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EARTHWORKS MANUAL

DESIGN AND CONSTRUCTION OF EARTH-STRUCTURES

PART 2 - SPECIFIC TECHNICAL DEVELOPMENTS
2A / MATERIALS

2A.I - NATURAL MATERIALS: SOILS AND ROCKS

TECHNICAL COMMITTEE 4.4 "*EARTHWORKS AND UNPAVED ROADS*"

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ABOUT THE AIPCR

The World Road Association (PIARC) is a non-profit organization, founded in 1909 to improve international cooperation and to encourage progress in roads and road transport.

The study, which is the subject of this Manual, was defined in the PIARC Strategic Plan 2012-2015 approved by the Council of the World Road Association, whose members represent the governments of its member countries. The members of the Technical Committee responsible for this report were chosen by the governments of these countries for their particular expertise.

The opinions, findings, conclusions and recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of the organizations or bodies to which they belong.

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1. INTRODUCTION

The optimal use and valorization of locally available materials on the road project site is today a priority both for economic reasons and as a contribution to sustainable development.

Materials from external quarries can thus be reserved for the most noble uses of pavement layers, especially when these resources become scarce due to depletion or environmental constraints.

The materials may be suitable for use either in their natural state or by treatment, mechanical process or with hydraulic binders, depending on their destination: common or special backfills, structuring layers, etc...

In this chapter, we consider the properties and geotechnical characteristics of natural soil and rock materials.

2. KNOWLEDGE OF MATERIALS FOR THE DESIGN OF EARTH STRUCTURES AND EARTHWORKS

- **Distinction between the design of earth structures and the design of earthworks**

We refer to the distinction made in “Part 1 - General Considerations" of the Earthworks Manual, in two phases.

The **design of earth structures** is defined in terms of stability and deformation. The geotechnical calculation and structural design define the requirements for the properties and functions of the completed structure.

The **earthworks design** defines the construction processes that allow the transformation of the in-situ terrain (soil or rock) and/or alternative materials into a compacted and durable fill with the required properties or stability (Case of earth structures fills).

Necessary information for the design of Earth structures

This information is collected at the stage of the geotechnical site survey, coordinated with the Earthworks one, as developed in § 5.

Information required for earthwork design

The description, identification, classification and characterization of soils, rocks and other fill materials are essential in the design of earthworks. They provide the information needed to determine the nature of each material, the best way to excavate it from cuts or pits, and the final state it can reach after compaction.

Reference Europe (Standrad "Earthworks")

Description of materials used

The description of natural and artificial materials used in earthworks covers:

- the presence of pollutants;
- the presence of organic matter and evolving or reactive minerals (gypsum, soluble salts, etc.);
- all information necessary to classify materials (Part 2):
  - soils: particle size, plasticity of clay soils, etc. ;
  - rocks: origin, degradability, fragmentability, etc. ;
  - other materials: components, mineralogy,, etc. ;
- information on the initial state of natural soils and rocks (density, water content and grain size of soils, faults in rocks, etc.) ;
- determination of optimal compaction characteristics for backfill materials ;
- if necessary, an estimate of the deformability, strength and permeability of the compacted material;
- any other information required for the project.

This information shall be integrated into the selected earthwork design system, based on specific full-scale compaction tests or the use of earthwork classifications.

3. GENERAL INFORMATION ON THE PROPERTIES OF NATURAL MATERIALS

In this paragraph we present general information on the properties of natural materials that are of interest to the field of Earth structures and Earthworks.

For more technical details, reference should be made to documents, treatises, articles that describe the classical basic concepts defining the properties of materials related to rock mechanics and soil mechanics.

The presentation made here of these properties refers to an *Article in the Journal* - Les Techniques de l'Ingénieur – "Propriétés des matériaux naturels" basé notamment sur le Guide TETR et les travaux du CFTR (France).

⇨ See Appendices Documentation / Material properties (§3)

The design and construction of earth structures implies knowledge of the properties of natural materials that are related to both rock and soil mechanics. This is the subject of the presentations in paragraphs 3.1 and 3.2, which present the main theoretical and/or empirical bases as well as reference rules for understanding the properties of materials and their behaviour under the influence of various factors.

In addition to these bases, paragraph 3.3 discusses the characteristic properties of soils and rocks from the perspective of geotechnical engineering suitable for earthworks. The parameters, resulting from the methodology applied in the field, are thus defined.

This approach is the essential vector for the earthmoving activity. It will make it possible to classify the materials according to their potential use in earthworks. These notions are decisive at the design stage as well as at the implementation stage.

The identification of materials requires a different approach for soils and rocks. We distinguish:

- Properties related to rock mechanics
- Soil mechanical properties
- Geotechnical properties applicable to Earthworks

3.1. PROPERTIES RELATED TO ROCK MECHANICS

The description and identification of the rocks resulting from these properties will be very useful for the rock extraction methodology: choice of materials, choice of ripping or mining operations in particular.

Refer to the Documentation Appendix I.1 "Properties of Natural Materials". The values proposed are usual values, subject to some variations if necessary.

3.1.1. Properties of rocks

❖ Definitions

In the field of rocks it is now classical to consider two different scales:

- The rocky massif scale: the scale of earthworks and foundations of structures (corresponding to at least a few tens of cubic metres).
- Rock itself scale: the scale of a sample and of the specimen submitted to the tests (corresponding to a few cubic decimetres)

Petrography classifies rocks into three major groups based primarily on genetic criteria.

❖ Identification

- **Eruptive rocks** are the result of the solidification of magma;
- **Sedimentary rocks** are formed from deposits of chemical or biochemical detrital elements;
- **Metamorphic rocks** result from changes to eruptive or sedimentary rocks when they are subjected to metamorphism due to huge pressures or temperatures.

Coherent rocks are particularly complex solids due to heterogeneity of constituents and structural defects.

Tableau 1 - Rocks family (1)		
MAGMATIC ROCKS (plutonics or volcanics)	Acid rocks family	Granit, granodiorite, syenite, microgranit,rhyolithe, trachyte
	Intermediary rocks family	Diorite, microdiorite, andesite, trachy-andesite
	Basic rocks family	Gabbro, dolerite, ophite, basalt, peridotite
METAMORPHIC ROCKS	Massive rocks	Quartzite, corneal, migmatite, marble
	Foliate rocks	Gneiss, ampibolite, leptynite, micaschist, slate
SEDIMENTARY ROCKS	Carbonated rocks family	Limestone, chalk, dolimite, travertine, blasenschiefer
	Rocks with dominant sileceous	Sandstone, arkose, argillite, millstone, radiolarite
	Silico-carbonated rocks family	Marl, molasse
	Evaporitic rocks family	Gypsum, anhydrite, rock salt
	Carbon rocks family	Coal, lignite
(1) Excerpt from TETR guidelines edited by the CFTR (cf. [Doc. C 5 360])		

Table 1: Rock family (Reference TETR)

❖ Porosity

Cracking, which plays a considerable role in the mechanical behaviour of rocks, is the consequence of the heterogeneous behaviour of the different constituents of a rock.

The volume of voids contained in a rock is expressed by the **porosity** n:

$$n\ (\%) = \frac{V_v}{V}$$

V_v volume of voids,

V total volume.

❖ Continuity index

Porosity is a poor expression of the cracked state of a rock, as the volume of voids associated with cracks is very small. Also, it appeared necessary to quantify, other than by porosity, the density of discontinuities present in a rock sample. The theory, like experimental measurements, shows that the speed of wave propagation in a solid is strongly influenced by the presence of discontinuities.

This is the reason why the **continuity index Ic** is used.

$$I_c = \frac{V_L}{V_L^{\bullet}} \times 100$$

V_L velocity of propagation of longitudinal waves in a rock,

$V_L^{\bullet}$ corresponding theoretical velocity for a perfect medium, of the same mineralogical composition as the rock, but without discontinuities.

❖ Mechanical resistance of rocks

Most often, the strength of a rock is given by the maximum resistance obtained in a tensile test or in a simple compression test.

- The tensile strength of rocks is little used. It is much weaker than the compressive strength; The most common tensile test is an indirect test, the Brazilian test.
- The most universally used mechanical characteristic for rocks is simple compressive strength σ. It is measured on a cylindrical slenderness specimen 2. Simple compressive strength is the basic parameter of most of the classifications offered in rock mechanics.

3.1.2. Properties of rock massifs

The mechanical characteristics of the rocks measured on samples are generally high, and much higher than those of the rock massifs. This difference can be explained by the presence of discontinuity surfaces.

❖ State of fracturing of a rocky massif

The state of fracturing of a rock mass is particularly useful in choosing the methods of realization.

Indexes are frequently used.

The most widespread index at present is the ***RQD*** (*Rock Quality Designation*).

It is determined by the cumulative length $\sum \mathbf{l}_i$ re elements greater than 10 cm in length in relation to the considered borehole length L. It is expressed in percent:

$$RQD = 100 \frac{\sum \ell_i \,(> 10 \text{ cm})}{L}$$

The **density of discontinuities** in the massif is often quantified by the **discontinuity interval** ***ID*** measured along a line and defined as the average interval between two successive discontinuities.

❖ State of weathering of a rocky massif

Rock weathering processes are extremely diverse. Depending on their intensity, the rock can have all the intermediate states between healthy rock and residual soil, for example granitic arenas. Most often, the weathering progresses through the surfaces of discontinuities.

❖ Abrasivity

Abrasivity is to be considered in the carrying out of rock excavations.

Reference France

A test carried out on a 4 to 6.3 mm aggregate (Standard reference XP P 18-579), quantifies the capacity of a rock to wear out the metal parts: this characteristic is therefore important for the drilling, the recovery phases of the blasted materials and the setting up.

❖ Prediction of extraction conditions for rock masses

It is important, for earthworks, to anticipate the conditions under which the material will be excavated. We come back to it in detail in the booklet "2D / Earthworks execution and controls".

3.2. PROPERTIES RELATING TO SOIL MECHANICS

We recall the basic notions that are relevant to the design of earth structures, particularly in terms of slope stability, structure foundation and deformation.

3.2.1. Intrinsic curve

Let us first recall that the **intrinsic curve** of a material is a curve limiting the elastic domain in a plane defined by the normal stress on the abscissa and the shear stress on the ordinate (Figure 1).

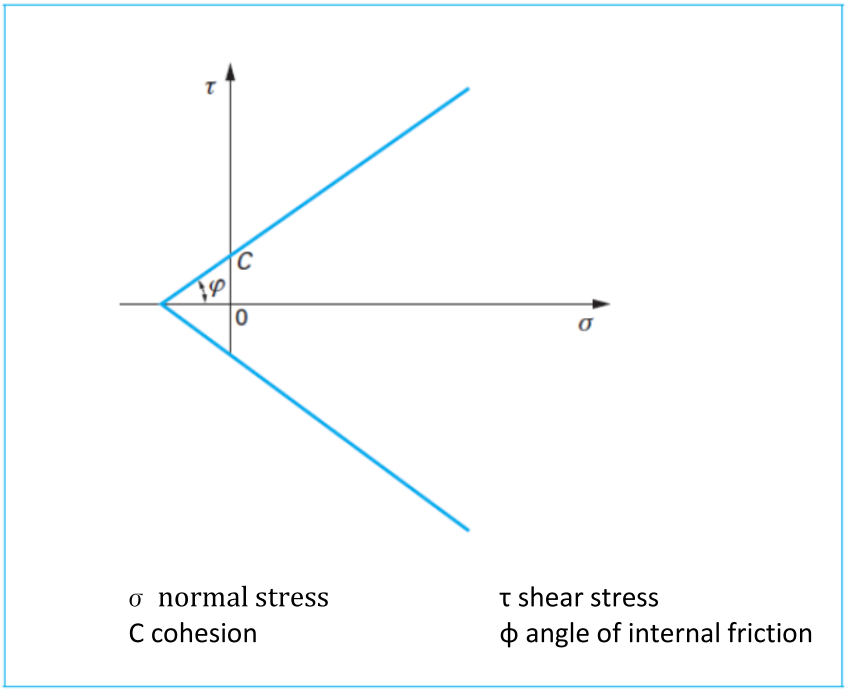


Figure 1: Intrinsic curve of a soil

This concept is applied in Soil Mechanics.

The two basic characteristics **C** and **ϕ** are not absolutely fixed values for a given soil, but are dependent on the condition of the soil and, in particular, its moisture content and the test conditions.

This intrinsic curve leads to a distinction between two kinds of soils: **pulverulent soils** for which C is zero and **cohesive soils** that have a certain shear strength C, even in the absence of any normal stress.

3.2.2. Dilatancy

This property is not very used for earthworks.

3.3. Geotechnical properties applicable to earthworks

3.3.1. The case of soils

As we have seen, the properties that refer to Soil Mechanics are generally important for the design of earth structures.

The field that interests us very directly is the **application of geotechnology to earthworks**.

This field involves the study of certain characteristic properties of soils, particularly with regard to **their nature and condition**.

Remember that cohesion and the angle of internal friction depend on these two major factors.

- General Definitions

Reference "Techniques of the Engineer" Article

Definition of a soil

A soil is a material consisting essentially of distinct solid grains and which may also contain water and air, the grains being on such a scale in relation to the volume of the soil in question that the law of large numbers is applicable.

This definition implies that a soil is a mixture of a dispersed solid phase consisting of rock grains, a liquid phase, either dispersed or continuous, and a gaseous phase, which may also be dispersed or continuous.

You do not change the nature of the soil by modifying the other two phases, increasing or decreasing the amount of water or air and modifying their distribution in the soil. All these modifications do not alter the nature of the soil but its state.

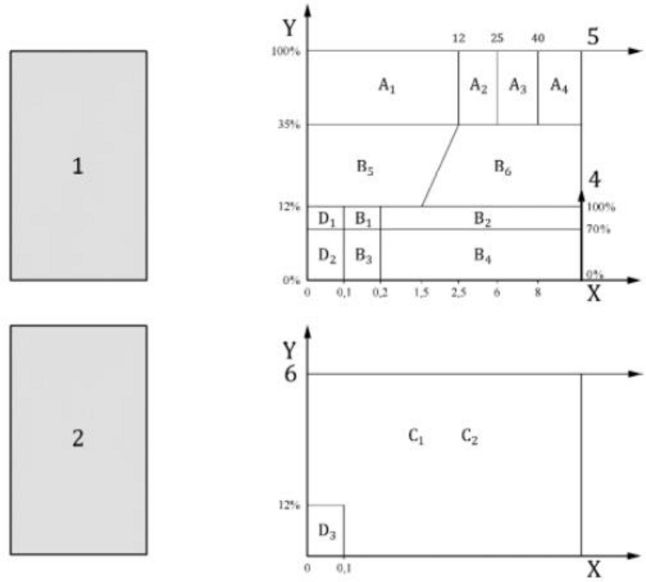
Nature of soil-identification

In order to define the nature of a soil, it may seem necessary to determine both the petrographic nature of the grains, their size, shape, etc., and the nature of the soil itself. In reality, from a geotechnical point of view, it is sufficient to know the Atterberg Limits (usually only the two limits of liquidity and plasticity) and the granularity of the soil. With these three characteristics, soils can be identified and classified.

❖ Parameters corresponding to geotechnical properties

See exemple GTR Synoptic Table

(Reference European Standard “Earthworks” Part 1 Appendix C):



3

Sedimentary rocks	Carbonate rocks	Chalk	R1
		Limestone	R2
	Argilaceous rocks	Marls, claystone, pelite, etc.	R3
	Siliceous rocks	Sanstone, puddigstone, breccia, etc.	R4
	Saline rocks	Rock salt, gypsum, etc	R5
Igneous and metamorphic rocks	Granite, basalt, andesite, gneiss, schist, slate, etc		R6

Key

X	percent passing 80 µm	4	percent passing 2 mm
Y	V_{BS}	5	Ip
1	soils Dmax ≤ 50 mm	6	fraction 0/50 mm
2	soils Dmax > 50 mm	C1	poorly structured rounded or angular materials with fraction 0/50 mm > 60-80 %
3	rocks	C2	strongly structured angular materials with fraction 0/50 mm < 60-80 %

Figure C.2 — French classification of soils and rocks for earthworks (NF P 11-300)

Synoptic table (GTR)

The classification and the possibilities of use of the materials require knowledge of the following parameters:

❖ Nature parameters

Nature parameters refer to **intrinsic characteristics**, i.e. those that do not vary or vary only slightly, neither over time nor during the various operations the soil undergoes during its implementation (See reference European Standard: intrinsic parameters).

The parameters of a fundamental nature for classification are:

- **Granularity** (Size distribution)
- **Argilosity** can be evaluated from two parameters which are the plasticity index $\mathbf{I_p}$, the most frequently used in the world, and the methylene blue value **VBS** (Reference France and Europe)

Where appropriate in case of doubt, an important parameter to consider:

- The organic content.

Other intrinsic properties

European Standard Reference

Other intrinsic properties can also be used to determine the suitability of materials:

- Mineralogy of grains;
- Genesis;
- Particle shape;
- Particle density;
- Optimum water content;
- Maximum dry density;
- Grain or particle strength;
- Particle mechanical soundness;
- Ease of weathering (including freeze thaw, shrinkage and water absorption) and particle durability;
- Frost heave potential;
- Chemical soundness and durability.

❖ State Parameters

State parameters are used particularly for implementation conditions. The most important parameter is: **Moisture content**

Characterization of hydric state

One of the following three parameters can be used to characterize the hydric state:

- The ratio of the natural water content $(\mathbf{W_N})$ to the **Optimum Normal Proctor OPN** $(\mathbf{W_{OPN}})$ of the material under study is the most reliable state parameter for characterizing very dry to medium water states.
- The most representative parameter of wet and very wet conditions is the **IPI** (Immediate Load-bearing Index) because it instantly reflects the difficulties of moving machinery.

- The soil consistency index **I_C** from which the natural water content (**W_N**) can be adjusted with respect to the Atterberg limits (**W_L et W_P**) by the relation:

$$IC = \frac{WL - Wnat}{IP}$$

This index allows for good characterization of very dry to very wet conditions, but only in the case of fine, moderately to very clayey soils with at least 80-90% 400 µm elements.

One can also consider the status parameter: e = the voids index

$$e = \frac{V_v}{V_s}$$

with V_v: Voids volume
V_s: Solids volume

Other parameters

Reference European Standard

The classification of soils for use in earthworks shall be based on the parameters of the following condition properties, as appropriate (List to be consulted as appropriate).

❖ Mechanical behaviour parameters

The mechanical behaviour parameters to be considered are as follows:

- Coefficient Los Angeles (LA)
- Micro Deval Coefficient in the presence of water (MDE)

❖ Variation of mechanical properties as a function of soil conditions

- Transmission of the stresses exerted on a ground

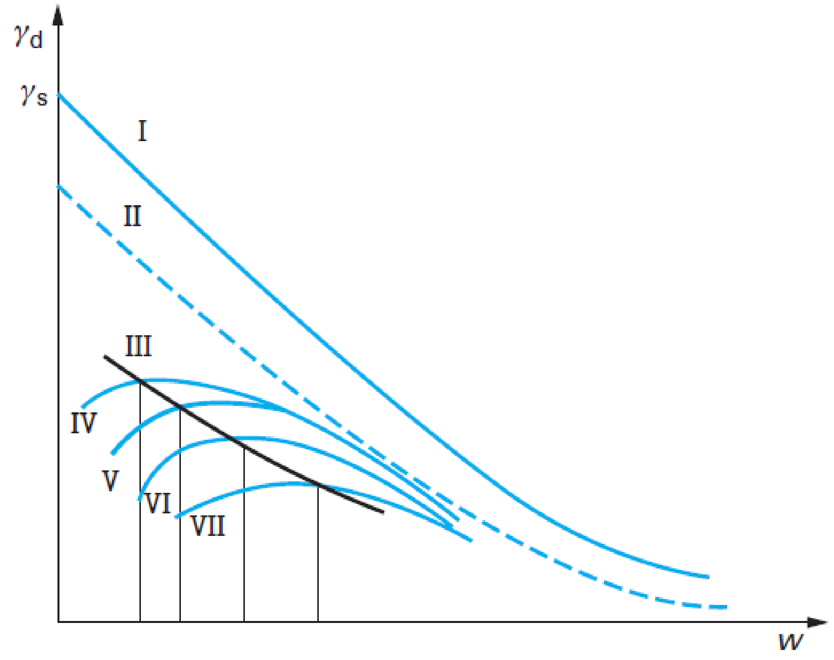
When a system of stresses is applied to aground, they are transmitted in different ways.

  - **Interstitial pressure** (pressure transmitted by water)
  - **Air pressure** (transmitted by air and steam)
  - **Intergranular stress**
  - **Suction**

Suction is caused by surface tension phenomena.

❖ Role of water and air in compaction

This role is fundamental in earthworks; it explains the influence of the state of the soil on its mechanical properties.



a air ratio

W water content

I saturation curve a = 0

II a = a0 ≈ 10%

III maxima area

*IV, V, VI, VII compa*ction energies

γ_d dry density

γ_s specific density of solid grains

Figure 2: Complete Proctor Diagram

❖ Proctor test taken into account

Reference GTR France

The water content and reference density of the materials of Dmax < 50 mm, used as embankment and as capping layer, are, according to the GTR specifications, those deduced from the **normal Proctor test** (NF P 94-093).

This test differs from the **modified Proctor** test (NF P 94-093) applicable to materials laid in underlayers and pavement layers.

For information, a fill is assumed to be properly compacted when the density of the material is 95% of the **Optimum Proctor Normal** (OPN) density, and a capping layer when the density of the material is 98% of the OPN density.

Optimum Proctor Normal corresponds to the optimum compaction characteristics of a material, its maximum dry density obtained with an optimum water content, measured under the conditions of the Proctor Normal test defined above.

As indicated above, management of the IPI is decisive because this parameter is "**the value used to assess the suitability of a soil or elaborated material to support the traffic of construction equiptment**".

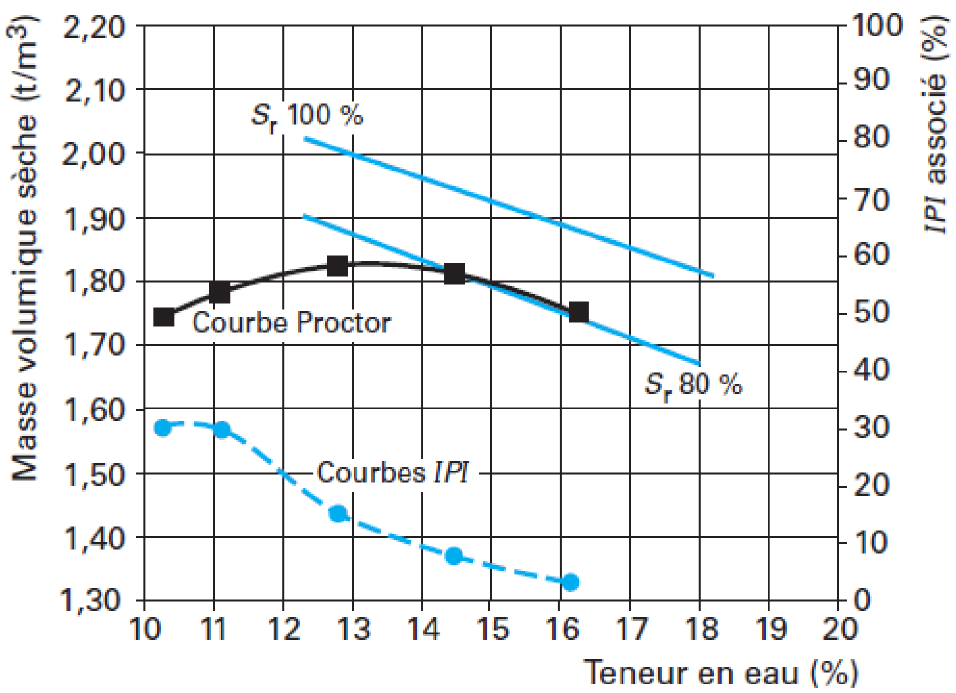


Figure 3: Example of a test carried out on fine silty sand

3.3.2. Case of rocks

❖ Parameters of the mechanical behaviour of a rock

The mechanical behaviour parameters are on the one hand, as for soils, the LA and MDE coefficients.

- Coefficient Los Angeles (LA)
- Micro Deval Coefficient in the presence of water (MDE)

On the other hand:

- Fragmentability coefficient *(*Reference France and Europe*)*

  It provides to assess the possibility of reusing evolving rock materials as backfill and certain more or less friable rock materials as bedding.

- Density of dehydrated rock in place (Reference France)

  Ce paramètre est très approprié en présence de craies et de calcaires tendres.

The essential parameter is the **Degradability Coefficient** (Reference France)

The interpretation of this parameter essentially concerns the possibilities of using materials from clayey rocks (marls, sedimentary shales, etc.) as backfill.

❖ State parameters used if applicable

- The natural water content

The influence of this parameter is only taken into account for certain very fragmentable chalk and clayey rocks.

- The content of soluble elements (% NaCl, gypsum...)

The interpretation of this parameter is obviously limited to the case of saline rocks.

4. MATERIAL IDENTIFICATION

The objectives of material identification are:

- on the one hand to determine the possibilities of reusing the materials
- on the other hand to provide the elements for the calculation of the stability of earth structures,
- as well as the elements necessary for the design of pavement structures

The identification of materials need essentially: Geological and hydrological studies and Geotechnical studies.

4.1. GEOLOGICAL AND HYDROLOGICAL STUDIES

These studies are carried out at the preliminary project and design stage of the earthworks.

Existing geological data generally provide indications of the formations encountered on the project site and an initial approach to the nature of the materials to be excavated and the risks that may arise.

The geological approach to the nature of the materials to be excavated is particularly important for rocks in order to determine the extraction methods: high-powered shovel, ripping, BRH equipment, mining, sorting ...

The analysis of natural geological and hydrological hazards will focus in particular on:

- flood or submersible areas,
- earthquake zones,
- slope stability,
- natural or man-made cavities,
- risk materials: swelling/shrinkage, erodability, liquefaction, collapsability, with disturbing elements (sulphates, organic materials, etc.)

Climatic conditions and the risks generated by climate change must also be taken into account.

The risks identified will most often require specific studies, such as:

- Physico-chemical analyses, particularly useful for detecting the content of penalizing elements
- Laboratory tests: material identification, formulation and processability studies,…

4.2. Geotechnical studies

Geotechnical studies are carried out, in whole or in part, at different stages of project's development:

- Preliminary draft
- Design
- Preparation period before work carrying out

❖ Study documents

Mainly:

- geological maps
- geotechnical reports and studies
- geotechnical long profile

4.3. Material reuse studies

The identification of materials makes it possible to predict which materials are suitable for use under certain conditions according to their nature and condition, by referring either to a material classification or to specific tests.

It is generally based on the following operations:

- geotechnical surveys: shovel, auger, core samples,...
- laboratory tests: identification, treatment study
- in situ tests: test bench (or test site), if justified by technical interest

5. GEOTECHNICAL SURVEY OF THE SITE

Geotechnical site investigation involves two approaches:

- Design of earth structures / Site survey for geotechnical calculations
- Earthworks Project / Specific Geotechnical Reconnaissance for Earthworks

These approaches need to be coordinated.

The geotechnical site investigation for a proposed earthwork project must provide adequate and sufficient information for the design and construction of the earth structures.

A geotechnical survey should include a theoretical study and a preliminary survey, in order to characterise the site in general terms.

Geotechnical survey must also include the detection and characterisation of particular areas: anthropolical embankment and storage areas, old mines or contaminated soils, polluted areas, etc.

The information is generally governed by standards, particularly with regard to the geotechnical calculation of the earth structures.

Reference Europe

Part of this information is governed by EN 1997-2 and should cover the design of the entire structure, as well as embankments and temporary structures created during construction. The geotechnical model must cover the area of influence of the structure, for mechanical analysis and for hydrological and hydrogeological analysis.

Reference France

Standard NFP 94-500 November 2013

Geotechnical engineering assignments – Classification

5.1. SITE RECONNAISSANCE FOR GEOTECHNICAL CALCULATIONS

In the case of Europe as an example, information is governed by two standards concerning:

- The design of the earth structures
- Investigations planning

Reference Europe

Earthworks create two types of earth-structures: "excavations and slopes" and "embankments". These structures must be designed according to the rules of EN 1997-1, which take into account the stability, deformation and durability of each structure.

EN 1997 2:2007 identifies the need to plan investigations in order to provide sufficient information for the different stages of design. Geotechnical measurements may be required to monitor stability, test design assumptions regarding terrain stability, monitor the behaviour of structures, and maintain evidence of adjacent structural works.

5.2. SPECIFIC GEOTECHNICAL RECONNAISSANCE FOR EARTHWORKS

The design of the Earthwork requires information on the nature and state of the materials encountered, soil or rock - and if necessary on alternative materials.

The geometry and volume of the natural terrain layers are based on geological analysis as well as drilling and other in-situ tests, including geophysical reconnaissance.

Part of this information is shared with the geotechnical calculation of the final structure and the main construction phases.

5.3. Geotechnical report

A geotechnical report is required. In this report, soils, rocks and other materials and their possible uses should be presented with the corresponding quantities and their possible interaction with the environment.

