]
5	Fine to middle sand	[ROD 61]
	Loose sand	[CAQ 66]
	Grove sand, gravel sand	[SCH 70b]
	Sandy silt	[SAN 65]
6	Clayey sand	[SAN 65]

Table 11.4. *Ratio $^{q_c}/_N$ according to various authors [VER 76]*

Soil type	F_R	qc/N
Clay, peat	>0.04	2
Silt	0.025< – < 0.04	3
Fine silty sand	0.017< – <0.025	3–4
Sand	0.012< – <0.017	4–5
Grove sand	0.007< – <0.012	5–8
Gravel	< 0.007	> 8

Table 11.5. *Range of the ratio $^{q_c}/_N$ related to the soil type*

For characterizing the soil type instead of F_R, Robertson *et al*. [ROB 83] proposed the relationship between the ratio of CPT to SPT and the mean grain diameter passing at 50% (Figure 11.1).

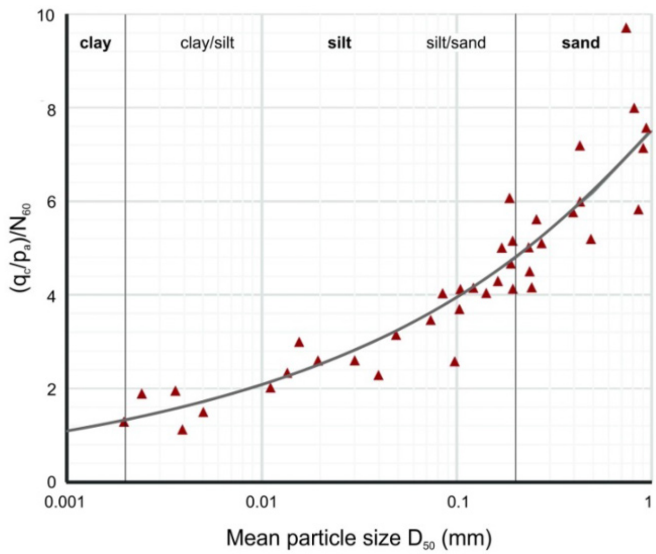

Figure 11.1. *CPT–SPT ratio related to the grain size, adapted from [ROB 83] and the curve of equation [11.27]*

As a result of a regression analysis of the points shown in Figure 11.1:

$$(q_c/\sigma_{atm})/N_{60} = 75D_{50}^{0.28} \; r^2 = 0.858 \qquad [11.27]$$

This is different from the one by Kulhaway and Mayne [KUL 90]:

$$(q_c/\sigma_{atm})/N = 5.44D_{50}^{0.26} \; r^2 = 0.702 \qquad [11.28]$$

The values derived from this equation are less than those obtained from Figure 11.1.

Also:

$$(q_c/\sigma_{atm})/N = 4.25 - {}^{F}/_{4.6} \; r^2 = 0.414 \qquad [11.29]$$

where F is the percentage passing on a No. 200 sieve, with $D = 74$ µm.

First, the low value of r^2 indicates a very poor correlation. Second, if F vanishes, the CPT–SPT ratio has an upper limit of 4.25 compared to the values up to 10 and above, as given in Table 11.4.

For very loose soils, the N values are often underestimated because of the effects of the weight of rods and accessories, resulting in overestimated values of q_c. Because of this and the low repeatability of the SPT, some authors such as [JEF 93] and [ROB 12] have suggested replacing N with N_{60} and characterizing the soil by the SBT index. Their equations are, respectively, given by:

$$(q_c/\sigma_{atm})/N_{60} = 8.5\,[1 - I_c/4.6] \qquad [11.30]$$

and

$$(q_c/\sigma_{atm})/N_{60} = 10^{(1.1268-0.2817 I_c)} \qquad [11.31]$$

The last equation may overestimate N_{60} for fine-grained sensitive soils.

11.2. PMT

11.2.1. *PMT and DPT*

Waschowski [WAS 82] proposed values for the ratios of DPT to PMT depending on the soil type:

$$q_d/p_{lm} = a \qquad [11.32]$$

and

$$q_d/E_m = b \qquad [11.33]$$

where for:

– NC clays, silts and muds, loose and low dense sands: $1.4 < a < 2.5$ and $0.1 < b < 0.3$;

– OC clays and silts: $3 < a < 5$ and $0.2 < b < 0.4$;

– dense to very dense sands, silty sands, clayey sands, gravels: $5 < a < 10$ and $0.4 < b < 1.5$.

Depending on the soil type, two correlations were proposed by Zhou [ZHO 97]:

– For loams:

$$6 = q_d/(p_{lm} - \sigma_h) \quad [11.34]$$

– For clays:

$$\mathbf{4.6 = q_d/(p_{lm} - \sigma_h)}. \quad [11.35]$$

For clays, Pilot [PIL 83] suggested a slightly lower value between 3 and 4.

11.2.2. *PMT and DMT*

For clays, Schmertmann in [MAR 01] suggested as a first evaluation:

$$p_0/p_L = 0.8 \quad [11.36]$$

and

$$p_1/p_L = 1.2 \quad [11.37]$$

A value closer to 1.25 was later proposed by Kalteziotis *et al.* [KAL 91], who also suggested that:

$$E_{MPT} = 0.4\, E_D \quad [11.38]$$

11.2.3. *PMT and SPT*

Some published correlations between the pressuremeter modulus and *N* values are of the general form:

$$E_M/\sigma_{atm} = aN^b \quad [11.39]$$

where a = 20.215 and b = 0.6253 for a fitting of data from Martin [MAR 77], a = 19.3 and b = 0.63 for clays and a = 9.08 and b = 0.66 for sands from [OHY 82]. All data present a large scatter as seen for the last two: $r^2 < 0.5$.

According to Pilot [PIL 83], with units expressed as MPa:

$$N_{60} / (p_L - \sigma_{v0}) = a \qquad [11.40]$$

and

$$N_{60} / E_M = b \qquad [11.41]$$

For different soils:

– clays: $15 < a < 20$ and $b = 1–1.5$;

– silts: $a = 30$ and $b = 3$;

– sands: $a = 20$ and $b = 1.5–2$.

Extended ranges of a and b values, including those of Pilot, were established by Monnet [MON 15].

Soil type	a	b
Clay	15–30	1–2.5
Silt	30–35	2.5–3
Sand	15–25	1.5–3
Marl	20–25	1.5–2.5
Chalk	5–20	0.7–1

Table 11.6. *a and b values according to [MON 15]*

11.3. DMT

11.3.1. *DMT and SPT*

The DMT is a more reliable test than the SPT, so values hereunder must be taken for evaluation.

From tests on sandy sites, Tanaka and Tanaka [TAN 98] suggested that:

$$N_{SPT} = 0.4\, E_D\ (MPa) \qquad [11.42]$$

According to [ISS 01], with the unit expressed as MPa:

$$N_{SPT} = M_{DMT}/3 = R_M E_D/3 \qquad [11.43]$$

From tests on sandy sites in the USA and Italy and from back calculations, with some scatter [SAB 02]:

$$E_D/\sigma_{atm} = 0.22\, N^{0.82} \qquad [11.44]$$

11.4. SPT

11.4.1. *SPT and DPT*

Cassan [CAS 88] reported correlations established by Waschkowski between these two dynamical tests and depending on the soil type:

$$q_d\ [MPa] = b\, N_{SPT} \qquad [11.45]$$

where b = 0.2 for overconsolidated clays and loams, b = 0.3 for sandy clays and loams, b = 0.4 for sands and b = 0.8 for gravelly sands.

11.5. PANDA dynamic penetration test

The "PANDA™" test is a light dynamic penetration test with variable energy commonly used in some Western European countries (France, Belgium, etc.) in the field of roads and highway construction. It allows control of the compaction and the bearing capacity of backfills and embankments up to a depth of 3–5 m. This specificity explains this separate treatment.

11.5.1. *PANDA and CPT*

The correlation has the same formulation as that between the CPT and the DPT:

$$q_d/q_c = a \qquad [11.46]$$

For sandy silts or silty clays, 1.0 < a < 1.1 for an electric tip [CHA 03, WEL 17] and a = 1.0 for a Gouda 20 kN penetrometer [ZHO 97].

11.5.2. *PANDA and DPT*

From comparisons with some heavy DPT, Welter *et al.* [CHA 03, WEL 17] concluded that:

$$0.93 < q_{d\,panda}/q_{DPT} < 1.02. \quad [11.47]$$

11.5.3. *PANDA and PMT*

For loamy and clayey soils from [ESC 94, WEL 17]:

$$3.7 < q_d/p_L < 4.2. \quad [11.48]$$

A good approximation is thus

$$p_l = {}^{q_d}\!/_4 \quad [11.49]$$

11.5.4. *PANDA and VST*

Correlations between PANDA and VST depend on whether the tests have been performed in the laboratory or the field [ESC 94, CHA 03]:

– From laboratory tests:

$$q_d = 11.4\ c_u \quad [11.50]$$

– From field tests:

$$q_d = 20.9\ c_u \quad [11.51]$$

The scatter is important, and the lateral friction on rods has a significant influence. Therefore, it is commonly agreed that [WEL 17]:

$$12 < q_d\,/c_u < 20 \quad [11.52]$$

12

Rocks

12.1. Introduction

"Rocks" are multi-scale materials with associated representative elementary volumes (REVs)[1] at millimetric scales (crystals), centimetric scales (rock material, rm) and metric or other scales (rock mass, RM).

At each scale, the properties of a "rock" depend on the properties of its lower scale constituents as well as on the properties of discontinuities between them. Hence, rock mechanics is defined as "non-continuous materials mechanics"[2]:

– The properties of a mineral depend not only on the mineralogical characteristics of the intact crystal but also on discontinuities, for example dislocations, glide, cleavage, twinning, micro- and nano-fissures, etc. This scale is increasingly being researched (micro- and nano-mechanics) but has not been used in daily engineering practices until now; hence, it will not be considered here.

– The properties of a rock material depend on those of the constituting minerals and fluids as well as on the properties of discontinuities, for instance fabric (texture, structure), porosity, microfissuration, etc.

– The properties of rock mass depend on those of a rock material as well as on the characteristics of joints, for example stratification, schistosity, diaclases, fractures, faults, etc.

1 The smallest volume over which a measurement can be made, which is representative of the whole [HIL 63].
2 Rock mechanics is "The scientific discipline that studies the response of *jointed rocks* when subjected to forces". L. Müller and F. Pacher; broadcast interview 24 May 1962.

Geotechnical Correlations for Soils and Rocks, First Edition.
Jean-Claude Verbrugge and Christian Schroeder.

In addition, different parameters do not necessarily react in the same way as the variations of a "geological" characteristic.

For example, in a stratified rock material, the unconfined compressive strength (UCS) is maximum when the load is applied perpendicularly to the stratification, whereas Young's modulus, E, and the sonic velocity, V_l, are minimum.

Therefore, it is difficult to provide relationships between different properties of a rock material or mass even though sharing the same REV.

The use of relationships that do not take into account the "geological" and "discontinuous" dimensions, which are not necessarily compulsory in soil mechanics, is thus extremely hazardous and can lead to serious errors.

If the "geological" dimension is not considered, scattering can be huge and the resulting correlation can be misleading, especially when the results are presented in the form of interpretations (regression curves or equations) (Figure 12.1). In all cases, it is important to know the coefficient of correlation, R (or r), or the coefficient of determination, R^2 (or r^2).

Even when the geological dimension is considered, the amount of causes of scattering is so important that, except in cases of homogeneous, isotropic, unweathered, non-fissured rock on the same lithology, the correlations must be considered with a critical look.

This is perfectly noted by Aydin [AYD 15] in the ISRM "Orange Book" [ULU 15]:

> "…it becomes obvious that correlations should ideally be established for a given rock type whose response falls within a single response domain. Nonlinear correlations simply indicate significant microstructural changes in that seemingly identical rock type…. When the aim is to derive a generic correlation function involving a large group of rock types (e.g. carbonates, mudrocks) it is essential to ensure that there are no large gaps across the entire range and all distinct microstructural varieties of each rock type are represented".

In contrast, the as-complete-as-possible description of the rock can lead to the distinction between rock classes and to a considerable reduction in the scattering [SCH 75].

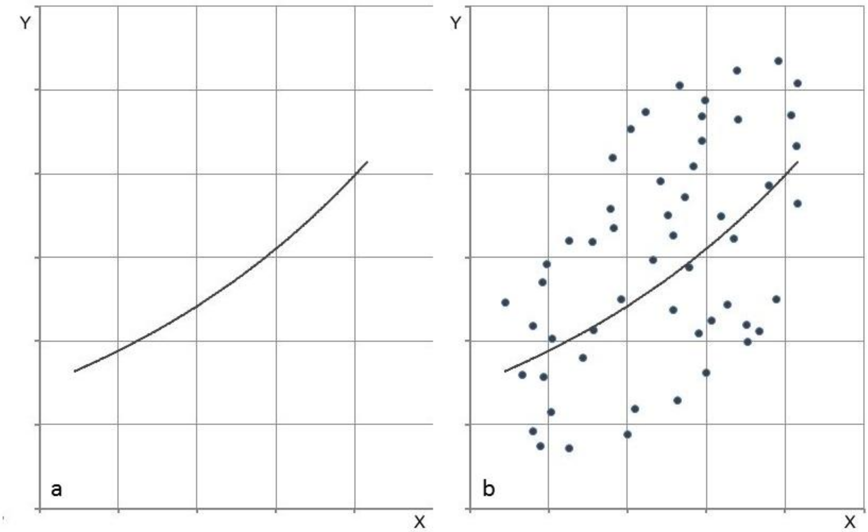

Figure 12.1. *The regression curve or its equation (a) sometimes does not reflect the scattering of the results, as shown in (b), mainly due to geological conditions that are not taken into account periodically. It is dangerous to trust the (a) model without seeing (b) (adapted from Moroney [MOR 51], cited in [COR 90])*

However, considering the role of constituents and discontinuities, it is possible to improve the value of the interpretation of laboratory or field tests by taking into account the "geological" characteristics.

For a rock material, the correlations between the results of different types of test performed *on the same lithological type*, in order to determine *the same characteristic* (modulus, strength, etc.), are often (but not always) more reliable because they only present different approaches to obtain the required parameter on a given rock.

For rock masses, *in situ* measurements are the best way to obtain the information (if tests are correctly processed). Indirect methods can be used if there is a lack of *in situ* tests or in order to confirm the range of the results.

The "rock" part presented in this book considers three REV scales:

– fundamental properties of intact minerals;

– rock materials (rm);

– rock masses (RM).

