(15)基準類リストHPへのアクセス方法

(15) 基準類リスト HP へのアクセス方法

SETRA の基準リストへのアクセス方法

仏国の交通・道路技術事務所(SETRA)で発行している各基準、技術ガイド類については SETRA のホームページでタイトル及び概要が確認可能な他、購入手続きも可能である。(HP は仏語/英語で閲覧可)

1. 基準・技術ガイド類のリスト検索方法：

道路関連の各基準、技術ガイド類については、仏国の道路に関する技術基準サイト(DTRF：La documentation des techniques routières françaises)で閲覧が可能である。

SETRA 発行図書のリスト：

SETRA 発行の技術ガイド、マニュアル等は下記のパンフレット(PDF で閲覧可)で確認することも可能である。

(http://www.setra.equipement.gouv.fr/IMG/pdf/0902w_repertoireOA.pdf)

SETRA の発行図書の中には、英語訳が施されていて全編がホームページからアクセス可能な図書(技術ガイド)もある。

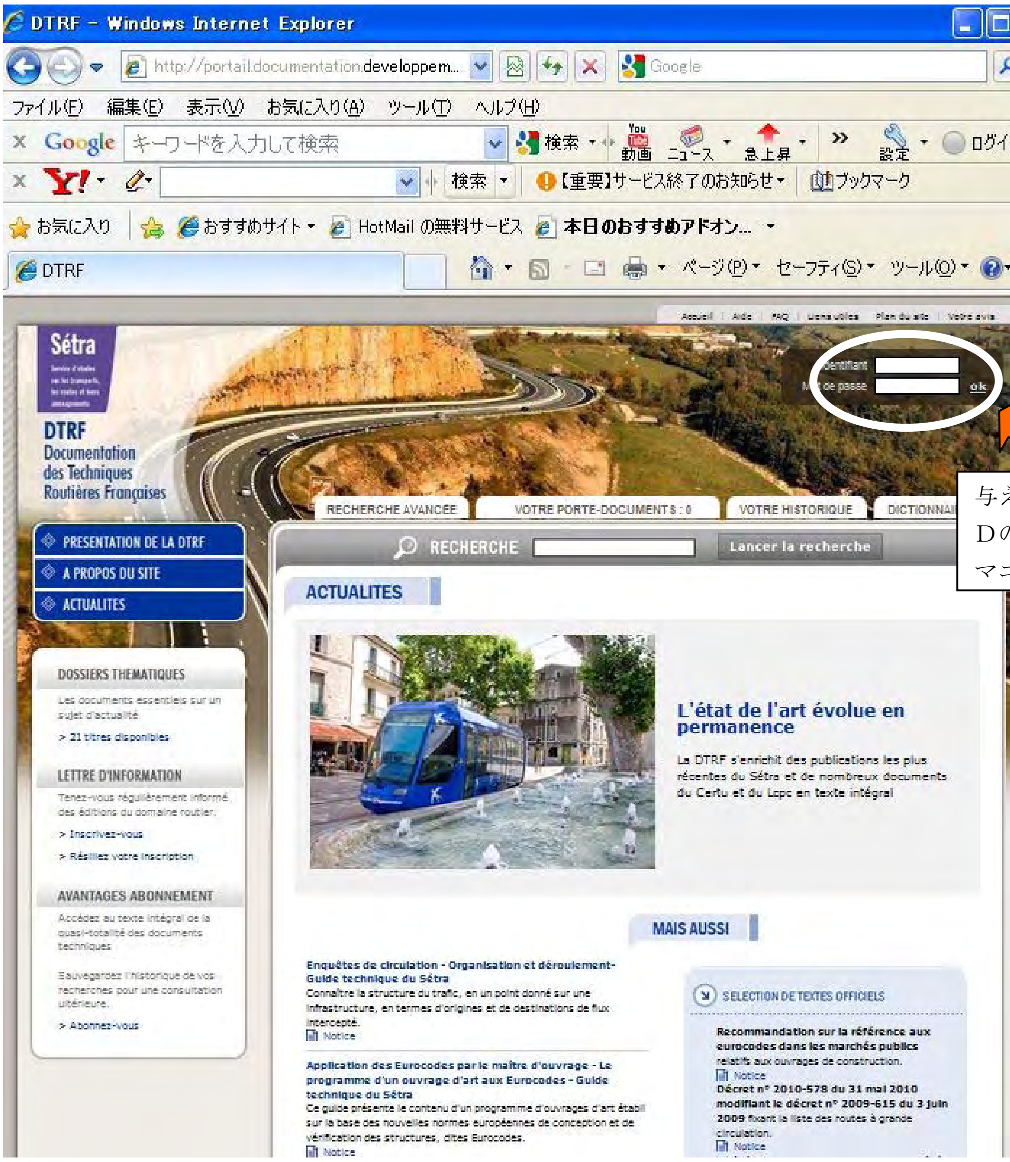
(http://www.setra.equipement.gouv.fr/Technical-guides.html)

2. 基準・技術ガイド類の年間購読契約：

SETRA では、SETRA が発行している技術ガイド、マニュアル等(通常は有料で購入の必要あり)の年間購読契約サービスがあり、DTRF のサイトからアクセスが可能である。年間購読はホームページからの申込みも可能で、1,000 ユーロの年間購読料を支払うことで 20 のライセンス(アクセス権)が与えられる。

SETRA の基準閲覧サイト(年間購読契約の場合)：

(http://portail.documentation.developpement-durable.gouv.fr/dtrf/)



与えられるパスワード及び I D の入力により、技術ガイド、マニュアル類の閲覧可

LCPC の基準リストへのアクセス方法

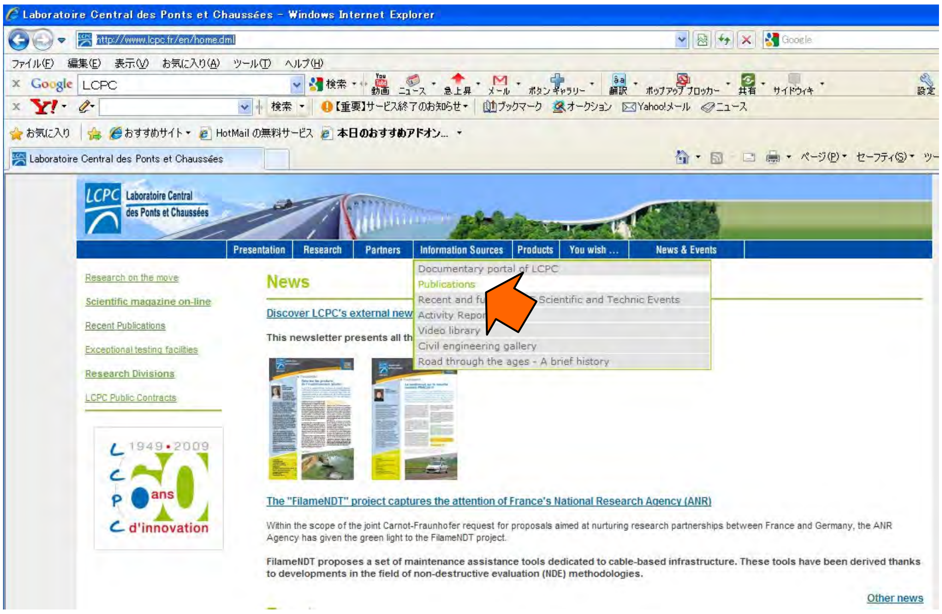
仏国の中央土木研究所(LCPC)で発行している各基準、技術ガイド類については LCPC のホームページでタイトル及び概要が確認可な他、購入手続きも可能である。(HP は仏語/英語で閲覧可)