The preparation of the reports will be carried out at different phases of the site survey, design and construction.

We come back in more detail to the design of earth structures and the design of earthworks in the booklet "2C / Earthworks project" of the Earthworks Manual.

6. MATERIAL CLASSIFICATION SYSTEMS

See APPENDICES I.2 References / Classification (§6)

- I.2.1 PIARC Reference The different types of material classifications in the world
  - Extract PIARC Technical Report 2012
- I.2.2 Reference European Standard "Earthworks"
  - Excerpts Classification tables
- I.2.3 Reference European Standard "Earthworks"
  - Excerpts Appendix to the standard / National practice European references
- I.2.4 Reference GTR Guide (France)
  - Extract / Classification and rock parameters

6.1. Use of natural materials

We saw it earlier:

The identification of materials makes it possible to predict which materials are suitable for use under certain conditions according to their nature and condition, by referring either to a material classification or to specific tests.

Reference to a material classification is recommended.

6.2. Need for a specific material classification for earthworks

Earthworks are an important study and work phase of a road operation. They require the management of many parameters, particularly those relating to geotechnics, a necessary management to avoid or limit the technical and financial hazards of the project.

The specificity of these parameters implies the need for a classification dedicated to earthworks.

6.3. History (Référence GTR)

The various geotechnical classification systems for soil and rock materials proposed in the world up to now have generally been established with the aim of covering all the different fields of Civil Engineering where these materials are concerned (earth structures, foundations, slope stability, road foundations or aggregate production, etc.).

The complexity of soil behaviour means that the properties that are significant for a certain use are often no longer the same as soon as one looks at another use. This has led to the recommendation of a specific classification for the use of materials in the field of Earthworks.

This type of classification was developed in part, first in the United States with a first normative system dedicated to Earthworks.

Some European countries have done so, in particular France, in the 1990s, which drew up guides (GTR) with a specific classification for Earthworks, with additional recommendations for the use of materials in fill and capping layers, depending on parameters.

Today, many countries (especially in South America and Asia) use ASHTOO or similar systems.

Finally, the European standard "Earthworks", which was published in 2019, includes a Part 2 devoted to a classification of materials specific to Earthworks and provides us with recent reflections on this topic.

6.4. Aims of a classification of materials dedicated to earthworks

The main interests are as follows:

- Specific management of project materials for optimal use
- Simplified studies for already known materials
- Use of feedback from local experience
- Recommendations for matérials implementation (up to a re-use grid with implementation modalities)

Reference Europe (Standard "Earthworks" Part 1)

Use of classification systems

The materials used for earthmoving must be classified:

- in such a way as to allow the planning and specification of construction procedures and quality control methods ;
- so that the material requirements for earth structures correspond to the design requirements in terms of bearing capacity, serviceability and durability.

The main objectives of the classifications for earthworks are as follows:

1) Sorting materials into different classes with similar properties and behaviour;

(2) define prescribed terminology to limit misunderstandings concerning soils, rocks and other materials in the exchange of geotechnical reports and other documents.

The classifications applied to earthworks should cover the corresponding properties of soils and rocks in terms of excavation, transport, disposal and compaction.

6.5. COMPARISON OF DIFFERENT MATERIAL CLASSIFICATION SYSTEMS AROUND THE WORLD

The PIARC Technical Reports 2003 and 2012 include a trial comparison of different classification and specification systems around the world, following a survey of member countries. This document is attached as an Appendix. The 2003 report written by Hervé HAVARD (France) distinguishes between 3 types of classification.

PIARC Reference 2003

It seems possible to distinguish three types of soil classification:

A: fairly general soil classification which is not directly connected to job specifications, but rather to privileged fields of use with reservations for soil classes likely to generate difficulties, or even whose use is not recommended. These are generally soil classifications derived fairly directly from the USCS or HRB classification. In this case, we can cite the example of Germany and Switzerland. With these classifications, it is up to the design office responsible for drawing up the project to define which soils will be usable for this project and under what conditions. The specification is therefore not generalizable to all projects, but adapted to each particular project under the responsibility of the geotechnical engineer.

B: classification of all soils likely to be encountered and connected directly to a grid of possible re-use as fill or as a layer of form, possibly with special implementation methods to make the soils acceptable as fill or as a capping layer. This is particularly the case for the soil classifications used by France and Portugal. These classifications are specialised for the field of earthworks and it often happens that in the same country, there is a different reference classification depending on whether one is interested in other problems (for example in France for soil mechanics or for the management of material resources).

C: a third type of classification seems to be that developed by England, which starts from categories of use (e.g. soils usable as fill, soils usable as backfill near structures, etc.) to define the characteristics that soils must possess. This type of classification reverses the reasoning which is no longer "what use can we make of this soil? "but rather "for a given need, what are the acceptable soil characteristics". This classification is ipso facto a specification for contracts and is also included in the Specification for Road Works (SHW) in the 600 series (earthworks). As in type B classifications, this classification is obviously dedicated to earthmoving projects and there is another one used in soil mechanics.

The approach developed in the Technical Report can be found in Appendix I.1.

This comparison test is to be completed to take into account the European system (Standard EN 16907-2) of which we present extracts in Appendix I.2.

6.6. HARMONIZATION OF MATERIAL CLASSIFICATIONS

6.6.1. Differences between classifications in the world

Classification systems differ around the world.

The PIARC Technical Reports clearly show this.

This can be understood when considering the diversity of soil and rock types and also the climatic conditions encountered in different countries.

Differences appear on certain parameters taken into account, threshold values or test methods. The systems also differ in the importance given to recommendations for use and the modalities of implementation.

However, it is noted that the approaches are relatively similar and that classification systems are not fundamentally contradictory and could be seen as complementary to cover the diversities encountered in the countries.

It should be noted that the objective is a common one:

The objective remains to have the best system to design and build earthworks, economical and meeting the requirements in terms of bearing capacity, serviceability and durability.

6.6.2. Our recommendation in this regard

In the context we have just described, we recommend a classification system **dedicated to earthworks** that covers all soils likely to be encountered in the country concerned or in larger geographical areas when the materials issue is similar.

This system should incorporate, where relevant in the country concerned, for reasons of economic and environmental performance, the following provisions:

- The classification should be connected directly to a grid of possible re-uses in earthworks: backfill and capping layer in particular.
- It may be accompanied by special implementation procedures to make the soil acceptable as fill or as capping layer.

6.6.3. The harmonisation achieved in Europe

The most current reference in the field of harmonisation is the European Standard, which reserves an independent part to the classification of materials used in earthworks.

The European Standard establishes a **common frame of reference for the description and classification of materials** to be used by all parties involved in the design, planning and execution of earthworks.

It specifies the classification principles, processes and properties to be used for the description and classification of earthworks materials. For this purpose, this standard proposes soil and rock classes based on material specifications for the different parts of earth structures.

In particular, the standard identifies classification principles and systems, taking into account different national practices.

The different regional situations in terms of geology and climate give rise to national differences in earthworks procedures.

Appendix I.2.3 gives two examples of European national practices (Spain and France) as an informative Appendix to the Standard.

The standard also identifies test procedures suitable for earthworks.

The following extract from the European standard "Earthworks-Part 2" describes the principles of the European classification.

Reference European standard
"Earthworks Part 2" (prEN 16907-2)

Classification of materials according to the European standard "Earthworks"

1. Principles of classification

Soils, rocks and other materials shall be placed into groups (based on intrinsic properties) and classes (based on state properties) which have similar behaviour for one or more earthworks procedures (excavation, transport, treatment, placement and compaction) and which will have similar engineering properties in an earth structure after completion of the earthwork procedures.

The system of classes or classification system shall be defined on the basis of experience from previous works and is influenced by the geological and climatic conditions that prevail in each country.

The classification system may represent national practices, local practices based on experiences with particular materials or project based. It may be limited to materials for which previous experience exists. Variations between countries are allowed to account for national experiences. The national regulations mentioned in the Appendices to EN 16907-1 may be used where appropriate.

When a new classification system is prepared or an existing system is used, experience shall refer to:

- the satisfactory behaviour of completed earth-structures made of a class of material using specific construction procedures,
- the suitability of particular construction procedures for a given class of material.

The engineering behaviour of the completed earth-structure shall be assessed according to8 the type of structure, see prEN 16907-1.

- cuts in soils or rocks,
- embankments for infrastructure or construction,
- reclaimed land, or
- of coatings.

The mineralogy and particle size distribution of soils and the origin, fragmentability and degradability of rocks are important properties from the point of view of engineering behaviour. The sensitivity of fine grained soils to water has a major effect on the execution of works and the water content of fine soils shall be considered, when relevant, in the classification system.

Since earthworks require large volumes of material, the classification of samples is not sufficient to characterise a given source of natural materials: the description of the massif is part of the information essential for the delimitation of homogeneous areas and therefore for the design of earthworks.

2. The description and classification process

General

The ground shall be described and classified into materials of similar material properties and homogenous areas, a process which is typically supported by testing. This should involve three stages as given in Table 1:

1) The soil and rock materials shall be described in their in situ condition;

2) Classification into groups of similar material properties and homogenous areas based on the intrinsic properties shall be made as part of the design and planning process. Classification of the materials generally requires an adequate suite of testing; and

3) Classification shall be made during the design, planning and construction stages, on the basis of the state properties. Classification by state properties should be used to plan, specify and control the works and to demonstrate that the product required by the design and the specification has been achieved.

Table 1: The steps of description and classification

Step	Definition	Base	Application
Description	Engineering geological description of the soil and rock to record those features of the ground that will control the ease of excavation and influence the use of the material but that may be destroyed in sampling or earthwork procedures such as layering, variability and discontinuities.	Observation in the field and laboratory. Description of material and mass characteristics.	Grouping into areas of similar characteristics; determine layers, areas or zones of similar properties or homogenous character. Stratum descriptions aid the scheduling of testing which leads to classification.
Classification (intrinsic) **see Table 3**	Classification based on the intrinsic properties, those which are not changed by the effects of sampling or the earthwork procedures Assignment of soil and rock materials to soil and rock groups.	Selection of appropriate tests in order to determine the mineralogy and properties such as grain size and plasticity.	Classification to determine material applicability in different zones of the proposed earthwork. Available tests can be selected from those listed in Appendix A.
Classification (state)	Characterization on the basis of the state of the ground, that is by those properties that can change by sampling or by the earthwork procedures	Selection of appropriate tests to determine properties such as water content, strength, stiffness and stability. The properties to be measured will depend on the particle size of the soil and the strength of the particles.	Classification to determine classes depending on engineering properties at excavation, transport, deposition and compaction. Available tests can be selected from those listed in Appendix A.

7. RECOMMENDATIONS FOR THE USE OF NATURAL MATERIALS SOIL AND ROCK

Materials are generally identified by a classification.

Depending on the country, the recommendations for the use of materials are the subject of recommendations, guides and possibly standards.

As pointed out in paragraph 6, it is desirable to have a grid of possible re-use depending on the geotechnical properties and characteristics of the materials, a grid connected to the classification system and taking into account the feedback from experience.

Specific studies to determine the nature of the materials can thus be limited to materials that are difficult to use, such as marginal materials.

In paragraph 6, we recalled the different types of classification existing in the world according to PIARC Technical Reports.

Based on the classification, the methodology applied for the recommendations for use is differentiated according to the countries. It does not provide the same degree of recommendations, in particular for:

- the conditions for the reuse and use of all types of materials as backfill or in the form of a form layer
- the practical arrangements for carrying out compaction.

Example of the European "Earthworks" Standard

The examples of European National Practices in the Appendix of the European Standard, Part 2 "Classification of Materials" illustrate classification systems in connection with reuse grids. See Appendix I.2.3

Example of the GTR (France)

The Technical Guide to "Embankment and capping layers" (GTR July 2000 France) is a methodological tool recommended for road infrastructure studies and works.

The guide proposes a standard classification of all soils and rocks (1) and the conditions for the application of materials as fill and as subgrade.

Today it is complete reference for everything that concerns the use of materials in the field of earthworks.

(1) In France we find a wide variety of soils and rocks

See Appendix I.2.4 GTR Extract / Classification and Rock Parameter

APPENDICES I.1

MATERIALS PROPERTIES

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APPENDIX I.1.1

Propriétés relevant de la Mécanique des roches

La description et l'identification des roches qui découlent de ces propriétés seront très utiles à la méthodologie d'extraction des roches : choix des matériels, opération de ripage ou de minage notamment. Les valeurs qui sont proposées sont des valeurs usuelles, susceptibles de quelques variations le cas échéant.

1 Propriétés des roches

Dans le domaine des roches il est désormais classique de considérer deux échelles différentes :

- celle de la **roche**, correspondant à quelques décimètres cubes; c'est l'échelle de l'échantillon et de l'éprouvette soumise aux essais de 1aboratotre;
- celle du **massif rocheux**, qui correspond à quelques dizaines de mètres cubes ; c'est l'échelle des travaux de terrassements et des fondations d'ouvrages.

1.1. Identification

La **pétrographie** classe les roches en trois grands groupes en se fondant essentiellement sur des critères génétiques (tableau **1**) :

- les **roches éruptives** proviennent de la solidification du magma ;
- les **roches sédimentaires** sont formées à partir de dépôts d'éléments détritiques chimiques ou biochimiques ;
- les **roches métamorphiques** résultent des modifications subies par les roches éruptives ou sédimentaires lorsqu'elles sont soumises au métamorphisme.

Les roches cohérentes sont des solides particulièrement complexes du fait de l'hétérogénéité des constituants et des défauts de structure.

Tableau 1 – Famille des roches (1)

ROCHES MAGMATIQUES (plutoniques ou *volcaniques*)	**Famille des roches acides**	Granite, granodiorite, syénite, microgranite, *rhyolite*, *trachyte*
	Famille des roches intermédiaires	Diorite, microdiorite, *andésite*, *trachy-andésite*
	Famille des roches basiques	Gabbro, dolérite, ophite, *basalte*, péridotite
ROCHES MÉTAMORPHIQUES	**Roches massives**	Quartzite, cornéenne, migmatite, marbre
	Roches foliées	Gneiss, amphibolite, leptynite, micaschiste, schiste, ardoise
ROCHES SÉDIMENTAIRES	**Famille des roches carbonatées**	Calcaire, craie, dolomie, travertin, cargneule
	Famille des roches à dominante siliceuse	Grès, arkose, argilite, meulière, silex, radiolarite
	Famille des roches silico-carbonatées	Marne, molasse
	Famille des roches évaporitiques	Gypse, anhydrite, sel gemme
	Famille des roches carbonées	Charbon, lignite

(1) Extrait du guide TETR édité par le CFTR (cf. [Doc. C 5 360]).

Ce sont des solides poly-cristallins hétérogènes formés, le plus généralement, de grains appartenant à plusieurs espèces minérales. Les cristaux d'un même minéral peuvent eux-mêmes se différencier par leur état d'altération.

Les roches sont également des milieux discontinus ; il existe des vides inter-granulaires nommés pores, des défauts planaires soit intra-cristallins, soit inter-cristallins appelés fissures. La fissuration, qui joue un rôle considérable dans le comportement mécanique des roches, est la conséquence de l'hétérogénéité de comportement des différents constituants d'une roche.

Le volume des vides contenu dans une roche est exprimé par **la porosité** *n* :

$$n\ (\%) = \frac{V_v}{V}$$

V_v volume des vides,

V volume total.

La porosité des roches est très variable ; elle peut atteindre 20 %, voire même 40 %, pour certaines roches sédimentaires comme la craie, alors que, pour les roches éruptives ou métamorphiques non altérées, exception faite de quelques laves, la porosité est inférieure à 1 %.

La porosité exprime mal l'état de fissuration d'une roche, car le volume de vides lié aux fissures est très faible. Aussi, il est apparu nécessaire de quantifier, autrement que par la porosité, la densité de discontinuités présentes dans un échantillon de roche. La théorie, comme les mesures expérimentales, montre que la vitesse de propagation des ondes dans un solide est très influencée par la présence de discontinuités.

C'est la raison pour laquelle on utilise l'**indice de continuité I_c** :

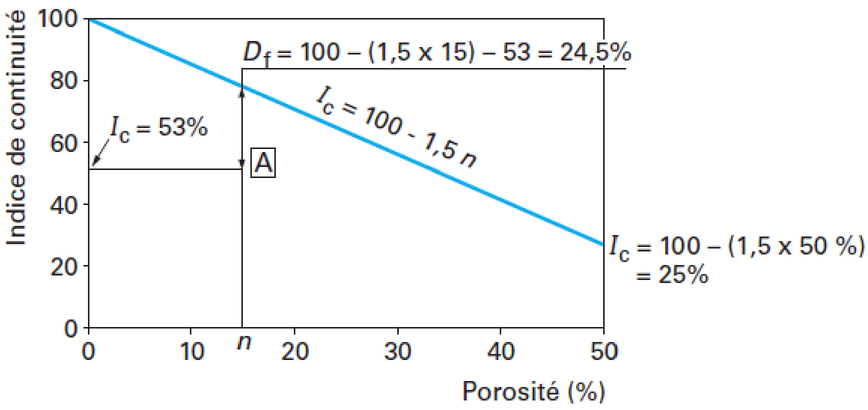
$$I_c = \frac{V_L}{V_L^\bullet} \times 100$$

V_L vitesse de propagation des ondes longitudinales dans une roche,

$V_L^\bullet$ vitesse théorique correspondante pour un milieu parfait, de même composition minéralogique que la roche, mais ne présentant pas de discontinuité.

Le guide technique TETR reprend ces notions et fait état de la relation établie de manière expérimentale représentée sur le graphique de l'Illustration 1.

La connaissance de l'indice de continuité et de la porosité d'une roche permet donc de caractériser son état de fissuration.



D_f : degré de fissuration

Sur ce diagramme, on a représenté la droite d'équation 100 – 1,5 *n* des milieux purement poreux (non fissurés), et le point A représentatif d'une roche de porosité *n* = 15 % et d'indice de continuité 53 % avec mise en évidence du degré de fissuration :
$D_f = 100 - 1{,}5\ n - I_c$ (*n* exprimé en pour-cent)

Illustration 1 – Relation entre indice de continuité et porosité d'une roche

Les roches sont souvent anisotropes : l'anisotropie peut être due à des orientations préférentielles de certaines espèces minérales ou de fissures. La mesure de la vitesse de propagation des ondes dans différentes directions permet également de caractériser l'anisotropie d'une roche.

1.2. Résistance mécanique des roches

Dans le domaine des contraintes usuelles, les roches ont un comportement fragile. C'est seulement dans des conditions de température et de pression élevées que les roches peuvent montrer une certaine ductilité.

Depuis les travaux de Griffith, il est bien établi que la rupture fragile est l'aboutissement du développement de fissures dans le solide. Il dépend des conditions opératoires que la propagation de la fissure puisse être ou non contrôlée. Le plus souvent, la résistance d'une roche est donnée par la résistance maximale obtenue dans un essai de traction ou dans un essai de compression simple.

La **résistance en traction** des roches est peu utilisée

Elle est beaucoup plus faible que la résistance en compression ;

Cette propriété peut être mise à profit dans toutes les opérations de cassage et de fragmentation des roches.

L'essai de traction le plus courant est un essai indirect, l'essai brésilien. Le plan de rupture sur lequel s'exercent les contraintes de traction est le plan diamétral de l'éprouvette.

Nous donnons ci-après quelques fourchettes de résistance en traction σ_t de roches, mesurée par l'essai brésilien :

- Calcaires...1 à 15 MPa
- Granites.. 10 à 20 MPa
- Basaltes.. 15 à 35 MPa

Sur le chantier, il est possible de caractériser la roche par un essai très rapide pouvant s'effectuer sur des carottes ou, même, sur des morceaux irréguliers. C'est l'essai sous charge ponctuelle.

La caractéristique mécanique la plus universellement utilisée pour les roches est **la résistance en compression simple** σ. Elle se mesure sur une éprouvette cylindrique d'élancement 2. La résistance en compression simple est le paramètre de base de la plupart des classifications proposées en mécanique des roches. Les intervalles et les termes utilisés dans la classification de base de la Société internationale de Mécanique des roches sont donnés dans le tableau **2** et repris dans le guide TETR avec l'indication des classes de résistances proposées par l'AFTES (Association française des travaux en souterrain).

Tableau 2 – Classes de résistance en compression simple des roches

Classe	**Résistance en compression uniaxiale σ_c** (MPa)	**Qualification de la résistance σ_c**
R_1	> 200	Très élevée
R_2	200 à 60	Élevée
R_3	60 à 20	Moyenne
R_4	20 à 6	Faible
R_5	< 6	Très faible

2 Propriétés des massifs rocheux

Les caractéristiques mécaniques des roches mesurées sur échantillons sont généralement élevées, et très supérieures à celles des massifs rocheux. Cette différence s'explique par la présence de surfaces de discontinuités.

Le terme général de discontinuité recouvre des surfaces d'origines très diverses :

- les plans de stratification des formations sédimentaires (calcaires, grès, marnes, pélites, etc.), qui ont une grande continuité spatiale ;
- les plans de schistosité de certaines roches métamorphiques (schistes, ardoises) ;
- les failles qui résultent d'une rupture par cisaillement avec déplacement relatif des épontes ;

- les diaclases dont l'origine est plus controversée, mais qui sont présentes dans pratiquement toutes les masses rocheuses ;
- les joints de retrait dus à un refroidissement rapide de laves ; on peut particulièrement les observer dans les coulées basaltiques.

L'étude structurale d'un massif rocheux consiste à hiérarchiser entre elles ces différentes familles, en distinguant les accidents majeurs isolés (failles) d'éléments répétitifs (plan de stratification, diaclases).

Pour chaque famille on s'efforce d'indiquer :

- l'orientation des surfaces (en indiquant l'azimuth et le pendage) ;
- leur extension ;
- l'état des épontes (planéité, rugosité, imbrication) ;
- l'existence et la nature du remplissage (remplissage bréchique, calciteux, argileux, etc.).

L'orientation des différentes surfaces de discontinuités est représentée sur un diagramme polaire en faisant une projection stéréographique.

On peut classer les massifs rocheux suivant le nombre de familles de discontinuités :

- massif avec très peu de discontinuités aléatoirement réparties ;
- massif avec une famille de discontinuités, avec ou sans discontinuités aléatoires (massif stratifié par exemple) ;
- massif avec deux familles de discontinuités, avec ou sans discontinuités aléatoires ;
- massif avec trois familles de discontinuités ;
- massif très fracturé avec de nombreuses discontinuités plus ou moins groupées en plus de trois familles.

2.1. État de fracturation d'un massif rocheux

L'état de fracturation d'un massif rocheux est particulièrement utile au choix des méthodes de réalisation.

L'étude complète de la structure d'un massif rocheux est souvent très longue et très difficile ; aussi utilise-t-on fréquemment des indices que l'on peut déterminer à partir de forages ou sur affleurements, et qui permettent de caractériser l'état de fracturation d'un massif.

L'indice le plus répandu actuellement est le ***RQD*** (*Rock Quality Designation*). Il se détermine par la longueur cumulée $\sum l_i$ des éléments de carottes supérieurs à 10 cm de long par rapport à la longueur ***L*** de sondage considérée. Il est exprimé en pour cent :

$$RQD = 100 \frac{\sum \ell_i \left(> 10 \text{ cm}\right)}{L}$$

À partir de ce seul indice, une classification peut être établie comme la suivante (Référence Don Deere) :

- RQD > 90 %..................État de fracturation du massif très faible
- 75 % < RQD < 90 % ....État de fracturation du massif faible
- 50 % < RQD < 75 % .... État de fracturation du massif moyen
- 25 %< RQD < 50 % .... État de fracturation du massif fort
- RQD < 25 %.....................État de fracturation du massif très fort

L'indice *RQD* apparaît souvent insuffisant, aussi la Société internationale de Mécanique des roches recommande-t-elle l'utilisation de l'espacement moyen des discontinuités. Le guide technique « *Terrassements à l'explosif dans les travaux routiers* » (France) utilise la notion de **densité**.

La **densité de discontinuités** dans le massif est souvent quantifiée par l'**intervalle de discontinuités *ID*** mesuré suivant une ligne (un sondage carotté, une ficelle tendue sur une paroi rocheuse...) et défini comme l'intervalle moyen entre deux discontinuités successives. Il est calculé sur une base glissante ***L*** (plusieurs mètres en général), ou pour chaque unité homogène reconnue. Il est conseillé d'utiliser les classes suivantes de l'intervalle ***ID***, proposées par l'AFTES (France) dans le tableau suivant :

Tableau 3 – Notion de *densité* selon le guide TETR

Classe	**Intervalle** (cm)	**Description**
1	> 200	Densité très faible
2	200 à 60	Densité faible
3	60 à 20	Densité moyenne
4	20 à 6	Densité forte
5	< 6	Densité très forte

2.2. État d'altération d'un massif rocheux

Les processus d'altération des roches sont extrêmement divers. Suivant leur intensité, la roche peut présenter tous les états intermédiaires entre une roche saine et un sol résiduel, par exemple les arènes granitiques. Le plus souvent, l'altération progresse par les surfaces de discontinuités. La description de l'état d'altération peut se faire à partir de la classification donnée dans le tableau **4**.

Tableau 4 – Classification des roches en fonction de l'état d'altération	
Roche	**Description de l'état d'altération**
Saine	Aucun signe apparent d'altération, avec éventuellement une légère décoloration sur les surfaces de discontinuités majeures
Légèrement altérée	Une décoloration montre l'altération de la roche et des discontinuités
Modérément altérée	Moins de la moitié de la roche est décomposée et/ou réduite en sol ; des blocs de roches saines ou décolorées sont présents et forment une structure discontinue
Très altérée	Plus de la moitié de la roche est décomposée et/ou réduite en sol ; présence de blocs de roche saine ou décolorée formant une structure très discontinue
Complètement altérée	Toute la roche est décomposée et/ou réduite en sol. La structure originelle du massif rocheux est encore largement intacte

2.3. Abrasivité

Un essai qui se pratique sur un granulat de 4 à 6,3 mm (Référence norme XP P 18-579), permet de quantifier la capacité d'une roche à user les pièces métalliques : cette caractéristique est donc importante pour les phases de foration et de reprise des matériaux abattus après le tir.

Selon cet essai, les calcaires purs ont une abrasivité voisine de 0 (la calcite n'est pas un minéral abrasif) ; à l'opposé, les roches siliceuses compactes telles que les quartzites ont une abrasivité qui peut dépasser 2 000 (tableau **5**).

Tableau 5 – Classes d'abrasivité suivant le guide TETR		
Classe	**Coefficient d'abrasivité *ABR*** (g/t)	**Description de l'abrasivité**
ABR_1	< 500	Très faible
ABR_2	500 à 1 000	Faible
ABR_3	1 000 à 1 500	Moyenne
ABR_4	1 500 à 2 000	Forte
ABR_5	> 2 000	Très forte

2.4. Prévision des conditions d'extraction des massifs rocheux

Il est important, pour les travaux de terrassement, de prévoir les conditions d'excavation des déblais. Nous y revenons de façon détaillée dans la partie « Réalisation ».

Rappelons à ce stade que les principales méthodes d'extraction sont :

- l'utilisation d'explosifs avec la foration de trous de mines ;
- le défoncement avec des tracteurs puissants ; les plus puissants dépassent 500 ch (≈ 370 kW) ;
- l'utilisation de pelles.

Ces deux dernières méthodes ne peuvent être utilisées que si les roches ne sont pas trop dures et le massif trop compact.

La prévision des conditions d'extraction des massifs rocheux est généralement faite par l'une ou la combinaison des méthodes suivantes :

- mesure de la vitesse de propagation des ondes longitudinales dans le massif
- indications sur les caractéristiques de la roche (indice de continuité) et sur l'état de fracturation du massif
- sondages carottés (onéreux)
- sondages destructifs dans lesquels on fait des diagraphies des vitesses sismiques
- carottages soniques.

En conclusion, nous donnons simplement dans le tableau **6** les indications sur les prévisions d'extraction fournies par les vitesses de propagation des ondes longitudinales obtenues par des essais de diagraphies sismiques.

L'Illustration 2 représente une allure de l'accroissement de difficulté en fonction de la vitesse du son, suivant la puissance du tracteur qui porte la dent de ripage.

Tableau 6 – Conditions d'extraction déterminées par sondage sismique (1)

Nature du banc rocheux vis-à-vis de son extraction	**Vitesse du son mesurée par sondage sismique** (m/s)
Rippable	$V < 1\,500$
Difficilement rippable	$1\,500 < V < 2\,500$
À extraire avec emploi d'explosif	$V > 2\,500$
(1) Les frontières de ce tableau sont approximatives.	