12.2. Fundamental properties of intact minerals

For geomechanical purposes, the main properties of minerals such as density, hardness and elastic properties are considered. The strength in itself is kept in the background.

Hardness is usually given by Mohs' arbitrary relative scale. There are other units of hardness based on mechanical measurements. The Vickers hardness is commonly used in mineralogy.

As the Vickers hardness scale is more extended than the Mohs' one, it is able to characterize the minerals more accurately and to make a clear distinction between them.

Therefore, using the Vickers hardness, it is possible to describe (obviously, in an approximate way) the mineralogical composition of a rock using only one figure. This parameter is called the "weighted average hardness" (WAH). Similarly, another parameter is defined using the Vickers hardness of constitutive minerals, that is, the "hardness contrast" (Hc). They are defined as follows [TOU 71b]:

$$WAH = \sum_i c_i \, . Vh_i \qquad [12.1]$$

$$Hc = \sum_i c_i \, . |Vh_i - Vh_b| \qquad [12.2]$$

where Vh_i is the Vickers hardness of the ith mineral in the rock, c_i its proportion and Vh_b is the Vickers hardness of the most present mineral.

The elastic properties considered here are as follows:

– Young's modulus (the elastic modulus, not the tangent one);

– Poisson's ratio;

– seismic wave velocities: compression wave, "sonic", V_p (or V_l) and shear wave V_s (or V_t).

Table 12.1 summarizes the values of density, hardness indices and the four elastic parameters for the major minerals.

Minerals	Density	Hardness		Young's modulus (GPa)	Poisson's ratio	Velocity of seismic waves	
		Mohs	Vickers (kgf/mm^2)			Compression V_p (km/s)	Shear Vs (km/s)
Plagioclase (Albite)	2.62	6.3	700			5.69	
Amphibole	3.25	5.5	650	128.8	0.28	7.21	3.99
Apatite	3.2	**5**	600				
Augite	3.4	5.5	650	143.7	0.24	7.20	4.17
Biotite	3	2.8	90	69.6	0.25	5.13	2.98
Calcite	2.71	**3**	110	81	0.28	6.66	3.39
Chlorite	2.7	2.5	730			6.50	
Corundum	4	**9**	2,080				
Diamond	3.51	**10**	10,000	1,050	0.1–0.2	18	
Dolomite	2.87	3.7	210			7.10	
Epidote	3.3	6.5	730	154.2	0.26	7.42	4.25
Fluorite	3.2	**4**	200			6.03	
Galena	7.58	2.5	80	71.7	0.3	3.58	1.92
Gypsum	2.3	**2**	70				
Hornblende	3.3	5.5	730			7.08	
Magnetite	5.17	6.0	1,100	230.8	0.26	7.41	4.20
Muscovite	2.9	2.8	90	78.9	0.25	5.01	3.36
Olivine	3.3	6.7	820	200.1	0.24	8.40	5.16
Orthoclase feldspar	2.54	**6**	720	67.2	0.27	5.69	3.26
Plagioclase (Oligoclase)	2.7	6.3	700	80.8	0.28	6.26	3.45
Pyrite	5	6.3	1,050			7.41	
Pyroxene	3.5	5.5	650			7.30	
Quartz	2.65	**7**	1,200	96.4	0.2	6.03	4.11
Talc	2.7	**1**	20				
Topaz	3.5	**8**	1,600				

Table 12.1. *Values of physical and elastic properties of main minerals*

Data are obtained from various sources: Taylor [TAY 49], Alexandrov *et al.* [ALE 66], Tourenq [TOU 66], Calembert *et al.* [CAL 81a] and Shuvalov [SHU 88].

It should be noted that measurements on crystals are extremely delicate and that the values given by different authors could slightly differ. The values presented here are a *practical "isotropic" average order of magnitude*.

12.3. Rock material (rm)

rm is composed of minerals of a given size, disposed in a given way (texture, including anisotropy) and including different voids: pores and (micro)fissures.

The behavior of rm thus depends on these parameters. Correlations between mechanical and physical properties must take all of them into account.

Ideally, a mechanical property of an rm can be expressed as:

$$P = a\,.Tm + b\,.Ts + c\,.Tt + d\,.Td + \ldots \qquad [12.3]$$

where:

– P is the considered property;

– Tm, Ts, Tt, Td, etc. are the values of several parameters: mineral composition, size, texture, voids (pores and fissures), etc.;

– a, b, c, d, etc. are empirical or theoretical factors.

This expression is rarely possible, but the following example shows how correlations can be made.

Le Berre [LEB 75] established a relationship between the traction strength (measured by the Brazilian test) and the mineralogical characteristics of a series of silicate rocks:

$$\log(R_t - 40) = 0.7\,\textstyle\sum_i c_i \log Vh_i - 0.4\log(\emptyset_m) - 0.29\,n_f + 1.18 \qquad [12.4]$$

where R_t is the traction strength in kgf/cm^2 and, for the ith mineral, Vh_i is the Vickers hardness in kgf/mm^2, c_i is the volumic proportion, $ø_m$ is the average grain size in microns and n_f is the porosity induced by fissures.

In this chapter, some correlations between parameters and lithology and mechanical parameters of rock materials (cores or aggregates) will be presented. As this chapter does not aim to explain in detail, only some main parameters will be considered.

12.3.1. *UCS*

12.3.1.1. *Relationship between UCS and mineralogical composition*

For “intact” rocks, that is, in which no void (mostly no microfracture) is present, the strength depends on the mineralogical composition and the texture, especially when phyllitic materials are present.

An attempt to establish the correlation between mineralogical composition and UCS was made by Polo-Chiapolini [POL 74] and Calembert *et al*. [CAL 80, CAL 81b] for several rocks types (Figure 12.2). All experimental factors were identical (orientation of load vs. stratification, size of samples, loading rate, moisture content, device, etc.). The mineralogical composition is given by the parameter WAH that clearly distinguishes the different lithological compositions.

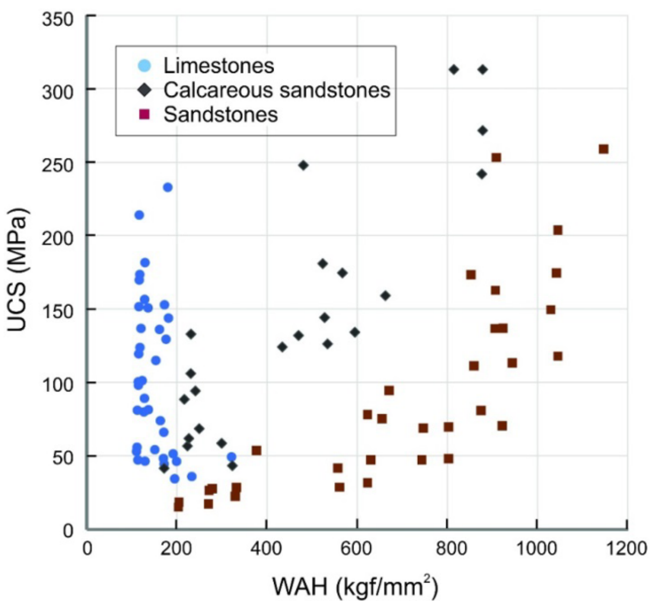

Figure 12.2. *Correlation between UCS and WAH (adapted from [CAL 81b]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

For limestones, all the tested samples are composed of only calcite, and the UCS values are not correlated with mineralogy. The UCS varies between 25 and 230 MPa, which is the same value as that of the WAH. This is explained by another geological factor: the intensity of microfissuration.

For carbonate sandstones, the increase in the amount of quartz shows a quasi-linear relationship with UCS, except for small values where the effect of microfissuration is important.

For sandstones with "clayey" cement, a nonlinear relationship exists; in a domain limited by two curves, the difference between them depends on the intensity of microfissuration [POL 74].

So, even if mineralogy is an important factor, its influence on the UCS can be hidden by other geological or experimental factor.

12.3.1.2. *Relationship between UCS and microfissuration*

The intensity of microfissuration can be estimated by the "continuity index", CI [TOU 71a], based on Hill's theory [HIL 63]: an elastic constant of a multi-crystalline material in an (weighted) average (in practice, arithmetic) of those of its constituents. For instance,

$$V_{p\ theoretical} = \sum_i c_i \,.\, V_{p\ i} \qquad [12.5]$$

where $V_{p\ theoretical}$ is the velocity of compression waves deduced from Hill's theory.

$V_{p\ I}$ is the velocity of compression waves of the ith constitutive mineral and c_i its proportion.

The "continuity index", CI, is the ratio between the theoretical value and the measured one:

$$CI = 100 \,.\, \frac{V_{p\ measured}}{V_{p\ theoretical}} \qquad [12.6]$$

where $V_{p\ measured}$ is the measured velocity of compression wave in the sample.

CI indicates the proportion of pores and fissures in the total porosity and thus the degree of microfissuration:

$$CI = 100 - 1.6\, n_p - 22\, n_f \qquad [12.7]$$

where n is the total porosity, n_p is the porosity due to pores and n_f is the porosity due to fissures.

12.3.1.3. *Relationship between UCS and traction strength*

A good way to check whether the results of compression (UCS) and traction (Rt) tests are coherent is to use the Griffith criterion [GRI 24] in a large sense. If

$$\frac{UCS}{R_t} \approx 8 \text{ to } 12 \qquad [12.8]$$

it is highly probable that there is no discrepancy.

Using a large database of French rocks, Serratice and Durville [SER 97] obtained an average ratio of 10.

12.3.1.4. *Relationship between UCS and porosity*

For high-porosity carbonate rocks (chalks and high porosity limestones), in the case of purely porous materials (no microfissuration), the UCS is correlated with a dry density, as shown in Figure 12.3. After eliminating the values which are obviously two low, the relation is written as:

$$UCS = 29\, \rho - 36\ (R^2 = 0.64) \qquad [12.9]$$

with UCS in MPa and ρ in 10^3 kg/m^3.

However, for a given density, the UCS value could vary in a ratio of 1 to 2.

If all types of carbonate rocks are considered, with a density up to 2.7 10^3 kg/m^3, the relation, shown in Figure 12.4, becomes

$$UCS = 0.08\, e^{2.88\, \rho}\ (R^2 = 0.90) \qquad [12.10]$$

Although this correlation looks fine (R^2 = 0.90), caution has to be taken for high values of density because, as expected, for high densities, the effect of (micro)fissuration becomes dominating.

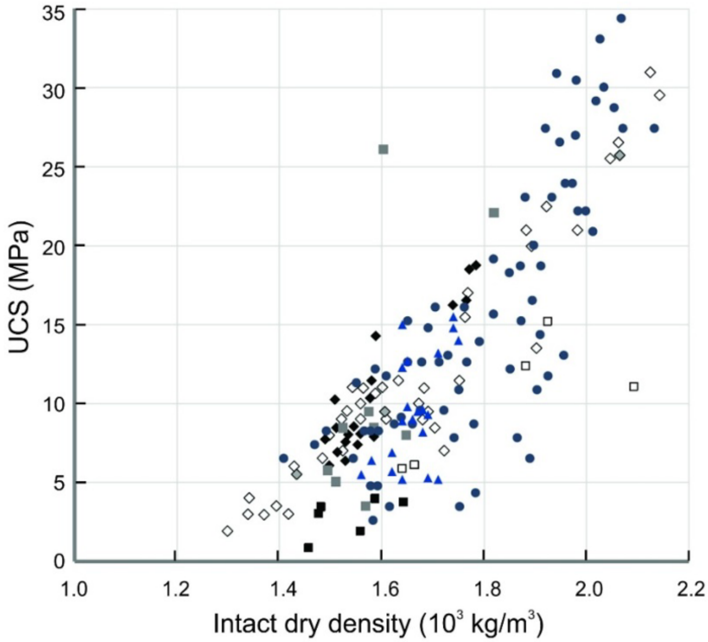

Figure 12.3. *Relationship between UCS and dry density, ρ, of an intact rock for six types of UK and France chalks (adapted from Duperret* et al. *[DUP 05]) and with the addition of data of carbonate rocks (blue-gray circles) from Datarock (database on rock and aggregate laboratory tests by the Laboratoire central des Ponts et Chaussées [DUR 91]) – density limited to 2.1. For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

For other rocks, the relationship is less evident [DEE 66], which results from the major importance of microfissuration on the strength of a rock while the amount of porosity induced by microfissuration remains small.

However, some data are reported by [FJÆ 08].

For sandstone, the upper bound of the UCS (in MPa) is [PLU 94]

$$UCS = 357\,(1 - 2.8\,\emptyset) \qquad [12.11]$$

where ø is the porosity.

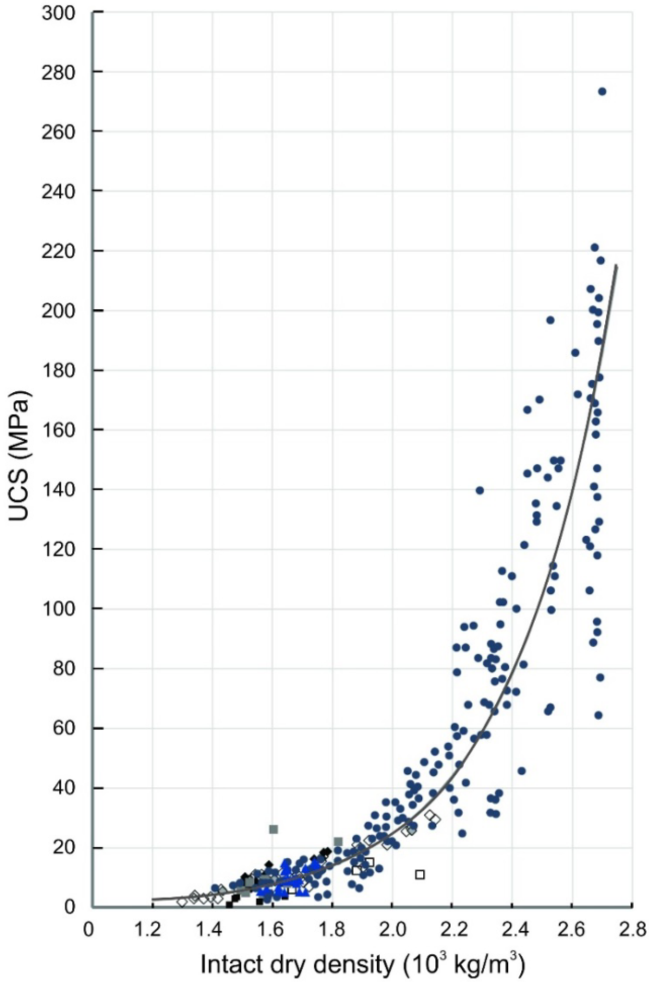

Figure 12.4. *Relationship between UCS and dry density, ρ, for carbonate rocks (adapted from Duperret et al. [DUP 05] and Serratice and Durville [SER 97]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

For North Sea chalk, Andersen, Havmøller, Foged and Engstrøm, cited in [FJÆ 08], gave (UCS in MPa):

$$UCS = 174\, e^{-7.57\, \emptyset} \quad [12.12]$$

For shales, Lashkaripour and Dusseault [LAS 93] gave the following relation:

$$UCS = 193\ \emptyset^{-1.14} \quad [12.13]$$

with UCS in MPa and ø in %.