1. 基準・技術ガイド類のリスト検索方法：

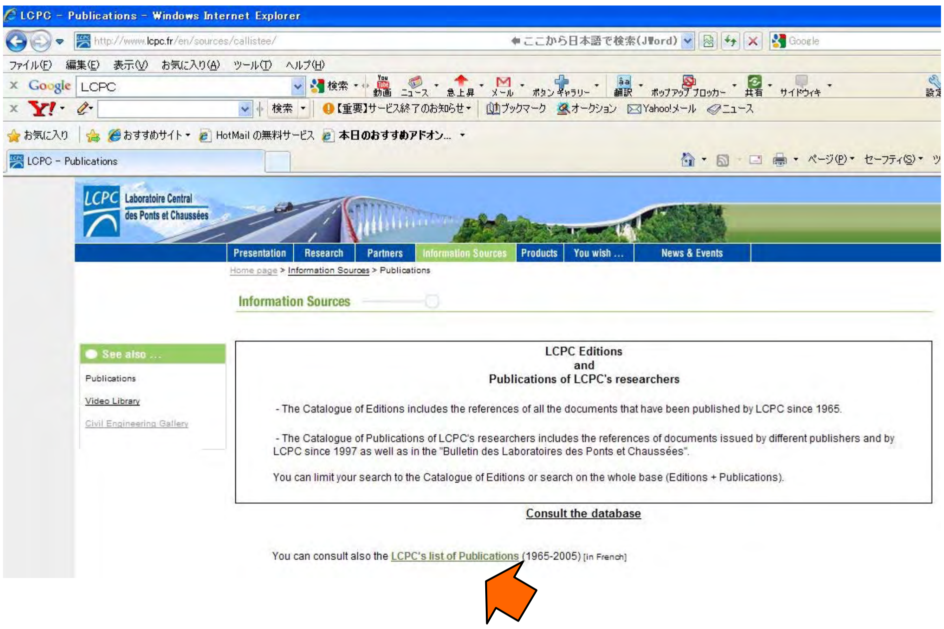
LCPC HP (http://www.lcpc.fr/en/home.dml)

└ “Information Sources” から “Publications” を選択

(http://www.lcpc.fr/en/sources/airs/index.dml)

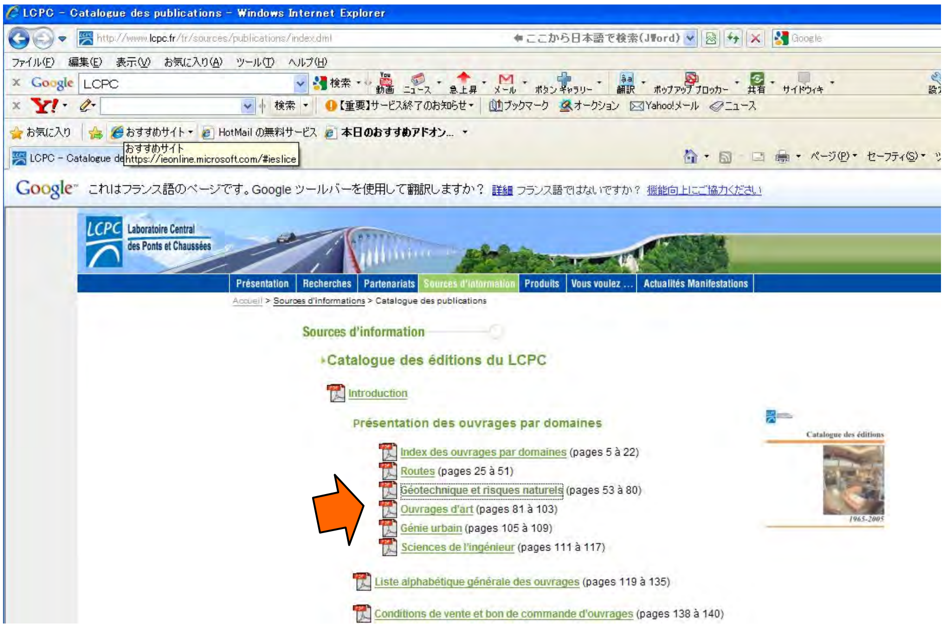


“Publications” 画面の下部の “LCPC’s list of Publications” をクリック



Catalogue des éditions du LCPC の画面

(http://www.lcpc.fr/fr/sources/publications/index.dml)



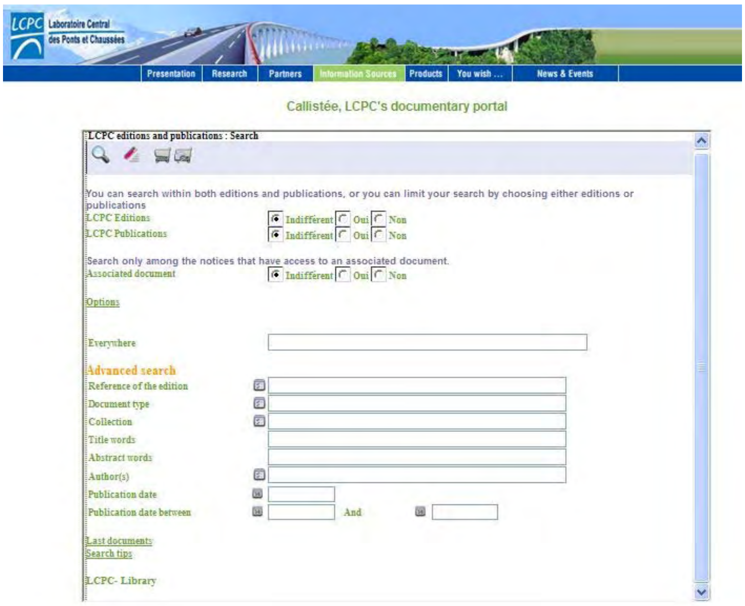
上記サイトでは、土木一般、都市土木、構造物、道路、地盤工学と自然災害などの分野に関連する個々の基準のタイトルと基準内容、購入の場合の価格情報等が示されている。