(Mesure longitudinale)

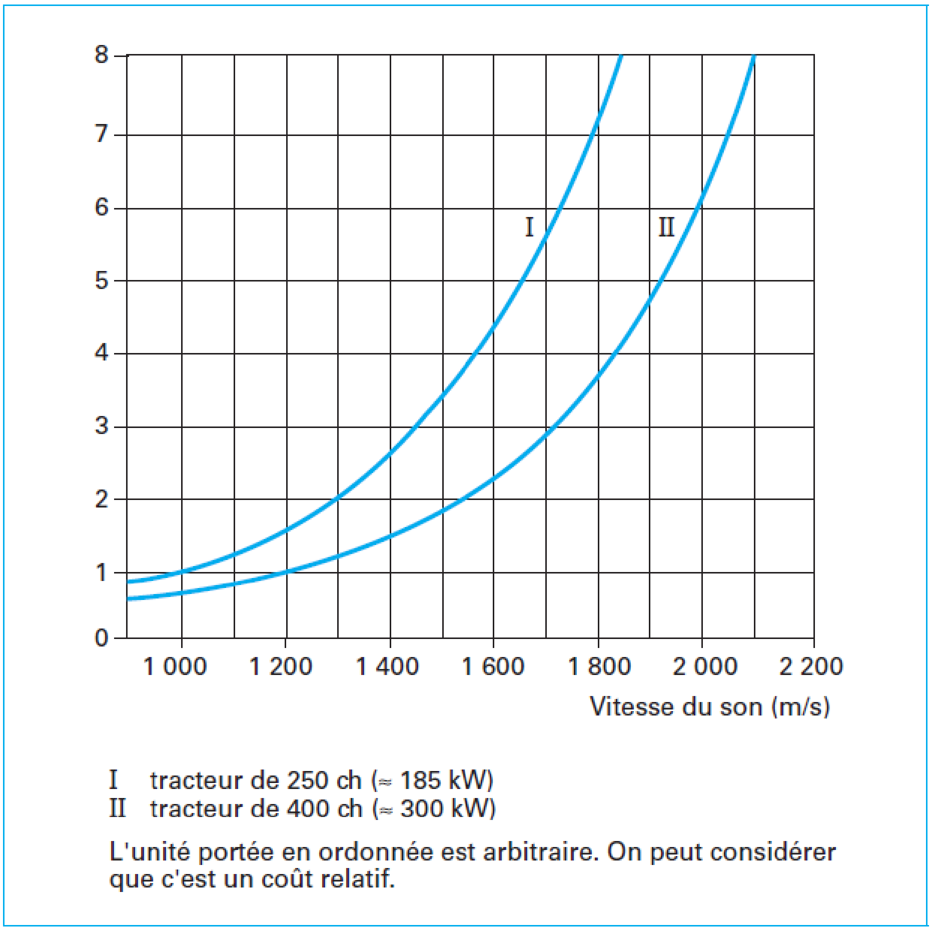


Illustration 2 – Ordre de grandeur de la difficulté à ripper un banc en fonction de la vitesse du son

APPENDIX I.1.2

Propriétés relevant de la Mécaniques des sols

Nous rappelons les notions de base qui intéressent la conception des ouvrages en terre, notamment en terme de stabilité des talus et de déformation.

1. Courbe intrinsèque

Rappelons d'abord que l'on appelle **courbe intrinsèque** d'un matériau une courbe limitant le domaine élastique dans un plan défini par la contrainte normale portée en abscisse et la contrainte de cisaillement portée en ordonnée (Illustration 1)

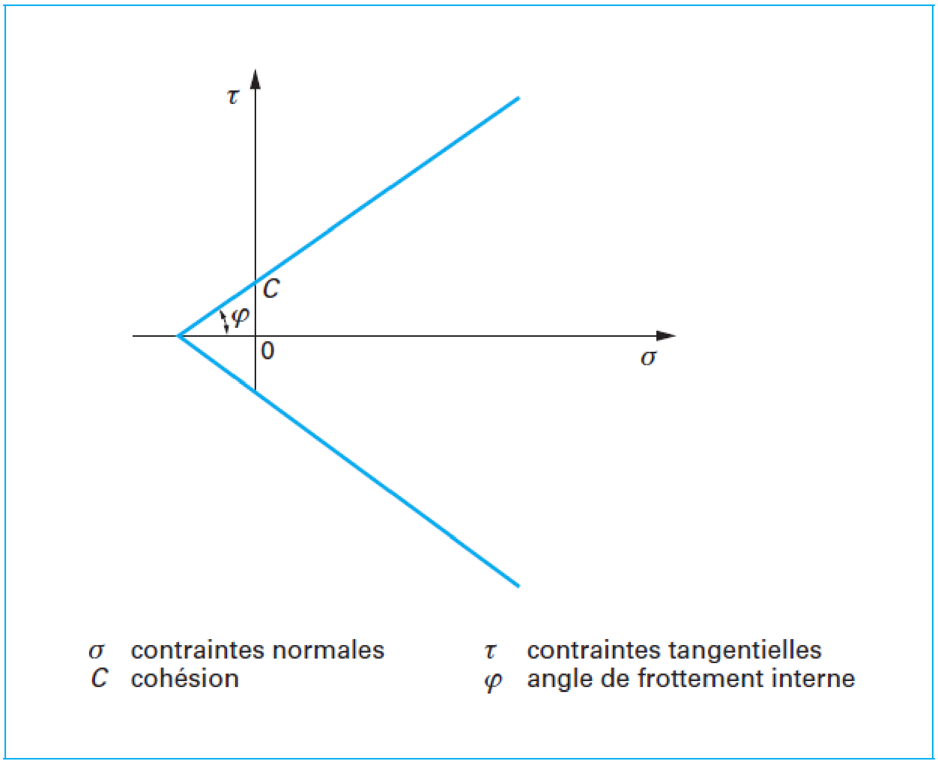


Illustration 1 – Courbe intrinsèque d'un sol

Cette notion, appliquée à la Mécanique des sols, a dû être quelque peu déformée car le domaine élastique de ces matériaux est très réduit et le domaine plastique au contraire très étendu : en Mécanique des sols, la courbe intrinsèque limite le domaine de stabilité.

L'expérience a montré que cette courbe est formée de deux demi-droites (Illustration 1) ; l'ordonnée à l'origine est la cohésion ***C*** et l'angle que fait la demi-droite supérieure avec l'axe des contraintes normales est l'angle de frottement interne ***φ***.

Les deux caractéristiques fondamentales ***C*** et ***φ*** ne sont pas des valeurs absolument fixes pour un sol donné, mais qu'elles dépendent de l'état du sol et, en particulier, de sa teneur en eau et des conditions de l'essai.

Cette courbe intrinsèque amène à distinguer deux sortes de sols : les **sols pulvérulents** pour lesquels ***C*** est nulle et les **sols cohérents** qui possèdent une certaine résistance ***C*** au cisaillement, même en l'absence de toute contrainte normale.

La forme et les caractéristiques de la courbe intrinsèque impliquent les conséquences suivantes sur la réalisation des terrassements.

Les **sols cohérents** peuvent présenter des parois verticales stables, tant que la hauteur de cette paroi est inférieure à :

$$\frac{2C}{\gamma \tan\left(\frac{\pi}{4}-\frac{\varphi}{2}\right)}$$

Formule dans laquelle ***C*** et ***φ*** ont les significations indiquées précédemment, ***γ*** étant la masse volumique du sol.

Par contre, les **sols pulvérulents** ne peuvent théoriquement jamais conserver une paroi verticale, fût-elle de très faible hauteur. En pratique, ils ne conservent de paroi verticale que de hauteur vraiment très faible.

Sols pulvérulents : sables et graviers sans fines

Lorsque le terrassement a été terminé, les massifs de terre, qu'ils soient de remblais ou de déblais, sont limités par des talus qui ne peuvent être stables que si les sols sont dans des états de contrainte inférieurs à la courbe intrinsèque. Le problème de la stabilité des pentes est une des premières difficultés du Terrassier.

On ne peut déformer un sol qu'en lui imposant des contraintes de cisaillement suffisantes. Lorsque, en particulier, on veut le compacter, il faut exercer sur le massif à densifier des contraintes de cisaillement.

2. Dilatance

L'expérience montre que lorsque l'on cisaille un sol, son volume varie et que le sens de cette variation dépend de l'état initial de ce sol.

Lorsque l'on cisaille un sol lâche (c'est-à-dire peu compact), il se densifie.

Au contraire lorsque l'on cisaille un sol compact, il se dilate.

Il existe donc, pour un sol donné, une densité critique telle que, en dessous de celle-ci, un cisaillement le compacte tandis qu'au-dessus de celle-ci un cisaillement le décompacte. On donne à ce dernier phénomène le nom de **dilatance**.

Cette propriété a peu d'implications dans le Terrassement.

APPENDIX I.1.3

Propriétés géotechniques applicables au terrassement

Les propriétés qui se réfèrent à la Mécanique des sols sont en général, nous l'avons vu, importantes pour la conception des ouvrages en terre.

Le domaine qui nous intéresse très directement est l'**application de la géotechnique** aux travaux de terrassement.

Ce domaine comporte l'étude de certaines propriétés caractéristiques des sols concernant en particulier **leur nature et leur état**.

Rappelons que la cohésion et l'angle de frottement interne, aussi bien que le phénomène de dilatance dépendent de ces deux grands facteurs.

Définition d'un sol

Un sol est un matériau constitué essentiellement de grains solides distincts et pouvant contenir, en outre, de l'eau et de l'air, les grains étant à une échelle telle, par rapport au volume du sol considéré, que la loi des grands nombres soit applicable.

Cette définition implique qu'un sol est un mélange d'une phase solide dispersée constituée par des grains de roche, d'une phase liquide soit dispersée, soit continue et d'une phase gazeuse, qui peut être, elle aussi, dispersée ou continue.

On ne change pas la nature du sol en modifiant les deux autres phases, en augmentant ou en diminuant la quantité d'eau ou celle d'air et en modifiant leur distribution dans le sol. Toutes ces modifications n'altèrent pas la nature du sol mais son état.

1. Paramètres correspondant aux propriétés géotechniques

Nous présentons ci-après les différents paramètres qui nous paraissent les plus appropriés pour définir des classes de matériaux et leurs possibilités d'emploi.

Les valeurs seuils sont différentes suivant les pays utilisant ces paramètres.

Comme exemple, nous citons ici surlignées en ocre les valeurs seuils du Guide Technique GTR (France).

Le guide technique de *« Réalisation des remblais et des couches de forme »* (GTR juillet 2000 France) est un outil méthodologique préconisé pour les études et travaux d'infrastructures routiers. Le guide propose une classification normalisée de tous les sols et roches et les conditions de mise en œuvre des matériaux en remblai et en couche de forme.

Il constitue aujourd'hui une référence approfondie et complète pour tout ce qui concerne l'utilisation des matériaux dans le domaine des terrassements.

Nous nous inspirons de ce Guide pour définir les propriétés géotechniques applicables au terrassement.

Le classement et la réutilisation des matériaux nécessitent la connaissance des paramètres suivants:

- paramètres de nature
- paramètres d'état
- paramètres de comportement mécanique

Ces différents paramètres seront exposés dans les paragraphes qui suivent en tenant compte, d'une part, de ceux qui sont jugés suffisants pour l'utilisation du Guide et, d'autre part, de l'ensemble des critères nécessaires à leur connaissance.

1.1. Paramètres de nature

Nous distinguerons les paramètres de nature des sols et des roches qui nous paraissent indispensables (en exemple : référence GTR) et certains paramètres pouvant compléter si nécessaire la connaissance approfondie du matériau.

Paramètres de nature retenus dans le Guide de référence GTR

> Les paramètres de nature se rapportent à **des caractéristiques intrinsèques**, c'est-à-dire qui ne varient pas ou peu, ni dans le temps ni au cours des différentes manipulations que subit le sol au cours de sa mise en œuvre.

1.1.1. Paramètres de nature des sols

Les paramètres intrinsèques de nature fondamentaux sont :

- La granularité
- L'argilosité

Granularité

Ces valeurs peuvent être sensiblement différentes dans les classifications d'autres pays dans le monde. On peut se référer en la matière aux comparaisons faites dans le Rapport Technique PIARC de 2003. Les principes restent toutefois les mêmes.

L'analyse granulométrique des matériaux doit permettre leur classement en tenant compte de leur diamètre maximal D_{max} et de leurs tamisats à **2 mm** et à **80 µm.**

Le D_{max} est la dimension maximale des plus gros éléments contenus dans le matériau étudié. C'est un paramètre très important pour préjuger des ateliers de terrassement envisageables, pour évaluer l'épaisseur des couches élémentaires, etc.

Le seuil de 50 mm est la valeur proposée pour distinguer les sols fins, sableux et graveleux, des sols blocailleux. Il peut aussi permettre de différencier des sols pouvant (ou non) être malaxés avec un liant.

En outre, le comportement d'un sol à éléments anguleux de D_{max} supérieur à **50 mm** peut être bien appréhendé à partir des essais de laboratoire réalisés sur sa portion de matériaux 0/50 mm, lorsque cette dernière est supérieure à 60 à 80 % de l'ensemble du matériau.

En revanche, cela n'est plus envisageable si cette fraction devient inférieure à 60 à 80 %.

Le tamisat à 2 mm permet de différencier les sols à tendance sableuse des sols à tendance graveleuse.

Le seuil de 70 % place au-delà de cette valeur les sols à tendance sableuse et en deçà les sols à tendance graveleuse.

Le tamisat à 80 µm permet de distinguer les sols riches en fines et d'estimer leur sensibilité à l'eau.

- **Le seuil de 35 %** est celui au-delà duquel le comportement du sol peut être considéré comme régi par celui de la fraction fine (< 80 µm).
- **Le seuil de 12 %** permet d'établir une distinction entre les matériaux sableux et graveleux pauvres ou riches en fines.

Argilosité

L'argilosité peut être évaluée à partir de deux paramètres qui sont l'indice de plasticité I_p, le plus fréquemment utilisé dans le monde, et la valeur de bleu de méthylène ***VBS***.

- **Indice de plasticité I_p**

L'indice de plasticité d'un sol permet de mesurer l'étendue du domaine de plasticité de ce sol, sachant que :

$$I_p = w_L - w_P$$

- w_L limite de liquidité qui sépare l'état liquide de l'état solide,
- w_P limite de plasticité qui sépare l'état plastique de l'état solide.

À titre indicatif, si la proportion de la fraction pondérale 0/400 mm d'un sol excède 50 % et si I_p > **12**, l'interprétation de ce dernier sera aisée.

En revanche, elle deviendra compliquée, voire impossible, si la proportion tombe au-dessous de 35 % et la valeur de I_p au-dessous de **7**.

Trois seuils sont retenus dans le GTR, à savoir :

- **12** : limite supérieure des sols faiblement argileux ;
- **25** : limite supérieure des sols moyennement argileux ;
- **40** : limite entre les sols argileux et très argileux.

- **Valeur de bleu de méthylène *VBS***

Contrairement à celle de **l'I_p**, l'application de la ***VBS*** à l'identification des sols, est récente.

La ***VBS*** exprime globalement la quantité et la qualité (ou activité) de l'argile contenue dans un sol (Fraction 0/50 mm). En effet, cet essai, qui permet de mesurer la quantité de bleu pouvant s'adsorber dans un sol, est directement lié à la surface spécifique de ce sol, elle-même en relation directe avec la surface des particules contenues dans sa fraction argileuse (passant à 2 µm).

Les seuils retenus dans le GTR sont donnés ci-après :

- **0,1** : seuil au-dessous duquel on peut considérer que le sol est insensible à l'eau si le tamisat à 80 µm est inférieur à 12 % ;
- **0,2** : seuil au-dessus duquel apparaît à coup sûr la sensibilité à l'eau ;
- **1,5** : seuil distinguant les sols sablo-limoneux des sols sablo-argileux ;
- **2,5** : seuil distinguant les sols limoneux peu plastiques des sols limoneux de plasticité moyenne ;
- **6** : seuil distinguant les sols limoneux des sols argileux ;
- **8** : seuil distinguant les sols argileux des sols très argileux.

- **Choix entre I_p et *VBS***

On pourra se référer à la partie X Contrôles pour la comparaison des deux méthodes.

La ***VBS*** est applicable à l'identification de tous les sols car elle exprime globalement la quantité et l'activité de l'argile contenue dans le sol.

Toutefois, I_p présente des avantages dans le cas de sols moyennement à très argileux :

- c'est un paramètre dont on dispose d'une longue expérience pour ce qui concerne son interprétation ;
- il est plus sensible que la ***VBS*** dès que les sols deviennent vraiment argileux ;
- I_p est un paramètre d'identification mais aussi un paramètre de comportement. Comme nous l'avons vu précédemment, il définit l'intervalle de teneur en eau dans lequel le sol reste souple et déformable, tout en conservant une certaine résistance au cisaillement. La connaissance de cet intervalle est très utile dans la conception des ouvrages en terre.

Nota :

Le paramètre **Équivalent de sable** a été également utilisé. Il l'est moins aujourd'hui, la valeur VBS est suffisante pour la classification.

1.1.2. Paramètres de nature des roches

Pétrographie

La pétrographie (discipline de la géologie fondamentale) est essentiellement utilisée pour caractériser la nature d'un massif rocheux.

La classification des roches différencie en général (Exemple GTR France) :

- les roches sédimentaires comprenant :
  - les roches carbonatées (calcaires, craies),
  - les roches argileuses (marnes, argilites, pélites...),
  - les roches siliceuses (grès, poudingues, brèches...),
  - les roches salines (sel gemme, gypse...)
- les roches magmatiques et métamorphiques (granites, basaltes, andésites, gneiss, schistes métamorphiques et ardoisiers...).

1.1.3. Autres paramètres de nature

Pour les sols

D'autres paramètres sont listés (par méthode d'essai) ci-dessous, de manière à donner au lecteur des possibilités lui permettant de bien appréhender et maîtriser la nature des sols.

- mise en évidence des matières organiques par colorimétrie
- détermination de la teneur pondérale en matières organiques d'un sol
- détermination de la granulométrie des sols fins par sédimentation

Pour les roches

- masse volumique sèche d'un élément de roche.

1.2. Paramètres d'état

Les paramètres d'état « **ne sont pas propres au matériau considéré, mais sont fonction de l'environnement dans lequel il se trouve** ».

Le présent paragraphe traite du paramètre d'état des sols. Par analogie, ce dernier peut être transposé aux roches fortement altérées ou naturellement friables ou dégradables dont la consistance et le comportement mécanique sont similaires à ceux d'un sol.

Quant aux paramètres d'état des roches massives dures, n'ayant quasiment aucune influence sur leur comportement aussi bien à l'extraction qu'à la mise en œuvre, bien que proches dans leur définition de ceux des sols, ils ne seront pas développés dans le présent document.

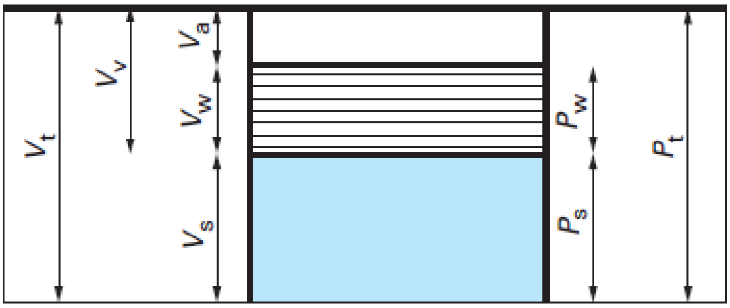


Illustration 1 – Schématisation des grains solides, de l'eau et de l'air dans un sol

1.2.1. Schématisation et définition des paramètres d'état

Un sol constitué de grains solides, d'eau et d'air peut être schématisé comme indiqué sur l'Illustration 1.

Les paramètres d'état sont particulièrement utilisés pour les conditions de mise en œuvre

- la teneur en eau : W = (Pw / Ps) × 100
- l'indice des vides : e = (Va + Vw) / Vs = Vv / Vs

1.2.2. Paramètres d'état retenus dans le GTR

État hydrique

Le seul paramètre d'état pris en compte dans la classification du GTR est l'**état hydrique**. En effet, « son importance est capitale vis-à-vis de tous les problèmes de remblai et de couche de forme ». Cinq états hydriques sont différenciés.

L'état « très humide » (th) : c'est un état qui ne permet pas la réutilisation du sol dans les conditions technico-économiques françaises actuelles.

L'état « humide » (h) : cet état d'humidité élevée autorise la réutilisation du sol en prenant des dispositions particulières telles que l'aération, le traitement, etc.

L'état « moyen » (m) : c'est l'état d'humidité optimal qui présente le minimum de contraintes pour la mise en œuvre.

L'état « sec » (s) : il s'agit d'un état d'humidité faible qui autorise encore une mise en œuvre des matériaux en prenant des dispositions particulières (arrosage, surcompactage, etc.).

L'état « très sec » (ts) : cet état d'humidité très faible n'autorise plus la réutilisation des sols dans les conditions technico-économiques françaises actuelles.

Néanmoins, dans le cas d'un mouvement de terre très déficitaire, la réutilisation de ces sols peut être envisagée moyennant des techniques appropriées telles que l'humidification dans la masse des matériaux (arroseuse enfouisseuse) et traitement à la chaux, *par exemple*.

<u>Caractérisation de l'état hydrique</u>

Le rapport de la teneur en eau naturelle **(W_N)** sur la teneur en eau à l'**optimum Proctor normal *OPN* (W_{OPN})** du matériau étudié est le paramètre d'état le plus fiable pour caractériser les états hydriques très secs à moyens

Référence GTR : états (m), (s) et (ts).

Le paramètre le plus représentatif des états humides et très humides est l'***IPI*** (indice portant immédiat) car il traduit instantanément les difficultés de circulation des engins.

Référence GTR : états (h) et (th)

Le paramètre *IPI* est mesuré dans un moule **CBR** *(Californian Bearing Ratio)*, sans surcharge ni immersion, sur une éprouvette de sol compacté à l'énergie Proctor normal et à sa teneur en eau naturelle. Se référer à la Partie Contrôles pour plus de détails sur l'IPI.

La prise en compte de cet indice est développée dans le paragraphe suivant qui traite des paramètres de comportement mécanique.

L'**indice de consistance I_C** du sol à partir duquel on peut recaler la teneur en eau naturelle (W_N) par rapport aux **limites d'Atterberg** (**W_L et W_P**) par la relation

$$I_C = (W_L - W_N) / I_P$$

Cet indice permet de bien caractériser les états très secs à très humides, mais seulement dans le cas de sols fins moyennement à très argileux comportant au moins 80 à 90 % d'éléments 400 µm**.**

1.3. Paramètres de comportement mécanique

1.3.1. Paramètres de comportement mécanique d'un sol

<u>Paramètres intrinsèques de comportement mécanique</u>

Ces paramètres ne sont généralement pris en compte que pour juger de l'utilisation possible des sols en couches structurantes comme la couche de forme. Il s'agit d'un critère essentiel principalement issu de l'expérience.

Ils permettent de mettre en évidence :

- les matériaux dont la fraction granulaire est susceptible de résister au trafic et au compactage et qui, de ce fait, peuvent être réutilisés en couche de forme ;
- les matériaux qui risquent de se fragmenter et de se transformer en un sol inutilisable en couche de forme dans son état naturel sans disposition particulière (traitement, par exemple).

Les paramètres de comportement mécanique qui sont considérés, notamment dans la classification GTR, sont les suivants :

- Coefficient Los Angeles (*LA*)
- Coefficient Micro Deval en présence d'eau (MDE) mesuré sur la fraction 10/14 mm ou, à défaut, sur la fraction 6 /10 mm.

Les seuils retenus sont de **45** pour les valeurs de *LA* et *MDE*.

Nota : Les normes correspondantes doivent être spécifiques aux travaux de terrassement. D'autres références de normes se rapportant aux coefficients *LA* et *MDE* existent dans le domaine des granulats.

À titre d'information, certains maîtres d'ouvrage imposent la réalisation de ces essais sur des fractions granulaires plus appropriées à l'usage des matériaux, comme la fraction 16/31,5 mm, par exemple, pour la mesure du *LA* d'une sous-couche ferroviaire, ou la fraction 25/50 mm pour la mesure du *LA* et du *MDE* sur des matériaux de zone inondable (ZI).

- **Le coefficient de friabilité des sables (FS),** mesuré sur la fraction 0/1 mm ou 0/2 mm peut être aussi considéré.

Le seuil retenu est de 60 pour les valeurs de FS.

<u>Variation des propriétés mécaniques en fonction de l'état du sol</u>

Voir Annexe I.1.4 du présent Livret - Compléments techniques

- Contrainte totale, pression interstitielle, pression d'air, contrainte inter-granulaire, succion
- Rôle de l'eau et de l'air dans le compactage
- Représentation de Proctor

1.3.2. Paramètres de comportement mécanique d'une roche

<u>Paramètres intrinsèques de comportement mécanique</u>

Pour caractériser un massif rocheux en vue de son emploi en remblai ou en couche de forme, le géotechnicien doit, après l'identification des paramètres de nature, définir le comportement mécanique des roches afin d'envisager avec le plus de précision possible quel sera leur comportement tout au long de leurs phases successives de terrassement : extraction, chargement, régalage, compactage, risques d'évolution une fois l'ouvrage en service, etc.

Cette démarche permettra le classement des roches étudiées selon les prescriptions de la classification GTR.

Les paramètres de comportement mécanique sont :

- comme pour les sols les **coefficients *LA* et *MDE*** ;
- **le coefficient de fragmentabilité** (FR – norme NF P 94-066) qui permet de juger de la possibilité de réemploi en remblai de matériaux rocheux évolutifs et en couche de forme de certains matériaux rocheux plus ou moins friables pour lesquels les coefficients LA et MDE manquent de sensibilité. Ce coefficient est déterminé par le rapport des D10 d'un échantillon de granularité initiale donnée, mesurés avant et après lui avoir fait subir un pilonnage conventionnel avec la dame Proctor normal ;
- **la masse volumique** de la roche déshydratée en place (norme NF P 94-064).Ce paramètre est très appréciable en présence de craies et de calcaires tendres, car

facilement corrélable avec le coefficient de fragmentabilité évoqué ci-avant ; il aide à mieux appréhender les possibilités de réutilisation de ces matériaux.

Variations des propriétés mécaniques en fonction de l'état des matériaux

Le paramètre essentiel est le **coefficient de dégradabilité** (*DG* – norme NF P 94-067).

Il s'exprime par le rapport des D_{10} d'un échantillon de granularité initiale donnée, mesurés avant et après l'avoir soumis à des cycles de séchage-immersion conventionnels.

L'interprétation de ce paramètre vise essentiellement les possibilités d'emploi en remblai des matériaux provenant de roches argileuses (marnes, schistes sédimentaires, etc.).

L'interprétation de ce coefficient est souvent associée à la connaissance de la teneur en eau des matériaux rocheux fragmentables et dégradables comme les craies et les roches argileuses.

Selon la nature de certains matériaux rocheux, d'autres investigations sont quelquefois nécessaires telles les recherches spécifiques de teneurs en éléments solubles comme, par exemple, le sodium et le gypse pour les roches d'origine saline.

Les **valeurs seuils** des paramètres d'état et de comportement des matériaux rocheux relativement nombreuses figurent de façon détaillée dans l'annexe 1 du fascicule II du GTR.

APPENDIX I.1.4

Variation des propriétés mécaniques en fonction de l'état du sol

1. Contrainte totale, pression interstitielle, pression d'air, contrainte inter-granulaire, succion

Lorsque l'on exerce sur un sol un système de contraintes, celles-ci se transmettent de différentes manières ; pour mettre cela en évidence, prenons une facette en un point du sol. Rappelons que l'on appelle ainsi, en élasticité, un élément de plan infiniment petit de surface dS (Illustration 1). Bien qu'infiniment petit vis-à-vis des dimensions du massif de sol, cet élément de plan pourra être assez grand pour intéresser un nombre notable de grains de sol. Nous déformerons d'ailleurs la facette pour qu'elle ne coupe pas de grain (Illustration 2). La surface dS coupera donc une partie χdS d'eau et $(1 - \chi)$dS d'air en appelant $\chi/(1 - \chi)$ le rapport des surfaces de dS occupées par l'eau et l'air. Ces deux fluides ne peuvent transmettre que des contraintes normales.

On appelle **pression interstitielle *u*** la pression transmise par l'eau, et **pression d'air *p*** celle transmise par l'air et la vapeur d'eau. De manière plus précise, comme le corps baigne dans l'atmosphère et que les forces extérieures devraient comprendre les forces de pression dues à la pression atmosphérique, on convient généralement que u (pression interstitielle) et p (pression d'air) sont respectivement l'excédent de la pression de l'eau et de la pression de l'air sur la pression atmosphérique ; il en résulte que les deux fluides transmettent d'un côté à l'autre de la facette une force normale à la facette et égale à :

$$[\chi u + (1 - \chi)p]\,\mathrm{d}S$$

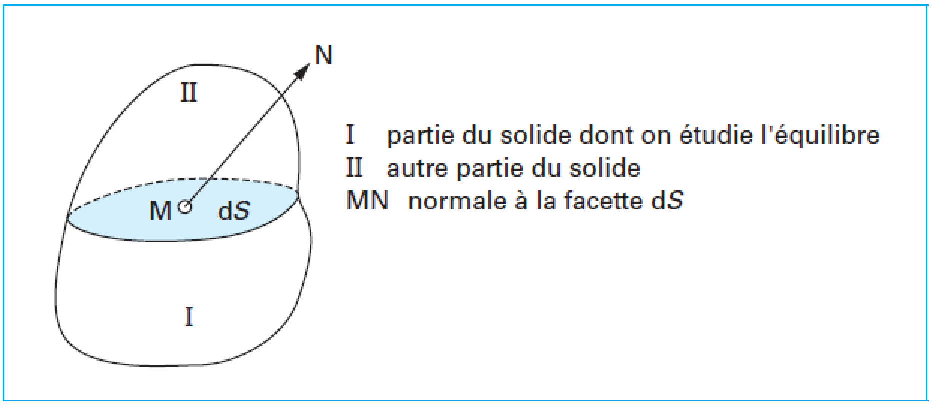
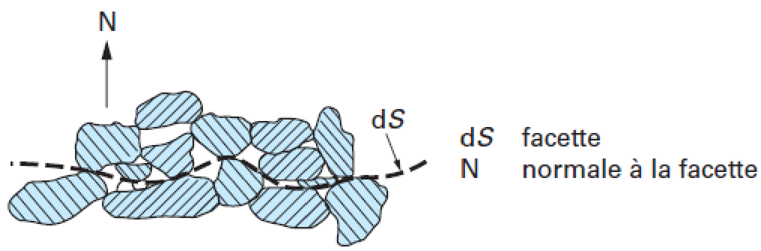


Illustration 1 – Définition d'une facette



Cette présentation est inspirée de E.L. Matyas et H.S. Radhakrishna qui introduisirent la notion de plan ondulant. En fait, χ a été introduit par Bishop comme un coefficient empirique.