12.3.1.5. *Relationship between UCS and E*

The UCS is well correlated with E_t (tangent modulus). This relation reflects the fact that the deformation at rupture is more or less constant with an order of magnitude of 2.10^{-3} to 4.10^{-3}.

This value is in accordance with the values of the modulus ratio (MR = E_t/UCS of intact rock) given by Deere [DEE 68] and Palmtröm and Singh [PAL 01] that ranges from 200 to 500, with an average of about 400.

The relationship between Deere and Miller [DEE 66], for all types of lithologies, is in imperial units (UCS in 10^3 psi and E_t in 10^6 psi), which is given by:

$$UCS = 0.0033.E_t - 2.886 \quad [12.14]$$

As the independent term is small, it can be neglected and the relation can be rewritten (in IS units) as

$$E_t = 0.286\,.UCS \quad [12.15]$$

with E_t in GPa and UCS in MPa.

For carbonates, the relation is [SAC 90]

$$E_t = 0.3752\,.UCS + 4.428 \quad [12.16]$$

with E_t in GPa and UCS in MPa.

For chalk, Monjoie and Schroeder [MON 89] obtained

$$E_t = 0.483\,.UCS - 0.39 \quad [12.17]$$

with E_t in GPa and UCS in MPa.

12.3.1.6. *Relationship between UCS and indirect measurements*

Other methods are commonly used to determine faster strength of the rm.

The two main ones are the Point load strength index (Franklin test), $I_{s,}$ and the Schmidt hammer rebound, N. Their values are more or less well correlated with UCS.

12.3.1.6.1. UCS–I_s

The basic relation between I_s and UCS (both in MPa) is

$$UCS = f \, . I_{s(50)} \quad [12.18]$$

where "f" is an experimental factor and $I_{s\,(50)}$ indicates that the measurement is done on NX core (54 mm) samples. Occasionally, a constant term is added to the equation.

In the case of a different diameter of the tested sample, a corrective factor must be applied [ISR 85]:

$$I_{s(50)} = I_s . \left(\frac{De}{50}\right)^{0.45} \quad [12.19]$$

where D_e is the (equivalent) diameter of the tested sample, in mm.

Torabi *et al*. [TOR 10] identified 42 different values in 30 publications. For the value of "f", the following will be retained:

– Broch and Franklin [BRO 72] and Bieniawski [BIE 74] proposed the value of *24*;

– Singh *et al*. [SIN 12] obtained somewhat different correlations. The proposed value varies from *21 to 24* for hard rocks ($I_{s(50)} > 5$ MPa) and from *14 to 16* for soft rocks ($I_{s(50)} < 5$ MPa);

– Deere and Miller [DEE 66] found

$$UCS = 21.2 \, I_{s(50)} + 3.179 \quad [12.20]$$

In imperial units (10^3 psi), a value of *21.2* for "f" was obtained, neglecting the constant term.

As with the others, these correlations should be applied carefully, considering the effect of the sample size, among other things.

12.3.1.6.2. UCS–N

Schmidt's hammer rebound is a fast and inexpensive way to determine the strength of a material.

Torabi *et al*. [TOR 10] identified 22 different relationships between N and UCS, some of which are linear or exponential, take the rock density into account, are applicable to all lithologies or are dedicated to a specific one. Some are presented in Figure 12.5.

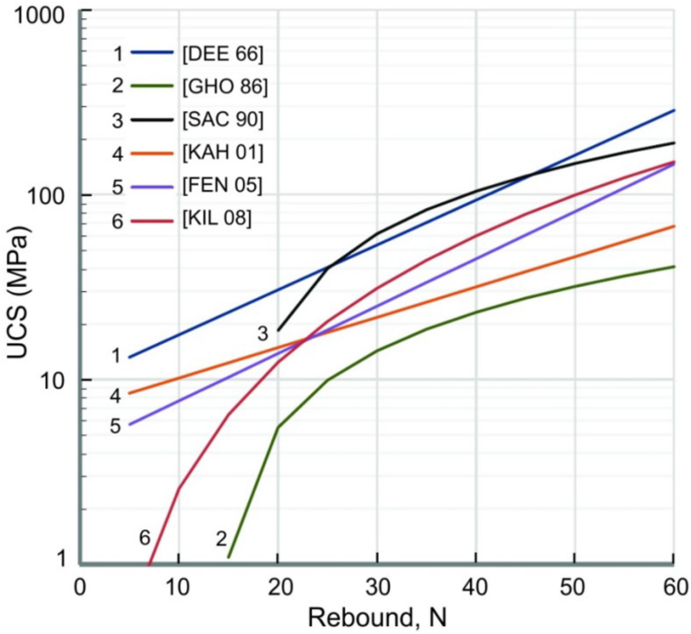

Figure 12.5. *Some correlations exist between UCS and rebound N, cited in [TOR 10]. For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

This amount of relations requires caution when using them. This has been underlined in the ISRM suggested method [AYD 15].

The relation that seems to be mainly used is the oldest one [DEE 66]:

$$UCS = 10^{(0.00014\,.\gamma.N+3.16)} \qquad [12.21]$$

where the UCS is expressed in psi and γ, and the weight per unit of volume, in pcf.

In SI units, for $\gamma = 27$ kN/m^3, with E_t in GPa and UCS in MPa, the relation becomes

$$UCS = 10\,.\,e^{0.056.N} \qquad [12.22]$$

12.3.2. *Abrasiveness*

Practically, the different methods for measuring abrasiveness are correlated as follows:

Description	Abrasiveness index		
	AIN value	ABR value	FPMs value
Extremely abrasive	>4	> 2,000	> 400
Very abrasive	2–4	1,500–2,000	150–400
Abrasive	1–2	1,000–1,500	50–150
Low abrasiveness	0.5–1	500–1,000	5–505
Very low abrasiveness	< 0.5	0–500	0–5

Table 12.2. *Classes of hardnesses and index values according to different methods [AFT 04]. AIN (CAI): CERCHAR-INERIS abrasiveness, ABR (LAC): LCPC abrasivity and FPMs (University of Mons) abrasivity [TSH 97]*

The relationship between AIN and ABR is more accurately described by the diagram in Käsling and Thuro [KÄS 10] (Figure 12.6). The correlation obtained by these authors is

$$ABR = 273\,.\,AIN \qquad [12.23]$$

A linear regression made on the whole data gives:

$$ABR = 277\,.\,AIN - 25\ (R^2 = 0.82) \qquad [12.24]$$

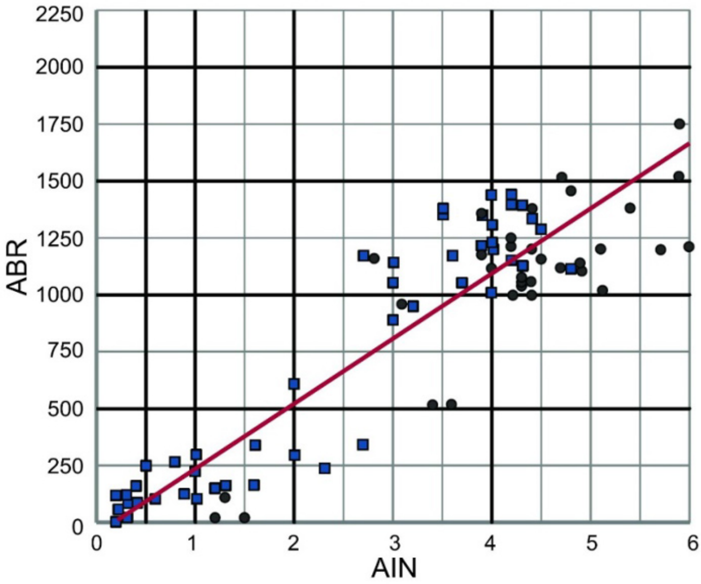

Figure 12.6. *CERCHAR and LCPC abrasivity index results – adapted from [KÄS 10] (squares: data from [BÜC 95], circles: data from [KÄS 10], in red linear regression [12.24]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

Other abrasion measurement methods exist in northern Europe [MAC 17] and Russia [OPA 15] but are not considered here.

The abrasiveness is linked to the mineralogical characteristics and expressed as a "Wear Factor", F_{schim}, defined by Schimazeck and Knatz [SCH 70a], cited in [CAL 74], as

$$F_{schim} = \frac{t}{100} \,.\, \emptyset \,.\, R_t \,.\, 1.4 \qquad [12.25]$$

where:

F_{schim} is the Schimazeck Factor, in N/mm;

t is the quartz or abrasive mineral (feldspars) content, in %;

ø is the mean diameter of grains, in mm;

R_t is the traction strength, in MPa (N/mm^2);

1.4 is a corrective factor for the grain dimensions.

12.3.3. *Attrition*

Many tests exist for testing attrition. In fact, almost every country has its own methods and standards.

However, the two commonly known tests are Los Angeles (LA), aggregate impact value (AIV) and Micro-Deval (MD or MDe) followed by several ways of dynamic and static fragmentation and other wear tests, including durability tests (for example, magnesium and sodium sulfate durability tests).

An extensive literature review (16 authors from 1991 to 2006) supported by experimental tests has been made by Cuelho *et al*. [CUE 08]. From this, it appears that the relationship between LA and MD is rather poor (Figure 12.7) when simultaneously considering all types of rocks.

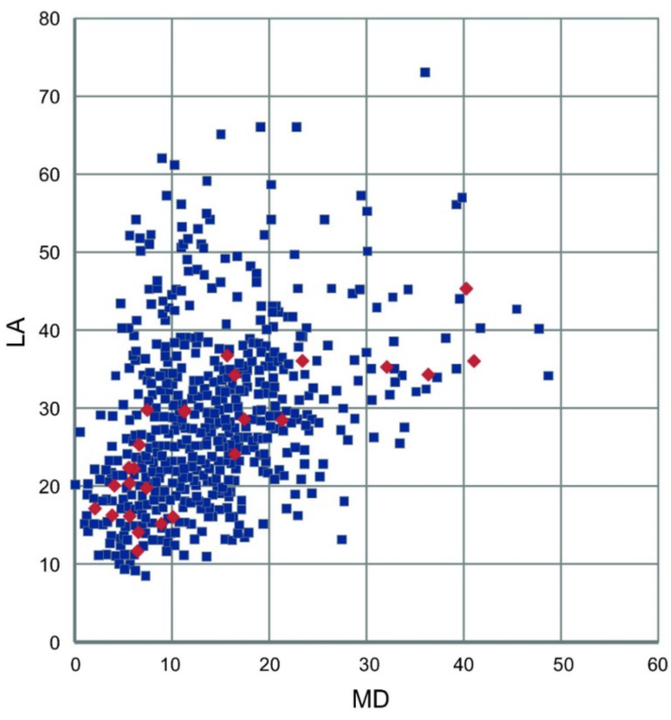

Figure 12.7. *Relationship between LA and MD from more than 500 experimental results – adapted from the results presented by Cuelho* et al. *[CUE 08] (blue: literature results, red: results from Cuelho* et al.*). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

The linear regression on this set of points is

$$LA = 0.48\,MD + 22 \qquad [12.26]$$

but with a very low determination coefficient, R^2, less than 0.2.

On the other hand, when lithology is taken into account, the relation LA/MD becomes a little more reliable. Benediktsson [BEN 15] made a distinction between the six types of tested rocks (Figure 12.8).

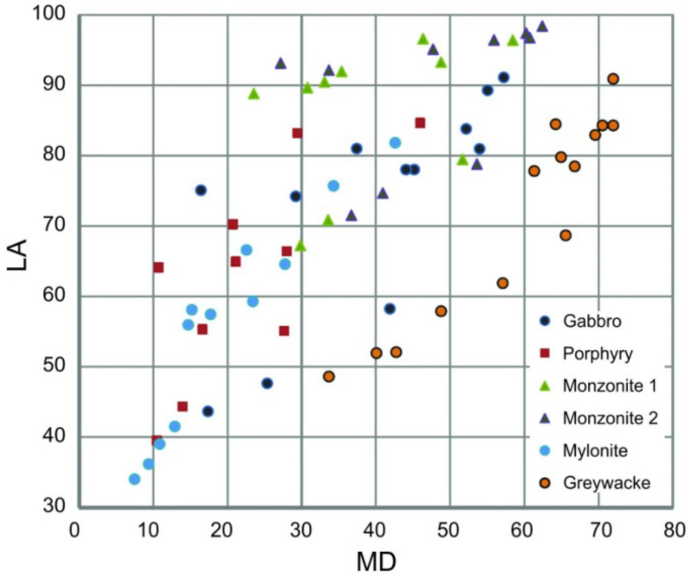

Figure 12.8. *Relationship between LA and MD for several types of rocks (adapted from the results presented by Benediktsson [BEN 15]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

From these data, relationships can be established.

Monzonites, coarse-grained rocks, have a small correlation coefficient.

For other lithologies, the relations are as follows:

Mylonite:

$$LA = 1.36\,MD + 29\ (R^2 = 0.88) \qquad [12.27]$$

Graywacke:

$$LA = 1.10\ MD + 8.2\ (R^2 = 0.89) \qquad [12.28]$$

All types of igneous rocks except monzonites:

$$LA = 0.9\ MD + 39\ (R^2 = 0.68) \qquad [12.29]$$

The average value of the angular coefficient (around 1) is similar to the one which could be intuitively deduced from the diagram of Figure 12.7.