本報告書では、資料編(13)「土工及び舗装関連技術ガイドリスト」にリストを掲載している。

2. 基準・技術ガイド類の購入申込み

下記サイトにて、ガイド類の購入注文が可能である。

(http://www.lcpc.fr/en/sources/callistee/iframe.php)



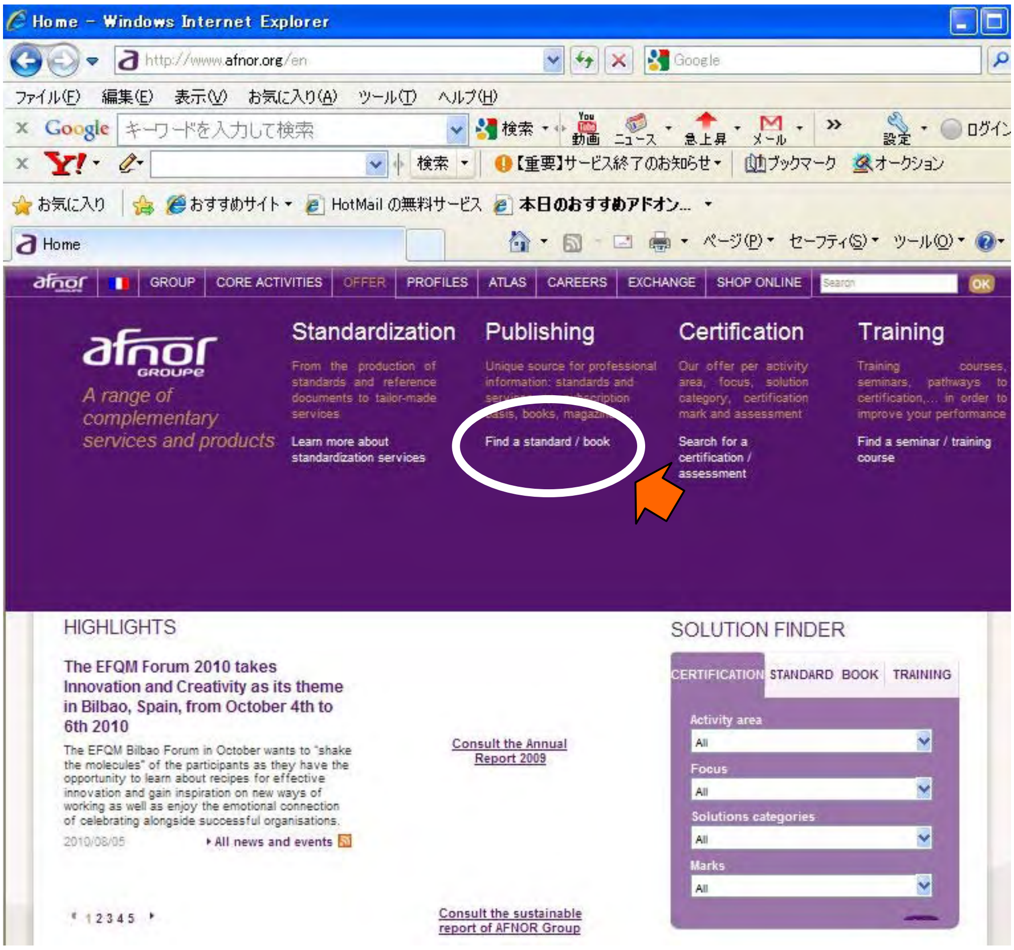
AFNOR の基準・規格リストへのアクセス方法

仏国の規格協会(AFNOR)で発行している各基準、規格類についても AFNOR のホームページでタイトル及び概要が確認可な他、購入手続きも可能である。(HP は仏語/英語で閲覧可)

1. 基準・規格類へのアクセス方法

AFNOR HP (http://www.afnor.org/en)

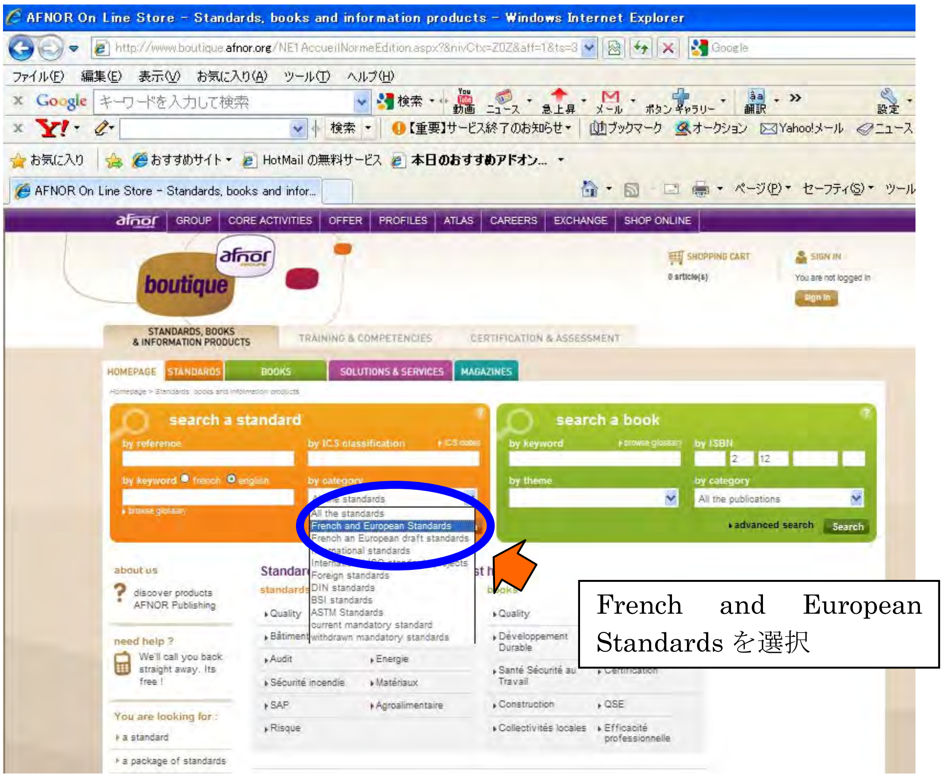
└ Pulldown メニューの“Offer”から“Publishing の Find a standard / book”を選択