Illustration 2 – Facette déformée

Si l'on appelle σ′d*S* et τ′d*S* les forces transmises à travers la facette par les contacts entre les grains (σ′d*S* force normale et τ′d*S* force tangentielle), la force totale transmise à travers la facette a les deux composantes suivantes :

- tangentielle : $\tau' \mathrm{d}S$;
- normale : $[\sigma' + \chi u + (1 - \chi)p]\mathrm{d}S$.

Il est clair que si σ et τ sont les composantes normales et tangentielles de la **contrainte totale**, on a :

avec σ′ et τ′ les deux composantes de la **contrainte intergranulaire**.

$$\left.\begin{aligned} \sigma &= \sigma' + \chi u + (1 - \chi)p \\ \tau &= \tau' \end{aligned}\right\}$$

(1)

On appelle **succion** la quantité $s = p - u$.

Cette succion est causée par les phénomènes de tension superficielle. En la faisant apparaître dans les formules, on obtient :

$$\left.\begin{aligned} \tau &= \tau' \\ \sigma &= \sigma' + p - \chi s \end{aligned}\right\}$$

(2)

2. Rôle de l'eau et de l'air dans le compactage

Ce rôle est fondamental en terrassement ; il explique l'influence de l'état du sol sur ses propriétés mécaniques.

Nous rappelons par un exposé qualitatif le comportement très différent, du point de vue de la compressibilité, des trois phases.

Sous un système de contraintes, la **phase solide** constituée par les grains du sol est peu compressible [ce qui signifie que le volume **(1− *n*)** varie peu et l'arrangement des grains entre eux ne peut varier que si la contrainte tangentielle est suffisante pour déformer le sol. Bien entendu, ce nouvel arrangement des grains modifie le volume des interstices, donc la porosité ***n***.

La phase liquide est, elle aussi, peu compressible, mais elle peut se déformer considérablement, c'est-à-dire que l'eau peut passer d'un interstice à un autre et cela d'autant plus facilement que le sol est plus perméable.

Quant à la phase gazeuse, elle est très compressible et encore plus fluide, c'est-à-dire que l'air peut passer encore plus aisément que l'eau d'un interstice à un autre, voire être chassé du sol (si du moins l'air n'est pas enfermé dans certains interstices). La perméabilité du sol à l'air a donc une influence encore plus grande que la perméabilité de l'eau.

On peut ainsi d'ores et déjà prévoir :

- qu'un sol perméable se compacte aisément si, du moins, son squelette n'offre pas une résistance trop grande au cisaillement (dans ce dernier cas, il faudra utiliser un engin puissant mais on pourra alors atteindre des densités sèches élevées) ;
- qu'un sol imperméable et saturé est incompactable.

Prenons le cas d'un sol à compacter pulvérulent.

Pour le compacter, on utilise un engin qui induit dans le sol un déviateur de contraintes $\boldsymbol{\sigma_1 - \sigma_3}$, exprimé en contraintes totales

σ_1 et σ_3 sont les contraintes principales et $\boldsymbol{\sigma_1 - \sigma_3}$ est le diamètre du **cercle de Mohr**.

Dans un sol pulvérulent, la succion est très faible et les relations (2) - indiquées dans le paragraphe précédent - entre les contraintes totales et les contraintes inter-granulaires deviennent :

$$\boldsymbol{p = u}$$

$$\boldsymbol{\sigma_1 = \sigma_1' + u}$$

$$\boldsymbol{\sigma_3 = \sigma_3' + u}$$

Supposons d'abord que le sol soit très perméable et que, par conséquent, la pression interstitielle soit nulle, l'eau étant en équilibre par perméabilité avec l'atmosphère.

En contraintes inter-granulaires, courbe intrinsèque et cercle de Mohr ont la configuration de l'Illustration **3**.

Pour qu'il y ait compactage, il faut que le cercle de Mohr coupe la courbe intrinsèque et que, par conséquent, le déviateur $\sigma_1 - \sigma_3$ soit élevé.

Comme la contrainte σ_3 est pratiquement fixée par le poids du sol situé au-dessus, le cercle de Mohr ne peut couper la courbe intrinsèque et, par conséquent, le compacteur ne peut cisailler le sol et le compacter que si le déviateur qu'il crée est élevé. Il faudra donc, pour compacter, un engin relativement puissant.

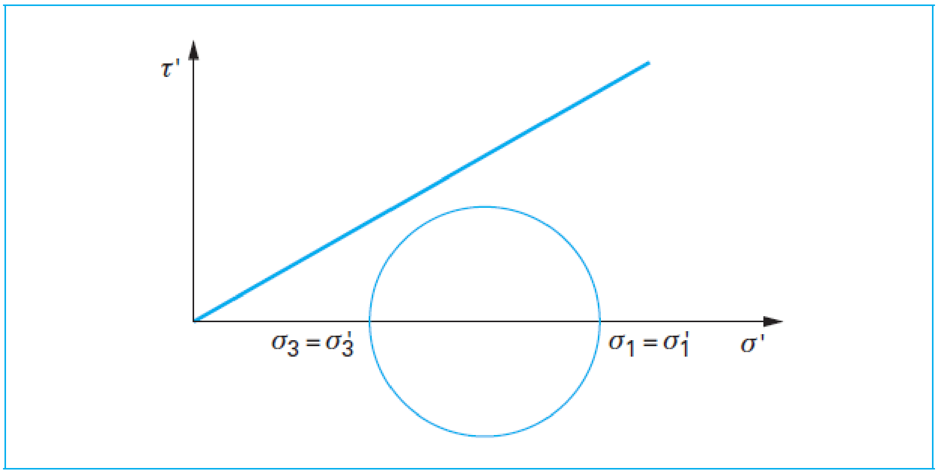


Illustration 3 – Courbe intrinsèque et cercle de Mohr d'un sol pulvérulent perméable

Un examen plus précis des différents cas de l'état du sol est présenté dans l'Annexe I.1.5 du présent Livret.

On peut résumer ce qui précède en indiquant qu'une élévation modérée de la teneur en eau facilite le compactage mais que, lorsque le degré de saturation s'approche de l'unité, le sol, s'il est perméable, perd son eau et, s'il est imperméable, devient incompressible.

3. Représentation de Proctor

Nous allons représenter les résultats que nous avons déduits de l'évolution du comportement du matériau au compactage en fonction de son état, c'est-à-dire de sa teneur en eau. Cette représentation, appelée **représentation de Proctor**, est obtenue en portant en abscisse la teneur en eau ***W*** et en ordonnée la densité sèche $\boldsymbol{\gamma_d}$ (Illustration **4**).

Prenons un sol de masse spécifique des grains solides égal à $\boldsymbol{\gamma_s}$; sa teneur en eau est ***W*** (masse spécifique de l'eau égal à γ_w).

Il est facile de voir que l'on a la relation :

$$\text{Pourcentage d'air} = \frac{\text{volume de l'air}}{\text{volume total apparent}} = a = 1 - \frac{\gamma_d}{\gamma_s} - W\,\frac{\gamma_d}{\gamma_w}$$

Dans la représentation de Proctor, les états du sol ayant le même pourcentage d'air *a* s'alignent sur une courbe d'équation *a* = Cte :

$$1 - \frac{\gamma_d}{\gamma_s} - W\frac{\gamma_d}{\gamma_w} = \text{Cte}$$

En particulier, la courbe *a* = 0 a pour équation, en faisant $\gamma_w = 1$:

$$\gamma_d = \frac{\gamma_s}{1 + W\gamma_s}$$

On appelle cette courbe, ***COURBE DE SATURATION***. Elle coupe l'axe des γ_d au point d'ordonnée γ_s. Comme toutes les courbes *a* = Cte, elle a pour asymptote l'axe des *W*.

3.1. Essai Proctor

C'est un essai de compactage pratiqué en plaçant le sol dans un moule normalisé et en employant un processus de damage normalisé et constant.

Le poids spécifique apparent du sol sec est mesuré égal à γ_{dr} et la teneur en eau de compactage est *W*.

L'opération est recommencée en modifiant la teneur en eau *W*. Chacune de ces expériences se traduit par un point dans le diagramme Proctor.

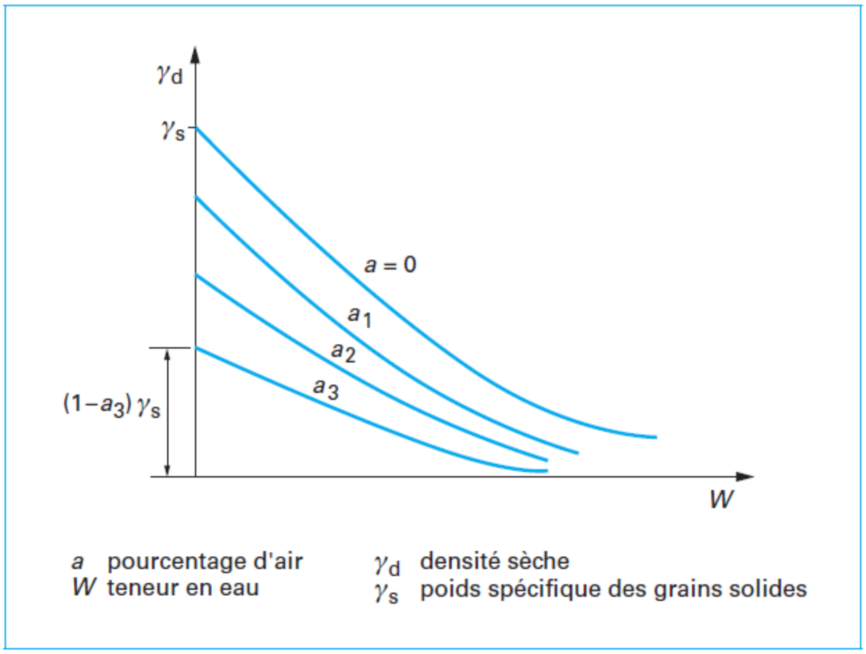


Illustration 4 – Courbe d'égal degré de vide dans la représentation de Proctor

L'expérience montre que ces points s'alignent sur une courbe que l'on appelle **courbe Proctor**. Elle présente un **maximum de densité sèche pour un optimum de teneur en eau**.

Si l'on modifie le processus de compactage en augmentant ou en diminuant l'énergie de compactage, les points obtenus sur le diagramme Proctor vont encore se placer sur une courbe présentant elle aussi un maximum.

Finalement, on constate que les courbes ont en général la forme de la Illustration **5**, c'est-à-dire :

- qu'elles s'emboîtent les unes dans les autres ;
- que les maximums croissent avec l'intensité du processus de compactage ;
- que les parties descendantes des courbes sont très proches les unes des autres, voire confondues.

La forme de cette courbe s'explique aisément par les propriétés que nous avons exposées. Au fur et à mesure que la teneur en eau s'élève (à partir d'une valeur basse), apparaît puis se développe une pression interstitielle qui déplace le cercle de Mohr vers la gauche et rend par conséquent plus facile la déformation du sol, donc son compactage (Illustration 5). Cela explique la partie ascendante de chaque courbe Proctor. Lorsque, du fait de l'augmentation de la teneur en eau, le degré de saturation devient élevé, les pressions interstitielles deviennent trop fortes pour qu'après compactage les grains de sol restent dans l'état où les a mis le compacteur. Ces pressions interstitielles écartent les grains de sol les uns des autres, c'est-à-dire qu'elles décompactent. L'efficacité décroît alors et la courbe Proctor descend. Il y a donc bien un maximum qui se produit à un degré de saturation S_{r0} suffisant pour que le phénomène d'instabilité mécanique apparaisse et qui est grossièrement le même pour toutes les courbes Proctor.

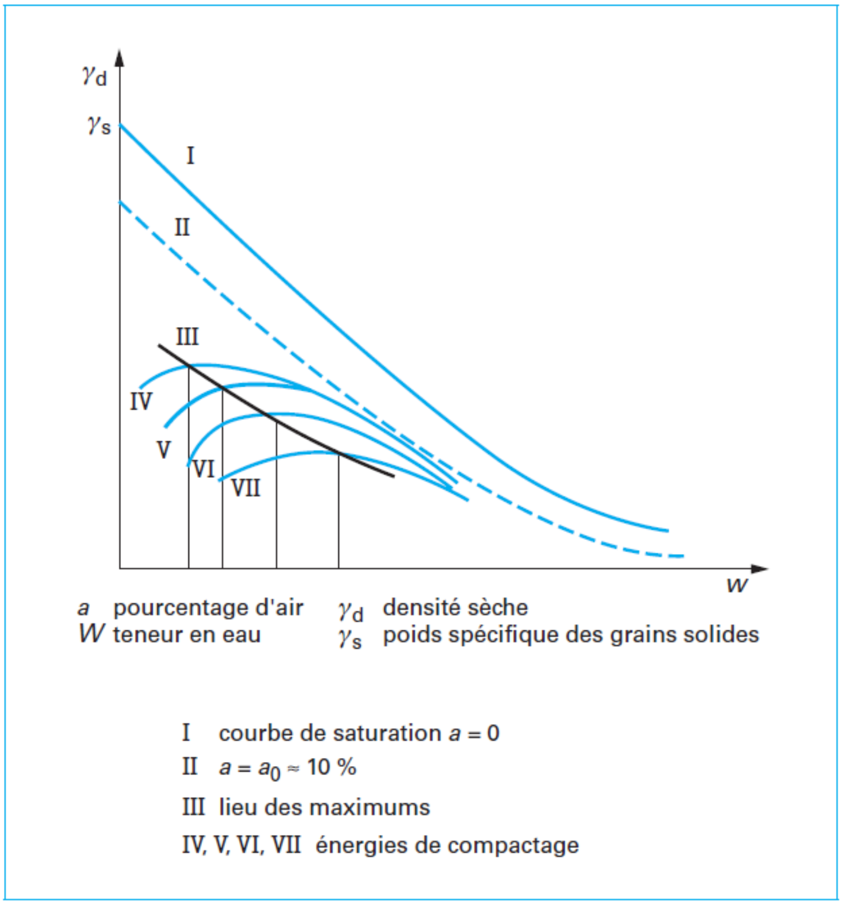


Illustration 5 – Diagramme Proctor complet

En faisant $S_r = S_{r0}$ dans les équations du paragraphe 3.2, on constate que le lieu des maximums vérifie l'équation :

$$\gamma_d = \frac{\gamma_s S_{r0}}{\gamma_s W + S_{r0}}$$

qui est évidemment une hyperbole équilatère. En fait l'expérience ne vérifie que très grossièrement l'équation de ce lieu, en particulier parce que S_{r0} n'est pas constant.

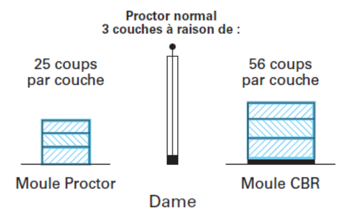
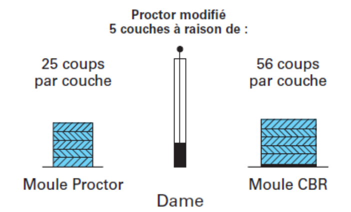
3.2. Prise en compte de l'essai Proctor dans le GTR

La teneur en eau et la densité de référence des matériaux de D_{max} < 50 mm, mis en œuvre en remblai et en couche de forme, sont, selon les prescriptions du GTR, celles déduites de l'**essai Proctor normal** (NF P 94-093).

Cet essai se différencie de l'essai **Proctor modifié** (NF P 94-093) applicable aux matériaux mis en œuvre en sous-couches et en couches de chaussée, par le compactage dans un moule Proctor ou CBR d'un échantillon de sol de D_{max} 20 mm en 3 couches, à raison de 25 coups de dame Proctor normal par couche, au lieu du compactage nettement plus intense dans un même moule Proctor ou CBR d'un même échantillon de sol à raison de 55 coups de dame Proctor modifié.

Le tableau ci-dessous, extrait de la norme NF P 94-093, illustre bien les « modalités d'exécution des essais Proctor « normal et modifié ».

Tableau 7 – Modalités d'exécution des essais Proctor normal et modifié

Nature de l'essai	Caractéristiques de l'essai	Moule Proctor	Moule CBR	Schéma récapitulatif
Essai Proctor normal	Masse de la dame	2 490 g	2 490 g	
	Diamètre du mouton	51 mm	51 mm	
	Hauteur de chute	305 mm	305 mm	
	Nombre de couches	3	3	
	Nombre de coups par couche	25	56	
Essais Proctor modifié	Masse de la dame	4 535 g	4 535 g	
	Diamètre du mouton	51 mm	51 mm	
	Hauteur de chute	457 mm	457 mm	
	Nombre de couches	5	5	
	Nombre de coups par couche	25	56	
Dimension des moules	Diamètre intérieur	101,5 mm	152 mm	
	Hauteur	116,5 mm	152 mm	

À titre indicatif, un remblai est supposé être correctement compacté lorsque la densité du matériau est 95 % de la densité de l'**Optimum Proctor Normal** (*OPN*), et une couche de forme lorsque la densité du matériau est 98 % de la densité de l'*OPN*.

L'**Optimum Proctor Normal** correspond à des caractéristiques optimales de compactage d'un matériau, sa masse volumique sèche maximale obtenue avec une teneur en eau optimale, mesurées dans les conditions de l'essai Proctor Normal défini ci-dessus.

L'*OPN* est déterminé par le point haut de la courbe Proctor tracée en reportant les valeurs de densité données par l'essai à cinq teneurs en eau différentes.

3.3. Prise en compte de l'indice portant immédiat (IPI) dans le GTR

Selon la norme NF P 94-078, le principe général de l'essai consiste à mesurer les forces à appliquer sur un poinçon cylindrique pour le faire pénétrer à vitesse constante dans une éprouvette de matériau. Les valeurs particulières des deux forces ayant provoqué deux enfoncements conventionnels sont respectivement rapportées aux valeurs des forces observées sur un matériau de référence pour les mêmes enfoncements.

L'indice recherché est défini conventionnellement comme étant la plus grande valeur, exprimée en pourcentage, des deux rapports énoncés ci-avant. La maîtrise de l'*IPI* est déterminante car ce paramètre est « **la grandeur utilisée pour évaluer l'aptitude d'un sol ou d'un matériau élaboré à supporter la circulation des engins de chantier** ».

La portance d'un matériau sous circulation d'engins de chantier est directement liée à sa nature (granularité, plasticité) et à ses paramètres d'état (teneur en eau, masse volumique sèche et degré de saturation).

L'*IPI* est donc mesuré à la teneur en eau du matériau au moment de sa mise en œuvre et en tenant compte de la masse volumique de ce matériau sec correspondant à la valeur obtenue lorsque l'on compacte le matériau à l'énergie Proctor normal.

Le graphique de l'Illustration **6**, qui est un extrait des essais de laboratoire effectués sur le chantier de l'autoroute A 89 (TOARCC 2.3) près de Mussidan sur un sable fin limoneux à mettre en remblai, illustre bien notre propos.

Dans ce cas particulier, la connaissance de l'*IPI*, tout à fait révélateur d'une chute de portance conséquente au-delà d'une teneur en eau naturelle de 14 % pourtant proche de celle de l'*OPN* (13,2 %), a conduit les équipes de chantier à apporter le plus grand intérêt au contrôle de la teneur en eau naturelle du matériau durant toute sa mise en œuvre, de façon à assurer un compactage efficace compatible avec une bonne traficabilité des engins de chantier.

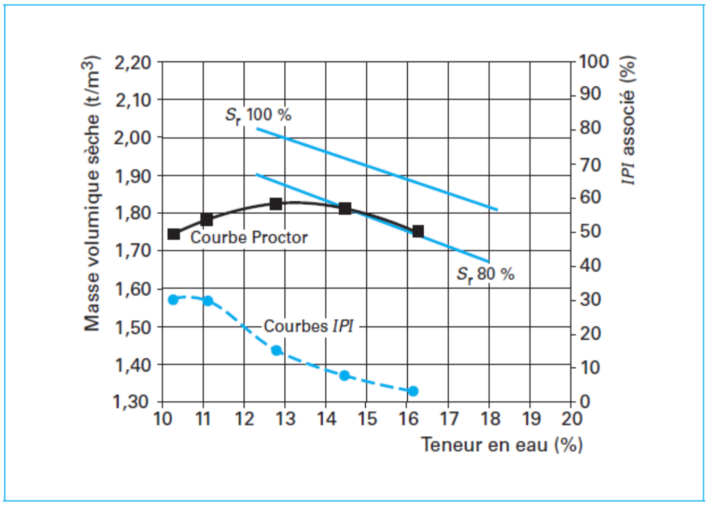


Illustration 6 – Exemple d'un essai effectué sur un sable fin limoneux

APPENDIX I.1.5

Variation des propriétés mécaniques en fonction de l'état du sol / Compléments techniques

1. Rôle de l'eau et de l'air dans le compactage

Nous présentons un examen détaillé de différents cas.

1er cas : supposons d'abord que le sol à compacter soit pulvérulent. Pour le compacter, on utilise un engin qui induit dans le sol un déviateur de contraintes $\sigma_1 - \sigma_3$, exprimé bien entendu en contraintes totales (σ_1 et σ_3 sont les contraintes principales et $\sigma_1 - \sigma_3$ est le diamètre du cercle de Mohr.

Dans un sol pulvérulent, la succion est très faible et les relations (2) du chapitre 1 de l' ANNEXE I.1.4 entre les contraintes totales et les contraintes inter-granulaires deviennent :

$$p = u$$

$$\sigma_1 = \sigma_1' + u$$

$$\sigma_3 = \sigma_3' + u$$

Supposons d'abord que le sol soit très perméable et que, par conséquent, la pression interstitielle soit nulle, l'eau étant en équilibre par perméabilité avec l'atmosphère.

En contraintes intergranulaires, courbe intrinsèque et cercle de Mohr ont la configuration de l'Illustration **1**.

Pour qu'il y ait compactage, il faut que le cercle de Mohr coupe la courbe intrinsèque et que, par conséquent, le déviateur $\sigma_1 - \sigma_3$ soit élevé.

Comme la contrainte σ_3 est pratiquement fixée par le poids du sol situé au-dessus, le cercle de Mohr ne peut couper la courbe intrinsèque et, par conséquent, le compacteur ne peut cisailler le sol et le compacter que si le déviateur qu'il crée est élevé. Il faudra donc, pour compacter, un engin relativement puissant.

2e cas : supposons au contraire, maintenant, **que ce même sol pulvérulent** (et dans lequel la succion est toujours supposée nulle) **soit suffisamment imperméable** pour que l'eau des interstices ne se mette pas en équilibre avec l'atmosphère ; nous supposerons de manière plus précise que les conditions de perméabilité du sol sont telles que la pression interstitielle prenne une valeur *u* non nulle mais modérée.

Par rapport à la Illustration **1**, le cercle de Mohr [4] (en supposant que l'on utilise le même engin) va se décaler vers la gauche de *u* (Illustration 1) puisque l'on peut écrire :

$$\sigma_1' = \sigma_1 - u$$

$$\sigma_3' = \sigma_3 - u$$

Cette fois le cercle de Mohr coupe la courbe intrinsèque, le compacteur cisaille le sol, le déforme et compacte.

3^{e} cas : supposons maintenant que le sol à compacter soit cohérent et à faible teneur en eau (très loin de la saturation). Il est alors impossible de négliger la succion, qui, dans les sols cohérents et relativement secs, est élevée.

Les relations s'écrivent alors :

$$\sigma_1 = \sigma_1' + p - \chi s$$
$$\sigma_3 = \sigma_3' + p - \chi s$$

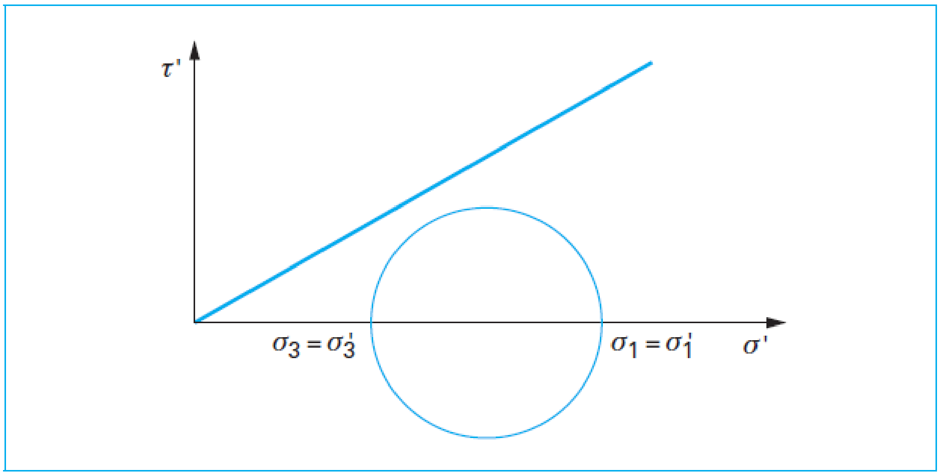


Illustration 1 – Courbe intrinsèque et cercle de Mohr d'un sol pulvérulent perméable

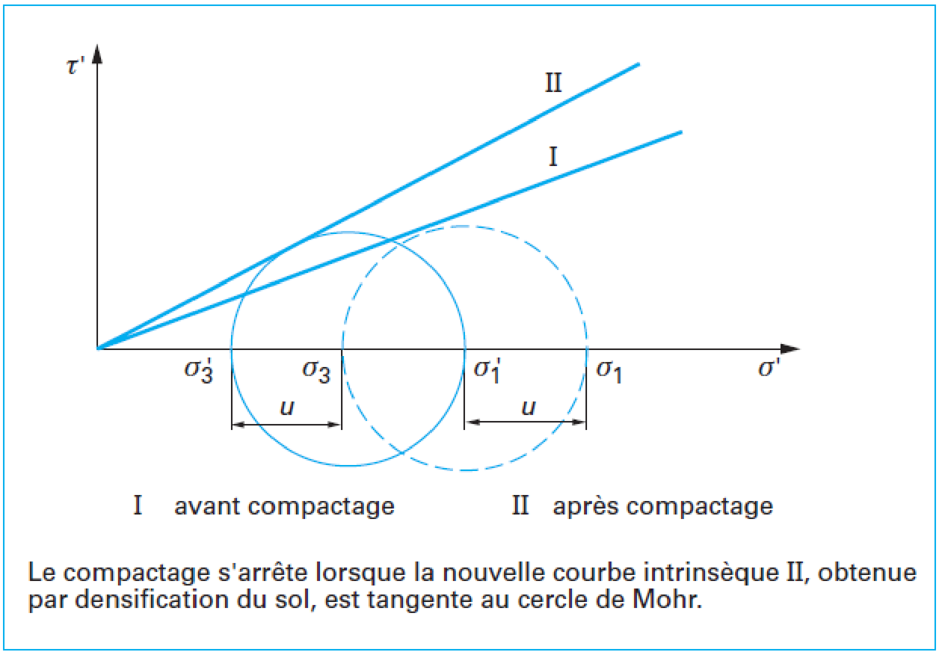


Illustration 2 – Courbe intrinsèque et cercle de Mohr d'un sol pulvérulent imperméable

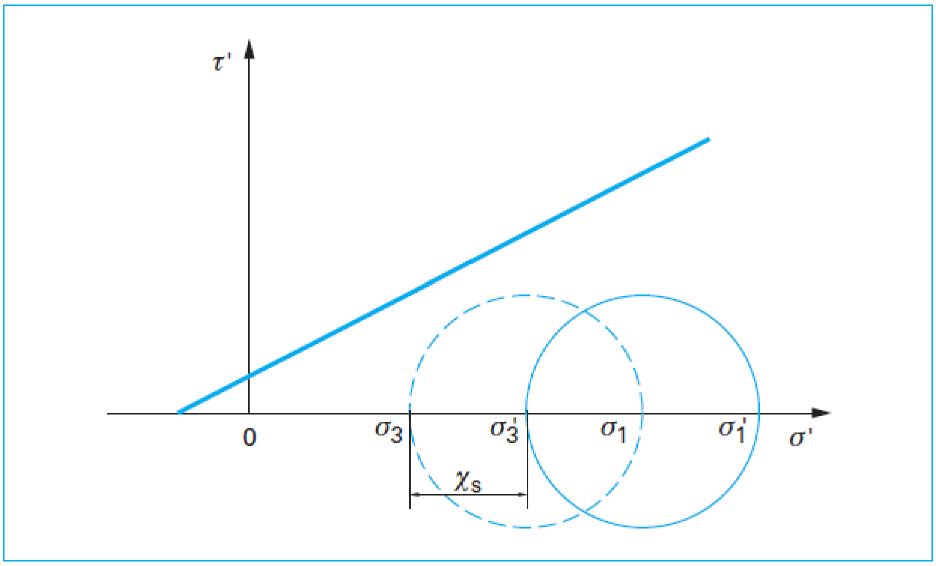


Illustration 3 – Courbe intrinsèque et cercle de Mohr d'un sol cohérent à faible degré de saturation

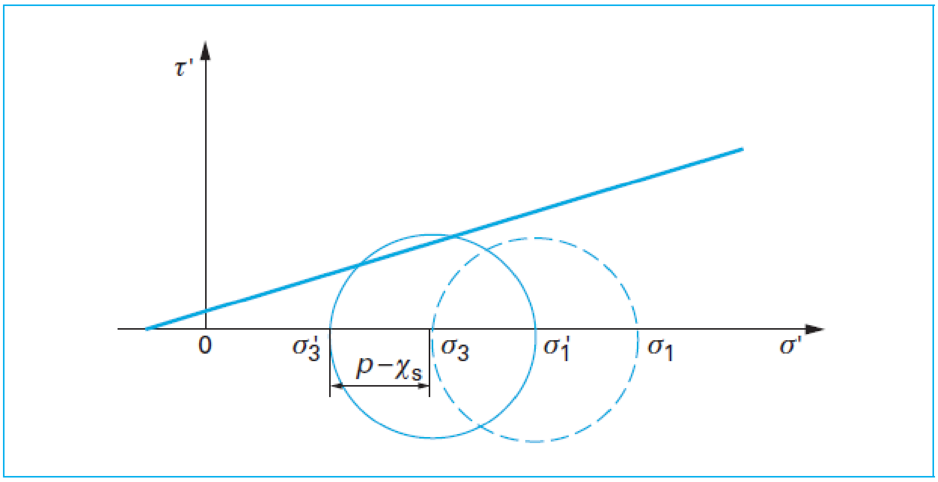


Illustration 4 – Courbe intrinsèque et cercle de Mohr d'un sol cohérent à degré de saturation notable

Les pores du sol comportent essentiellement de l'air et le volume peut diminuer de façon importante, sans que la pression du gaz *p* s'élève notablement (il suffirait pour s'en rendre compte d'appliquer la loi de Mariotte). Nous négligerons donc *p*. En contraintes intergranulaires, la courbe intrinsèque et le cercle de Mohr ont alors la configuration de l'Illustration **3**.