The relationship between AIV and LA is given by Senior and Rogers [SEN 91] (Figure 12.9):

$$LA = 1.36\ AIV - 4\ (for\ AIV > 10)\ (R^2 = 0.64) \qquad [12.30]$$

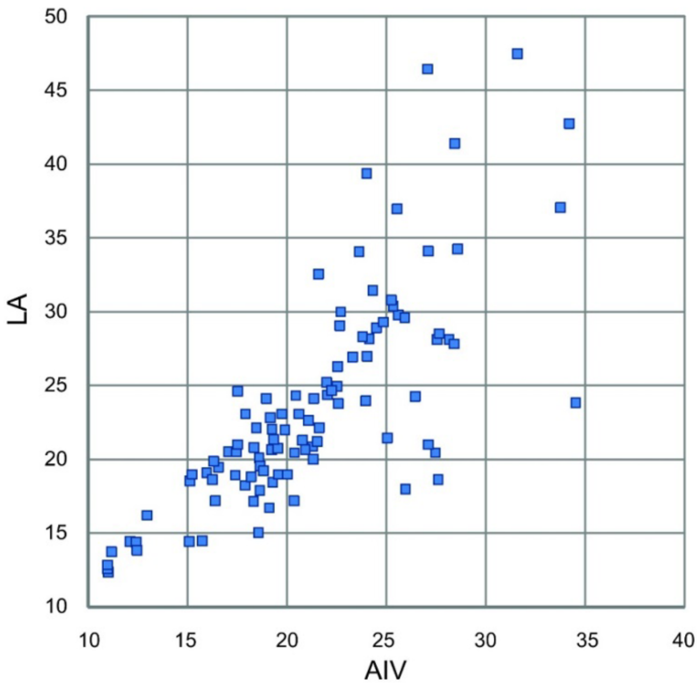

Figure 12.9. *Relationship between AIV and LA for several types of rocks (adapted from the results presented by Senior and Rogers [SEN 91])*

The same authors [SEN 91] present a relationship between magnesium sulfate soundness (MSS) and MD (Figure 12.10):

$$MD = 0.63\ MSS + 11.3\ \mathrm{R}r^2 = 0.72) \qquad [12.31]$$

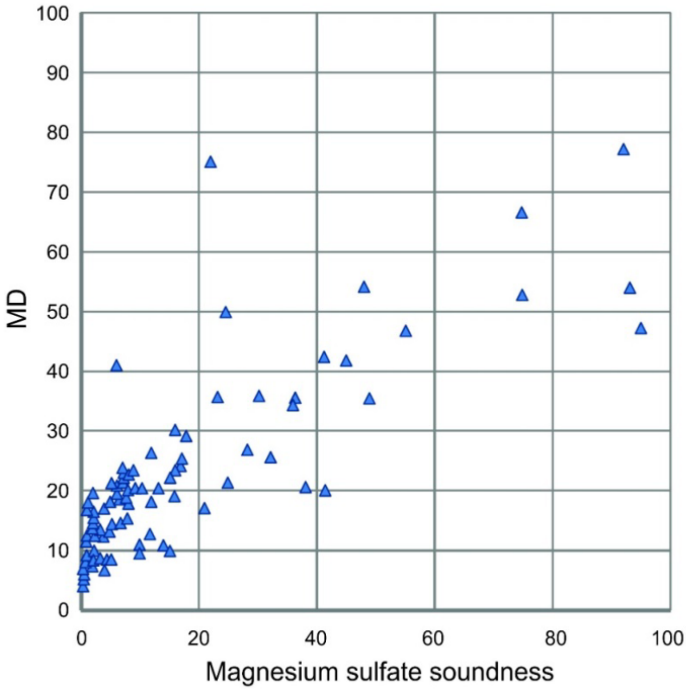

Figure 12.10. *Relationship between MSS and MD for several types of rocks (adapted from the results presented by Senior and Rogers [SEN 91])*

12.3.4. *Polished stone value (PSV)*

Apparently, no direct correlation with other parameters exists for PSV.

Nevertheless, a rather good correlation exists with the mineralogical composition expressed by Hc, the hardness contrast and the average weighted hardness WAH.

For a set of various rocks (limestones, sand and siltstone), Tourenq and Fourmaintraux [TOU 71b] proposed (Figure 12.11) the following relation:

$$PSV = 0.075\,Hc + 35 \qquad [12.32]$$

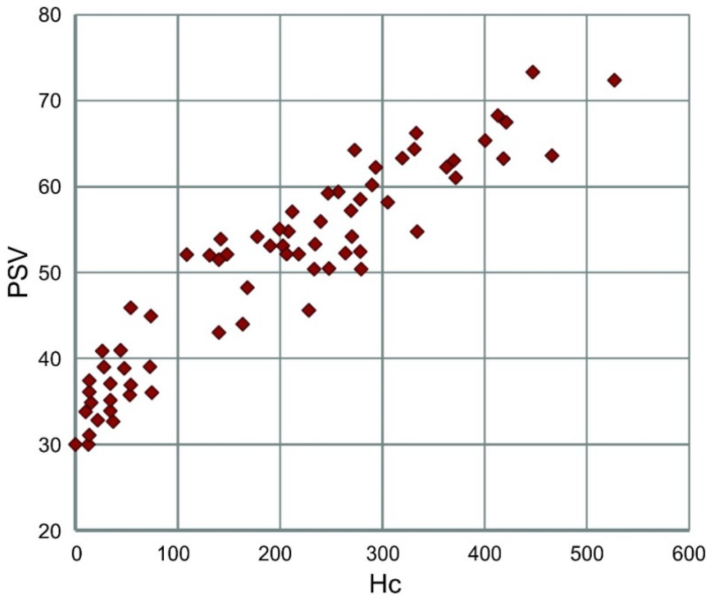

Figure 12.11. *Relationship between PSV and Hc (adapted from [TOU 71b])*

Another relation is given by Fourmaintraux [FOU 70], including the WAH:

$$PSV = a + b\,.WAH + c\,.Hc \qquad [12.33]$$

with a, b and c being experimental parameters.

Table 12.3 lists the values of a, b and c:

– for all kinds of rocks: Fourmaintraux [FOU 70];

– for limestones: Archimbaud and Tourenq [ARC 74];

– for several Belgian rocks (limestone to sandstone): Schroeder and Vanden Eynde [SCH 78], [VAN 83].

Description	Empirical factor values		
	a	b	c
All types of rocks [FOU 70]	30	0.015	0.07
Limestones [ARC 74]	33	0.009	0.062
Belgian rocks [SCH 78], [VAN 83]	30	0.02	0.03

Table 12.3. *Values of the empirical factors a, b and c*

12.4. Rock masses (RMs)

Rock masses (RMs) are composed of rock materials (rm) separated by discontinuities (families of discontinuities).

The behavior of the RM is determined by the characteristics of the different constituting rm (considered in the previous section) and several discontinuities affecting the RM:

– when the amount of discontinuity families is small, the stability analyses are based on geometrical considerations, and the most important mechanical parameter is the shear strengths of the different discontinuity families;

– on the other hand, and/or for practical problems (tunnels, slopes, etc.), the RM (or each distinct RM part of the total studied area) can be considered as a homogeneous whole. The RM has thus global mechanical properties that can be used for calculations and modeling with the soil mechanic tools or with empirical methods.

This characterization of RM is made by the classification systems that describe the geological (l.s.), and, for some of them, other geotechnical and hydrogeological conditions. The RM quality is then described by only one "global" figure. Mechanical properties of the RM are thus calculated from these parameters.

12.4.1. *Shear strength of discontinuities*

The major characteristic of discontinuities is their shear resistance, depending on their geological conditions. The compression strength is not taken into account. If it must be considered in computation, the traction strength is generally considered to be equal to zero.

Shear resistance is generally based on the Coulomb criterion where, for safety reasons, cohesion is often neglected.

12.4.1.1. *Barton–Bandis approach*

The internal friction angle is measured *in situ* or in the laboratory, and can also be calculated from some geological parameters [BAR 90]:

$$\tau = \sigma_n \,.\, tan\left[JRC \,.\, log_{10}\left(\frac{JCS}{\sigma_n}\right) + \varphi_r\right] \qquad [12.34]$$

where:

JRC is the joint roughness coefficient [BAR 73, BAR 76] determined by abacus or direct measurements;

JCS is the joint wall compressive strength;

φ_r is the residual friction angle [BAR 77]:

$$\varphi_r = (\varphi_b + 20°) + 20 \,.\, \left(\frac{N_{fracture}}{N}\right) \qquad [12.35]$$

φ_b is measured by the tilt test on a freshly sawed surface (normal range 25–35°);

N is the Schmidt hammer rebound on a dry non-weathered surface;

$N_{fracture}$ is the Schmidt hammer rebound on a fracture surface.

Another expression for φ_r is

$$\varphi_r = \varphi_b + \psi \qquad [12.36]$$

where Ψ is the dilatancy angle (range 0° to 10°).

The dilatancy angle can be estimated by [SET 09]

$$\psi = 0.8 \,.\, \delta_{max} \qquad [12.37]$$

where δ_{max} is the maximum slope of asperities on an average fracture plane.

12.4.1.2. *RMR correlation*

The frictional strength of discontinuities can be inferred from RMR [BIE 89] or rather from the component "Rating for Conditions of Discontinuities", J_{Cond89} (Table 12.4).

J_{Cond89}	30	25	20	10	0
Completely dry	45	35	25	15	10
Damp	43	33	23	13	<10
Wet	41	31	21	11	<10
Dripping	39	29	19	10	<10
Flowing	37	27	17	<10	<10

Table 12.4. *Frictional shear strength of discontinuities (°)*

12.4.2. *RM classification systems*

More than 30 different classification systems have been reported by several authors, among them are Palmström [PAL 95], Hack [HAC 02], Edelbro [EDE 04], Aksoy [AKS 08] and Hashemi *et al*. [HAS 10].

This huge amount of RM classification systems proves that the problem is not solved and enhancements of the classification systems are still in progress.

Nevertheless, some systems are more widely used. Only the important ones will be considered here even though many others are useful[3]:

– RQD (Rock Quality Designation): Deere and Miller [DEE 66];

– RMR (Rock Mass Rating): Bieniawski [BIE 73], [BIE 89];

– Q (Quality index): Barton *et al*. [BAR 74], [NGI 15];

– GSI (Geological Strength Index): Hoek *et al*. [HOE 95], [HOE 02], [MAR 00]. GSI is not only a classification (description) of the RM but also the main element of the Hoek and Brown criterion [HOE 97], [HOE 02], [HOE 07b].

3 a.o.: RMS, Rock Mass Strength, Stille *et al*. [STI 82]; SMR, Slope Mass Rating, Romana [ROM 93]; RMi, Rock Mass index, Palmström [PAL 95]; SSPC, Slope Stability Probability Classification, Hack [HAC 97].

The theory and methods of determination of RQD, RMR, Q and GSI will not be presented here.

12.4.2.1. *Note on RQD*

RQD is the commonly used basic quantification of the amount of discontinuities, even though it has to be used with caution, especially when the rock is highly fractured [PEL 17] and/or when the core size differs from NX standard samples.

When no coring has been made, RQD can be calculated from the outcrop observations (working face, slope face, etc.) [PRI 76]:

$$RQD = 100\, e^{0.1\,\lambda}\,(0.1\,\lambda + 1) \qquad [12.38]$$

where λ is the average number of discontinuities per meter.

Another relation is given by Palmström [PAL 05]:

$$RQD = 110 - 2.5\,J_v \qquad [12.39]$$

where J_v is the volumetric joint count, a measure of the number of joints within a unit volume of rock mass [PAL 95]:

$$J_v = \sum_i 1/S_i \qquad [12.40]$$

where S_i is the joint spacing in meters for the *i*th joint set.

12.4.2.2. *Correlations between RM classification indices*

12.4.2.2.1. Correlation between RMR and Q

The correlation between RMR and Q is the subject of many publications. Overall, there are more than 20 correlations. Hashemi *et al.* [HAS 10] presented 11 of them; Palmström and Broch [PAL 06] added their own data and some data from Bieniawski [BIE 84] and Jethwa [JET 81].

A few correlations that are different from them are presented in Figure 12.12, as well as in [PAL 06].

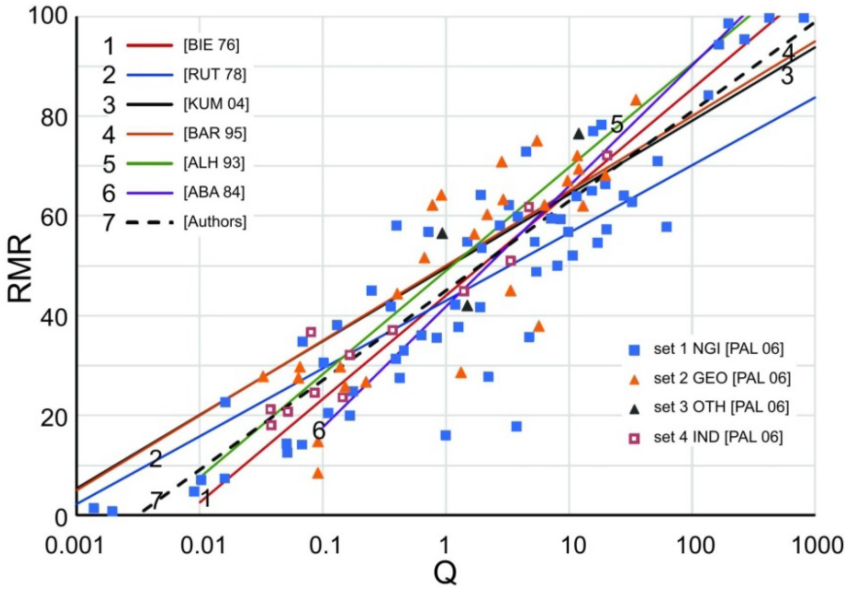

Figure 12.12. *Correlations between RMR and Q. For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

The equations corresponding to the presented relations are as follows:

Bieniawski [BIE 76]:

$$RMR = 9\ ln(Q) + 44 \qquad [12.41]$$

Rutledge and Preston [RUT 78]:

$$RMR = 5.9\ ln(Q) + 43 \qquad [12.42]$$

Kumar *et al*. [KUM 04]:

$$RMR = 6.4\ ln(Q) + 49.6 \qquad [12.43]$$

Barton [BAR 95]:

$$RMR = 15\ log(Q) + 50 \qquad [12.44]$$

Al-Harthi [ALH 93]:

$$RMR = 9\ ln(Q) + 49 \qquad [12.45]$$

Abad *et al.* [ABA 84]:

$$RMR = 10.5\, ln(Q) + 41.8 \quad [12.46]$$

A least-square logarithmic regression on the whole of the presented data has been added, where the equation is given by:

$$RMR = 7.8 \ln(Q) + 44.7\ (R^2 = 0.79) \quad [12.47]$$

12.4.2.2.2. Correlation between RMR and GSI

Hoek and Diederichs [HOE 06] proposed that:

$$GSI = RMR_{89} - 5 \quad [12.48]$$

In [SET 09], the RMR_{89} index is replaced in the same equation by RMR'_{89}, that is, RMR_{89} without the adjustment function of the orientations of discontinuities and with the score for the "water conditions" being equal to 5.