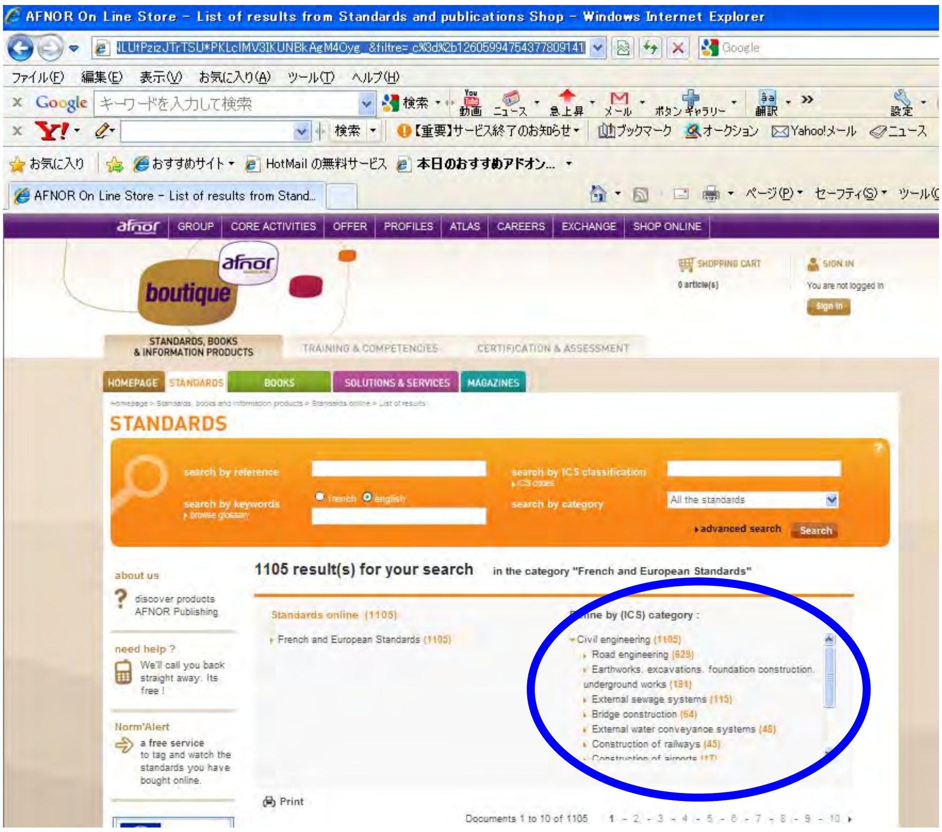
次頁に示す基準及び図書の検索画面が現れる。

(http://www.boutique.afnor.org/NE1AccueilNormeEdition.aspx?&nivCtx=Z0Z&aff=1&ts=3217344)

このサイトでは、仏国基準のみならず、国際基準のスタンダード、AASHTO や BS スタンダードの閲覧も可能となっている。



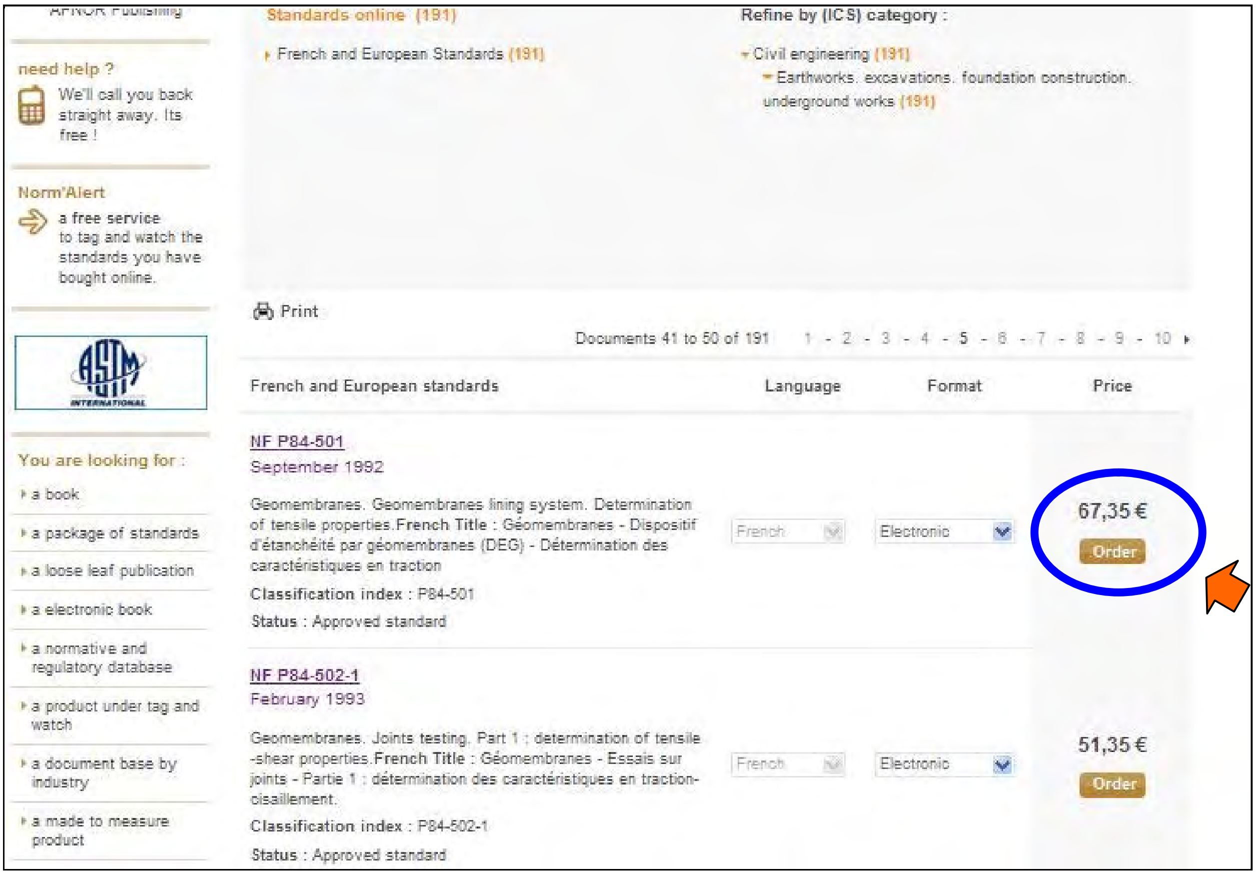
“Search a standard”の Pulldown メニューから“French and European Standards”を選択する。又は、ICS コードや基準(規格)No.を直接入力することでもアクセスは可能。



“Refine by (ICS) category”メニューから“Civil engineering”を選択すると、道路施工関連の基準、規格のサイトへアクセスが可能である。