Cette Illustration montre nettement que, du fait de la cohésion et du fait de la succion, on ne peut obtenir que le cercle de Mohr coupe la courbe intrinsèque (c'est-à-dire que le compacteur cisaille le sol) qu'avec un très fort déviateur. Il faut donc, pour compacter, un engin puissant.

4^e^ cas : prenons maintenant le même sol cohérent mais augmentons le degré de saturation

Certes la succion diminue, mais le coefficient χ augmente. En fait, s diminue plus vite que χ n'augmente, si bien que le produit χs diminue. Mais un autre phénomène très important se produit : dès lors qu'un début de compactage se serait amorcé, les pores diminuent de volume. Comme la plupart de ces sols sont imperméables aussi bien à l'air qu'à l'eau et que le volume occupé par le gaz est faible, toute diminution de volume des pores (diminution qui porte obligatoirement sur le volume du gaz) entraîne, comme le montrerait la loi de Mariotte, une augmentation assez importante de *p*. En contraintes intergranulaires, la courbe intrinsèque et le cercle de Mohr auront alors la configuration de la Illustration 4.

On voit alors que le compactage est facilité. Donc, dans ces sols une augmentation de la teneur en eau facilite le compactage, du moins tant qu'elle reste modérée.

5^e^ cas : supposons enfin que, dans ce même sol cohérent, le degré de saturation s'élève encore et que, sans atteindre la valeur 1, il s'en approche.

Dans ces sols, la succion est pratiquement nulle, si bien que le compactage va s'amorcer aisément. Mais toute diminution du volume du sol se traduit intégralement par une diminution du volume des pores et celle-ci se traduit tout aussi intégralement, puisque l'eau est incompressible, par une diminution du volume de l'air. Comme le matériau est imperméable et que l'air ne peut s'échapper, cette diminution de volume va se traduire par une augmentation considérable de la pression du gaz. Sitôt que le compacteur sera passé et ne sera donc plus là pour contrebattre cette pression du gaz, le matériau va regonfler. Autrement dit, le sol va se comporter comme un pneumatique qui ne s'affaisse que lorsque l'on appuie dessus, mais se regonfle lorsque l'on supprime le poids du véhicule. C'est le phénomène du coussin de caoutchouc. En fait, on constate le phénomène suivant : lorsque le compacteur passe sur une zone donnée, le sol s'affaisse sous lui et reflue en avant et en arrière du compacteur comme s'il était refoulé et, effectivement, lorsque le compacteur est passé, le sol se regonfle. Autrement dit, le sol se comporte comme un coussin de caoutchouc qui se déprime sous le poids de la tête et se gonfle autour de celle-ci. On constate de plus, en surface du sol, l'existence d'une sorte de croûte souvent feuilletée et qui joue le rôle de la paroi du coussin de caoutchouc, le sol sous-jacent liquéfié par le phénomène que nous avons analysé jouant le rôle de l'air.

Au fur et à mesure que l'on continue le compactage, le phénomène ne fait d'ailleurs que s'amplifier. Mieux vaut arrêter dès que celui-ci s'amorce.

L'explication précédente justifie bien la perte de résistance du sol mais ne met pas en évidence le mécanisme de création de la croûte ; celui-ci vient de la dilatance. En surface, immédiatement sous le poids du compacteur, les contraintes sont beaucoup plus élevées que dans le corps du matériau ; le cisaillement dans cette zone de surface produit une dilatance, c'est-à-dire un décompactage du sol. L'air va trouver de la place, la pression d'air diminue considérablement ; de ce fait, tout se passe en surface comme si le sol y était plus résistant et formait une peau. L'expérience prouve bien que le coussin de caoutchouc se produit essentiellement dans les sols doués de dilatance.

On peut résumer ce qui précède en indiquant qu'une élévation modérée de la teneur en eau facilite le compactage mais que, lorsque le degré de saturation s'approche de l'unité, le sol, s'il est perméable, perd son eau et, s'il est imperméable, devient incompressible.

On voit donc bien, sur cet exemple, l'influence considérable de l'état du sol sur ses propriétés mécaniques et rhéologiques.

APPENDICES I.2

CLASSIFICATION OF MATERIALS

TABLE OF CONTENTS APPENDICES I.2

APPENDIX I.2.1

The different types of materials classification in the world

Reference PIARC / Extract Technical Report PIARC 2012R37EN
"Innovative approaches towards the use of locally available natural marginal materials"

"3.4.1 The different types of material classifications"

As mentioned in section 1.4, the interpretation of the survey responses was difficult at times because of the Jack of precise identification of the proposed materials. Reference to an internationally recognised classification is desirable and provides a well-documented comparison between the various materials proposed by the countries which responded to the survey.

According to the report of the committee TC12 cited in section 1.1 and published in 2003 (36], there are tluee types of classification used in the international community:

"*A. General classification of soils which is not connected direct/y to specifications of employrnent, but rather with privileged fields of application with concerning the classes of soi/ likely to generate difficulties, even whose employment is disadvised. They are generally the classifications of soils der.ived rather directly from classification USCS or HRB. In this case, one can quote the example of Gennany and Switzerland. With these classifications, it returns to the engineering and design departrnent charged to draw up the project to define the soifs which will be employable for this project and which condition. So the specification is not generalizable with all the projects, but has to be thus adapted to each particular project under the responsibility of the geotechnical enginee1.*

B. b. Classification of all the soils suitable to be met and connected direct/y to a grid of the possible re-employrnent as fil/ or capping layer possibly supplied with particular rnethods of implementation to make the soifs acceptable as Jill or capping laye,: It is in particular the case of the classifications of soils used by France and Portugal. These classifications are specialised for the field of the earthworks and it fi-equently happens that in the same countly, one has a classification of diffe1-ent reference according to specific problerns (for example in France for the soi/ rnechanics or the management of1-esources materials.

C. c. A third type of classification appears to be developed by England which starts fi·om categories of employment (for example soifs usable as fil/, soifs usable as fil/ close to a bridge...) to define the characteristics that the soi/ must have. This type of classification opposites the reasoning which is not any more "what use can one makes with this soil?", but rather ' for a detennined need, what m-e the characteristics of the acceptable soil?". So, this classification constitutes a specification for the contJ•acts and is integrated besides in the Specification for Highway Works (SHW} in series 600 (earthworks). As in the classifications of the type B, this classification is obviously dedicated to the projects of earthworks and there is another used in soil mechanics. "

These three types of classification are confirmed by the survey returns and are explained below:

Type A

The USCS and AASHTO (or HRB) classifications developed in the USA are slightly different:

La référence de base pour "the Unified Soil Classification System" est l'ASTM D2487.

The basic reference for "the Unified Soi/ Classification System" is ASTM D2487 (figure 12). This class ification is based on the laboratory detennination of particle size characteristics, liquid limit and plasticity index (including the Casagrande chart for fine soils). The 0,075 mm sieve defines the transition from ("fine grained soils" which have a percentage greater than 50%) to ("coarse grained soils"). Each class is represented by a symbol of 2 letters:

G gravel – gravier

S sand – sable

M silt – limon

C clay – argile

O organic – organique

P_t highly organic soil

P poorly graded – mal gradué

W well graded – bien granule

H high plasticity – plasticité élevée

L low plasticity – plasticité faible Unified Soil Classification (USC) System

(from ASTM D 2487)

Major Divisions			**Group Symbol**	**Typical Names**
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

- *The classification AASHTO or HRB (Highway Research Board), developed by "the American Association of State Highway and Transportation Officiais" is used as the guide for the classification of soils and soil/aggregate mixmres for specific needs in highway constmction and earthworks stmctures. It is described in ASTM D3282 or AASHTO Ml 45. It defines 7 groups (Al to A7) which are different soils based, as the classification USCS, on a laboratory detennination of the particle size distribution, liquid liinit and plastic index. The 0,075 mm sieve size defines the transition between ("silt-clay mater.ials" with a percentage passing greater than 35%) and ("granular materi als”).*

AASHTO Soil Classification System (from ASTM M 145)

General Classification	Granular Materials 35% or less passing the 0.075 mm sieve							Silt-Clay Materials >35% passing the 0.075 mm sieve			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	---	---	---	---	---	---	---	---	---	---
0.425 (No. 40)	30 max	50 max	51 max	---	---	---	---	---	---	---	---
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit	---		---	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[a]
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

[a]Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30.Plasticity index of A-7-6 subgroup is greater than LL – 30

Among the responses received from countries with respect to national classifications it is possible to identify those who use type A, for example, those of Canada (Québec) (Norme 1101 - Classification des sols - Tome VII, ch. 1) , of Slovakia (Slovak technical standards STN 736244 and STN 736133), of Bolivia, of Brazil, of Ivory Coast, of Greece, of Italy (Classificatione delle terre C.N.R. – U.N.I. 10006/1963), of Tchec Republic (CSN 736133).

- ***Canada***

Tome VII | Chapitre 1 | Norme 1101 | Page 4 de 4 | Date 95 05 26

CLASSIFICATION DES SOLS

1.1 Classification des sols

Classification des sols

Directeur général adjoint
Infrastructures et technologies
Jean-Pierre Tremblay, ing.

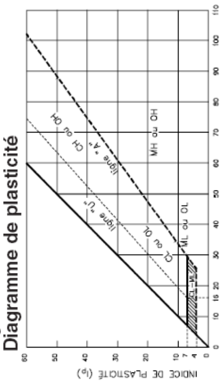
Gouvernement du Québec
Ministère
des Transports

NORME

Tableau 1101–1
Classification des sols

		Principales divisions		Symboles code	Symbole	Description	Critères de classification	
NIVEAU 1	Sols à grains extra-gros > 80 mm	Blocs (B) Éléments > 300 mm		B		Blocs	Si moins de 20 % des éléments sont inférieurs ou égaux à 300 mm, donner au sol la classification «blocs», B.	
				BQ		Blocs et cailloux	Si de 20 % à 50 % des éléments sont inférieurs ou égaux à 300 mm, donner au sol la classification «blocs et cailloux», BQ.	
		Cailloux (Q) Éléments > 80 mm et ≤ 300 mm		QB		Cailloux et blocs	Si de 50 % à 80 % des éléments sont inférieurs ou égaux à 300 mm, donner au sol la classification «cailloux et blocs», QB.	
				Q		Cailloux	Si 80 % ou plus des éléments sont inférieurs ou égaux à 300 mm, donner au sol la classification «cailloux», Q.	
NIVEAU 2	Sols à gros grains (1) Moins de 50 % passent le tamis de 80 µm	Graviers (G) Moins de 50 % passent le tamis de 5mm(2,3)	Graviers propres	GW		Gravier bien gradué, mélange gravier–sable. Peu ou pas de grains fins.	$C_u \geq 4$ et $1 \leq C_c \leq 3$	
				GP		Gravier mal gradué, mélange gravier–sable. Peu ou pas de grains fins.	$C_u < 4$ ou $1 > C_c > 3$	
			Graviers avec grains fins	GM		Gravier silteux, mélange gravier-sable-silt.	Limites de consistance sous la ligne «A» du diagramme de plasticité de la figure 1101–1 ou indice de plasticité inférieur à 4.	Les limites de consistance situées dans la zone hachurée du diagramme de plasticité de la figure 1101–1 requièrent un symbole double.
				GC		Gravier argileux, mélange gravier-sable-argile.	Limites de consistance au-dessus de la ligne «A» du diagramme de plasticité de la figure 1101–1 et indice de plasticité supérieur à 7.	
		Sables (S) 50 % ou plus passent le tamis de 5mm(2,3)	Sables propres	SW		Sable bien gradué, mélange sable–gravier. Peu ou pas de grains fins.	$C_u \geq 6$ et $1 \leq C_c \leq 3$	
				SP		Sable mal gradué, mélange sable–gravier. Peu ou pas de grains fins.	$C_u < 6$ ou $1 > C_c > 3$	
			Sables avec grains fins	SM		Sable silteux, mélange sable-silt.	Limites de consistance sous la ligne «A» du diagramme de plasticité de la figure 1101–1 ou indice de plasticité inférieur à 4.	Les limites de consistance situées dans la zone hachurée du diagramme de plasticité de la figure 1101–1 requièrent un symbole double.
				SC		Sable argileux, mélange sable-argile.	Limites de consistance au-dessus de la ligne «A» du diagramme de plasticité de la figure 1101–1 et indice de plasticité supérieur à 7.	
	Sols à grains fins (1) 50% ou plus passent le tamis de 80 µm	Silts et argiles $W_L \leq 50$		ML		Silt inorganique, poussière de roche, sable très fin silteux ou argileux, silt argileux de faible plasticité.		
				CL		Argile inorganique de faible plasticité, argile graveleuse, sableuse, silteuse.		
				OL		Silt organique, mélange silt-argile organique de faible plasticité.		
		Silts et argiles $W_L > 50$		MH		Silt inorganique et sable très fin ou silteux de grande plasticité.		
				CH		Argile inorganique de grande plasticité.		
				OH		Argile organique et silt organique de plasticité moyenne à grande.		
		Sols organiques		f PT a		Terre noire et autres sols très organiques, tourbe. PTf = fibreuse PTa = amorphe		

Figure 1101–1
Diagramme de plasticité



Pour la classification des sols à grains fins et de la fraction fine des sols à gros grains.
Les limites de consistance situées dans la zone hachurée requièrent un symbole double.
Équation de la ligne «A», horizontale à $I_P = 4$ jusqu'à $W_L = 25.5$, puis $I_P = 0.73\ (W_L - 20)$
Équation de la ligne «U», verticale à $W_L = 16$ jusqu'à $I_P = 7$, puis $I_P = 0.9\ (W_L - 8)$

Notes : 1. Fraction de sol passant le tamis de 80 mm.
2. Fraction de sol passant le tamis de 80 mm et retenue sur le tamis de 80 µm.
3. Classification basée sur le pourcentage de silt et argile : moins de 5 % passent le tamis de 80 µm : GW, GP, SW, SP. Plus de 12 % passent le tamis de 80 µm : GM, GC, SM, SC. De 5 % à 12 % passent le tamis de 80 µm : symbole double.

Type B

A typical example is the French classification for materials.

This classification was implemented in Belgium for work on the highspeed railways.

As noted earlier (paragraph 3.2.2), the main technical specifications for highway earthworks in France are defined in the GTR technical guide (construction of embankments and capping layers).

The classification of soils (classes A, B, C and D) and rocks (classes R) presented in the GTR guide are standardised (norm AFNOR NF-P-11-300).

The classification of materials used in ear thworks is established using the following parameters:

Nature (intrinsic) characteristics:

- Granularity (Dmax, grading curve).
- Clay content (plasticity index - methylene blue value- sand equivalent).

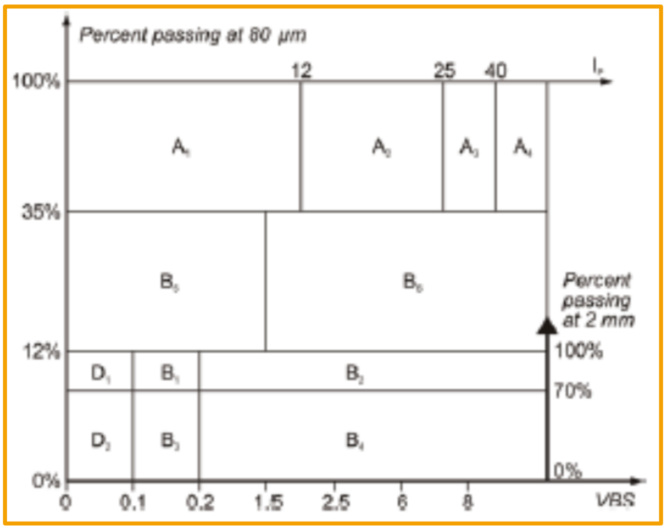
Mechanical characteristics:

- Los Angeles (LA)
- Micro Deval in the presence of water (MDE)
- Friability of sands (FS)
- Fragmentability (FR)
- Degradability (DG).

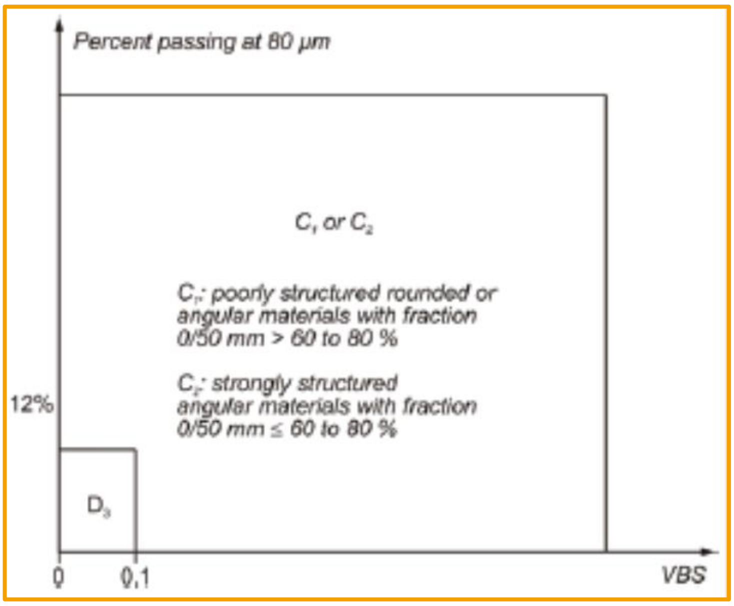
State characteristics:

- Moisture state of the soils (water content according to the reference PROCTOR and/or
- Value of the inunediate CBR index (IPI).

The schematic representation of the GTR classification of materials is shown on the followingpage:



Soil classification ($D_{max} \leq 50$ mm)



Soil classification (D_{max} > 50 mm)

Sedimentary rocks	Carbonate rocks	Chalk	**R1**
		Limestone	**R2**
	Argilaceous rocks	Marls, claystone, pelite, etc.	**R3**
	Siliceous rocks	Sandstone, puddingstone, breccia, etc.	**R4**
	Saline rocks	Rock salt, gypsum, etc.	**R5**
Igneous and metamorphic rocks	Granite, basalt, andesite, gneiss, schist, slate, etc.		**R6**

Rocks classification

For the record, the GTR classification aiso distinguishes orgamc materials and industrial by-products (class F).

Use of materials

Depending upon the classification of materials, the GTR guide indicates their possible uses and defines the specifications for use as filler in PST and capping layer.

In the case of materials which are marginal and where there are important economic or environmental issues, the GTR guide provides recommendations on the specific studies required to clarify the limitations of using these materials. Tests are recommended and trial sites may be necessary.

Type C

Type C is essentially identified with the classification from the UK set out in series 600 "Earthworks" in the SH\.V(Specification for Highway \.Vorsk) and in particular clause 601 which defines acceptable materiais and their conditions of use for both general fill and selected fill - for example, capping layers or backfill to structures).

An example is given for materiais used as general fill in following page.

Acceptable Earthworks Materials: Classification and Compaction Requirements

Class				General Material Description	Typical Use	Permitted Constituents (All Subject to Requirements of Clause 601 and Appendix 6/1)	Material Properties Required for Acceptability (In Addition to Requirements on Use of Fill Materials in Clause 601 and Testing in Clause 631)				Compaction Requirements in Clause 612	Class		
							Property (See Exceptions in Previous Column)	Defined and Tested in Accordance with:	Acceptable Limits Within: Lower	Upper				
GENERAL COHESIVE FILL	2	A	-	Wet cohesive material	General Fill	Any material, or combination of materials, other than chalk.	(i) grading	BS 1377 : part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 1 except for materials with liquid limit greater than 50, determined by BS1377 : Part 2, only deadweight tamping or vibratory tamping rollers or grid rollers shall be used.	2	A	-
							(ii) plastic limit (PL)	BS 1377 : part 2	-	-				
							(iii) mc	BS 1377 : Part 2	PL -4%	App 6/1				
							(iv) MCV	Clause 632	App 6/1	App 6/1				
							(v) Undrained shear strength of remoulded material	Clause 633	App 6/1	App 6/1				
	2	B	-	Dry cohesive material	General Fill	Any material, or combination of materials, other than chalk	(i) grading	BS 1377 : Part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 2	2	B	-
							(ii) plastic limit (PL)	BS 1377 : Part 2	-	-				
							(iii) mc	BS 1377 : Part 2	App 6/1	PL -4%				
							(iv) MCV	Clause 632	App 6/1	App 6/1				
							(v) undrained shear strength of remoulded material	Clause 633	App 6/1	App 6/1				

A new event to be considered in the inventory of classifications and specifications across the worldis the process of European standardisation. A Technical Committee CEN TC 396 "Earthworks" was created in Europe in 2009.

APPENDIX I.2.2

European standard Reference

NF EN 16907-2 DÉCEMBRE 2018

Extracts / Classification tables

Table 2 — Soil groups for earthworks - Materials containing particles larger than 63 mm

Group name	Soil group symbol	Grouping parameters	Comments
Very coarse soil	**VC1**	The range of particle sizes present and their proportions by mass are estimated in the field by description. The behaviour in fill is controlled by the fraction > 63 mm. The fractions≤ 63 mm should be placed into groups as an additional parameter according to Table 3.	Size of blocks shall be considered for excavation and placement Soil may need to be processed before using in earth structures
Soil with very coarse particles	**VC2**	The range of particle sizes present and their proportions by mass are estimated in the field by description. The behaviour in fill is controlled by the fraction ≤ 63 mm. The fraction ≤ 63 mm shall be placed into groups according to Table 3.	

Table 3 — Soil groups for earthworks – Materials containing particles not larger than 63 mm and with organic matter content not more than 2 %
Table 3a — Soils with fines content not more than 15 % - Coarse soils and Composite soils

Main Group	Group name	Soil group symbol	Particle fractions			Further Grouping Parameters (intrinsic properties)	Comments
			Fines content $C_{0,063}$	Sand fraction 0,063 mm < D ≤ 2 mm	Gravel fraction 2 mm < D ≤ 63 mm	Uniformity coefficient C_U	
Coarse soil	Gravel widely graded	G1	< 5 %	less than gravel fraction	more than sand fraction	≥ 6	Normally soil is usable in earth structure.
	Gravel narrowly graded	G2				< 6	
	Sand widely graded	S1		more than gravel fraction	less than sand fraction	≥ 6	
	Sand narrowly graded	S2				< 6	
Composite soil	Gravel-Fines mixture widely graded	G3	5 to 15 %	less than gravel fraction	more than sand fraction	≥ 6	
	Gravel-Fines mixture narrowly graded	G4				< 6	
	Sand-Fines mixture widely graded	S3		more than gravel fraction	less than sand fraction	≥ 6	
	Sand-Fines mixture narrowly graded	S4				< 6	

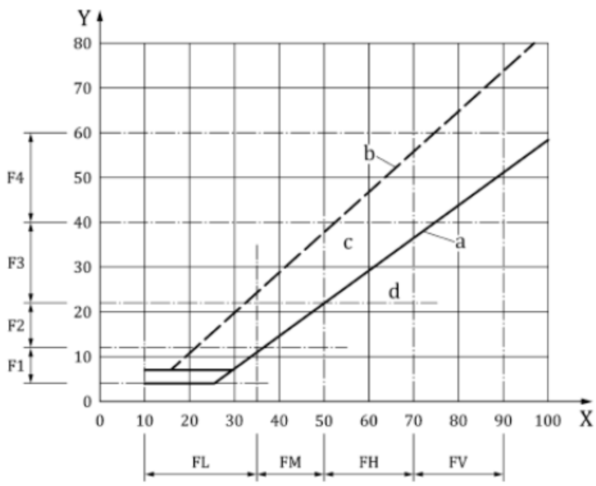
NOTE The coefficient of curvature C_C is also useful for classification. Approaches and limiting values vary widely in different countries.

Table 3b — Soils with fines content $C_{0,063}$ of more than 15 % - Intermediate Soils and Fine Soils

Main Group	Group name	Soil group symbol	Fines Content $C_{0,063}$	Further Grouping Parameters (see Note 1)		Comments
				Liquid Limit w_L	Plasticity Index I_P (Methylene Blue Value V_{BS})	
Intermediate soil (see Note 2)	Intermediate soil — low plasticity	I1	> 15 to 35 %		≤ 12 % (≤1,5)	Normally soil is usable in earth structure
		IL		≤ 35 %		
	Intermediate soil — medium to high plasticity	I2			> 12 % (>1,5)	
		IM		> 35 %		
Fine soil	Low plasticity fine soil	F1	> 35 %		≤ 12 % (≤2,5)	Normally soil is usable in earth structure. Distinction between Silt and Clay in fine soils may be made on the basis of the A line (see Figure 1)
		FL		≤ 35 %		
	Medium plasticity fine soil	F2			> 12 to 22 % (>2,5 to 6)	
		FM		> 35 to 50 %		
	High plasticity fine soil	F3			> 22 to 40 % (>6)	
		FH		> 50 to 70 %		
	Very high plasticity fine soil	F4			> 40 % (n/a)	These soils should be presumed to be not usable unless testing, local experience, or treatment demonstrate otherwise
		FV		> 70 %		

NOTE 1 Intermediate soils and fine soils may be grouped using Liquid limit or Plasticity Index or Methylene Blue Value V_{BS} (but only using one parameter at a time); the parameter used in classification is stated using the group codes above.

NOTE 2 More detailed grouping of intermediate soils may be achieved by adding the group of the coarse fraction according to groups given in Table 3a and the group of fine fraction according to groups of fine soils in Table 3b (e.g. IM/S1/FH).



Key

X Liquid Limit (w_L), a A-line Y = (X-20)*0.73, c Clay (above A-line)

Y Plasticity Index (I_P), b U-line Y = (X-8)*0.9, d Silt (below A-line)

Soil group symbol	Liquid Limit w_L [%]	Soil group symbol	Plasticity Index I_p [%]
FL	< 35	F1	< 12
FM	35 to 50	F2	12 to 22
FH	50 to 70	F3	22 to 40
FV	> 70	F4	> 40

NOTE Fine soils with liquid limits above 70 % and plotting below the A-line can be called 'very high plasticity silt' but such materials are unlikely to occur in practice.