Cebalos *et al.* [CEB 14] concluded that the relationship comprises the following two values:

$$GSI = RMR_{89} + 5 \quad [12.49]$$

$$GSI = RMR_{89} - 15 \quad [12.50]$$

In addition, these authors made a distinction between lithological types:

Igneous:

$$GSI = 1.08\, RMR_{89} - 10.44 \quad [12.51]$$

Metamorphic:

$$GSI = 0.95\, RMR_{89} - 10.44 \quad [12.52]$$

Sedimentary:

$$GSI = 1.30\, RMR_{89} - 20.19 \quad [12.53]$$

12.4.2.2.3. Correlation between GSI, RMR and RQD

GSI (written GSI_{2013} by Pells *et al*. [PEL 17]) is correlated [HOE 13] with a part of the RMR and RQD definitions [HOE 13]:

$$GSI = 1.5\, J_{Cond89} + 0.5\, RQD \qquad [12.54]$$

where J_{Cond89} is the joint condition rating of the RMR classification [BIE 89].

12.4.2.3. *RM deformability*

12.4.2.3.1. RQD

There is no direct relationship between E_{mass} and RQD. Nevertheless, Coon and Merrit [COO 70], cited in [DEE 89], gave the relationship with RMR, which they call the "modulus ratio" (written in lower case and which should not be confused with the Modulus Ratio, MR, used by other authors in different contexts) defined as:

$$modulus\ ratio = \frac{E_{mass\ (in\ situ)}}{E_{core\ (lab)}} \qquad [12.55]$$

of course, for an RM composed by only one rm!

These authors also indicate the relationship with the "velocity index", which is defined as the square of the ratio between velocities of compression waves measured, respectively, *in situ* (MR) and on samples, in the laboratory (mr) (Table 12.5).

$$Velocity\ Index = \left(\frac{V_{p\ in\ situ}}{V_{p\ on\ sample}}\right)^2 \qquad [12.56]$$

RQD	RM quality	Velocity index	Modulus ratio
0–25	Very Poor	0–0.20	< 0.20
25–50	Poor	0.20–0.40	< 0.20
50–75	Fair	0.40–0.60	0.20–0.50
75–90	Good	0.60–0.80	0.50–0.80
90–100	Excellent	0.80–1.00	0.80–1.00

Table 12.5. *Velocity index and "modulus ratio" function of RQD [COO 70]*

A more precise estimation of the RM modulus was given by Zhang and Einstein [ZHA 04] in [ZHA 16]:

$$modulus\ ratio = 10^{0.0186\ RQD - 1.91} \quad [12.57]$$

12.4.2.3.2. RMR

There are many relationships between E_{mass} and RMR. Hoek and Diederichs [HOE 06] presented eight of them. In Figure 12.13, the data, some of the relations presented in [HOE 06] and a relationship from data with RMR > 50 are shown. E_{mass} is expressed in GPa.

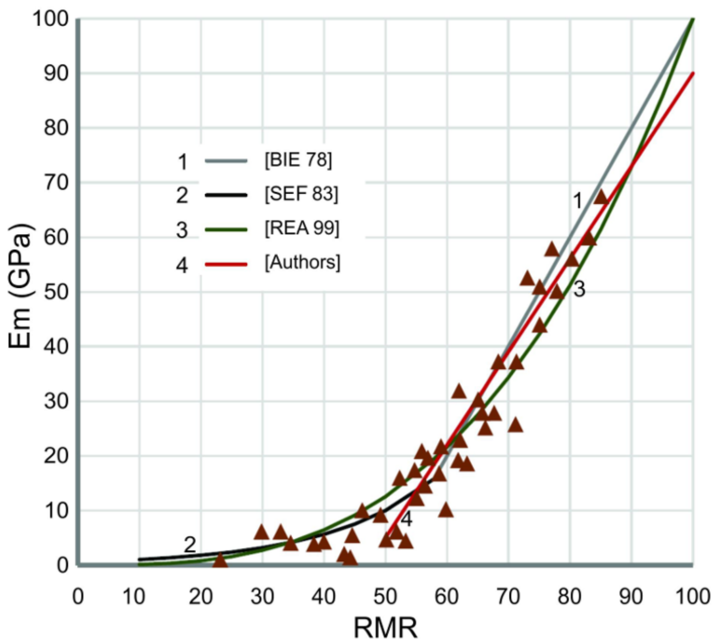

Figure 12.13. *Some relationships between RMR and E_{mass} (Em) (partially adapted from [HOE 06]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

The equations corresponding to these graphs are as follows:

Bieniawski [BIE 78]:

$$RMR > 58\ \ E_{mass} = 2\ RMR - 100 \quad [12.58]$$

Sefarim and Pereira [SEF 83]:

$$RMR < 58 \; E_{mass} = 10^{\frac{RMR-10}{40}} \quad [12.59]$$

Read *et al*. [REA 99]:

$$E_{mass} = 0.1 \left(\frac{RMR}{10}\right)^3 \quad [12.60]$$

For RMR > 50, the authors of this book established that:

$$E_{mass} = 1.7\, RMR - 80 \; (R^2 = 0.90) \quad [12.61]$$

12.4.2.3.3. Q

Using the normalized index, Q_c [BAR 02], Q is defined as:

$$Q_c = Q \, \frac{\sigma_c}{100} \quad [12.62]$$

where σ_c is the uniaxial compressive strength (UCS) in MPa

The relation is given by [BAR 02]:

$$E_{mass} = 10 \, Q_c^{\frac{1}{3}} \quad [12.63]$$

or it is given by [BAR 92], cited in [HOE 95]:

$$E_{mass} = 25 \, log(Q) \quad [12.64]$$

where E_{mass} is expressed in GPa.

The deformation modulus of the RM can also be calculated from the value of the compression sonic wave velocity Vp, which is correlated with Q.

Attention! The relation Q/Vp assumes that V_p is > 3.5 km/s, which is often the case for igneous RM but seldom for sedimentary RM.

Using Q [BAR 91], it is valid only for a hard rock:

$$V_p = 3.5 + \log Q \quad [12.65]$$

Using Qc [BAR 02], it is valid for all types of rocks:

$$V_p = 3.5 + \log Q_c \qquad [12.66]$$

$$E_{mass} = 10\,.\,10^{\frac{V_p - 3.5}{3}} \qquad [12.67]$$

with E_{mass} in GPa and V_p in km/s.

If V_p is calculated from field measurements, the relation E_{mass}/V_p remains significant.

12.4.2.3.4. GSI

The relation between the modulus of the RM and E_{mass} (in GPa) is given by Hoek [HOE 02].

For $\sigma_{ci} \leq 100$ MPa:

$$E_{mass} = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}}\,.\,10^{\frac{GSI-10}{40}} \qquad [12.68]$$

For $\sigma_{ci} > 100$ MPa:

$$E_{mass} = \left(1 - \frac{D}{2}\right).\,10^{\frac{GSI-10}{40}} \qquad [12.69]$$

D is the disturbance factor (depending on the roughness of the excavation process).

Palmström and Singh [PAL 01] gave

$$E_{mass} = 0.5\,MR\,\sigma_i \qquad [12.70]$$

MR is the modulus ratio defined [DEE 68] as

$$MR = \frac{E_i}{\sigma_i} \qquad [12.71]$$

E_i and σ_i being, respectively, the deformation modulus (tangent modulus) and the UCS of the intact rock.

12.4.2.4. *RM strength*

12.4.2.4.1. RQD

Peck *et al.* [PEC 74], cited in [DEE 89], provided the values of the allowable contact pressure for jointed rocks for a 0.5" settlement as a function of RQD, which are given in Table 12.6.

RQD	**Allowable contact pressure for 05" settlement**	
	psi	MPa
100	4,170	28.8
90	2,780	19.2
75	1,660	12.4
50	970	6.70
25	410	2.83
0	140	0.97

Table 12.6. *Allowable contact pressure*

12.4.2.4.2. RMR

A correlation between RMR and c and φ of the RM is given by Bieniawski [BIE 89] using the data, among others, from [SER 83] in Table 12.7.

RMR	100–81	80–61	60–41	40–21	<20
Cohesion c (kPa)	>400	300–400	200–300	100–200	<100
Friction angle (°)	>45	35–45	25–35	15–25	<15
Modulus (GPa)	>56	18–56	5.6–18	1.8–5.6	<1.8

Table 12.7. *Geomechanical classification (adapted from [BIE 89])*

It is also presented in the form of the following equations:

$$\varphi = 5 + \frac{RMR}{2} \quad [12.72]$$

$$c = 5\,.\,RMR \quad [12.73]$$

with ϕ in ° and c in kPa.

12.4.2.4.3. Q

The global UCS of the RM is given by [BAR 02], cited in [EDE 04]:

$$\sigma_{rm} = 5\,\rho\,Q_c^{\frac{1}{3}} \qquad [12.74]$$

In this relation, σ_{rm} is in MPa and ρ, the rock density, is in tons/m^3.

Barton [BAR 07] proposed the following values of the Coulomb criterion parameters, based on the components of the Q definition:

– *cohesive component (c in MPa)*:

$$c = \left(\frac{RQD}{Jn} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right) \qquad [12.75]$$

– *frictional component (φ in)*:

$$\varphi = tan^{-1}\left(\frac{Jr}{Ja} \times \frac{Jw}{1}\right) \qquad [12.76]$$

where:

Jn is the rating for the number of joint sets;

SRF is the rating for faulting, strength/stress ratios, squeezing and swelling;

Jr is the rating for joint surface roughness;

Ja is the rating for joint alteration and discontinuity filling;

Jw is the rating for water softening, inflow and pressure effects.

12.4.2.4.4. GSI and Hoek and Brown criterion

The Hoek and Brown failure criterion [HOE 97], [HOE 02], [HOE 07b] is given as follows:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s\right)^a \qquad [12.77]$$

where:

σ'_1 and σ'_3 are the major stresses;

σ_{ci} is the UCS of intact rock sample;

m_b is a parameter for the rock mass:

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14\,D}\right)} \qquad [12.78]$$

m_i is a characteristic of the intact rock (function a.o. of the UCS);

D is the disturbance factor;

s and a are constants depending on the RM characteristics, GSI and D:

$$s = e^{\left(\frac{GSI-100}{9-3\,D}\right)} \qquad [12.79]$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \qquad [12.80]$$

The compression strength (UCS) of the RM is

$$\sigma_{cm} = \sigma_{ci}\,.\,s^a \qquad [12.81]$$

The traction strength (UCS) of the RM is

$$\sigma_{tm} = -\frac{s\,.\sigma_{ci}}{m_b} \qquad [12.82]$$

For RM with very poor quality (GSI < 25), the Hoek and Brown criterion can be used [HOE 97]:

$$s = 0 \qquad [12.83]$$

$$a = 0.65 - \frac{GSI}{200} \qquad [12.84]$$

The parameters m_b and s can also be calculated from RMR [HOE 88]:

For disturbed rock masses:

$$\frac{m_b}{m_i} = e^{\frac{RMR-100}{14}} \qquad [12.85]$$

$$s = e^{\frac{RMR-100}{6}} \qquad [12.86]$$

For undisturbed or interlocking rock masses:

$$\frac{m_b}{m_i} = e^{\frac{RMR-100}{28}} \quad [12.87]$$

$$s = e^{\frac{RMR-100}{9}} \quad [12.88]$$

The Coulomb criterion can match the Hoek and Brown constitutive law for a given stress interval (see, for instance, [HOE 02], [HOE 07b] or RocLab™).

12.4.2.5. *Hydraulic conductivity of RM*

The hydraulic conductivity K is related to the intensity of fracturation, which can be expressed by RQD, RMR or Q. However, it should be noted that the hydraulic conductivity in fractured media is proportional to the thickness of the power three; therefore, it is subject to changes of two or even three orders of magnitude, even for a tough rock.

12.4.2.5.1. RQD and RMR

El-Naqa [ELN 01] gave correlations between RMR and K and RQD and K, with K in UL (Lugeon units) using data obtained from boreholes (bh) or from field mapping (fm).

1 UL ≈ 10-7 m/s (in hard, jointed, clay-free, rock masses):

$$K = 5.10^6.e^{-0.193\,RMR}\ (R = 0.74)(bh) \quad [12.89]$$

$$K = 3166\,.\,e^{-0.07555\,RMR}\ (R = 0.84)(fm) \quad [12.90]$$

$$K = 177.45\,.\,e^{-0.0361\,RQD}\ (R = 0.64)(bh) \quad [12.91]$$

$$K = 890.9\,.\,e^{-0.0559\,RQD}\ (R = 0.87)(fm) \quad [12.92]$$

Qureshi *et al*. [QUR 14] proposed the following relation with K, in cm/s:

$$K = 0.01382 - 0.003\,.\,ln(RQD) \quad [12.93]$$

After unit conversions, values closer to this are presented in [ELN 01] (bh).

However, as shown in Figure 12.14, the scattering is too important for practically using this formula. The scattering is mainly due to limestones which exhibit a high hydraulic conductivity for RQD = 100. Perhaps it is due to the presence of the karstic phenomenon that increases the hydraulic conductivity strongly but locally.

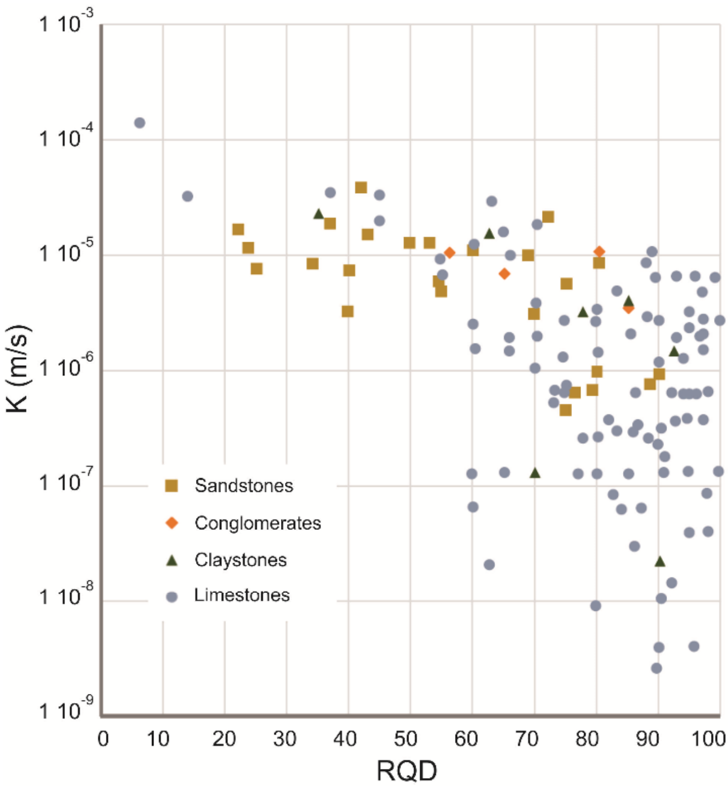

Figure 12.14. *Relationship between hydraulic conductivity, K, and RQD (adapted from [QUR 14]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

If the limestones and two obviously incorrect points are excluded, the whole of the 61 points from both publications [ELN 01] and [QUR 14] gives a rather satisfactory correlation that allows the comparison of the results obtained by different methods. This is shown in Figure 12.15.