2. 基準・技術ガイド類の購入申込み

各基準(規格)類は、下記の個々の基準類が示されているサイトで購入可能である。



(16)アフリカ各国の適用基準と舗装厚計算例

(16)アフリカ各国の適用基準と舗装厚計算例

1.アフリカ各国の適用基準及び耐用年数

2005 年以降の無償及び有償案件の報告書で調査した各国(各プロジェクト)の軸重制限、舗装耐用年数と使用された基準を示す。

- 軸重制限は、東アフリカ諸国では 10 t が多い。
- COMESDA 基準における軸重制限は 8t であるが、エチオピア、ケニアは国際回廊を考慮して 10t を採用している。
- チュニジア、セネガルはフランスと同じ 13t を採用している。

表 アフリカ各国(各プロジェクト)の軸重制限・舗装耐用年数・使用された基準

国名	件名	軸重制限	耐用年数	軸重制限基準	適用舗装設計基準	備考
マリ-セネガル	マリ-セネガル		10年		AASHTO舗装設計指針 (フランス基準と同等として使用)	
チュニジア	-	軸重規制 :11.5t 設計軸重 :13t 総重量 :51t	-		-	
ブルンジ			10年		AASHTO舗装設計指針	
マラウイ	ブランタイヤ市道路網	軸重規制 :10t 隣接軸重 :16t 総重量 :56t	15-20年	SADC基準/COMESA基準	舗装設計施工指針・舗装設計便覧(日本道路協会)	
タンザニア	ニューバガモヨ	軸重規制 :10t 隣接軸重 :18t 総重量 :56t	15年	SADC基準	Pavement and Materials Design Manual 1999 (タンザニア基準) AASHTO舗装設計指針で検証	
	キルワ		15年		SATCC 1998版	
	マサシ-マンガッカ				Pavement and Materials Design Manual1999 (タンザニア基準)	
マダカスカル					GUIDE PRATIQUE DE DIMENSIONNEMENT DES CHASUSSEES POUR LES PAYS TROPICAUX(1984 年版)	
ガーナ		軸重規制 :11.5t 設計軸重 :13t 総重量 :51t	15年	ECOWAS基準	AASHTO舗装設計指針	
エチオピア		軸重 :10t	20年	COMESA基準 エチオピア基準	Pavement Design Manual (Draft) 2001(エチオピア基準) (英国TRL参照:日本の「舗装設計要領」と同様、SN法)	AASHTO、BS参照
ウガンダ	-		-		-	
モザンビーク	ナンプラ-クアンバ間道路				SATCCを適用した舗装とナンプラ-ナカラ間の簡易舗装を比較	簡易舗装を採用
ザンビア		軸重規制 :10t 隣接軸重 :18t 総重量 :56t	10年	SADC基準	AASHTO舗装設計指針 / SATTC1998年版	
コンゴ		総重量 :46t	15年		OFFICE DES ROUTES, PROJET DES NORMES ROUTIER AASHTO舗装設計指針で検証	
ケニア		軸重規制 :10t 隣接軸重 :16t 総重量 :56t	-	COMESA基準	基準改定中	
SADC基準	-	軸重規制 :10t 隣接軸重 :18t 総重量 :56t	-	-	-	-
COMESA基準	-	軸重規制 : 8t 隣接軸重 :16t 総重量 :53t	-	-	-	-

2.「GUIDE PRATIQUE DE DIMENSIONNEMENT DES CHASUSSEES POUR LES PAYS TROPICAUX」及び「SATTC」による舗装厚計算例

サブサハラにおいて参照されている「GUIDE PRATIQUE DE DIMENSIONNEMENT DES CHASUSSEES POUR LES PAYS TROPICAUX」と南部アフリカで使用されている SATTC にて下記の設計条件のもとで舗装構成の計算例を行った結果を示す。また、SATTC は AASHTO を参照して作成されているが、舗装構成と舗装厚の選択がチャート式になっているため、同様の設計条件での AASHTO での設計計算例も参考までに掲載する。

－　設計条件

設計交通量　　：　11,000 台/日/方向

設計軸重量　　：　12.3 百万　標準軸重累積通過数(ESAL)

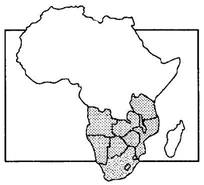
路床支持力　　：　CBR９以上

耐用年数　　：　15 年

表　舗装構成計算例

GUIDE PRATIQUE DE DIMENSIONNEMENT DES CHASUSSEES POUR LES PAYS TROPICAUX	SATTC	AASHTO(参考)
Classes de traffic : T5 Classes de portance des sols : S2 (表・基層) Beton bitumineux：10cm (上層路盤) Grave ciment ：22cm (下層路盤) Grave ameliores au ciment：25cm	Traffic classes : T7 Subgrade classification : S4 Dry Regions (表・基層) アスファルトコンクリート：5cm (上層路盤) 粒度調整砕石 : 15cm (下層路盤) 現地発生材＋セメント安定処理：20+10cm	【Layer coefficient】 surface =0.44 base =0.14 sub base =0.13 【Drainage coefficient】 base/sub base =0.11 【Reliability】 surface/base/sub base =-1.282 【Combined standard error of traffic prediction and performance prediction】 surface/base/sub base =0.4 【Performance Serviceability Index】 (Initial) surface/base/sub base =4.2 (terminal) surface/base/sub base =2.0 【Resilient modulus for surface】 surface =350000 base = 31000 sub base = 18600 Sub grande=13500 (表・基層) アスファルトコンクリート：7cm (上層路盤) 粒度調整砕石：20cm (下層路盤) 現地発生材＋セメント安定処理：26cm

SATTC の舗装基準については次頁以降に添付する。



SATCC

Draft

Code of Practice
for the
Design of Road Pavements

September 1998
(Reprinted July 2001)

Prepared by the Division of Roads and Transport Technology, CSIR

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

1.1 Background

This pavement design guide for new trunk roads is intended to provide a simple and easily applied method for determining an appropriate pavement structure for the expected design criteria. It is based on the use of a comprehensive design catalogue which enables the pavement designer to rapidly select possible structural configurations that should meet the design criteria. Suggested designs have been checked against current mechanistic analysis methods for suitability.

It must be noted at the outset that certain limitations apply to the use of this guide and the principal ones are given as follows:

- This guide is not for either concrete or gravel roads.
- This guide does not cover special considerations for urban pavements.
- This guide is not for design trafficking of more than 30 million equivalent standard axles.
- This guide does not specifically cover existing subgrade conditions for which the nominal California Bearing Ratio (CBR) is less than 2 per cent.

Guidance is, however, given on some practical aspects of pavement design and construction which may assist in addressing these other conditions.

The catalogue structures have been developed from current practice deemed appropriate to the region, primarily as exemplified by the Transport Research Laboratory's Overseas Road Note 31 (RN31)[1] and the South African pavement design guide TRH4[2]. These documents in particular were identified from a broader review, including the previous SATCC guidelines and two Australian approaches, as being most apt for the purpose.