Figure 1 — Plasticity Chart for Subdivision of Fine Soils

Table 4 — Soil groups for earthworks - Soils with organic matter content C_{OM} greater than 2 %

Main Group	Group Name	Soil group symbol	Organic matter content C_{OM} (see Notes 1 and 2)	Comments
Organic soil	Low organic soil	O1	> 2 to 6 %	Normally only used as structural fill with organic contents up to about 6 %, although national practices in testing methods and specifications vary Classification of the material shall be completed with Tables 2 and 3
	Medium organic soil	O2	> 6 to 20 %	
	Peat	O3	> 20 %	normally not used in earthworks

NOTE 1 These organic matter contents are based on using a loss of ignition test method. Other test methods, such as, addition to sodium hydroxide or titration or oxidation with $KMnO_4$ will give different values.

NOTE 2 There is no European Standard for determination of the organic matter content; therefore national test methods need to be used.

Table 5 — Soil groups for earthworks: - Anthropogenic materials

Main Group	Group Name	Soil group symbol	Particle size	Examples	Comments
Anthropogenic material	Natural materials processed mechanically	AN	all particle sizes	embankment fill, drainage layer, crushed rock, washed sand	Grouping and classification of materials should follow the that given in Table 2 to 4
	Manufactured materials (including secondary manufactured materials)	AM		ash, slag, expanded lightweight aggregates	Grouping and classification of materials may follow that given in Tables 2 to 4. Site or material specific grouping and classification may be required.
	Recycled materials	AR		crushed concrete, brick rubble, tyres, road planings, cold planings, debris	

Table 6 — Rock material groups (Strength)

Rock group		Point Load Index I_{s50} Assuming ($I_{s50} = \sigma_u/25$) MPa	Compressive strength σ_u MPa	Rock examples
Rock group symbol	Strength term			
RES	Extremely strong rock	> 10,0	> 250	volcanic rock, plutonic rock, metamorphic rock
RVS	very strong rock	4,0–10,0	100–250	volcanic rock, plutonic rock, metamorphic rock
RS	strong rock	2,0 to 4,0	50 to 100	sandstone, limestone, volcanic rock, plutonic rock, metamorphic rock
RMS	medium strong rock	1,0 to 2,0	25 to 50	sandstone, marlstone, limestone, schists, metamorphic rock
RW	weak rock	0,2 to 1,0	5 to 25	claystone, siltstone, sandstone, marlstone, limestone, schists, gypsum-stone, coal
RVW	very weak rock	less than 0,2	1 to 5	weathered claystone, siltstone, sandstone, gypsum — stone, coal
REW	extremely weak rock		0,6 to 1	weathered claystone, siltstone, sandstone

NOTE 2 Other intrinsic properties of rock used in classification may include mineralogy and density.

Table 7 — Rock groups for use in earthworks

Indicative strength	Geological nature	Group Symbol	Parameters (Intrinsic properties) Fragmentability Index	Degradability Index	Los Angeles Coefficient	Micro deval Coefficient	Dry Density	Behaviour
			I_{FR} (see note 1)	I_{DG} (see note 1)	C_{LA}	C_{MDE}	ρ_d	
Extremely high	Volcanic and plutonic rocks	R1 Vo			< 25	< 10		as a granular soil
	Metamorphic rocks	R1 Me						
Very high	Volcanic and plutonic rocks	R2 Vo			< 35	< 25		
	Metamorphic rocks	R2 Me						
High	Clay Rocks	R3 Cld	< 7	> 5 / > 2		< 45		evolutive or degradable rock
	All other rocks	R3 Xxd [a]	< 7	> 5 / > 2		< 45		
	Clay Rocks	R3 Cl	< 7	< 5 / < 2		< 45		non-evolutive or non-degradable rock
	Limestone	R3 Li	< 7	< 5 / < 2		< 45		as a granular soil
	Sandstone	R3 Sa	< 7	< 5 / < 2	< 45	< 45		
	Conglomerate	R3 Co	< 7	< 5 / < 2	< 45	< 45		
	Volcanic and plutonic rocks	R3 Vo			< 45	< 45		
	Metamorphic rocks	R3 Me			< 45	< 45		
Inter-mediate	Clay Rocks	R4Cld	< 7	> 5 / > 2		> 45		evolutive or degradable rock
	All other rocks	R4 Xxd [a]	< 7	> 5 / > 2		> 45		
	Clay Rocks	R4 Cl	< 7	< 5 / < 2		> 45		non-evolutive or non-degradable rock
	Limestone	R4 Li		< 5 / < 2		> 45	> 1,8	
	Sandstone	R4 Sa	< 7	< 5 / < 2	> 45	> 45		depending on the earth work procedure
	Conglomerate	R4 Co	< 7	< 5 / < 2	> 45	> 45		
	Volcanic and plutonic rocks	R4 Vo	< 7	< 5 / < 2	> 45	> 45		
	Metamorphic rocks	R4 Me	< 7	< 5 / < 2	> 45	> 45		
Low	Clay rocks	R5 Cl	> 7					as a soil after extraction
	Limestone	R5 Li				> 45	< 1,8	
	Sandstone	R5 Sa	> 7					
	Conglomerate	R5 Co	> 7					
	Volcanic and plutonic rocks	R5 Vo	> 7					
	Metamorphic rocks	R5 Me	> 7					

NOTE 1: The values for I_{FR} and I_{DG} will vary with the test procedures adopted. In this table parameters for I_{FR} are given according to French Standard NF P 94-066; parameters for I_{DG} are given according to French Standards NF P 94-067(first value) and Spanish Standard UNE 146510 (second value).

NOTE 2 Clay rocks include rocks formed by mixtures of clay and calcite minerals.

[a] Xx denotes any other rock type designated using two letters as appropriate, for example Sa for Sandstone, Li for limestone.

Table 8 — Classification of chalk groups

Group name	Group Symbol	IDD (Mg/m3)
Chalk of very high density	**CH1**	> 1,95
Chalk of high density	**CH2**	1,7 < IDD ≤ 1,95
Chalk of medium density	**CH3**	1,55 < IDD ≤ 1,7
Chalk of low density	**CH4**	≤ 1,55

Table 9 — Salt rock groups

Group Symbol	Content of salt mineral (in mass percentage)	Conditions for use in earthworks	Range of solubility of salt mineral (examples)
	Any	No special conditions due to solubility.	Low (calcite $CaCO_3$, s = 0,01 g/l for T = 20 °C)
	Less than 0,2 %	May be used in any area of the earth fill. No special conditions due to solubility.	Medium (gypsum $CaSO_4 \cdot 2H_2O$, s = 2,40 g/l for T = 20 °C)
SR1	0,2 to 2 %	May be used in the core of the earth fill. No special precautions are needed when constructing the crown and haunches.	
SR2	2 % to 20 %	Limited use in the core of the earth fill provided that: The core shall constitute a compact and impermeable mass. There are drainage measures and waterproofing to prevent access of surface water and groundwater into the fill.	
SR3	More than 20 %	Generally shall not be used. Its use shall be limited to those cases in which no other soil is available and provided that this is specified and duly justified in the design	
	Less than 0,2 %	May be used in any area of the earth fill. No special conditions due to solubility.	High (Halite NaCl, s = 360 g/l for T = 20 °C)
SR4	Less than 1 %	May be used in the core of the earth fill. Special precautions shall need to be taken when constructing the crown and haunches.	
SR5	More than 1 %	Not to be used.	

Table 10 — Fracture spacing

Group	Rock class symbol	Length Width Height in mm
Wide	**W**	600 to 2 000
Medium	**M**	200 to 600
Close	**C**	60 to 200
Very close	**VC**	20 to 60
Extremely close	**XC**	less than 20

Table 11 — P wave Seismic velocity

Rock class symbol	m/s
V1	> 4 000
V2	3 000 to 4 000
V3	2 000 to 3 000
V4	1 000 to 2 000
V5	< 1 000

Table 12 — Characteristics for execution of earthworks

Earthwork procedure	Examples of characteristics for soils	Examples of characteristics for rock
Excavation	Soil group, particle size distribution, undrained strength	Rock group, rock type, Compressive Strength, Rock Quality Designation, Rock Mass Rating, seismic velocity
Load and transport	Soil group, particle size distribution, water content, density, undrained strength	Rock group, block size after excavation, density
Traffic on site roads	Soil group, water content, plasticity, undrained strength, CBR, , moisture condition value	As for soil, shape of blocks
Handling or storage of materials	Soil group, frost susceptibility, erodability, solubility, weathering, evolutivity, mechanical soundness, chemical soundness	Rock group, (for argillaceous rocks or similar see prEN 16907-3)
Stabilisation (ease of mixing)	Soil group, particle size distribution, water content, plasticity, chemical constituents	As for soil
Placement	Soil group, particle size distribution, water content, plasticity, undrained strength	As for soil
Compaction	Soil group, particle size distribution, water content, plasticity, Proctor density, CBR, undrained strength	As for soil
Treatment with binders	Soil group, particle size distribution, water content, plasticity, chemical constituents, Proctor density, CBR, undrained strength, hydraulic conductivity, deleterious materials	—
NOTE Mechanical soundness includes swelling and collapse properties.		

Table 13 — Characteristics for use in earth structures

Position in earth structure	Examples of characteristics
Starter layer – Foundation layer	soil group, particle size distribution, water content, plasticity, drained shear strength, stiffness, CBR, moisture condition, Proctor density, chemical constituents, deleterious materials, material durability
Drainage layer	soil group, particle size distribution, drained shear strength, hydraulic conductivity, deleterious materials, material durability, chemical constituents
General fill (and embankment core)	soil group, particle size distribution, water content, plasticity,, Proctor density, CBR, undrained strength, bearing capacity, moisture condition, chemical constituents, organic content, swelling and collapse potential, mechanical soundness, chemical soundness, sensitivity and activity of clays
Fill behind structures (transition zones)	soil group, particle size distribution, water content, plasticity, Proctor density, CBR, undrained or drained shear strength, hydraulic conductivity, chemical constituents, deleterious materials, material durability
Outside slopes (shoulders)	soil group, particle size distribution, water content, plasticity, strength, stiffness, Proctor density, CBR, chemical constituents, deleterious materials, material durability
Reinforced earth	soil group, particle size distribution, water content, plasticity, Proctor density, CBR, drained shear strength, hydraulic conductivity, chemical constituents, deleterious materials
Liner	soil group, particle size distribution, water content, plasticity, undrained strength, Proctor density, hydraulic conductivity, chemical soundness, sensitivity and activity of clays
Upper layer beneath capping (sub-grade)	soil group, particle size distribution, water content, plasticity, strength, stiffness, CBR, chemical constituents
Capping layer	soil group, particle size distribution, water content, plasticity, frost susceptibility, undrained strength, Proctor density, CBR, chemical constituents, organic content, deleterious materials, frost heave potential

NOTE 1 Mechanical soundness includes swelling and collapse properties.

NOTE 2 The behaviour of rocks once excavated from the ground and incorporated into the earthwork process are normally characterised as soils.

APPENDIX I.2.3

Reference European Standard "Earthworks"

NF EN 16907-1: Part 1: Principles and general rules – DECEMBER 2018

Extracts: Informative Appendices / European national Practices

Preservation of national experiences in application of the "Earthworks" Standard

This objective is explained in the Standard - Part 1 document, as follows:

"14 Use of national experience and non-conflicting rules"

"14.1 General"

Earthworks rules and suggested procedures, which are the heart of the different parts of this standard, are either principles, which apply to any situation, or experience-based rules, which usually differ for different materials and climates and can result from local or national practice, including specific ways of collecting experience and expressing the corresponding rules. The collection and preservation of national experiences is one of the fundamentals of this document. Informative Appendices devoted to national practices aim at sharing information on existing systems and offering the possibility to the standard users to benefit from these experiences.

The existence of national standards or well established and accepted procedures complementing the present one, by non-conflicting rules, is allowed.

"14.2 Informative examples of experience-based national practices"

Existing practices cover the whole process of design, execution and control of earthworks, to transform materials from different sources into earth-structures with specified properties. This process consists of three successive, though interrelated stages:

Stage 1: Based on the available information, selection of the materials which can be used to meet the needs (in terms of volumes and required properties) of the earth-structure:

- classification (assignment) of materials into groups based on intrinsic properties;
- definition of possible uses without and with treatment (experience based classification or new tests);
- choices and preliminary optimization (volumes and destination).

Stage 2: Organization (design) of earthworks (including extraction, transport, eventual treatment, compaction):

- further specific classification including state and other parameters;
- definition of the compaction process as a function of initial state, climate, treatment, preferred or available equipment, etc.;
- final optimization (equipment, schedule), which produces the “earth movement” and optimal use of equipment.

Stage 3: Organization of the control of earthworks:

- selection of tools (method and equipment);
- definition of acceptance criteria for the completed earth-structure.

The detailed implementation of these staged practices, which cover the contents of EN 16907-2, EN 16907–3, EN 16907–4, EN 16907–5 and EN 16907–6, has been described in different ways. Informative Appendices B to H explain how these practices have been organized in different European countries.

List of informative Appendices presented by different countries

Appendix B (Informative) Summary of national practice – Austria
Appendix C (Informative) Summary of national practice - France
Appendix D (Informative) Summary of national practice - Germany
Appendix E (Informative) Summary of national practice - Norway
Appendix F (Informative) Summary of national practice - Spain
Appendix G (Informative) Summary of national practice – Sweden
Appendix H (Informative) Summary of national practice - United Kingdom

All Appendices can be found in the Standard. In the Earthwork Manual, we present the examples of two countries, Spain and France, often cited as references in different PIARC technical reports.

Examples of informative Appendices, summary of national practices

A France

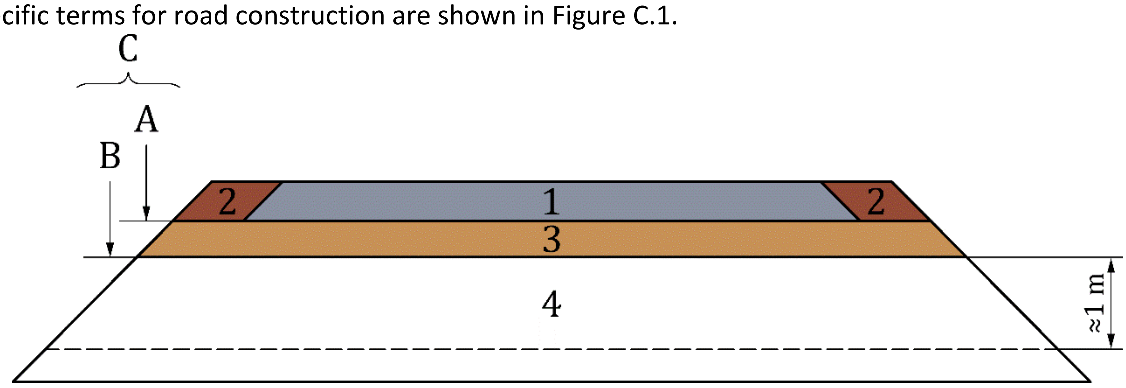
B United Kingdom

A Informative Appendix
Summary of national practice - France

B.1 Introduction

Earthworks rules applied in France are based on test and classification standards and guides prepared mainly by public organisations. These documents follow the same stages as indicated in the present standard: global geotechnical study, material classification, execution of works design using classification and other parameters and associated execution procedures. The core system was established for road construction; similar rules exist for railways, dams and the building sector.

Specific terms for road construction are shown in Figure C.1.



Légende

A	road support platform (PF)
B	fill surface (AR)
1	pavement layers
2	shoulders
3	capping layer
4	upper part of fill (embankment) or natural ground (cutting) (PST)

Figure C.1 — Definitions for road structure

B.2 Classification of materials

Site investigations are when possible organized in successive stages: covering site inspection, use of topographic and geological maps, aerial photos, drilling and excavation for soil and rock sampling, groundwater measurements, field and laboratory tests and geophysical investigations. These investigations aim at defining a geological model, identifying and studying geotechnical issues (compressible zones, karstic zones, unstable slopes, etc.), defining the conditions for extracting materials, their potential use and conditions of use, and making an estimation of the mass earthworks balance.

Some 40 years ago, much effort was put in expressing the national experience of earthworks in the form of a classification system, where materials are gathered in groups with similar behaviour during the execution of embankments and capping layers. This system, which is similar to the classification system presented in EN 16907-2, is described in the "Guide for the execution of embankments and capping layers" (SETRA/LCPC, 1992) and in French standard NF P11-300. It is based on the intrinsic properties of the materials.

Three main groups were established:

- soils;
- rocks;
- organic soils and industrial by- or co-products

A. Soils group

Soils are natural materials, made up of grains which can easily be separated simply by crushing or by the action of a stream of water. Those containing a percentage of organic matter larger than 3 % are defined as organic soils.

The soils group is divided into 4 main classes:

- class A - Fine soil;
- class B - Sandy and gravelly soil with fines;
- class C - Soil containing fines and large elements;
- class D - Soil insensitive to water.

These classes are sub-divided into sub-classes: A1 to A4, B1 to B6, C1 to C2 and D1 to D3, on the basis of particle size distribution and interaction of fine soils with water:

- three particle size characteristics are used: the size Dmax of the largest element contained in the ground, the fines percentage (C80µm) and the percentage of particles with a size less than 2 mm (C2mm);
- the plasticity index (IP) and the methylene blue value (VBS). Both values can be measured on every soils, but VBS is preferred for soils with limited clay activity (VBS < 2,5) and IP for soils with a moderate to high clay activity (IP > 12).

For further classification, other parameters such as hydric state, friability of particles and their behaviour under attrition are measured. These are used to select the most appropriate earthworks processes (C.3).

B. Rocky materials group

The classification of rocky materials is primarily based on geology.

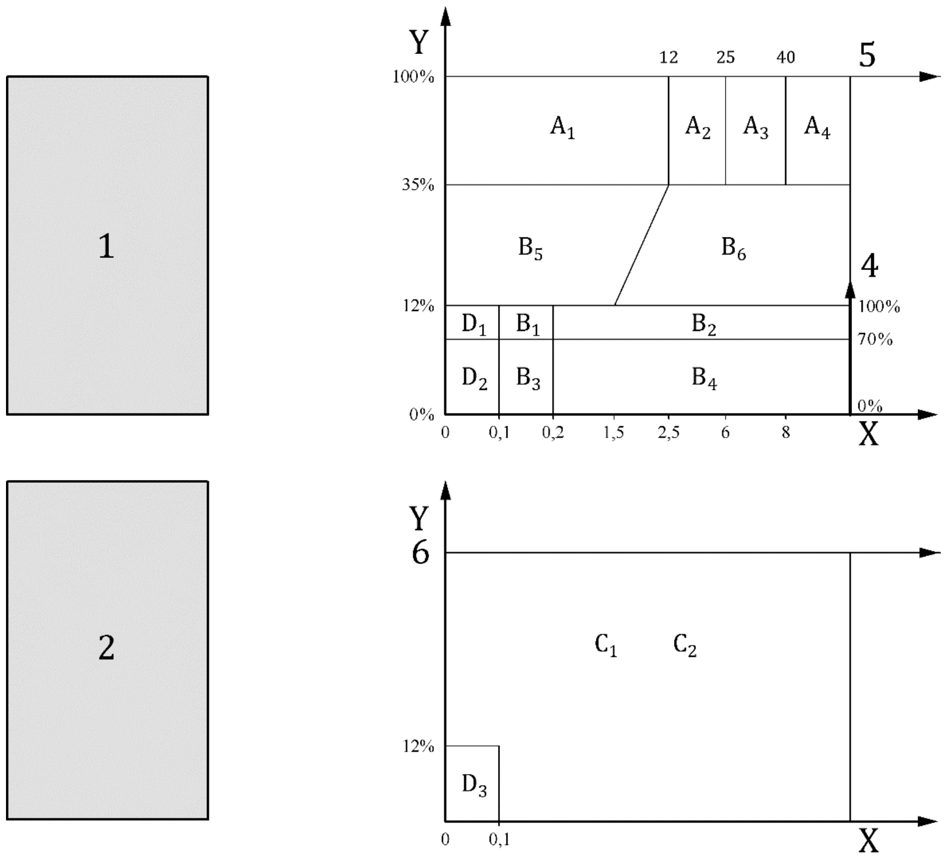
Sedimentary rocks are divided into 5 classes:

- R_1 class is the class of Chalks. These are classified according to their dry density ρ_d and their moisture content w_n. Based on these parameters, the class is divided into three sub-classes (R_{11}, R_{12} and R_{13});
- R_2 class concerns the sundry calcareous rocks. The more compact calcareous rocks are classified according to their behaviour under attrition (micro-Deval test), while softer rocks are classified according to their bulk unit weight. The sub-classes go from R_{21} to R_{23}.

- R_3 class concerns the argillaceous rocks. The scalable nature of these rocks is determined by their fragmentation characteristic (FR test) and their weathering degradability (DG test). The sub-classes go from R_{31} to R_{34}.
- R_4 class concerns the siliceous rocks. The more compact siliceous rocks are classified according to their behaviour under attrition (micro-Deval test and Los Angeles test), while softer rocks are classified according to their fragmentability. The sub-classes go from R_{41} to R_{43}.
- R_5 class concerns the saline rocks. In mechanical terms, this class of materials is like classes R_2 and R_3 but materials are more soluble in water. Critical parameters are fragmentation characteristic and soluble salt content and two sub-classes are defined, R_{51} and R_{52}.

Igneous and metamorphic rocks are considered together in the R_6 class. The more compact rocks are classified according to their behaviour under attrition (micro-Deval test and Los Angeles test), while softer rocks are classified according to their fragmentation characteristic. The sub-classes go from R_{61} to R_{63}.

The French classification of soils and rocks for earthworks is summarized in Figure C.2. Symbols C_1 and C_2 are not used alone. They are added to another symbol from the A or B series: for example, C_1A_3.



3	Sedimentary rocks	Carbonate rocks	Chalk	R1
			Limestone	R2
		Argilaceous rocks	Marls, claystone, pelite, etc.	R3
		Siliceous rocks	Sanstone, puddigstone, breccia, etc.	R4
		Saline rocks	Rock salt, gypsum, etc	R5
	Igneous and metamorphic rocks	Granite, basalt, andesite, gneiss, schist, slate, etc		R6

Figure C.2 — French classification of soils and rocks for earthworks
(NF P 11-300)

Key

X	percent passing 80 µm	4	percent passing 2 mm
Y	VBS	5	Ip
1	soils Dmax ≤ 50 mm	6	fraction 0/50 mm
2	soils Dmax > 50 mm	C1	poorly structured rounded or angular materials with fraction 0/50 mm > 60-80 %
3	rocks	C2	strongly structured angular materials with fraction 0/50 mm < 60-80 %

C. Organic soils and industrial by- or co-products group

This category (class F) concerns specific materials which may prove interesting from a technical and economic point of view when used in embankments and capping layers, if they are acceptable with respect to the environment.

There are nine families. The F_1 sub-class concerns the natural materials containing organic matter. The F_2 sub-class concerns the silica-aluminous airborne dusts from thermal power plants. The F_3 sub-class concerns the coal schist. The F_4 sub-class concerns the schist from potash mines. The F_5 sub-class concerns the phosphogypsum. The F_6 sub-class concerns the clinkers or the slags resulting from incineration of household waste. The F_7 sub-class concerns the demolition wastes. The F_8 sub-class concerns the blast-furnace slags. And the F_9 sub-class concerns the other wastes and industrial by- and co-products.

For each sub-class, the possibility of use depends on specific parameters.

In addition to technical issues, these materials are concerned by environmental ones. These issues are discussed in a Guide “Acceptability of alternative materials in road construction – Environmental assessment” (SETRA, 2011).

The first parameter is the type of layer and type of covering (permeable material or natural soil). The study of these polluted materials is done gradually, to assess their possibility of use: leaching tests first, then percolation tests or a particular study for those who did not comply with the tests.

These materials cannot be used at a distance less than 30 m of a river, in a karst area, in sensitive areas nor in a catchment area.

B.3 Design of Earthworks

B.3.1 Introduction

As already mentioned, the philosophy of the French GTR system is to identify the nature of the ground (soils, rocks, other materials), then make specific tests to enter classification tables which lead to experience-based instructions for performing works. In case the ground does not fit into the classification system, preliminary full scale tests are performed to validate an execution procedure (material, extraction or compaction process, eventual treatment) which produces the expected earth-structure. This procedure is used to define the work to be done (for method specification or for defining the work process leading to specified end results).

B.3.2 Specification of the mechanical properties to be obtained

The required properties of the earth structure are defined at the level of the road support platform (PF) and are linked to the pavement design (in case of road construction).

Four classes of platforms are used in France, as indicated in Table C.1. Class PF2 was subdivided in 2014, by adding a new threshold 80 MPa. Future documents will have five classes: PF1, PF2 (50-80 MPa), PF2qs (80-120 MPa), PF3 and PF4.

Table C.1 — Classes of road support platforms (PF)

Deformation modulus E_{v2}	20–50 MPa	50–120 MPa	120–200 MPa	> 200 MPa
Class of PF	PF1	PF2	PF3	PF4

The attainable PF class is derived from the properties of the upper part of the fill (PST) and the resulting AR class, and from the capping layer design (C.3.5). These depend on the classification of the materials, their hydric state and the drainage. There are 7 classes of PST (Table C.2) and 5 classes of AR (Table C.3).

Table C.2 — Definition of PST classes

PST case	Types of materials	Comments
PST n° 0	A, B_2, B_4, B_5, B_6, C_1 in a very humid hydric state (th)	Peaty, swampy or flood zones. The deformability may be very high at some moments of the construction or life of the road.
PST n° 1	A, B_2, B_4, B_5, B_6, C_1, R_{12}, R_{13}, R_{34} and some C_2, R_{43} and R_{63} in a humid hydric state (h)	PST made of water sensitive materials with high deformability during the installation of the capping layer and with no later improvement.
PST n° 2	A, B_2, B_4, B_5, B_6, C_1, R_{12}, R_{13}, R_{34} and some C_2, R_{43} and R_{63} in a medium hydric state (m)	PST made of water sensitive materials with good stiffness during the installation of the capping layer, with a possible lowering of this stiffness in case of water infiltrations.
PST n° 3	Same materials as for PST n° 2	PST made of water sensitive materials with good stiffness during the installation of the capping layer, with a possible lowering of this stiffness in case of water infiltrations.
PST n° 4	Same materials as for PST n°1, provided that their particle size allows for treatment	PST made of water sensitive materials, improved with lime or hydraulic binders on 0,30 to 0,50 m thickness.
PST n° 5	B_1, D_1 and some rocky materials of class R_{43}	PST made of sandy materials, insensitive to water, above the water table, with trafficability problems.
PST n° 6	D_3, R_{11}, R_{21}, R_{22}, R_{32}, R_{33}, R_{41}, R_{62}, and some materials from classes C_2, R_{23}, R_{43} and R_{63}	PST made of gravelly or rocky materials, insensitive to water, but with levelling or trafficability problems.

Table C.3 — Definition of AR classes

AR classes	AR0	AR1	AR2	AR3	AR4
Deformation modulus	< 20 MPa	20 MPa	50 MPa	120 MPa	200 MPa

The hydric states are defined in C.3.3.

The correspondence between PST classes and AR classes is shown in Table C.4. The correspondence between AR classes and PF classes is shown in C.3.5 (Table C.7).

Table C.4 — Link of PST classes and AR classes

PST	Possible AR	Comments
PST n° 0	AR 0	Such zones need to be improved (substitution and/or drainage) to allow to reevaluate the AR class to at least AR 1.
PST n° 1	AR 1	In this case, two solutions: either improve the upper part of the material (at least 0,5 m) by lime treatment (to obtain AR 2, 3 or 4), or make a thick capping layer using granular materials, insensitive to water.
PST n° 2	AR 1	A capping layer is generally necessary. In case of ground water lowering at sufficient depth, see PST 3.
PST n° 3	AR1 or AR2	AR 1, similar to PST n° 2, in the absence of drainage of the pavement base and sealing of the AR. AR 2 in case of drainage of the pavement base and sealing of the AR.
PST n° 4	AR 2	The need for a capping layer depends on the project and on the measured values of the bearing capacity of the treated material.
PST n° 5	AR 2 or AR 3	The selection of AR2 or AR3 depends on the nature of materials and the measured value of E_{V2}. Short-term and long-term stiffness values are equal. No capping layer is usually needed except for trafficability.
PST n° 6	AR 3 or AR 4	The selection of AR3 or AR4 depends on the nature of materials and the measured value of E_{V2}. Short-term and long-term stiffness values are equal. No capping layer is usually needed except for levelling and short-term trafficability. A levelling layer may be sufficient.

B.3.3 Classification of hydric state of materials and weather conditions

To specify the execution procedure for embankment construction, the French rules consider two main factors:

a) weather conditions during the embankment construction. Provisions should be made for favourable and unfavourable conditions depending on the season during which the earthworks start (this shall be noted in the risk analysis document);

Four possibilities are considered in the classification tables:

- heavy rainfall: symbol used ++ ;
- light rainfall: symbol used + ;
- weather conditions free from any significant rainfall or evaporation: symbol used = ;
- weather conditions causing significant evaporation: symbol used -

b) water sensitivity and water content (state characteristics) of the material. In fact, there are two main types of materials (soils or rocks), those which are water-sensitive and the others. This sensitivity is defined on the basis of clay activity (percentage below 80 µm and methylene blue value or plastic index I_P).