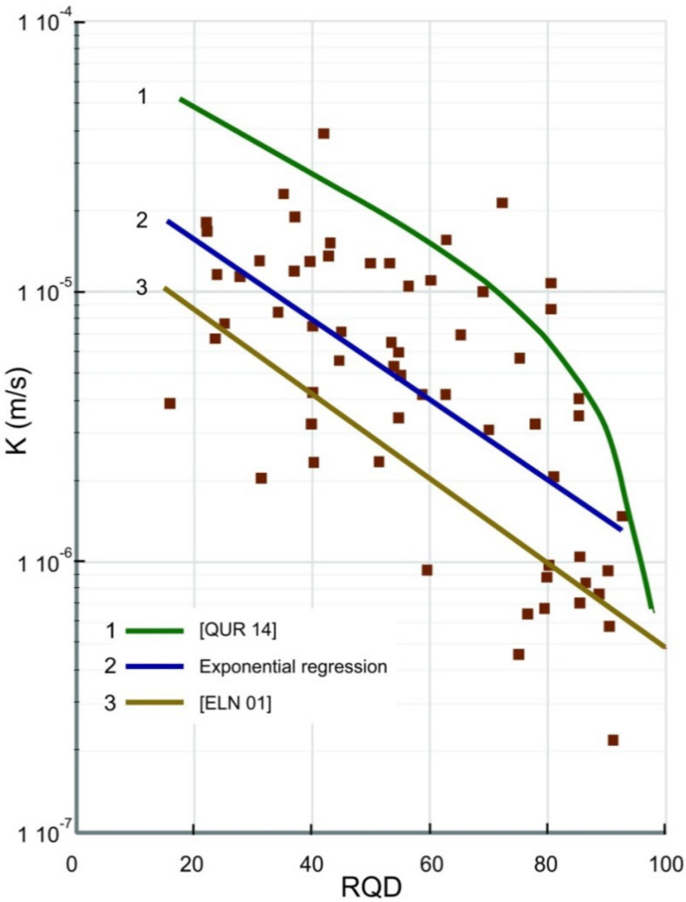

Figure 12.15. *Relationship between RQD and hydraulic conductivity, K, in m/s (data from [ELN 01] and [QUR 14]). For a color version of the figure, please see www.iste.co.uk/verbrugge/soils.zip*

The exponential regression is shown in the equation (K in m/s):

$$K = 3\,.\,10^{-5}\, e^{-0.034\,RQD}\ (R^2 = 0.41) \qquad [12.94]$$

12.4.2.5.2. From Q

Barton [BAR 07] gave the following relation:

$$K \approx \frac{1}{Q_c} \qquad [12.95]$$

where K is expressed in UL and Q_c is the normalized index.

In the same paper, generally, when the depth/stress dependence is taken into account and the wall strength JCS is considered, the Q index is modified in Q_{H2O}:

$$Q_{H2O} = \frac{RQD}{Jn} \times \frac{Ja}{Jr} \times \frac{Jw}{SRF} \times \frac{100}{JCS} \qquad [12.96]$$

From this, the hydraulic conductivity, K, is given by

$$K \approx \frac{0.002}{Q_{H2O} . D^{5/3}} \qquad [12.97]$$

with K in m/s and D, the depth, in m.

13

Usual Values of Soils and Rock Parameters

It is important to consider the fact that the name given to a soil depends on the standards used and can therefore vary from one country or administration to another. Hence, some differences may arise between the values given in the tables of this chapter, depending on the sources. Therefore, the name of a soil type must be seen as a generic one and the associated value as an order of magnitude. Values are also given in the chapters dealing with soil and rock identification.

13.1. Physical parameters

13.1.1. *Plasticity, unit weights and porosity*

Soil type		w_L (%)	I_P (%)	γ (kN/m³)	n (%)
Gravel	Clean	NA	NA	19–20	30–32
	Silty	15–20	<5	21–22	28–30
	Clayey	20–30	5–10	19.5–22	28–32
Sand	Clean	NA	NA	17–20	36–38
	Silty	15–25	<5	18–21	32–40
	Clayey	20–30	5–10	19–21	32–40
Silt		30–60	5–25	18–20	32–60
	Clayey	20–35	15–25	18–21	35–50
Clay		>35	>25	15–20	45–70

Table 13.1. *Plasticity, unit weights and porosity (orders of magnitude)*

Geotechnical Correlations for Soils and Rocks, First Edition.
Jean-Claude Verbrugge and Christian Schroeder.

13.1.2. *Consistency and related strength parameters*

13.1.2.1. *Granular soils*

Consistency	DR (%)	γ (kN/m³)	q_c (MPa)	N (blows/ft)	φ' (*) (°)	φ' (**) (°)	E (*) (MPa)
Very loose	0–15	11–16	0–2.5	0–4	26–30	29–32	<10
Loose	15–35	14–18	2.5–5	5–10	28–34	32–35	10–20
Medium	35–65	17–20	5–10	11–24	30–40	35–37	20–30
Dense	65–85	17–22	10–20	25–50	33–46	37–40	30–60
Very dense	85–100	20–23	>20	<50	40–50	40–42	60–90

*Eurocode 1997-3/B1 [DYS 01]; **AASHTO 1988 [SAB 02].

Table 13.2. *Consistency and related parameters for granular soils*

13.1.2.2. *Cohesive soils*

Consistency	CI	γ (kN/m³)	S_u (kPa)	N (blows/ft)	q_c (MPa)
Very soft	<0.5	<14	0–25	0–1	<0.5
Soft	0.5–0.75	14–17	25–50	2–4	0.5–1.5
Medium	0.5–0.75	16–19	50–100	5–8	0.5–1.5
Stiff	0.75–1	18–21	100–200	9–15	1.5–3
Very stiff	1–1.5	19–22	200–400	16–30	3–6
Hard	>1.5	21<	>400	>30	>6

Table 13.3. *Consistency and related parameters for cohesive soils according to the AASHTO [SAB 02, p. 110]*

13.1.3. *Soil indices*

Soil type	Ic	*Ic
Organic soil – clay	>3.6	>3.22
Clays – silty clay to clay	2.85–3.6	2.82–3.22
Silt mixtures – silty sand to silty clay	2.60–2.85	2.54–2.82
Sand mixtures – silty sand to sandy silt	2.05–2.60	1.90–2.54
Sands – clean sand to silty sand	1.31–2.05	1.25–1.90
Gravelly sand to dense sand	<1.31	<1.25

Table 13.4. *Soil types according to Ic and *Ic indices*

13.1.4. *Soil and rock resistivity*

Material	Resistivity (Ω^{-m})
Sea water	0.25
Drinking water	12–18
Clay	0.5–30
Marl	10–60
Silt, loam	20–100
Sand and gravel (dry)	400–2,000
Sand (moist)	50–200
Sand (sea water)	2–15
Gravel	150–500
Shale (compact)	100–300
Shale (weathered)	50–100
Sandstone (compact)	1,000–3,000
Sandstone (fissured)	500–1,500
Limestone (compact)	2,000–5,000
Limestone (weathered)	50–2,000

Table 13.5. *Soil and rock resistivity*

13.1.5. *Wave velocity*

Material		Velocity (m/s)
Sand and gravel	Moist	200–900
	Immerged	1,400–2,300
Clay, clay mixtures		800–2,300
Shale	Sound	2,500–4,500
	Weathered	500–2,000
Sandstone	Sound	1,500–4,300
	Weathered	600–3,000
Limestone, chalk	Sound	1,800–6,000
	Fractured	800–2,000
Igneous rock	Sound	3,600–6,000
	Fractured	1,000–2,500
Metamorphic rock	Sound	3,000–5,000
	Fractured	600–3,000

Table 13.6. *Compressive wave velocity*

The shear and compressive wave velocities are related to the equation:

$$\frac{V_p}{V_s} = \sqrt{\frac{2(1-\nu)}{1-2\nu}}$$

13.1.6. *Clay minerals and CEC*

Clay mineral	CEC (meq/100 g)
Kaolinite	3–15
Smectite	80–150
Illite	10–40
Chlorite	10–40
Vermiculite	100–150

Table 13.7. *Clay minerals versus CEC values*

13.2. Hydraulic parameters

13.2.1. *Hydraulic conductivity*

Soil type	Ic	k (m/s)
Sensitive fine-grained	NA	3.10^{-10} to 3.10^{-8}
Organic soil – clay	>3.6	1.10^{-10} to 1.10^{-8}
Clay	2.95–3.6	1.10^{-10} to 1.10^{-9}
Silt mixture – silty sand to silty clay	2.60–2.95	3.10^{-9} to 3.10^{-7}
Sand mixture – Silty sand to sandy silt	2.05–2.60	1.10^{-7} to 1.10^{-5}
Sands – clean sand to silty sand	1.31–2.05	1.10^{-5} to 1.10^{-3}
Gravelly sand to dense sand	<1.31	1.10^{-3} to 1
Very dense or stiff soil	NA	1.10^{-8} to 1.10^{-3}
Very stiff fine-grained soil	NA	1.10^{-9} to 1.10^{-7}

Table 13.8. *Hydraulic conductivity related to soil index [ROB 90]*

13.2.2. *Water storage capacity*

For a free water table n' < n:

– Coarse alluvial deposits without clay: n' = 30–40%;

– Gravel: n' = 20–25%;

– Sand, sandy gravel: n' = 15–20%;

– Fine sand: n' = 5–10%;

– Clayey or cemented gravel: n' = +/−5%;

– Silt, loam: n' = 2–5%;

– Clay, sandy clay: n' = 3%.

For a confined aquifer:

$$n' = H\gamma_w/E \qquad [13.1]$$

where H is the thickness of the aquifer.

13.3. Strength parameters

For details, the reader is referred to section 13.1.2.

13.4. Deformation parameters

13.4.1. *Compression index*

Compressibility	Cc
Uncompressible	<0.02
Very low	0.02–0.05
Low	0.05–0.1
Medium	0.01–0.2
Very compressible	0.2–0.3
High	0.3–0.5
Very high	>0.5

Table 13.9. *Compressibility versus compression index*

13.4.2. *Soil modulus*

Soil type	Compactness	E (MPa)
Clay	Soft	2.5–15
	Medium	15–50
	Stiff	50–100
Silt		2–20
Fine sand	Loose	8–12
	Medium	12–20
	Dense	20–30
Sand	Loose	10–30
	Medium	30–50
	Dense	50–80
Gravel	Loose	30–80
	Medium	80–100
	Dense	100–200

Table 13.10. *Soil modulus values adapted from AASHTO 1996 [SAB 02, p. 148]*

13.4.3. *Poisson's ratio*

Soil type	Quick loading	Slow loading
Gravel	0.30	0.30
Sand	0.35	0.30
Silt and silty clay	0.45	0.35
Stiff clay	0.45	0.25
Plastic clay	0.50	0.40
Compacted clay	0.45	0.30

Table 13.11. *Poisson's ratio after Poulos and Small [POU 00]*

The behavior of dilatant soils is inelastic so ν may exceed 0.5.

13.4.4. *Small strain modulus*

Soil type	G_0 (MPa)
Soft clays	2.75–13.75
Firm clays	7–34.5
Silty sands	27.5–138
Dense sands, gravels	69–345

Table 13.12. *Order of magnitude of small strain modulus for cohesive and granular soils*

13.5. Consolidation parameters

13.5.1. *Primary consolidation*

Liquid limit w_L (%)	Intact (10^{-8} m^2/s)	Remolded (10^{-8} m^2/s)
40	30	6
60	10	2
80	4	1.5
100	2	1
120	1.2	0.7
140	0.7	0.6

Table 13.13. *Order of magnitude of c_v for clays*

Material	c_v (10^{-8} m^2/s)
Kaolinite	20–40
Illite	10–20
Montmorillonite	2–10
Sandy clays	±10

Table 13.14. *Order of magnitude of c_v for clay minerals*

13.6. *In situ* test parameters

13.6.1. *CPT*

Soil type	q_c (MPa)	FR (%)
Soft clay	<0.8	4<
Sandy silt	0.4–3	1–7
Silt	0.7–2	1.5–4
Clayey silt	<1	3–9
Clayey silt	1<	<3
Sand	1.5<	<2
Silty sand	0.8–4	1.1–5
Clayey sand	0.3–6	0.5–8
Sandy gravel	20<	<2
Clayey gravel	10<	<5

Table 13.15. *Domains of CPT cone resistance and friction ratio for various soils*

13.6.2. *PMT*

Soil type	E_M(MPa)	p_L (MPa)
Mud, peat	0.2–1.5	0.02–0.15
Soft clays	0.5–3	0.05–0.3
Plastic clays	3–8	0.3–0.8
Stiff clays	8–14	0.6–2
Marls	5–10	0.6–6
Muddy sands	0.5–2	0.1–0.5
Silts, loams	2–10	0.2–1.5
Sands, gravels	8–100	1.2–5
Sedimentary sands	7.5–40	1–5
Chalks	8 <	3–10 <
Recent embankments	0.5–1	0.05–0.3
Old embankments	4–15	0.4–1
Gravely embankments	20–15	1–2.5

Table 13.16. *Domains of pressuremeter modulus and limit pressure for various materials*

13.6.3. *DMT*

[GUS 15] Soil type	Plasticity index	[GUS 15] ID	Marchetti ID	Marchetti Soil type
Fat clay – heavy	>27	<0.09	0.1–0.6	Clay
Fat clay – silty	17–27	0.09–0.37	0.1–0.6	Clay
Fat clay – sandy	17–27	0.37–0.43	0.1–0.6	Clay
Lean clay – heavy silty	12–17	0.43–0.74	0.1–1.8	Clay/silt
Lean clay – heavy sandy	12–17	0.74–0.84	0.6–1.8	Silt
Lean clay – light silty	7–12	0.84–1.46	0.6–1.8	Silt
Lean clay – light sandy	7–12	1.46–1.64	0.6–1.8	Silt
Silty clay	1–17	1.64–1.88	0.6 - (10)	Silt/sand
Silty clay – sandy	1–17	1.88–2.00	1.8 - (10)	Sand
Fine sand – silty	x	2.00–2.20	1.8 - (10)	Sand
Fine sand	x	2.20–2.50	1.8 - (10)	Sand
Medium sand	x	2.50<	1.8 - (10)	Sand

Table 13.17. *DMT soil classification according to Guskov and Gayduk [GUS 15]*

13.6.4. *SPT*

For details, the reader is referred to section 13.1.2.