In order to keep this guide readily usable, much of the information has been pared to essentials. Whilst efforts have been made to ensure that all practical design considerations are addressed, the user is actively encouraged to become familiar with other more comprehensive pavement design documents which may provide additional insight into the process.

Similarly the user should regard the catalogue structures here as sound suggestions for the assumed conditions, but should make use of other design methods as both a check and a means of possibly refining the structure to suit specific conditions. In the same way, the Engineer should draw on local knowledge of materials and techniques which have proven to be satisfactory and substitute these where deemed appropriate for the more generic material classifications used in this guide. The nominal materials details are given in Appendix A, which provides general guidance on their usage.

1.2 Pavement Structure and Cross-section

Figure 1.1 shows both a typical pavement cross-section and the nominal pavement structure, in order to define some of the terminology used. The specific geometry of the pavement section is defined separately by applying either SATCC or other acceptable regional standards.

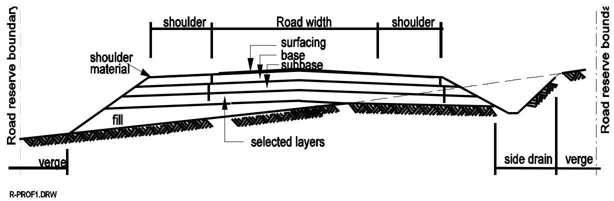


Figure 1.1: Geometry and nomenclature for the pavement

This guide focuses only on deriving the most appropriate layer configuration to form the pavement structure, but the following inherent conditions must be met if the pavement is to function properly:

- The road has an impervious surfacing.
- The road has at least 1m width shoulders.
- The road has a minimum road camber or crossfall of 2 per cent.

Appendix B gives more details on road drainage and shoulders.

1.3 Design Process

The design process in this guide is defined in five steps. These are:

(i) Estimating the cumulative traffic loading expected during the design life.
(ii) Defining the strength of the subgrade (soil) over which the road will be built.
(iii) Defining the nominal operating climate (wet or dry).
(iv) Determining any practical aspects which will influence the design selection.
(v) Selecting possible pavement structures.

These steps are given in the five sections following.

2. ESTIMATING DESIGN TRAFFIC LOADING

2.1 General

The design life is the period during which the road is expected to carry traffic at a satisfactory level of service, without requiring major rehabilitation or repair work. It is implicit, however, that certain maintenance work will be carried out throughout this period in order to meet the expected design life. This maintenance work is primarily to keep the pavement in a satisfactory serviceable condition, and would include routine maintenance tasks and periodic resealing as necessary.

Absence of this type of maintenance would almost certainly lead to premature failure (earlier than the design life) and significant loss of the initial investment.

A maximum design life of 20 years is recommended for these pavements, at which stage the road would be expected to need strengthening but would still have a good residual strength (and value). Conversely, a minimum design life of 10 years is recommended as a practical limit for economic justification in most cases.

The selection of design life will depend on a number of factors and uncertainties, and must be specified by the designer based on all available information, but most times should be either 15 or 20 years. Table 2.1 provides some guidance on selection.

Table 2.1: Pavement design life selection guidance

Design data reliability	Importance/level of service	
	Low	High
Low	10 - 15 years	15 years
High	10 - 20 years	15 - 20 years

It is important to note that there may be little difference in pavement structure for two distinctly different design periods, and it is always worthwhile to check the estimated design traffic for different periods.

2.2 Design Traffic Loading

The pavement design process requires the estimation of the average daily number of ESAs on one lane at the opening of the new road to traffic, which is then projected and cumulated over the design period to give the design traffic loading. There are principally two ways to do this as outlined below in 2.3(a) and (b) respectively. The most important thing is that the data used be checked carefully for reasonableness and accuracy. Where deemed necessary, additional traffic and axle survey data must be obtained. More complete details regarding the processes and factors involved can be obtained from RN31[1] or TRH4[2].

a) Traffic Count Data and Static Vehicle Axle Load Survey Data

(i) *Determine the baseline Annual Average Daily Traffic (AADT).*
This is defined as the total annual traffic summed for **both** directions and divided by 365, applicable at time of opening of the new road. This should be derived from available traffic count data, and should take cognisance of the possibility for diverted traffic (existing traffic that changes from another route) and generated traffic (additional traffic generated from the development).

(ii) *Estimate the numbers of vehicles in different categories that comprise the baseline AADT.*
Normal categories are cars/small pick-ups; light goods vehicles (including 4-wheel drives, panel vans and minibus/combis); trucks (which normally includes several sub-classifications to differentiate rigid and articulated vehicles, trucks with trailers, and various multi-axle configurations typical to the area), and buses. These classifications will already exist in a particular region and should form the basis for this estimate.

(iii) *Forecast the* ***one-directional*** *cumulative traffic flow for each category expected over the design life.*
This means taking half the value in step (ii) and projecting it at a selected growth rate, and cumulating the total over the design period. Growth rates will normally be in the range of 2 to 15 per cent per annum, and selected values should be based on all available indicators including historical data, and socio-economic trends.

The following formula, using the average daily traffic flow for the first year (not the value at opening to traffic, but the projected average for the year), gives the cumulative totals:

$$DT = T * 365 * \frac{[1 + r/100]^p - 1}{r/100} \qquad \text{......Equation 1}$$

where
DT is the cumulative design traffic in a vehicle category, for one direction, and
T = average daily traffic in a vehicle category in the first year (one direction)
r = average assumed growth rate, per cent per annum
p = design period in years

(iv) *Use static axle load data to determine average vehicle damage factors (ESAs per vehicle class).*
These are determined from converting the surveyed axle loads to ESAs/vehicle classification, and then deriving a representative average value. **In some cases, there will be distinct differences in each direction and separate vehicle damage factors for each direction should be derived.**

No average vehicle damage factors for different vehicle classes are given in this document, as vehicle classifications, usage, degrees of overloading and legal limits are likely to differ throughout the region. These will all influence the average factors, and it is considered injudicious to propose values in this document which are likely to be inappropriate.

The following formula is used for converting real axle loads to ESAs:

$F = [P/8160]^n$ (for loads in kg) or $F = [P/80]^n$ (for loads in kN)
......Equation 2

where
F is the load equivalency factor in ESAs, and
P = axle load (in kg or kN)
n = relative damage exponent

For vehicles using multi-axle configurations (such as tandems and tridems), some agencies introduce further factors to derive modified load equivalencies on the basis that these axle groupings may be less damaging than the sum of the individual axles as derived above. Within the bounds of current knowledge and data reliability, and to keep the calculation straightforward, it is recommended that no such additional processing be adopted at this stage.

The value of 4 for the exponent n is often used, in line with early findings and the commonly cited "fourth-power damage effects" of heavy axle loads. It is now clear that the value is influenced by various factors, with the most significant being the pavement configuration.