To define the state characteristics (the water content), one needs:

- the value of the soil consistency index I_c;
- the soil IPI value;
- the natural water content (w_n) at a given time, in relation to the optimum water content (w_{OPN}) determined from the standard Proctor test on the fraction smaller than 20mm.
- the only exception for defining the hydric state is made for chalk, for which the hydric state is defined from the dry density and the moisture content.

Five hydric states are considered (Figure C.3):

- the normal state (m) is the best condition for placement ; in particular, it allows appropriate compaction to be achieved;
- the wet (h) and very wet (th) states indicate soils for which trafficability and compaction are difficult (a very wet soil is not normally trafficable for a standard earthmoving plant);
- the dry (s) and very dry (ts) states correspond to soils which are difficult to compact to form stable fill structures: with these kinds of soils the compaction energy has to be adapted. But this is outside of the GTR recommendations, which only consider compaction energies similar to those required in the Normal Proctor test.

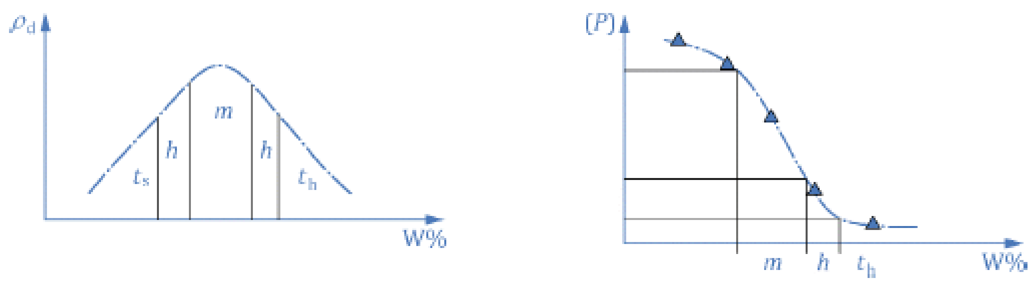


Figure C.3 — Hydric states

B.3.4 Fill material

The GTR prescribes, according to the materials classification and their hydric state, and depending on the weather expected during the earthworks, the conditions in which these materials may or may not be used in the embankment. These conditions depend on various factors, namely:

- extraction: in layer or front loading;
- granularity: disposal of some granular categories (above 800 mm or 250 mm) or complementary fragmentation after extraction;
- moisture correction: this is to add water (humidification or watering) or to dry the material (aeration, wringing);
- treatment: with lime or other suitable reagent;
- layer thickness: thin or moderate (20-30 cm or 30-50 cm or any);

- compaction: weak, average, intense;
- height of the embankment: less than 5 m, 5 – 10 m, more than 10 m.

The treatment of materials using lime or hydraulic binders is defined in a Guide "Treatment of soils with lime or hydraulic binders for embankments and capping layers" – GTS (SETRA/LCPC, 2000).

The treatment of a material with binders consists in mixing, more or less thoroughly, the material with lime, or hydraulic binder, or both of them, and optionally with additional water. The objective is to enhance the properties of materials (soils) with poor characteristics in earth structures.

Two aims can be developed for studies and works:

- soil improvement: reducing water content in the soils for reusing them in the embankments; the mixing of soils and lime (generally), or soils and hydraulic binders, shall lead to a water content between normal and wet states;
- soil stabilization: improving mechanical properties by mixing of soils and lime, or soils and hydraulic binders, or both of them, to improve specified parameters like:
    - CBR after immersion > immediate bearing index for embankment concerned by area liable to flooding = resistance to water;
    - cohesion and internal friction of the soils, for embankment stability;
    - compressive strength for high embankments and technical embankments;
    - modulus of elasticity and tensile strength for capping layers, etc.

— Soil improvement

In this case, the addition of a binder is aimed at improving the physical properties of the soil (short-term), or, more generally, a material, such as water content, plasticity, water and frost susceptibility, compactibility and swelling potential. The quantity of binder added may not be sufficient to induce significant permanent properties.

— Soil stabilization

The aim of soil stabilization consists in obtaining an homogeneous mixture of soil with binder(s), and optionally with water, which, when properly compacted, significantly changes (generally in the medium or long-term) the characteristics of the soil in a way that renders it stable, particularly with respect to the action of water and frost. It creates a permanent behaviour, which can be measured by methods typical of solid materials.

The methods used to study soil treatment are listed in the GTS guide (SETRA/LCPC, 2000).

The first step is to search for soils characteristics, which could hinder or exclude the existing treatment techniques (feasibility):

- soils with blocks. The feasibility of treatment depends on the D_{max} and number of the blocks. For stabilization, D_{max} is generally smaller than 100 to 150 mm, depending on the equipment. Treatment with lime or hydraulic binders concerns in generally soils (D_{max} < 50 mm) or soils with blocks in minority (class C_1);
- any constituent in the soils which would adversely affect setting, hardening, performance and volumetric stability of the treated material (organic matter, soils with sulfur, sulfide, sulfate, etc.

This part of the study is generally done by experienced geotechnical engineers.

Then, for soil improvement, the studies consist in laboratory tests to determine for the mixture (soils and binders): Proctor references, immediate bearing index with objectives giving by GTS, CBR after immersion when resistance to water is needed.

For soil stabilization, the second step is to determine the suitability for treatment, using accelerated swelling tests (NF P 94-100).

This test should take place at the earliest possible stage in the project to identify any risks of swelling or setting failure. This study shall determine the indirect tensile strength (Rit) and volumetric swelling. The suitability for treatment is given in Table C.5 for soils with hydraulic binder and in Table C.6 for soils with lime only.

Table C.5 — Criteria for interpreting suitability for treatment for soils with hydraulic binder

Soils with hydraulic binder	Volumetric swelling G_v (%)	Tensile strength after 7 days conservation in water R_{it} (MPa)
Suitable	$G_v < 5$ % and	$R_{it} > 0{,}2$ MPa
Doubtful	5 % $\leq G_v \leq$ 10 % or	0,1 MPa $\leq R_{it} \leq$ 0,2 MPa
Unsuitable	$G_v > 10$ % or	$R_{it} < 0{,}1$ MPa

Table C.6 — Criteria for interpreting suitability for treatment for soils with lime only

Soils with only lime	Volumetric swelling G_v (%)	Tensile strength after 7 days conservation in water R_{it} (MPa)
Suitable	$G_v < 5$ %	No value
Doubtful	5 % $\leq G_v \leq$ 10 %	
Unsuitable	$G_v > 10$ %	

Additional tests like CBR after immersion, compressive strength (R_c), cohesion and internal friction, can be realized. The aim is to analyse specifics properties in the medium or long-term.

Criteria for technical (next to structures) embankments, high embankments, upper part of earthworks (PST), etc. are given in the GTS. For example, for a high embankment, GTS recommends to obtain a compressive strength larger than 0,5 or 1 MPa (depending of the height of embankment).

Finally, the treatment study can determine long-term modulus and tensile strength (28 days – 90 days or 180 days) to be used for designing capping layers made of materials treated with hydraulic binders.

B.3.5 Capping layer

For the materials used in capping layers, the French rules consider the hydric state of the material, the weather expected during the earthworks, and the PST/AR couple classification. With these data, the GTR indicates the possibility for re-use of materials, the thickness of the capping layer, how the capping layer should be constructed and the platform class (PF1 to PF4) that will be obtained. In particular, the following can be selected:

- the granularity according to the water sensitivity, by carefully adjusting the mixing used in the layer;
- the water content correction;
- the treatment: the use of lime or other binders will make some very humid (th) soils suitable for use as capping layer material. A special study shall always be made to determine the benefits and feasibility of treatment, application rates, and associated difficulties if they exist;
- the protective measures: protective coating, etc.

If the chosen capping layer is thinner than the thickness recommended in the GTR tables, the PF class is kept the same as the AR class. If it is thicker, the PF class may be increased.

Three cases are considered:

- capping layer made of untreated granular material. With some kinds of material, the PF3 class will be obtained directly, with adequate capping layer thickness (Table C.7).

Table C.7 — Example of specifications for capping layer made of untreated granular material

AR class	PF class	Material used	Capping layer thickness
AR1	PF3	B_{31}, C_1B_{31}, C_2B_{31}, D_{21}, D_{31}, R_{21}, R_{41}, R_{61} C_1B_{11}, C_2B_{11}, R_{11}, R_{42}, R_{62} (provided the PF stiffness is checked)	0,80 m (a reduction of 0,10 to 0,15m may be accepted in case an adequate geotextile is put between the PST and the capping layer)
AR2	PF3	The same as above	0,50 m

- capping layer made of lime or cement treated clayey soils, treated *in situ*. The PF3 class above AR1 or AR2 will then be obtained directly, when following the indications of Table C.8.

Table C.8 — Example of specifications for capping layer made of treated clayey soil

AR class	PF class	Material used	Capping layer thickness
AR1 (for PST 2 and PST 3 only)	PF3	A_3 treated with lime, only	0,70 m (in two layers)
		A_1, A_2, A_3 treated with lime+cement or eventually cement alone	0,50 m (in two layers)
AR2	PF3	A_3 treated with lime, only	0,50 m (in two layers)
		A_1, A_2, A_3 treated with lime+cement or eventually cement alone	0,35 m

- capping layer made of hydraulic binder treated materials. The expected PF class above AR1 or AR2 is defined in Table C.9, on the basis of the mechanical class of the treated material and the thickness of the capping layer.

Table C.9 — Specifications for capping layer made of materials treated using hydraulic binder

AR class		AR1			AR2	
		Thickness of the capping layer				
Mechanical class of the treated material used	3	< 30 cm	30 cm	40 cm	25 cm	30 cm
	4	30 cm	35 cm	45 cm*	30 cm	35 cm
	5	35 cm	50 cm*	55 cm*	35 cm	45 cm*
PF class obtained		PF2	PF3	PF4	PF3	PF4
* two layers will usually be needed to obtain the required compacity at the bottom of the capping layer						

- The definition of the mechanical class of the materials treated with hydraulic binders is based on direct traction strength R_t and deformation modulus E obtained from tests on treated material, after 90 days maturation. The values of R_t and E are put on the diagram of Figure C.4, which defines five zones. The zone in which Rt and E are located is used to define the mechanical class, depending on whether the treatment will be done in place or in a plant (Table C.10).

Table C.10 — Definition of the mechanical class of materials treated with hydraulic binders

Treatment in plant	Treatment in place	Mechanical class obtained
Zone 1		1
Zone 2	Zone 1	2
Zone 3	Zone 2	3
Zone 4	Zone 3	4
Zone 5	Zone 4	5

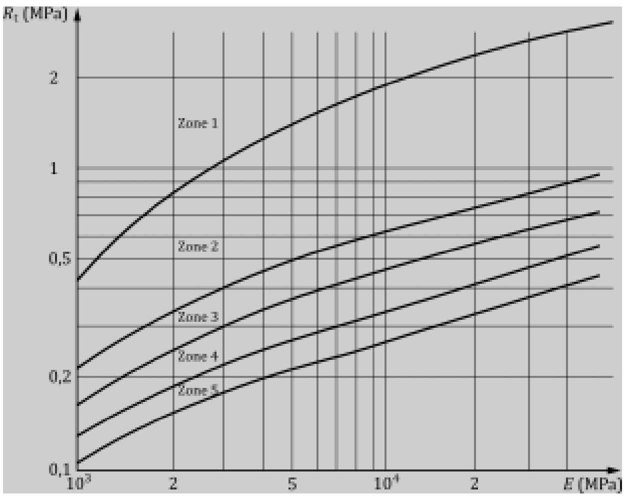


Figure C.4 — Zones of mechanical properties for materials treated with hydraulic binders

B.3.6 Compaction of fill

The GTR recommendations, once the conditions in which the materials may be used are defined, indicates the compaction energy needed to achieve the densification objectives.

The GTR classification system, according to nature (intrinsic properties) and state of materials, describes soil classes such that, within each class, the densification energy needed to obtain a stable fill is roughly the same. In this way, the compaction energy can be set beforehand for each specific job, along with appropriate construction method.

The required compaction energy is expressed by two parameters:

- maximum thickness (compacted thickness, not bulk thickness) of constituent layers of fill;
- Q/S ratio in m_3 per m_2, a measure of the ratio between the compacted soil volume placed in a given time (say, one day) Q, and the area of fill covered by the compaction machine in the same time S. Volume Q is calculated from the number of round trips by haulage plant of known capacity or beforehand from the estimated geometrical volume of the embankment to calibrate the haulage plant. Area S is obtained from the effective width of the compaction machine multiplied by the distance covered by the machine, usually read from the mileage counter or, better, from a tachograph fitted to the machine.

The next step is to classify the compaction plant according to its performance. This classification refers to French standard NF P 98-736. Its basic principles are set forth below:

- the rollers considered have a compaction width of 1,30m or more;
- the basic types of compacting plant addressed are:
- pneumatic tyred rollers Pi;
- smooth vibrating drum rollers Vi;
- tamping rollers VPi;
- static tamping rollers SPi;
- vibrating plate compactors PQi;
- i is the class number; it increases with compaction efficiency within each type category.

The final step of the GTR method is to fix the number of load applications for a given material, a given plant, a given compacted thickness and a given speed of the plant. This information is given in tables, given for each class of material. Table C.11 shows an example of compaction specifications for material classes A_2 and C_1A_2.

Table C.11 — Example of GTR recommendations for compacting A_2 and C_1A_2 soils

A_2, C_1, A_2 (*)																					
Compacteur	Modalités	F1	F2	F3	V1	V2	V3	V4		V5		VP1	VP2	VP3	VP4	VP5	SP1	SP2	PQ3	PQ4
Energie de compactage moyenne	Q/S	0.030	0.050	0.070		0.035	0.050	0.065		0.080			0.035	0.065	0.080	0.105	0.035	0.060		
	e	0.20	0.25	0.35	0	0.20	0.30	0.30	0.40	0.30	0.45	0	0.20	0.30	0.30	0.30	0.20	0.30	0	0
	V	5.0	5.0	5.0		2.0	2.0	2.5	2.0	3.0	2.0		2.0	2.0	2.5	3.0	3.0	3.0		
Code 2	N	7	5	5		6	6	5	7	4	6		6	5	4	3	6	5		
	Q/L	150	250	350		70	100	165	130	240	160		70	130	200	315	280	480		

* D_{max} shall be less than 2/3 of the compacted layer

NOTE 1 Q/S (m), layer thickness e (m), roller speed V (km/h), number of passes N (-), Q/L (m^3/h/m)
NOTE 2 (2) plan an additional operation to erase the prints in case of predicted rain after the work (take off the upper centimetres or use another type of compacting plant)
NOTE 3 0 – compaction plant is not adequate

B.3.7 Extraction and transportation of soil and rocks

The recommended techniques are summarized in Table C.12 (for fine materials) and Table C.13 (for rocky materials).

Table C.12 — Extraction and transportation of fine soils

Excavation (with or without stockpiling)	Shovel excavator or front loader			Motorscrapers
Transportation	Articulated dump truck	Rigid dump truck	Road lorry	
Haul road	Deformable	Slightly deformable	Non deformable	Slightly deformable
Distance	Short distance (<3000 m)	Long distance (<5000 m)	Long distance (>5000m)	Short distance (<1500 m)

Table C.13 — Extraction and transportation of rocky materials

Excavation	Ripper or Blasting	
Loading	Shovel excavator or front loader	
Transportation	Articulated dump truck	Rigid dump truck
Haul road	Deformable	Slightly deformable
Distance	Short distance (2000 m)	Long distance (<5000 m)

B.3.8 Compaction of materials

Suitable equipment for various types of materials are indicated in Table C.14.

Table C.14 — Summary of suitable compaction equipment

Compaction equipment	Wheeled roller	Vibratory smooth roller	Vibratory tamping roller	Deadweight tamping roller	Vibrating plate compactor
Fine silty or clayey soils with less than 30 % of particles > 50mm	possible	adapted	adapted	adapted	non adapted
Fine silty or clayey soils with blocks	possible	adapted	possible	possible	possible
Sandy gravels	possible	adapted	non adapted	non adapted	possible
Strong rocks	possible	adapted	non adapted	non adapted	possible
Evolutive rock and chalk	possible	adapted	possible	possible	possible

B.4 Control of earthworks

B.4.1 Introduction

This clause describes:

- the actions and current practices in France for monitoring the execution of earthworks;
- the ways and procedures to check this execution.

Two kinds of controls are implemented:

- controls by tests and trials; these are defined by their type and frequency;
- controls by visual observations; these visual checks shall be done by a skilled officer.

B.4.2 Technical processes and control methods

B.4.2.1 Identification of materials

Before the beginning of the earthworks in the field, materials are identified (as described by GTR). The conditions for their reuse, as a function of their hydric state, are then defined.

During the construction operations, the hydric state of the materials shall be checked day by day.

From time to time, a check of the maximal grain size and clay activity can be done to ensure the homogeneity of the deposits from which materials are extracted.

<u>Relevant standardized tests for identification of materials</u>

Particle size analysis NF P94-056

Water content NF P94-050

Atterberg's limits NF P94-051

Methylene blue absorption capacity NF P94-068

Compaction test (Proctor tests) NF P94-093

CBR after immersion - Immediate CBR - Immediate bearing ratio NF P94-078

Fragmentability coefficient of rocky material NF P94-066

Degradability coefficient of rocky material NF P94-067

Resistance to fragmentation (Los Angeles) NF EN 1097-2

Resistance to wear (Micro-Deval) NF EN 1097-1

Sand friability coefficient NF P 18-576

Particle density NF P 94-054

Density of a dehydrated rock sample NF P 94-064

Water reactivity for quick lime NF P 98-102

B.4.2.2 Préparation des matériaux

— Actions on the hydric state;

For the materials that are too dry, the water content can be increased by spraying to bring it up closer to the optimum water content (w_{OPN}).

For materials that are too wet, the water content can be decreased by mechanical aeration or lime treatment.

— Actions on particle size distribution;

The maximum particle size of materials shall be relevant to the thickness of the layer and its required characteristics.

In all cases, blasting shall be rationalized to avoid, as far as possible, further processing of the rocky materials. After blasting, actions on particle size can be:

- sorting after quarrying;
- screening of rough materials;
- crushing of materials;
- use of hydraulic rock breaker to comminute blocks.

For the capping layers, if a central crushing station is used, rates and storage areas shall be integrated in the project.

The check of these actions is usually visual, but measurements of water content and particle size can be done.

B.4.2.3 Earthmoving program

It is important to ensure that the earthmoving program in the field is consistent with the project. The monitoring can be done by means of the compaction checks.

B.4.2.4 Compaction

Compaction aims at ensuring a minimum required density to avoid long-term settlements and to reduce the voids ratio.

a) Long-term settlements

The method used in France to measure the compaction levels is called the “e-Q/S” method. For a given thickness e of a compacted layer, the ratio Q/S (m_3/m_2) between the compacted material volume Q and the covered surface S. This couple (e, Q/S) is then compared with theoretical values.

Checks on compaction can be:

- check of the compaction tools and the compliance with the contract documents;
- check of the harmony between compaction tools and rates of implementation;
- check of the recorded Q/S on compactors;
- check of relevant distribution of compaction and the compliance with the layer thickness.

In addition, further measurements can be done:

- density measurements;
- for granular materials, measurements with a dynamic penetrometer to check the layer thickness and consistency of compaction.

b) Decrease of the void ratio

The inherent features of the materials (cohesion and angle of internal friction) increase when the void ratio decreases. For the improvement of the mechanical features of cement-treated soils, it is more frequent to adjust the dosage in cement to reach the aimed performances, than to increase the density. This last solution is possible, but subject to realizing a full scale test.

<u>Relevant standardized tests for compaction control</u>

Measurement of mechanical features on drill cores:

- compressive strength: NF EN 13286-41;
- modulus of elasticity: NF EN 13286-43;
- diametral compression test: NF P 98-232-3.

Determination of density of materials on site - gammadensitometer: NF P 94-061-1

Determination of density of materials on site - Method for large materials: NF P 94-061-4

Measurement of density on site - Gamma- gamma ray log: NF P 94-062

Compaction quality control - Constant energy dynamic penetration test method: NF P 94-063

Inspection of compaction quality - Method using a variable energy dynamic penetrometer: NF P 94-105

Compactors. Evaluation of the eccentric moment: NF P 98-761

Pneumatic tyre compactors. Evaluation of the contact pressure on soil: NF P 98-760

B.4.2.5 Deformability

An adequate deformability is needed for the construction traffic. Pavement design is also based on this parameter.

a) Deformability for construction traffic

It is required to check that hydric state of materials is relevant for construction traffic.

b) Deformability for design

It is required to construct a test section in order to adjust operating parameters and to construct a full-scale reference in order to adjust checking parameters. For granular materials, it is recommended to build a full-scale reference with a high thickness in order to compare the materials modulus and the design modulus.

Measurements of the deformability of the roadbed (PF) and formation level (AR) can be done by means of a plate load test, a Dynaplaque, a deflectograph, a Benkelman beam, or a portancemetre.

For lime/cement treated layers, bearing capacity is measured with a deflectograph or curviameter. For lime/cement treated layers, it is also required to check the amount of used binders. At least two ways can be proposed:

- tarpaulins or trays are placed in the field before the binders spreading. After the spreading, the amount of collected binders is measured;
- axles of the spreading truck are weighted before spreading and after spreading on a defined surface.

Relevant standardized tests for deformability measurement

Formation level bearing capacity - Part 1: Plate test static deformation module (E_{V2}): NF P 94-117-1

Formation level bearing capacity - Part 2: Dynamic deformation module: NF P 94-117-2
Measurement of a rolling load deflection: NF P 98-200-1, 2, 3, 4 and 7

B.4.2.6 Drainage

a) Construction drainage

The aim of construction drainage is to allow the evacuation of run-off waters (precipitation and intercepted groundwater).

Attention shall be paid to:

- cross-sections and longitudinal sections have to allow rational run-off;
- avoid crossing water outfall during cuttings;
- protect the top of the earthworks by compaction;
- building of temporary ditches;
- collection of groundwater.

b) Permanent drainage

During construction process, compliance checks between earthworks and project shall be done.

After works, the efficiency of drainage shall be checked.

B.4.2.7 Geometrical setting out of the works

Non respect of project layout can lead to costly rework. The Land surveyors marks shall be respected. Furthermore, the widths of platforms, embankment slopes and altimetry of structure shall be checked.

a) Environment

The aim is to minimize the impact of earthworks on the environment. It is important to avoid soaring of dust or soaring of dry cement. Trucks and other earthworks vehicles shall be cleaned in a dedicated place. All waters from construction site have to decant before salting out in the environment. Vibrations resulting from mining or from compaction and noise shall be limited.

For these points, the checks are visual, but some measurements can also be done.

b) Specific building methods

For these works, the recommendations specific to these techniques shall be followed.

B.5 References

AFNOR (1992). NF P 11-300: Exécution des terrassements. T2: Classification des matériaux utilizables dans la construction des remblais et des couches de forme d’infrastructures routières.
SETRA (2007). Conception et réalisation des terrassements – Guide technique.
Fascicule 1: études et exécution des travaux.
Fascicule 2: organization des contrôles.
Fascicule 3: méthodes d'essais.

The English translation of this document is available and can be downloaded from the Cerema website.
SETRA (2007) Design and execution of earthworks.
Section 1: Design and execution of work.
Section 2: Organization of checks.
Section 3: Test procedures.
SETRA (2011). Acceptabilité de matériaux alternatifs en technique routière – Évaluation environnementale – Guide méthodologique.
SETRA-LCPC (1992-2000). Réalisation des remblais et des couches de forme – Guide technique.
Fascicule 1: Principes généraux.
Fascicule 2: Annexes techniques.
An English abbreviated version of this document is available and can be downloaded from the Cerema website.
LCPC (2003). Practical manual for the use of soils and rocky materials in embankment construction.
SETRA-LCPC (2000). Traitement des sols à la chaux et/ou aux liants hydrauliques – Application à la réalisation des remblais et des couches de forme – Guide technique.
SETRA-LCPC (2007). Traitement des sols à la chaux et/ou aux liants hydrauliques – Application à la réalisation des assises de chaussée – Guide technique.
The English translation of this document is available and can be downloaded from the Cerema website.
SETRA (2008). Treatment of soils with lime and/or hydraulic binders – Application to the construction of pavement base layers.
SETRA (2006). Drainage routier – Guide technique.
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SETRA (2007). Road Drainage.
SETRA-LCPC (2000). Organization de l'assurance qualité dans les travaux de terrassements – Guide technique.
CFTR (2002). Terrassements à l'explosif dans les travaux routiers – Guide technique.

Tests standards listed in the text.

B Informative Appendix

Summary of national practice - United Kingdom

"H.1 Introduction"

This informative Appendix briefly summarizes the standards that cover the field of earthworks in the United Kingdom. The UK practice satisfies the requirements of the set of TC396 standards.

The British Standard that covers the field of earthworks is BS 6031:2009 Code of practice for earthworks [3], it reflects the widespread UK practice of using the Specification for Highway Works (SHW) 600 series for the construction of earthworks [1]. This Appendix to TC396 EN 16907-1 has been prepared as a brief summary of these documents to enable engineers who are not familiar with the UK earthworks system to consider utilizing it on projects where appropriate.

The EN 16907 series has been developed to address the elements of an earthworks project in terms of the following overall phases of activity:

a) selection of materials: fill material identification and classification (EN 16907-2);
b) design: process to determine how to utilize the fill materials available, including the preparation of the earthworks project Specification (EN 16907-1), and determination of the appropriate methods of earthworks construction to meet the Specification criteria (EN 16907-3); and
c) QA/QC testing to control the works (EN 16907-5).

The UK system has been developed since 1945 specifically to address the task of earthworks construction. The three phases (a) to c)) listed above are all covered within that system, with the phases being inter-linked and inter-dependent. Therefore, it is not appropriate for an earthworks project to use one element of the UK system in isolation; for example, the classification system (a)) is developed specifically to suit the design based approach required (b)). This informative Appendix summarizes the phases of design input required by the UK system for earthworks, and how these satisfy the requirements of EN 16907-1 to −5.

BS 6031:2009 is the UK's national code of practice for earthworks. It is an all-encompassing code of practice; the document has been developed to enable it to cover all earthworks projects, with the exception of dams. BS 6031:2009 is aligned with Eurocode BS EN 1997-1:2004 and its supporting family of documents. As a code of practice, BS 6031 is not a normative standard and takes the form of guidance and recommendations, which should not be quoted as if it were a specification. However, there is an expectation that any project will address its requirements.

"H.2 Classification of materials"

BS 6031:2009, 3.1 defines the basic principles and terminology to be used for identification and classification of soils for use as earthworks fill materials. Clause 5 of the BS provides guidance on issues to be considered during the planning stage of an earthworks project, and Clause 6 sets out the requirements for site assessment and investigation, including the description and classification of soils and rocks (which is in accordance with the EN ISO 14688 series and EN ISO 14689, respectively), and details the requirements for classification of earthworks fill materials (following excavation), which is in accordance with Tables 6/1 and 6/2 of the SHW (extracts are provided for illustration at Clause H.3.8 of this Appendix).

The SHW standard classification system for earthworks materials includes various classes of fill material as presented in Table H.1. These classes fall into nine main groups of fill materials that have different intended end uses. These groups can be summarized as follows:

- “General fill” is used for the construction of bulk earthworks (i.e. the main body of an embankment) and these can comprise Class 1 “granular fill” (with less than 15 % fines), Class 2“cohesive fill” or Class 3 “chalk fill”;
- Class 4 “Landscape fill” is material that can be used to construct landscape areas; the classification is set broadly to permit the use of as wide a range of materials as possible;
- Class 5 “Topsoil” is sub-divided depending on whether it is site won or imported;
- Class 6 “Selected granular fill” is a group of tightly controlled granular fill materials, each being defined to suit a specific end use (e.g. Class 6F1 to 6F5 are options for capping material);
- Class 7 “Selected cohesive fill” are a group of fill materials with greater than 15 % fines that can be utilized for particular purposes (e.g. Class 7E and 7F are fills for stabilization to form capping);
- Class 8 is a broad range of materials that can be used for trench fill;
- Class 9 “stabilised materials” are formed following the addition of a binder (and meet the requirements of EN 16907-4).

Some intrinsic properties of fill materials are defined within the SHW classification system (e.g. SHW Table 6/2 defines grading requirements), although there is scope for the designer to identify additional classes of fill material with differing state properties if these are shown to satisfy the required engineering use. The SHW classification system requires the designer to determine the acceptable range of state properties to suit the intended end use for many of the classes of fill materials. This system has been developed to maximize the proportion of site won material as fill for the project, while continuing to facilitate acceptable earthwork construction. Therefore, the classification process continues through the fill design process described at H.2 below.