13.7. Rock parameters

The values presented herein are those of intact, dry and assumed isotropic rock samples unless otherwise mentioned. An accurate and reliable determination of rock properties requires a complete geological and experimental definition, as, for example, made in the catalogue of French rocks published by the LCPC [HAB 69].

13.7.1. *Rock materials*

13.7.1.1. *Strength*

13.7.1.1.1. Field estimation (Hoek)

Term	UCS (MPa)	I_s (MPa)	Field estimate of strength	Examples
Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Basalt, chert, diabase, gneiss, granite, quartzite
Very strong	100–250	4–10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff
Strong	50–100	2–4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, phyllite, sandstone, schist, shale
Medium strong	25–50	1–2	Specimen cannot be scraped or peeled with a pocket knife, but can be fractured with a single blow of a geological hammer	Claystone, coal, concrete, schist, shale, siltstone
Weak	5–25	NA	Specimen can be peeled with a pocket knife with difficulty, shallow indentation made by a firm blow with a point of a geological hammer	Chalk, rock salt, potash
Very weak	1–5	NA.	Specimen crumbles under firm blows with a point of a geological hammer, and can be peeled by a pocket knife	Highly weathered or altered rock
Extremely weak	0.25–1	NA	Specimen can be indented by thumbnail	Stiff fault gouge

NA: Point load tests (Is) on rocks with uniaxial compressive strength less than 25 MPa are likely to yield highly ambiguous results.

Table 13.18. *Field estimates of uniaxial compressive strength (modified from Hoek [HOE 07a] and RockLab 2007™)*

13.7.1.1.2. Laboratory results

	Traction strength σ_t			
	No. of samples	Mean (MPa)	Stand dev. (MPa)	CV (%)
Shales	15	**2.9**	0.6	20
Sandy shales	17	**6.2**	2.7	44
Sandstones	22	**11**	3.2	30

Table 13.19. *Traction strength of several carboniferous Belgian rocks (intact rocks) (adapted from Calembert* et al. *[CAL 74])*

	Compression strength σ_c				Mean $\sigma c/\sigma t$
	No. of samples	Mean (MPa)	Stand dev. (MPa)	CV (%)	
Shales	14	**25**	16.3	65	8.6
Sandy shales	32	**54**	27.9	52	8.7
Sandstones	27	**152**	39.1	26	13.8

Table 13.20. *Compression strength of several carboniferous Belgian rocks (intact rocks) (adapted from Calembert* et al. *[CAL 74])*

	Traction strength σ_t					
	No. of samples	Mean (MPa)	Min (MPa)	Max (MPa)	Stand dev. (MPa)	CV (%)
All rocks	46	11.6	4	37.4	6.8	58
Sandstones	16	17.0	5.0	37.4	7.9	46.2
Limestones	24	8.1	4.2	15.3	3.5	43.0
Dolomites	3	12.4	11.0	15.3	2.5	20.0

Table 13.21. *Traction strength of several Devonian Belgian rocks (intact rocks) (data from author's personal database)*

	Compression strength σ_c						
	No. of samples	**Mean (MPa)**	**Min (MPa)**	**Max (MPa)**	**Stand dev.(MPa)**	**CV (%)**	**Mean $\sigma c/\sigma t$**
All rocks	205	**132**	22	387	71	54	11.3
Sandstones	48	**154**	34.4	387	68.6	45.1	8.9
Limestones	88	**100**	22	233	50.3	50.5	12.3
Dolomites	6	**140**	59	205	54.2	38.7	11.3
Porphyry	28	**206**	100	365	70.2	34.1	-

Table 13.22. *Compression strength of several Devonian and igneous Belgian rocks (intact rocks) (data from author's personal database)*

13.7.1.2. *Deformation and sonic velocity*

	Deformation modulus E_d			
	No. of samples	**Mean (MPa)**	**Stand dev. (MPa)**	**CV (%)**
Shales	14	**6.0**	3.8	64
Sandy shales	32	**13**	7.4	57
Sandstones	27	**31**	7.8	26

Table 13.23. *Deformation modulus of several carboniferous Belgian rocks (intact rocks) (adapted from Calembert* et al. *[CAL 74])*

	Deformation modulus E_d					
	No. of samples	**Mean (GPa)**	**Min (GPa)**	**Max (GPa)**	**Stand dev. (GPa)**	**CV (%)**
All rocks	269	38	13.6	64.1	8.8	23.3
Sandstones	54	30.6	15.4	54.2	8.4	27.4
Limestones	101	40	19.5	64.1	8.9	22.3
Dolomites	2	35	22.7	46.4	16.7	48.4
Porphyry	42	40	26.2	53.2	6.0	15.1

Table 13.24. *Deformation modulus of several Devonian and igneous Belgian rocks (intact rocks) (data from author's personal database)*

	Compression wave velocity V_p					
	No. of samples	**Mean (m/s)**	**Min (m/s)**	**Max (m/s)**	**Stand dev. (m/s)**	**CV (%)**
All rocks	210	5,500	2,270	6,900	670	12.2
Sandstones	37	4,720	2,270	6,000	780	16.5
Limestones	75	5,790	3,700	6,900	510	8.8
Dolomites	2	4,910	4,210	5,600	980	20.0
Porphyry	46	5,550	4,340	6,080	370	6.7

Table 13.25. *Compression wave (sonic) velocity of Devonian and igneous Belgian rocks (intact rocks) (data from author's personal database)*

13.7.2. *Rock masses*

13.7.2.1. *Shear strength of discontinuity surfaces*

13.7.2.1.1. Barton–Bandis

Rock	**Moisture**	**σ_n (MPa)**	**ϕ_b (°)**
Amphibolite	Dry	0.1–4.2	32
Basalt	Dry	0.1–8.5	35–38
	Wet	0.1–7.9	31–36
Conglomerate	Dry	0.3–3.4	35
Chalk	Wet	0–0.4	30
Dolomite	Dry	0.1–7.2	31–37
	Wet	0.1–7.2	27–35
Gneiss (schistose)	Dry	0.1–8.1	26–29
	Wet	0.1–7.9	23–26
Granite (fine g.)	Dry	0.1–7.5	31–35
	Wet	0.1–7.4	29–31
Granite (coarse g.)	Dry	0.1–7.3	31–35
	Wet	0.1–7.5	31–33
Limestone	Dry	0–0.5	33–39
	Wet	0–0.5	33–36
	Dry	0.1–8.3	37–40
	Wet	0.1–8.3	35–38
Porphyry	Dry	0–13.3	31
Sandstone	Dry	0–0.5	26–35
	Wet	0–0.5	25–33
	Wet	0–0.3	29

	Dry	0.3–3.0	31–33
	Dry	0.1–7.0	32–34
	Wet	0.1–7.3	31–34
Shale	Wet	0–0.3	27
Siltstone	Wet	0–0.3	31
	Dry	0.1–7.5	31–33
	Wet	0.1–7.2	27–31
Slate	Dry	0–1.1	25–30

Table 13.26. *Basic friction angle for various rocks, obtained from sand-blasted, rough-sawn and residual surfaces (data from Barton [BAR 71, BAR 76])*

13.7.2.1.2. Shear strength of filled discontinuities and filling materials

Rock	**Description of joint and filling**	**Peak**		**Residual**	
		c'	**φ'**	**c'**	**φ'**
		MPa		**MPa**	
Basalt	Clayey basaltic breccia, wide variation from clay to basalt content	0.242	42		
Bentonite	Bentonite seam in chalk	0.015	7.5		
Bentonite	Thin layers	0.09–0.12	12–17		
Bentonite	Triaxial tests	0.06–0.1	9–13		
Bentonitic shale	Triaxial tests	0–0.27	8.5–29		
Bentonitic shale	Direct shear tests			0.03	8.5
Chalk	80 mm seams of bentonite (montmorillonite) clay in chalk	0.016–002	7.5–11.5		
Clay shale	Triaxial tests	0.06	32		
Clay shale	Stratification surfaces			0	19–25

Clays	Overconsolidated, slips, joints and minor shears	0–0.018	12–18.5	0–0.003	10.5–16
Coal measure rocks	Clay mylonite seams, 10–25 mm	0.012	16	0	11–11.5
Diorite, granodiorite and porphyry	Clay gouge (2% clay, PI = 17%)	0	26.5		
Dolomite	Altered shale bed, ±150 mm thick	0.04	15	0.02	17
Granite	Clay-filled faults	0–0.1	24–45		
Granite	Sandy loam fault filling	0.05	40		
Granite	Tectonic shear zone, schistose and broken granites, disintegrated rock and gouge	0.26	45		
Greywacke	1–2 min clay in bedding planes			0	21
Lignite	Layer between lignite and underlying clay	0.014–.03	15–17.5		
Lignite/marl	Lignite/marl contact	0.1	10		
Limestone	6 min clay layer			0	13
Limestone	10–20 mm clay fillings	0.1	13–14		
Limestone	< 1 mm clay filling	0.05–0.2	17–21		
Limestone	Marlaceous joints, 20 mm thick	0	25	0	15–24
Limestone, lignites	Interbedded lignite layers	0.08	38		

Quartz/kaolin / pyrolusite	Remolded triaxial tests	0.042–.09	36–38		
Schists, quartzites	100–250 mm thick clay filling	0.03–0.08	32		
Siliceous schists	Stratification with thin clay	0.61–0.74	41		
Siliceous schists	Stratification with thick clay	0.38	31		
Slates	Finely laminated and altered	0.05	33		

Table 13.27. *Peak and residual values of cohesion and internal friction angle of filled discontinuities and filling materials (from Barton [BAR 73] and Hoek [HOE 07a])*

13.7.2.2. *Classification of rock masses: Hoek and Brown criterion*

Nature	Class	Group	Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerates*	Sandstones	Siltstones	Claystones
			(21 ± 3)	17 ± 4	7 ± 2	4 ± 2
			Breccias		Greywackes	Shales
			(19 ± 5)		(18 ± 3)	(6 ± 2)
						Marls
						(7 ± 2)
	Non-clastic	Carbonates	Crystalline	Sparitic	Micritic	Dolomites
			Limestone	Limestones	Limestones	(9 ± 3)
			(12 ± 3)	(10 ± 2)	(9 ± 2)	
		Evaporites		Gypsum	Anhydrite	
				8 ± 2	12 ± 2	
		Organic				Chalk**
						7 ± 2

Metamorphic	Non-foliated		Marble	Hornfels	Quartzites	
			9 ± 3	(19 ± 4)	20 ± 3	
				Meta-		

				sandstone		
				(19 ± 3)		
	Slightly foliated		Migmatite	Amphibolites		
			(29 ± 3)	26 ± 6		
	Foliated		Gneiss	Schists	Phyllites	Slates
			28 ± 5	12 ± 3	(7 ± 3)	7 ± 4

Igneous	Phaneritic	Light	Granite	Diorite		
			32 ± 3	25 ± 5		
			Granodiorite			
			(29 ± 3)			
		Dark	Gabbro	Dolerite		
			27 ± 3	(16 ± 5)		
			Diabase	Peridotite		
			(15 ± 5)	(25 ± 5)		
	Porphyritic		Porphyries (20 ± 5)			
	Aphanitic and glassy			Rhyolite (25 ± 5)	Dacite (25 ± 3)	Obsidian (19 ± 3)
				Andesite 25 ± 5	Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

*Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone to values used for fine-grained sediments.

**Strongly depending on porosity.

NB: These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if the specimen is tested in a different orientation and/or if failure occurs along a weakness plane.

Table 13.28. *Hoek and Brown criterion for estimating m_i values (modified from Hoek [HOE 07a] and RockLab 2007™)*

List of Symbols

Some symbols are the same for both soils and rocks, but differ in their definition. In such cases, to avoid any confusion, an (*) indicates the valid definition for rocks.

$(N_1)_{60}$	Normalized SPT blow count at 60% efficiency
$*I_C$	SBT classification index including B_q
A or A_C	Activity
a or a_n	Net cone tip area ratio (0.70<a<0.85)
a	Parameter of the Hoek and Brown criterion (*)
B_q	Normalized pore water pressure parameter [$= (u_2 - u_0)/q_n$]
c'	Effective cohesion intercept
c_a	Capillary cohesion
C_c	Virgin compression index
CF	Clay fraction (<2μm)
CI	Consistency index
CV	Coefficient of variation

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Jean-Claude Verbrugge and Christian Schroeder.