Table 2.2 indicates recommended n values to be used for the pavements in this guide, and Table 2.3 gives load equivalency factors for different axle loads and n values derived from Equation 2. The pavement base/subbase combination of cemented/granular is not used in this guide, nor recommended, based on many examples of poor performance deriving from premature cracking and deterioration of the cemented base.

Table 2.2: Recommended relative damage exponents, n[2]

Pavement base/subbase	Recommended n
Granular/granular	4
Granular/cemented	3
Cemented/cemented	4.5
Bituminous/granular	4
Bituminous/cemented	4

Table 2.3: Load equivalency factors for different axle load groups, in ESAs

Axle loads measured in kg				Axle loads measured in kN			
Axle load range (kg)	**n = 3**	**n = 4**	**n = 4.5**	**Axle load range (kN)**	**n = 3**	**n = 4**	**n = 4.5**
Less than 1500	-	-	-	Less than 15	-	-	-
1500 - 2499	.02	-	-	15 - 24	.02	-	-
2500 - 3499	.05	.02	.01	25 - 34	.05	.02	.01
3500 - 4499	.12	.06	.05	35 - 44	.13	.06	.05
4500 - 5499	.24	.15	.12	45 - 54	.24	.15	.12
5500 - 6499	.41	.30	.26	55 - 64	.42	.32	.28
6500 - 7499	.64	.56	.52	65 - 74	.66	.58	.55
7500 - 8499	.95	.95	.94	75 - 84	.99	.99	1.00
8500 - 9499	1.35	1.51	1.59	85 - 94	1.41	1.59	1.69
9500 - 10499	1.85	2.29	2.55	95 - 104	1.94	2.42	2.71
10500 - 11499	2.46	3.34	3.90	105 - 114	2.58	3.55	4.16
11500 - 12499	3.20	4.72	5.75	115 - 124	3.35	5.02	6.15
12500 - 13499	4.06	6.50	8.22	125 - 134	4.26	6.92	8.82
13500 - 14499	5.07	8.73	11.46	135 - 144	5.32	9.3	12.31
14500 - 15499	6.23	11.49	15.61	145 - 154	6.54	12.26	16.79
15500 - 16499	7.56	14.87	20.85	155 - 164	7.94	15.88	22.45
16500 - 17499	9.06	18.93	27.37	165 - 174	9.53	20.24	29.50
17500 - 18499	10.76	23.78	35.37	175 - 184	11.32	25.44	38.15
18500 - 19499	12.65	29.51	45.09	185 - 194	13.31	31.59	48.67
19500 - 20499	14.75	36.22	56.77	195 - 204	15.53	38.79	61.32

(v) *Convert one-directional cumulative traffic flows to the cumulative total ESAs in each direction*
The total ESAs in each direction are the sum of the ESAs from each vehicle category, derived from step (iii) above, using the factors from step (iv).

The actual design traffic loading (ESAs) is then calculated from the above, using the design carriageway widths and type of road to finalise the probable design needs. Table 2.4 gives the basis for design traffic loading using the nominal totals for each direction as determined above.

Table 2.4: Factors for design traffic loading

Road type	Design traffic loading	Comment
Single carriageway		
Paved road width 4.5 m or less	Up to twice the sum of the ESAs in each direction*	At least the total traffic must be designed for as there will be significant overlap in each direction. For widths of 3.5m or less, double the total should be used due to channelisation
Paved road width 4.5 m to 6.0 m	80% of the sum of the ESAs in each direction	To allow for considerable overlap in the central section of the road
Paved road width more than 6.0 m	Total ESAs in the most heavily trafficked direction	No overlap effectively, vehicles remaining in lanes
Dual carriageway		
Less than 2,000 commercial vehicles per day in one direction	90% of the total ESAs in the direction	The majority of heavy vehicles will travel in one lane effectively
More than 2,000 commercial vehicles per day in one direction	80% of the total ESAs in the direction	The majority of heavy vehicles will still travel in one lane effectively, but greater congestion leads to more lane switching
* Judicious to use double the total ESAs expected, as normally these are low trafficked roads and this may give little difference in pavement structure.		

For dual carriageways it is not recommended to adopt different designs for the different lanes for the main reason that, apart from practical issues, there are likely to be occasions when traffic is required to switch to the fast lane or other carriageway due to remedial needs. This could then lead to accelerated deterioration of the fast lanes and any initial cost savings could be heavily outweighed by future expenditure and loss of serviceability.

b) Weigh-in-motion (WIM) Axle Load Survey Data

(i) *Determine the baseline average daily traffic loading, ESAs, in each direction.*
Reliable WIM survey data provides a direct measurement of axle loads, and these can be converted to ESAs directly as outlined above. In this case, it is not necessary to know the vehicle details.

(ii) *Convert the baseline ESAs to cumulative ESAs in each direction during the design life.*
Equation 1 can be used, in which the average daily ESAs expected during the first year from step (i) are used for term T. The result, DT, is then the total cumulative ESAs for the particular direction.

The design traffic loading is then derived from Table 2.4 in the same manner as before.

2.3 Design Traffic Class

The pavement structures suggested in this guide are classified in various traffic categories by cumulative ESAs expected. Table 2.5 gives these classifications, and the design traffic determined from Section 2.2 is used to decide which category is applicable.

Table 2.5: Traffic classes[1]

	Traffic class designation							
Traffic ranges (million ESAs)	T1	T2	T3	T4	T5	T6	T7	T8
	< 0.3	0.3 - 0.7	0.7 - 1.5	1.5 - 3	3 - 6	6 - 10	10 - 17	17 - 30

If calculated design values are very close to the boundaries of a traffic class, the values used in the forecasts should be reviewed and sensitivity analyses carried out to determine which category is most appropriate.

The lowest traffic class T1, for design traffic of less than 0.3 million ESAs, is regarded as a practical minimum since realistic layer thicknesses as well as materials specifications tend to preclude lighter structures for lesser traffic. The current level of knowledge on pavement behaviour, in any case, limits the scope for rational design of such lighter structures.

However, in the unlikely case that design traffic is estimated at less than 0.1 million ESAs (that is, traffic significantly less than the lowest class T1), since this guide is aimed primarily at the Regional Trunk Road Network, the Engineer is recommended to also consider alternative designs proven locally for this very light trafficking.

3. DETERMINING SUBGRADE STRENGTH

3.1 Background

The subgrade strength is the other most important factor, apart from traffic loading, which governs the pavement structural configuration. It is assumed in this guide that the first stages of determining nominally uniform sections in terms of subgrade condition will have been undertaken. This can be based on geological and soil property assessments, in conjunction with other physical assessments such as the Dynamic Cone Penetrometer (DCP) test or in situ bearing tests, or any other means that allows realistic delineation. Section 5.8 discusses the general use of the DCP.