Table H.1 — Classification of earthworks materials in the UK by the SHW (reproduced from BS 6031:2009 Table 8)

Type	Class	Description	Typical use
General granular fill	1A	Well graded granular material	General fill
	1B	Uniformly graded granular material	
	1C	Coarse granular material	
General cohesive fill	2A	Wet cohesive material	General fill
	2B	Dry cohesive material	
	2C	Stony cohesive material	
	2D	Silty cohesive material	
	2E	Reclaimed pfa cohesive material	
General chalk fill	3	Chalk	General fill
Landscape fill	4	Various	Fill for landscape areas

Topsoil fill	5A	Topsoil or turf existing on site	Topsoiling	
	5B	Imported topsoil		
Selected granular fill	6A	Selected well graded granular material	Below water	
	6B	Selected coarse granular material	Starter layer	
	6C	Selected uniformly graded granular material	Starter layer	
	6D	Selected uniformly graded granular material	Starter layer	
	6E	Selected granular material	For cement stabilization to form capping – class 9A	
	6F1	Selected granular material (fine grading)	Capping	
	6F2	Selected granular material (coarse grading)	Capping	
	6F3	Selected granular material recycled (bituminous/asphaltic materials, etc.)	Capping	
	6F4	Selected/imported (unbound) granular material that conforms to BS EN 13285 (fine grading)	Capping	
	6F5	Selected/imported (unbound) granular material that conforms to BS EN 13285 (coarse grading)	Capping	
	6G	Selected granular material	Gabion filling	
	6H	Selected granular material	Drainage layer	For reinforced soil
	6I	Selected well graded granular material	Fill	And anchored earth
	6J	Selected uniformly graded granular material	Fill	structures
	6K	Selected granular material	Lower bedding for:	Corrugated steel
	6L	Selected uniformly graded granular material	Upper bedding for:	buried structures
	6M	Selected granular material	Surround to:	
	6N	Selected well graded granular material	Fill to structures	
	6P	Selected granular material	Fill to structures	

Type	Class	Description	Typical use	
	6Q	Well graded, uniformly graded or coarse granular material	Overlying fill for corrugated steel buried structures	
	6R	Selected granular material	For stabilization with lime and cement to form capping – class 9F	
	6S	Selected well graded granular material	Filter layer below subbase	
Selected cohesive fill	7A	Selected cohesive material	Fill to structures	
	7B	Selected conditioned pfa cohesive material	Fill to structures and reinforced soil	
	7C	Selected wet cohesive material	Fill to reinforce soil	
	7D	Selected stony cohesive material	Fill to reinforce soil	
	7E	Selected cohesive material	For stabilization to	Lime – class 9D
	7F	Selected silty cohesive material	form capping to:	Cement – class 9B
				Cement – class 9C
	7G	Selected conditioned pfa cohesive material	Overlying fill for corrugated steel buried structures	
	7H	Wet, dry, stony or silty cohesive material and chalk		
	7I	Selected cohesive material	For stabilization with lime and cement to form capping – class 9E	
Miscellaneous fill	8	Class 1, class 2 or class 3 material	Lower trench fill	
Stabilized materials	9A	Cement stabilized well graded granular material	Capping	
	9B	Cement stabilized silty cohesive material		
	9C	Cement stabilized conditioned pfa cohesive material		
	9D	Lime stabilized cohesive material		
	9E	Lime and cement stabilized cohesive material		
	9F	Lime and cement stabilized well graded material		
NOTE Clays and cohesive soils are shown in shaded rows.				

"H.3 Design of earthworks"

H.3.1 General

The earthworks design process involves the identification of appropriate fill material properties to be included within the earthworks Specification to ensure that the completed embankment satisfies the required end use requirements for the earth structure. Therefore the main output of the earthworks fill material design process is the project specification.

BS 6031:2009 includes the following clauses that relate to the determination of the earthworks projects “Specification”; both reflect the approach required by EN 16907-1:

- Clause 7 describes all aspects of the “Design of Earthworks”, including a summary of the requirements for selection of the required properties of earthworks fill materials properties (at Clauses 7.6.2 and 7.6.4), which is a fundamental design stage required to enable the implementation of the SHW form of Specification;

- Clause 8 sets out the various approaches that can be implemented for the Specification of earthworks fill materials, with the requirements being based on either: method, end product or performance. Clause 8.2 sets out the requirements of the SHW approach, which are summarized below.

A core part of the SHW is the use of “method compaction” for most classes of fill, with “end-product” compaction being required for a number of particular classes of fill. The SHW provides earthworks designers with an introduction to the approach and the documents that need to be referenced for implementation on a project. The intention is to enable designers to utilize the SHW’s well-established method compaction approach on a project wherever the designer considers that it will be an appropriate form of earthworks specification.

The defined “method compaction” requirements of layer thickness and number of passes for a wide range of different types of compaction plant were developed based on extensive research by the Transport Research Laboratory using full scale testing of plant for the geological strata and climatic conditions encountered in the British Isles [10]. The TRL research considered the significant variation in these geological and climatic conditions from the southeast of England to the northwest of Ireland or Scotland. The SHW system accommodates this wide range of conditions by giving the designer flexibility to determine the earthworks specification acceptability limits, based on relationship testing, for the fill materials available for use on the project. Details on the use of relationship testing for earthworks design purposes are provided by Nowak and Gilbert [9].

H.3.2 BS 6031:2009, Clause 8.2: Specification of earthworks by SHW approach

When the SHW [1] forms the basis of the earthworks specification, the document should preferably refer to the SHW rather than repeat the clauses from the SHW (e.g. “The Specification for earthworks shall be Series 600 of the Specification for Highway Works dated 20xx”).

The content of the SHW [1] is updated on a regular basis by the Highways Agency. Users of the SHW should download the up to date version from: http://www.dft.gov.uk/ha/standards/mchw/.

The SHW [1] should be used to satisfy the compaction requirements of BS EN 1997-1:2004, 5.3.3 and the testing requirements of BS EN 1997-1:2004, 5.3.4.

Engineered fills which are used to produce suitably shaped landforms for building structures should be constructed to high standards to minimize the risk of ground movements causing damage to property built on shallow foundations. Specifications based on those developed for highway embankments are not necessarily appropriate for fills on which *buildings* will be founded, since acceptable settlement is likely to be significantly smaller for a building than for a road; hence a more stringent specification might be necessary than for highway purposes (see BS 6031:2009, 7.2).

H.3.3 SHW required documentation within project earthworks specification

The appropriate appendices should be completed by the earthworks Designer and provided with the Specification to enact several of the SHW [1] clauses. As a minimum, the following appendices should be provided:

- Appendix 1/5: Testing to be carried out by the Contractor (see Table H.2 below);
- Appendix 6/1 (including Table 6/1): Requirements for acceptability and testing, etc. of earthworks materials;
- Appendix 6/2: Requirements for dealing with unacceptable materials; and
- Appendix 6/3: Requirements for excavation, deposition, compaction.

Appendices 6/8, 6/12, 6/14 and 6/15 should also usually be provided. Other appendices should only be supplied when the specific works covered are proposed as part of the scheme. Reference to specific clauses in the DMRB [4] may also be provided.

H.3.4 Compaction requirements (classification, design and construction aspects)

SHW Table 6/1 indicates which classes of fill are appropriate for either “method” or “end product” compaction, and stipulates the appropriate method to be adopted where method compaction is appropriate. In some cases, the Designer may decide to modify the compaction requirements, in which case site trials to confirm the proposed approach may be necessary. Attempting to combine elements of both “method” and “end product” compaction for a class of fill is not appropriate.

The method compaction requirements that are stated within Table 6/4 *of the SHW (see* Table H.5 *for an example) were developed based on extensive testing trials undertaken at the Transport Research Laboratory between 1945 and 1990 (as published in full by A W Parsons, [10]). Method compaction is designed to deliver 90 % compaction by* BS 1377, *2,5 kg compaction test for general fill and 95 % for most class 6 fills .The compactive effort stipulated in SHW [1]* Table 6/4 *is designed to produce an adequate state of compaction (less than 10 % air voids for many fills, and less than 5 % for certain class 6 fills) at a conservative (low) water content for the particular class of soil (see SHW [1] NG 612 for more details). See* BS 6031:2009, 7.6.4 *for further explanation.*

H.3.5 Alternative specifications

BS 6031:2009 sets the SHW [1] as the default specification for earthworks, but it does allow a project to use an alternative specification if the following minimum level of information is provided:

a) types of materials permitted for use in the earthworks together with material properties;
b) performance requirements to be met;
c) requirements for the disposal of unsuitable material;
d) requirements for placement, spreading and compaction of the earthworks materials;
e) requirements for the treatment of exposed surfaces;
f) requirements for the testing and verification of compliance.

H.3.6 Additional requirements for deep fill areas/buildings and structures

Collapse compression upon groundwater inundation is a major hazard for buildings and other structures on significant thicknesses of fill; therefore the specification of placement and compaction of the fill should be designed to eliminate collapse potential.

NOTE The risk of collapse upon inundation is particularly high where fill is placed below the potential groundwater level (e.g. infilling of a quarry), but is present at many other sites due to risk of inundation following a water main burst.

BS 6031:2009 reminds the designer that the collapse potential of some fills will not be eliminated despite the achievement of a field dry density equivalent to at least 95 % of the maximum dry density achieved using the BS 1377 2,5kg compaction test. In such cases, dry density should not be relied upon to provide an adequate measurement for compaction specification. Where there is an unacceptable risk of collapse upon inundation the specification should include a requirement for all fill to be compacted to < 5 % air voids. See Charles et al. [7] and BRE Digest 427 [8].

H.3.7 Selection of fill material properties (earthworks fill design)

Research at TRL led to the development of the compaction requirements of Table 6/4 of the SHW [1], which are intended to achieve an appropriate degree of compaction for the relevant classes of fill provided that the water content of the material is appropriate. Further guidance on the degree of compaction achieved using the methods specified in Table 6/4 is provided in BS 6031 [3] and HA44/91 [5] which describes how to set the parameters:

[http://www.dft.gov.uk/ha/standards/dmrb/vol4/section1/ha4491.pdf].

An important activity for every earthworks project is the selection of material properties for the fill; this is considered to be a design activity regardless of whether it is undertaken by a Contractor or a consultant. The material properties should be chosen to ensure that the engineering design assumptions are satisfied as well as addressing construction practicalities.

The material properties for earthworks fill should be selected to ensure that:

- the material can be trafficked, placed and compacted during construction of the earthworks;
- the earthworks will be stable during and after construction;
- excessive settlement or heave will not take place.

For the majority of fill materials, the acceptable material properties should be related to limits applied to either water content, MCV or shear strength (e.g. see Table 6/1 extract at Table H.3 below). It is strongly recommended that only one of these properties is used for a particular acceptability limit.

For most coarse soils, the upper and lower acceptability limits should be selected by reference to a particular ratio of dry density to the maximum dry density. The values are determined from dry density/water content relationship tests, which are illustrated in general terms in BS 6031:2009, Figure 9. The most commonly adopted criteria are 95 % of the maximum dry density determined from the 2,5 kg light dynamic compaction test or 90 % of the maximum dry density determined from the vibrating hammer test for bulk earthworks fill. A higher value up to 100 % of the maximum dry density is required for fill that will support structures where settlement is more critical. It is recommended that the air voids content at the proposed lower acceptability limit is checked to ensure that excessive air voids will not remain within the fill at the chosen compaction ratio; however, an air void content less than 10 % may not be feasible with some uniformly graded coarse soils.

It is important to note that the maximum dry density and optimum water content are not fundamental soil properties and the values are dependent on the compactive effort imparted to the material.

For fine soils the upper acceptability limit (see BS 6031:2009 Figure 10) e.g. minimum MCV, should be chosen in relation to the requirements for placement of the fill, stability of slopes, and settlement of the fill due to internal loading (see BS 6031:2009, 7.6.3). These requirements may vary for different end uses of the earthworks, which will determine the fill properties of greatest importance, e.g. permeability for a flood bund, or *in situ* density for structural fill. The lower acceptability limit (minimum water content, maximum MCV or maximum shear strength) should be selected to reduce the air voids in the material to a value that will restrict the potential for excessive movement after compaction. A maximum of 10 % air voids for bulk earthworks fill and 5 % air voids for earthworks fill that is to support structures are commonly specified values. Research at TRL led to the development of the compaction requirements of Table 6/4 of the SHW [1], which are intended to achieve these values for the relevant classes of fill provided that the water content of the material is appropriate. Further guidance on the degree of compaction achieved using the methods specified in SHW Table 6/4 is provided in HA44/91.

The preferred method of specifying water limits on clays in the UK is the Moisture Condition Value (MCV), which is quick to measure on site and for which there is a substantial base of experience. The appropriate MCV values should be determined for the particular project based on site-specific laboratory relationship testing to relate MCV to fundamental properties. General guidance on appropriate ranges of MCV values based on experience across many sites is provided within BS 6031:2009 [3]. The MCV test is commonly used as the basis for quality control testing in the UK for fine soils (including sands), but it should not be used for stoney clays if there is insufficient matrix (typically less than 50 %–55 %) for the test and in such cases reliance on water content is necessary (Oliphant and Winter [6]).

Guidance on selection of classification and acceptability criteria and the appropriate frequency of testing is given in HA 44/91 [5] and BS 6031:2009 Clause 8 [3], see Table H2 below for a general summary. In addition to this guidance, the project specification and method of working on site should be developed to reflect local knowledge of materials and experience of particular equipment.

BS 6031:2009 *comments on some fill materials that require special* cons*ideration, for example:*

- In the case of Class 2C fill materials of well graded soil (e.g. Glacial Till), the designer is encouraged to consider the implication of the fills grading (the relative proportion of gravel sized particles and soil matrix < 425 µm) and how this may influence the fill behaviour at a particular water content;

- many of the standard tests included in BS 1377 cannot be usefully employed on some of the UK soils primarily due to their coarse nature (>10 % of material is retained on the 37,5 mm sieve) requiring a flexible approach to identifying suitable acceptance criteria.

Table H.2 — Classification and acceptability tests (reproduced from BS 6031:2009, Table 10)

Test	Applicable material type	Purpose	Reference
Water content	All	Classification/stabilization	BS 1377-2, BS 812-109
Atterberg limits	Cohesive	Classification	BS 1377-2
Particle size distribution	All	Acceptability/classification	BS 1377-2[a)]
MCV	Cohesive and/or some granular	Acceptability/trafficability	BS 1377-2, Clause 632 of SHW [1], TRRL LR 1034, TRRL LR 130, TRRL LR 90
Maximum density and optimum water content	Mainly granular	Acceptability/compatibility	BS 1377-2, BS 812-109
CBR	All except coarse granular	Trafficability/stabilization/ classification	BS 1377-2, BS 1924 (both parts)
Triaxial (quick)	Cohesive	Acceptability/trafficability	BS 1377-2
Chemical tests	All	Acceptability	BS 1377-2
Relationship testing[b)]	All	Acceptability	BS 1377-2

a) The requirements of BS 1377-2 may be added, to include all sieve sizes quoted in Table 6/2 of the SHW [1].

b) Testing soils at various water contents to study the change in soil properties.

When there are specific requirements to limit the internal settlement for large bodies of fill that will carry structures (as described in BS 6031:2009, 7.6.3) then the approach of selection of design parameters beyond that which would normally be considered under the SHW [1] approach may be developed. One methodology that may be used is proposed in BRE Digest 427 [8], whereby:

- the water content upper and lower acceptability limits of the fill are chosen based on OMC from both the standard Proctor (2,5 kg rammer) and the modified Proctor (4,5 kg rammer) compaction tests (i.e. relatively dry material for fine soils), see Nowak and Gilbert [36] for further details; and
- the method of compaction is selected to ensure heavy compaction is delivered (which is likely to be in excess of the SHW standard methods); and
- the earthworks are monitored to ensure a high *in situ* density and low air voids are achieved.

Further information on the approach required to select material properties for earthworks fill is provided in BS 6031:2009 Clause 7.6.4 (the first half of which is reproduced as the text given above).

Particular design requirements associated with treatment of fill with lime or cement binders is set out within HA74/07 [11].

H.3.8 Extracts from core tables of UK Specification for Highway Works (SHW)

The following extracts from the core tables of the SHW have been included to help illustrate the layout of the system that is used in the UK. These are only short extracts; the reader shall refer to the original document for the full version which can be obtained freely from Highways England website: *http://www.dft.gov.uk/ha/standards/mchw/*.

One page extracts are provided here of each of the following tables (value in brackets shows the length of the full document):

- Table 6/1: Acceptable Earthworks Materials: Classification and Compaction Requirements (original is 30 pages, extract included as Table H.3 below);
- Table 6/2: Grading Requirements for Acceptable Earthworks Materials (original is 2 pages, extract included as Table H.4 below); and
- Table 6/4: Method Compaction for Earthworks Materials: Plant and Methods (original is 4 pages, extract included as Table H.5 below).

It shall be recognized that these tables form part of a full Specification and shall be read with in conjunction with the 600 series clauses of the SHW. Guidance on the completion of the project earthworks specification appendices are provided in *Notes for Guidance to the Specification for Highway Works* [2].

The Designer is required to finalise Table 6/1 for the project Specification based on the design process described above. The designer shall assess the properties to be used to control the earthworks and is required to determine each of the Acceptable Limits that are marked “App 6/1” and update the table with project specific values. If the earthworks are to be controlled by water content, then this will normally be achieved relative to the optimum water content, in which case the required compaction test will need to be inserted into the project specific Table 6/1.

Table H.3 — extract of Table 6/1 (reproduced from SHW, November 2009)

Class				General Material Description	Typical use	Permitted Constituents (All Subjects to Requirements of Clause 601 and Appendix 6/1)	Property (See Exceptions in Previous Column)	Defined and Tested in Accordance with:	Acceptable Limits Within:		Compaction requirements Clause 612	Class		
									Lower	Upper				
GENERAL COHESIVE FILL	2	C	-	Stony cohesive material	General Fill	Any material, or combination of materials, other than chalk	(i) grading	BS 1377: part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 2	2	C	-
							(ii) plastic limit (PL)	BS 1377: part 2	-	-				
							(iii) mc	BS 1377: part 2	App 6/1	App 6/1				
							(iv) MCV	Clause 632	App 6/1	-				
							(v) undrained shear strength of remoulded material	Clause 633	App 6/1	-				
	2	D	-	Silty cohesive material	General Fill	Any material, or combination of materials, other than chalk	(i) grading	BS 1377: part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 3	2	D	-
							(ii) mc	BS 1377: part 2	App 6/1	App 6/1				
							(iii) MCV	Clause 632	App 6/1	App 6/1				
							(iv) undrained shear strength of material	Clause 633	App 6/1	App 6/1				
	2	E	-		General Fill		(i) mc	BS 1377: part 2	To enable compaction Clause 612		End product 95 % of maximum dry density of BS 1377: Part 4 (2.5 kg rammer method)	2	E	-
							(ii) bulk density	BS 1377: part 9	App 6/1	App 6/1				

Table H.4 — extract of Table 6/2 (reproduced from SHW, November 2009)

Percentage by Mass Passing the Size Shown																					
Class	Size (mm)		Size (mm) BS Series												Size (microns) BS Series				Size (micron)	Class
	500	300	125	90	75	37.5	28	20	14	10	6.3	5	3.35	2	1.18	600	300	150	63	2	
1A		100	95–100																< 15		1A
1B			100																< 15		1B
1C	100		10–95													0–25			< 15		1C
2A&2B			100											80–100					15–100		2A&2B
2C			100											15–80					15–80		2C
2D			100																80–100	0–20	2D
6A	100									0–100		0–85				0–45			0–5		6A
6B	100		0–10																		6B
6C			100								0–100		0–35	0–10		0–2					6C
6D										100		89–100		60–100	30–100	15–80	5–4	0–15 except 0–20 for crushed rock			6D
6E&6R			100	85–100						25–100						10–100			< 15		6E&6R
6F1					100	75–100				40–95		30–85				10–50			< 15		6F1
6F2			100	80–100	65–100	45–100				15–60		10–45				0–25			0–12		6F2
6F3			100	80–100	65–100	45–100				15–60		10–45				0–25			0–12		6F3
6H								100				60–100			15–45	0–25		0–5			6H
6I&6J			100		85–100				25–100					15–100		9–100			< 15		6I&6J
6K								100											0–10		6K
6L										100		89–100		60–100	30–100	15–100		0–65 except 0–20 for crushed rock			6L

Table H.5 — extract of Table 6/4 (reproduced from SHW, November 2009)

Type of Compaction Plant	Ref No.	Category	Method 1		Method 2		Method 3		Method 4		Method 5		Method 6		
			D	N#	D	N#	D	N#	D	N	D	N	N for D = 110 mm	N for D = 150 mm	N for D = 250 mm
Smoothed wheeled roller (or vibratory roller operating without vibration)		Mass per meter width of roll:													
	1	over 2100 kg up to 2700 kg	125	8	125	10	125	10*	175	4	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 2700 kg up to 5400 kg	125	6	125	8	125	8*	200	4	unsuitable		16	unsuitable	unsuitable
	3	over 5400 kg	150	4	150	8	unsuitable		300	4	unsuitable		8	16	unsuitable
Grid roller		Mass per meter width of roll:													
	1	over 2700 kg up to 5400 kg	150	10	unsuitable		150	10	250	4	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 5400 kg up to 8000 kg	150	8	125	12	unsuitable		325	4	unsuitable		20	unsuitable	unsuitable
	3	over 8000 kg	150	4	150	12	unsuitable		400	4	unsuitable		12	20	unsuitable
Deadweight tamping roller		Mass per meter width of roll:													
	1	over 4000 kg up to 6000 kg	225	4	150	12	250	4	350	4	unsuitable		12	20	unsuitable
	2	over 6000 kg	300	5	200	12	300	3	400	4	unsuitable		8	12	20
Pneumatic-tyred roller		Mass per wheel:													
	1	over 1000 kg up to 1500 kg	125	6	unsuitable		150	10*	240	4	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 1500 kg up to 2000 kg	150	5	unsuitable		unsuitable		300	4	unsuitable		unsuitable	unsuitable	unsuitable
	3	over 2000 kg up to 2500 kg	175	4	125	12	unsuitable		350	4	unsuitable		unsuitable	unsuitable	unsuitable
	4	over 2500 kg up to 4000 kg	225	4	125	10	unsuitable		400	4	unsuitable		unsuitable	unsuitable	unsuitable
	5	over 4000 kg up to 6000 kg	300	4	125	10	unsuitable		unsuitable		unsuitable		12	unsuitable	unsuitable
	6	over 6000 kg up to 8000 kg	350	4	150	8	unsuitable		unsuitable		unsuitable		12	unsuitable	unsuitable
	7	over 8000 kg up to 12000 kg	400	4	150	8	unsuitable		unsuitable		unsuitable		10	16	unsuitable
	8	over 12000 kg	450	4	175	6	unsuitable		unsuitable		unsuitable		8	12	unsuitable
Vibratory tamping roller		Mass per metre width of a vibrating roll:													
	1	over 700 kg up to 1300 kg	100	12	100	12	150	12	100	10	unsuitable		unsuitable	unsuitable	unsuitable
	2	over 1300 kg up to 1900 kg	125	12	125	12	175	12*	175	8	unsuitable		12	unsuitable	unsuitable
	3	over 1900 kg up to 2300 kg	150	12	150	12	200	12*	unsuitable		unsuitable		8	12	unsuitable
	4	over 2300 kg up to 2900 kg	150	9	150	9	250	12*	unsuitable		400	5	6	10	unsuitable
	5	over 2900 kg up to 3600 kg	200	9	200	9	275	12*	unsuitable		500	6	6	10	unsuitable
	6	over 3600 kg up to 4300 kg	225	9	225	9	300	12*	unsuitable		600	6	4	8	unsuitable
	7	over 4300 kg up to 5000 kg	250	9	250	9	300	9*	unsuitable		700	6	3	7	12
	8	over 5000 kg	275	9	275	9	300	7*	unsuitable		800	6	3	6	10

The Method number shall be as stated for the class of fill in Table 6/1, "D" is maximum depth of the compacted layer and "N" is the minimum number of passes (see SHW cl. 612 for full details and plant operation requirements).

"H.4 Control of earthworks during construction"

BS 6031:2009, Clause 9 sets out general requirements for management of earthworks construction, an important part of which is the Quality Assurance/Quality Control (QA/QC) testing to be undertaken during construction which shall be in accordance with the project earthworks specification (see H.3 above). This earthworks control testing will include testing for both fill classification and construction control.

The Designer should stipulate the testing requirements for earthworks materials during construction as part of the QA/QC process, including any source approval testing and routine fill control testing during the works. Under the SHW this information is presented within Appendix 1/5 which may be in the form presented in Table NG1/1 of SHW [1]. The designer should include either the frequency or number of tests dependent on the size or duration of the works being undertaken.

All earthworks materials will be controlled by testing to check the acceptability of the fill material prior to placement in the works, to confirm compliance with the project specification. The main difference in quality control approach between the two main forms of compaction control are as follows:

- Earthworks managed by method specification focus on checking that the number of passes, and resulting compacted layer thickness, satisfy the compaction method for the type of compaction plant that is defined at SHW Table 6/4. In addition, a limited number of *in situ* density tests are undertaken to check that the standard compaction method is appropriate for the fill and site conditions.
- Fill materials managed by end product specification are simply tested for *in situ* dry density and/or stiffness of the compacted fill, with tests undertaken at regular intervals through the earthwork to check that acceptable results are consistently achieved.

The use of full scale site compaction trials at the start of the construction works is encouraged to confirm that the intended methods of working achieve the expected results.

Detailed records shall be maintained of all the QA/QC testing undertaken, along with details of the location of tests within the works and relevant site observations. In the UK, the approach of summarizing the construction records within a Geotechnical Feedback Report is encouraged to capture “lessons learned” in order to improve future methods of working.

"H.5 References"

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EN 16907-7, *Earthworks - Part 7: Hydraulic placement of waste1*

APPENDIX I.2.4

Reference Guide GTR (France)

Extract / Rock classification and parameters

Classification of rock materials according to their condition and mechanical characteristics

Knowledge of the petrographic nature of the rock from which a rock material is derived alone is generally not sufficient to foresee all the problems that its use as fill or as a capping layer.

Apart from the question of the choice of extraction method, which is not dealt with here, the aspects to be considered are:

- the ability of the material to fragment under the stresses applied during the various phases of implementation and in particular the possibility of producing a proportion of fine elements sufficient to have a water-sensitive soil behaviour,
- the potential for post-implementation evolution under the action of mechanical constraints alone or combined with those of water and frost,
- the water content in the case of highly fragmentable materials such as certain chalk, marl, sedimentary shales, etc., which may contain in their structure a large amount of water that will inevitably be communicated to the fine elements produced during the excavation,
- the content of soluble elements in the case of saline rocks.

It is therefore necessary to characterize the rock materials with respect to these aspects using different parameters, of which the following are considered the most representative.

Parameters of state and mechanical behaviour retained in the classification of rock materials

- ❖ The Los Angeles coefficient (LA) (Standard P 18-573).
- ❖ The micro-Deval coefficient in the presence of water (MDE) (standard P 18-572).

  These two parameters are introduced for relatively hard rocks: granite, gneiss, limestone and hard sandstone. Their interpretation is essentially aimed at the possibilities of using these materials in capping layer, or even in a pavement layer. (Standard P 18-101).
- ❖ The density of the dehydrated rock in place (ρd) (standard P 94-064).

  This parameter, which has the advantage of being easily measurable, is closely correlated with the fragmentability of materials such as chalk and soft limestone. Its interpretation focuses on the potential use of these materials as fill.
- ❖ The fragmentability coefficient (FR) (standard P 94-066)

  This coefficient is determined from a fragmentation test. It is expressed as the ratio of the D10 of a sample of a given initial granularity, measured before and after it was subjected to conventional crushing with the normal Proctor compaction.

The interpretation of this parameter concerns the possibilities of use as fill for evolving rock materials and as a capping layer for certain more or less friable rock materials for which the LA, MDE coefficients lack sensitivity...

- The coefficient of degradability (DG) (standard P 94-067)

  This coefficient is expressed as the ratio of the D10 of a sample of a given initial granularity, measured before and after subjecting it to conventional drying-immersion cycles. Its interpretation is mainly aimed at the possibilities of using materials derived from clayey rocks (marls, sedimentary shales, etc.) as fill.

- The natural water content (wn) (standard NF P 94-050)

  The influence of this parameter is only taken into account in the classification for certain very fragmentable chalk and clayey rocks.

- The content of soluble elements (% NaCl, gypsum...)

  The interpretation of this parameter is obviously limited to the case of saline rocks.

Threshold values selected for the state and behaviour parameters of rock materials:

<u>Sedimentary rocks</u>

<u>Magmatic and metamorphic rocks</u>

FR ≤ 7 Poorly fragmentable rock

FR > 7 Fragmentable rock

DG ≤ 5 Poorly degradable rock

5 < DG ≤ 20 Medium degradable rock

DG > 20 Very degradable rock

LA and MDE ≤ 45 Hard rock

LA or MDE > 45 and FR ≤ 7 Rocks of medium hardness

Soluble salt content / Poorly soluble saline rocks

- ≤ 5 to 10% in the case of salt rock
- ≤ 30 to 50% in the case of gypsum

Soluble salt content / Highly soluble saline rocks

- 5 to 10% in the case of salt rock
- >30 to 50% in the case of gypsum

Carbonate rocks

$\rho d > 1,7$ dense chalk

$1,5 < \rho d \leq 1,7$ medium density chalk

$\rho d \leq 1,5$ lime chalk

MDE ≤ 45 hard limestone

MDE > 45 et $\rho d > 1,8$ medium density limestone

$\rho d \leq 1,8$ fragmentable limestone

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