C_s	Swelling compression index
c_u or c	Apparent cohesion intercept, equivalent to s_u
C_U	Coefficient of uniformity
c_v	Coefficient of primary consolidation
C_α	Rate of secondary consolidation
D	Depth
D	Disturbance factor (*)
D	Grain diameter
D_e	Equivalent diameter of the tested sample (in mm) in the Franklin test
Dn	n percentage diameter
D_R	Relative density of sand
E or M	Modulus of linear deformation, vertical drained constraint modulus
e	Void ratio
E	Young's modulus (*)
E'	Young's modulus of the soil skeleton
e_0	Initial void ratio
E_0	True elastic modulus (very small strains, $\varepsilon<+/-10^{-5}$)
E_D	DMT dilatometer modulus
E_e	Young's modulus of elasticity (*)
E_i	Modulus for intact rock (*)

E_m	Pressuremeter modulus
E_{mass}	Modulus (tangent) of rock mass
E_s	Short-term Young's modulus
e_{max}	Void ratio at the loosest state
e_{min}	Void ratio at the densest state
E_t or E_d	Tangent Young's modulus or deformation modulus
E_u	Undrained modulus
F or FOS	Factor of safety
F	Normalized sleeve friction
FC	Field capacity
F_R or R_f	Friction ratio of CPT
f_s	Measured cone sleeve friction
F_{schim}	Schimazeck factor (abrasiveness)
g	Gravitational constant (9.8 m/s^2)
G	Modulus of shear deformation
G_0	Small strain shear modulus (very small strains, $\varepsilon < +/-10^{-5}$)
h	Hydraulic head or potential
i	Hydraulic gradient
I_C	Soil behavior type index (SBT index) for soil classification from CPT
I_D	Density index (*)
I_D	Material index from DMT

I_L or LI	Liquidity index
I_P or PI	Plasticity index
I_r	Rigidity index
I_s	Point load strength index (Franklin test)
$I_{s\,(50)}$	Point load strength index measured on NX core (54 mm)
Ja	Q rating for joint alteration, discontinuity filling
J_{Cond89}	Joint condition rating
Jn	Q rating for the number of joint sets
Jr	Q rating for joint surface roughness
J_v	Volumetric joint count
K	Hydraulic conductivity (*)
k	Hydraulic conductivity
K_0	Coefficient of earth pressure at rest
K_D	Lateral/horizontal stress index from DMT
ln	Natural logarithm
log	Logarithm base 10
M or E	Modulus of linear deformation, vertical drained constraint modulus
M_R	Resilient modulus
m_b	Parameter of the Hoek and Brown criterion
m_q	Cone resistance depth ratio $[= \Delta q_t/\Delta z \cong q_t/z]$
m_v	Coefficient of volume change

N	Schmidt hammer rebound (*)
N	Blow count from SPT
n	Porosity
N_{60}	SPT blow count at 60% efficiency
N_{dc}	Dynamic cone blow count
nf	Part of porosity due to fissures
np	Part of porosity due to pores
ø	Diameter (of grains) (*)
ø	Porosity (*)
p_0	First DMT corrected reading
p_1	Second DMT corrected reading
p_l or p_L	Pressuremeter limit pressure
q_0	Dilatometer tip resistance
q_c	Measured static cone tip resistance
q_d	Dynamic probe resistance
q_e	Effective static cone tip resistance $[= q_t - u_2]$
q_n	Net cone resistance $[= q_t - \sigma_{v0}]$
q_t	Corrected cone tip resistance $[= q_c + (1 - a_n)u_2]$
Q_t	Normalized cone resistance for n=1
q_{t1}	Normalized cone tip resistance
Q_{tn}	Normalized cone resistance
R or r	Coefficient of correlation

R	Particle roundness (very angular = 0<R<very round = 1)
R^2 or r^2	Coefficient of determination
Rc or σ_c	Compression strength
R_t or σ_t	Traction strength
S	Joint spacing
s	Parameter of the Hoek and Brown criterion (*)
s	Time in seconds
S_r	Degree of saturation
S_t	Sensitivity
s_u	Undrained shear strength, equivalent to c_u
t	Quartz or abrasive mineral content (*)
t	Time
T_v	Time factor
U	Degree of consolidation
u	Pore pressure
u*	Normalized pore water pressure
u_0	Initial pore pressure
u_1	Pore water pressure measured at the midface of the cone tip
u_2	Pore water pressure measured behind the cone tip
u_a	Pore air pressure
u_c or s	Matrix suction
u_w	Pore water pressure

v	Flow velocity
V_p or V_l	Sonic wave velocity (compression, primary, longitudinal wave)
V_s or V_t	Sonic wave velocity (shear, secondary, transversal wave)
V_{s1}	Overburden corrected shear wave velocity
w	Water content by weight
w_L or LL	Liquid limit
w_{opt}	Proctor optimum moisture content by weight
w_P or LP	Plastic limit
w_s	Shrinkage limit
γ	Weight per unit of volume (density)
λ	Average number of discontinuities per meter
σ_c or R_c	Compression strength
σ_i or σ_{ci}	Compression strength of intact rock
σ_{rm}	Compression strength of rock mass
σ_t or R_t	Traction strength
σ_{tm}	Traction strength of rock mass
Λ	Plastic volumetric strain potential
α	Ratio of soil modulus to cone tip resistance
γ'	Submerged unit weight
γ_s	Dry unit weight
γ_s	Unit weight of solid particles

γ_s	Unit weight of water
γ_{sat}	Saturated unit weight
γ or γ_t	Total unit weight
φ or ϕ	Apparent angle of internal friction
φ' or ϕ'	Effective angle of internal friction
ϕ_b	Basic friction angle for rock discontinuities
φ_{cs}' or ϕ_{cs}'	Critical state angle of internal friction
φ_p' or ϕ_p'	Peak angle of internal friction
φ_r' or ϕ_r'	Residual angle of internal friction
ν	Poisson's ratio
θ	Water content by volume
ρ	Density
ρ'	Submerged density
ρ_d	Dry density
$\rho_{d(max)}$	Maximum dry density
$\rho_{d(min)}$	Minimum dry density
ρ_s	Density of solid particles
ρ_{sat}	Saturated density
ρ_t	Total moist density
ρ_w	Density of water
σ'	Effective normal stress

σ	Total normal stress
σ'_p	Effective preconsolidation pressure
σ'_{v0}	Effective overburden pressure
σ_{atm}	Atmospheric pressure (100 kPa)
σ_c	Compressive strength
σ_t	Tensile strength
σ_{v0}	Total overburden pressure
τ	Shear stress
τ_f	Peak shear strength
τ_r	Remolded shear strength
τ_R	Residual shear strength
ψ	Angle of dilatancy

List of Equations

Cone resistance depth ratio: $m_q = \Delta q_t / \Delta z \approx q_t / z$

Continuity index: $CI = \frac{V_{p\,measured}}{V_{p\,theoretical}}$

Corrected tip resistance: $q_t = q_c + (1 - a_n)u_2$

DMT dilatometer modulus: $E_D = 34.7(p_1 - p_0)$

DMT horizontal stress index: $K_D = (p_0 - u_0)/\sigma'_{v0}$

DMT material index: $I_D = \frac{p_1 - p_0}{p_0 - u_0}$

Effective cone resistance: $q_e = q_t - u_2$

Friction ratio: $F_R = {f_s}/{(q_n)} \cong {f_s}/{q_t}$

Hardness contrast: $Hc = \sum_i c_i \,.\, |Vh_i - Vh_b|$

Net cone resistance: $q_n = q_t - \sigma_{vo}$

Normalized blow count at 60% efficiency: $(N_1)_{60} = \left(\frac{\sigma_{atm}}{\sigma'_{v0}}\right)^n N_{60}$

with n = 1 for clays and 0.5 < n < 0.6 for sands.

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Normalized cone resistances:

$$q_{t1} = \left(\frac{q_t}{\sigma_{atm}}\right) \Big/ \left(\frac{\sigma'_{v0}}{\sigma_{atm}}\right)^{0.5}$$

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{\sigma_{atm}}\right)\left(\frac{\sigma_{atm}}{\sigma'_{v0}}\right)^{n}$$

where

$$n = 0.381 I_c + 0.05\left({\sigma'_{v0}}/{\sigma_{atm}}\right) - 0.15 \ \leq 1$$

If n=1,

$$Q_t = {(q_t - \sigma_{v0})}/{\sigma'_{v0}}$$

Normalized pore water pressure: $u* = \frac{\Delta u}{\sigma'_{v0}}$

Modulus ratio: $MR = \frac{E_i}{\sigma_i}$

Modulus ratio: $modulus\ ratio = \frac{E_{mass\ (in\ situ)}}{E_{core\ (lab)}}$

Overburden corrected shear wave velocity: $V_{s1} = V_s\left({\sigma'_{v0}}/{\sigma_{atm}}\right)^{0.25}$

Plastic volumetric strain: $\Lambda = 1 - {C_s}/{C_c}$

Pore pressure ratio: $B_q = (u_2 - u_0)/q_n$

Q_c: $Q_c = Q \frac{\sigma_c}{100}$

Velocity index: $Velocity\ Index = \left(\frac{V_{p\ in\ situ}}{V_{p\ on\ sample}}\right)^2$

Volumetric joint count: $J_v = \sum_i 1/S_i$

$V_{p\ theoretical}$: $V_{p\ theoretical} = \sum_i c_i \,.V_{p\ i}$

Weighted average hardness: $WAH = \sum_i c_i \,.Vh_i$

Young's modulus versus constraint modulus: $E' = \frac{(1+\nu)(1-2\nu)}{(1-\nu)}\ E$

List of Abbreviations and Acronyms

ABR or LAC	LCPC abrasivity
AIN or CAI	CERCHAR-INERIS abrasiveness
AIV	Aggregate Impact Value index
ASTM	American Society for Testing and Materials
CCT	Calibration Chamber Tests
CI	Continuity Index
CPT	Cone Penetration Test
CPTu	Piezocone Test
DMT	Dilatometer Test
DPT	Dynamic Probe Test
DSS	Direct Simple Shear test
ECPT	Electric CPT
GSI	Geological Strength Index
Hc	Hardness contrast
HOC	Heavily Overconsolidated

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JCS	Joint Wall Compression Strength (in Q classification)
Jw	Q rating for water softening, inflow and pressure effects
LA	Los Angeles index
LCPC	Laboratoire Central des Ponts et Chaussées
LOC	Lightly Overconsolidated
MD or MDe	Micro-Deval index
MPO	Modified Proctor Optimum
MR	Modulus Ratio
MSS	Magnesium Sulfate Soundness
NC	Normally Consolidated
NGI	Norwegian Geotechnical Institute
OCR	Overconsolidation Ratio
PMT	Pressuremeter Test
PSV	Polished Stone Value
Q	Quality index
Q_c	Normalized Q index
Q_{H2O}	Modified Q index for hydraulic conductivity
REV	Representative Elementary Volume
RM	Rock Mass
rm	Rock Material
RMR	Rock Mass Rating
RMR_{89}	Rock Mass Rating (1989 revision)

RQD	Rock Quality Designation
SBMPT	Self-Boring Pressuremeter Test
SBT	Soil Behavior Type
SCPT	Seismic Cone Penetration Test
SDMT	Seismic Dilatometer Test
SPO	Standard Proctor Optimum
SPT	Standard Penetration Test
SRF	Q rating for faulting, strength/stress ratios, squeezing, swelling
UCS	Unconfined Compressive Strength
UL	Lugeon Unit
Vh	Vickers hardness
VST	Vane Shear Test
WAH	Weighted Average Hardness

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Index

A, C, D

activity, 3, 11, 32, 62, 69
angle
  critical friction, 39
  internal friction, 12, 125, 156, 166
  of dilatancy, 37, 41
  peak friction, 39
  residual friction, 39, 125
anisotropy, 21, 23, 26, 32, 70, 108
Atterberg limits, 3
CBR, 53, 79
classification system, 124, 126
clay, 3, 8, 11, 14, 16, 19, 23, 24, 27, 28, 43, 44, 49, 54, 59, 61, 62, 82, 95, 137, 142–149, 154–156
  remolded, 16, 32, 44, 69
  soft, 2, 28, 42, 46, 47, 55, 58, 66, 67
  varved, 31, 46, 63
coefficient
  of earth pressure at rest, 73
  primary consolidation, 69
cohesion, 25, 27, 30, 32, 33, 86, 125, 156, 159, 160
  apparent, 86
  capillary, 86
collapse, 61
conductivity
  hydraulic, 21, 23, 137–140, 174
cone
  electric, 27, 89, 90
  mechanical, 89
consolidation, 4, 41, 46, 69–72, 147, 160, 164
  anisotropic, 26, 32
  degree of, 72
  isotropic, 32
constant of compressibility, 47
daturation
  degree of, 85, 87
dilatometer, 30, 51, 160, 169
discontinuities, 21, 103, 105, 124, 126, 127, 129, 154, 156, 165, 166
drying path, 85
dynamic probe testing (DPT), 91, 97, 100, 101, 173

E, G, H

effective cohesion, 33
expansion, 60, 61
glacial tills, 68, 75
gravel, 16, 24, 34, 37, 49, 52, 92, 93, 95, 143, 145, 148
hardness, 106–108, 122, 171, 175
humus carbon, 32

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I, L, M

identification
  from CPT, 1, 3, 12, 13, 15, 19, 22, 27, 28, 35, 36, 38, 42, 49, 56, 64, 74, 75, 89, 91–98, 100, 101, 148, 161, 173, 174
  from DMT, 2, 7, 18, 19, 30, 38, 51, 57, 65, 68, 75, 79, 93, 98, 99, 149, 160–163, 169, 173
  from SPT, 7, 17, 18, 29, 37, 52, 57, 65, 67, 74, 94–100, 149, 159, 163, 175
  from tests, 11, 25, 33, 60
index
  compression, 43, 46, 70, 145, 159, 160
  consistency, 4, 26, 82, 110, 111, 142, 159, 173
  liquidity, 26, 162
  rigidity, 5, 162
  swelling, 48
intermediate stress, 40, 41
lithology, 104, 105, 109, 114, 116, 120, 129
loading rate, 54, 109
loam, 24, 83, 87, 143, 145, 155
local friction, 12, 94
microfissuration, 103, 110–112
modulus
  small strain shear, 56, 57, 65
  soil, 48, 55, 79, 165
  tangent, 114, 133
modulus of subgrade reaction, 59
moisture content, 2, 43, 61, 81, 83, 86, 109, 165

O, P, Q

organic matter, 32, 71
overconsolidated soils, 31, 51
peat, 43, 63, 72, 95, 148
plane strain, 40
porosity, 21–23, 87, 103, 108, 111, 112, 141, 157, 163
preconsolidation pressure, 63–65, 167
Proctor test, 77, 78
  modified, 77, 78
quasi-saturated, 84, 85

R, S, T

ratio
  overconsolidation (OCR), 5–7, 27, 31, 66–68, 174
  Poisson, 58
  void, 22, 32, 33, 43, 67, 160
relative density, 5, 7, 12, 18, 33, 74
relative humidity, 82
rock mass, 103, 105, 106, 127, 136, 137, 156, 161, 165
rock material, 103–106, 109, 124
sand, 12, 14, 16, 19, 24, 49, 54, 59, 90, 92, 93, 95, 99, 141–146, 148, 149
Schmidt hammer rebound, 115, 125, 163
silt, 8, 14, 16, 19, 27, 43, 49, 54, 56, 95, 142, 144, 148, 149
soil
  behavior type (SBT) index, 51, 97, 161
  classification index, 3, 12
stress, 4, 5, 12, 27, 30, 32, 33, 38, 41, 49, 51, 56, 60, 65, 68, 71, 72, 74, 83–85, 135, 137, 140, 162, 166, 167, 169, 175
suction, 23, 81–84, 86, 164
swell, 9, 11, 43, 48, 60–62, 135, 175
tip resistance, 12, 27, 65, 79, 89, 90, 163, 165, 169
triaxial, 5, 26, 30, 32, 40, 41, 60, 156

U, W

Unconfined Compressive Strength (UCS), 104, 109–117, 132, 133, 135, 136, 150, 175
undrained shear strength, 25, 32
unit
  volume, 1
  weight, 1, 165, 166
wave velocity, 7, 36, 87, 132, 143, 153, 165, 170

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