This section therefore focuses on the classification of these sections in terms of the California Bearing Ratio (CBR) to represent realistic conditions for design. In practice this means determining the CBR strength for the wettest moisture condition likely to occur during the design life, at the density expected to be achieved in the field.

The classification of subgrade condition in this guide is similar to RN31 and is shown in Table 3.1.

Table 3.1: Subgrade classification[1]

	Subgrade class designation					
Subgrade CBR ranges (%)	S1	S2	S3	S4	S5	S6
	2	3 - 4	5 - 7	8 - 14	15 - 29	30+

Since the combination of density and moisture content wholly governs the CBR for a given material, it is firstly clear that changes in moisture content will alter the effective CBR in the field, and it is therefore clear that particular effort must be taken to define the design subgrade condition.

The result of incorrect subgrade classification can have significant effects, particularly for poorer subgrade materials with CBR values of 5 per cent and less. If the subgrade strength is seriously overestimated (ie, the support is actually weaker than assumed), there is a high likelihood of local premature failures and unsatisfactory performance. Conversely, if the subgrade strength is underestimated (ie, the support is stronger than assumed), then the pavement structure selected will be thicker, stronger and more expensive than needed.

By the same token, there will always be considerable variation between results from samples, which makes it difficult to decide on a design value. This is further complicated by the requirement that the assumed subgrade strength is available to some depth: a thin, nominally high strength, material layer over a far weaker material will not provide the good support expected.

These guidelines are purposely kept as simple as possible, which means that limited details are provided. If more detailed information is required, RN31[1] is suggested as a primary reference source.

3.2 Representative Subgrade Moisture Content

The estimation of the wettest subgrade condition likely to occur, for design purposes, is the first stage in determining the design subgrade CBR. It is well known that moisture contents in subgrades are prone to variation due to natural effects, including rainfall, evaporation, and proximity of water table, as well as material type.

Any available local knowledge of the subgrade, locale, and prevailing conditions, should be drawn on first in determining the nominal design moisture content. Direct sampling should be undertaken if there is a clear understanding of how the sampled moisture content relates to the probable wettest condition to be encountered. If such specific information is not available, or it is felt necessary to supplement the available information, the following approach is suggested to estimate design moisture content.

a) Areas where water-tables are normally high, regular flooding occurs, rainfall exceeds 250 mm per year, conditions are swampy, or other indicators suggest wet conditions occurring regularly during the life of the road leading to possible saturation.

   Design moisture content should be the optimum moisture content determined from the AASHTO (Proctor) compaction test T-99 for the design moisture content.

b) Areas where water-tables are low, rainfall is low (say less than 250 mm per year), no distinct wet season occurs, or other indicators suggest that little possibility of significant wetting of the subgrade should occur.

   Use the moisture content determined from the following formula based on the optimum moisture content (OMC) determined from the AASHTO (Proctor) compaction test T-99:

   Design moisture content (%) = 0.67 * OMC (%) + 0.8 .....Equation 3

   where
   OMC is the optimum moisture content from the AASHTO (Proctor) compaction test T-99, and the simple relationship was derived from a comprehensive investigation into compaction characteristics (Semmelink, 1991[3]).

3.3 Classifying Design Subgrade Strength

The subgrade strength for design should reflect the probable lowest representative CBR likely to occur during the life of the road. As noted in Section 3.1, the value will be influenced by both density achieved and moisture content. For practical purposes, it is important that the highest practical level of density (in terms of Maximum Dry Density, or MDD) be achieved from the subgrade upwards in order to minimise subsequent deformations due to further densification under the traffic loading.

Clearly if insufficient compaction is achieved during construction then the longer term performance of the road is likely to be negatively affected, so it is critical to ensure that good compaction is attained. It is also critical to ensure that the subgrade has been compacted to a reasonable depth in order to avoid the possibility of the road deforming due to weakness of the deeper underlying material.

The following guidance (Table 3.2)is suggested for determining subgrade CBR classification according to Table 3.1, for minimum subgrade compaction requirements and for a control check on subgrade compaction during construction.

Table 3.2: Method for classifying subgrade design CBR

Expected subgrade conditions	Sample conditions for CBR testing*
Saturation is likely at some periods (high rainfall areas, distinct wet season, low-lying areas, flooding, high water-table, etc)	Specimens compacted at OMC (AASHTO T-99), to 100%** MDD. CBR measured after 4 days soaking***.
Saturation unlikely, but wet conditions will occur periodically (high rainfall areas, distinct wet season, water-table fluctuates, etc)	Specimens compacted at OMC (AASHTO T-99), to 100%** MDD. CBR measured with no soaking***.
Dry conditions (low rainfall areas, water-table low)	Specimens compacted at OMC (AASHTO T-99), to 100%** MDD. Specimens dried back to the design moisture content from Equation 3. CBR measured with no soaking***.

Notes:
* A minimum of six (6) representative samples per uniform section would be expected for classification purposes
** See (a) below regarding the use of other test moisture content/density requirements
*** Cohesive materials with Plasticity Indexes (PIs) greater than 20 should be stored sealed for 24 hours before testing to allow excess pore pressures to dissipate

a) Minimum subgrade compaction requirements
   The method for classification in Table 3.2 assumes that a <u>minimum</u> field compaction density of 100 per cent Proctor MDD (or 95 per cent modified AASHTO MDD) will be attained. In most cases, with current compaction equipment, this minimum should be readily achieved.

   Where there is evidence that higher densities can be realistically attained in construction (from field measurements on similar materials, from established information, or from any other source), a higher density should be specified by the Engineer. The higher density should also be used in the CBR classification in Table 3.2 in place of the 100 per cent MDD value.

   There may be cases where, because of high field compaction moisture contents (higher than OMC), material deficiencies or other problems, the CBR sample conditions are not realistic. In such cases, the Engineer <u>must</u> specify a lower target density and/or higher moisture content to be substituted for the sample conditions in Table 3.2 to represent probable field conditions more realistically.

b) Specifying the design subgrade class
The CBR results obtained in accordance with Table 3.2 are used to determine which subgrade class should be specified for design purposes, from Table 3.1.

In some cases, variation in results may make selection unclear. In such cases it is recommended that, firstly, the laboratory test process is checked to ensure uniformity (to minimise inherent variation arising from, for example, inconsistent drying out of specimens). Secondly, more samples should be tested to build up a more reliable basis for selection.

Plotting these results as a cumulative distribution curve (S-curve), in which the y-axis is the percentage of samples less than a given CBR value (x-axis), provides a method of determining a design CBR value. This is illustrated in Figure 3.1, from which it is clear that the design CBR class is realistically S2, or 3 - 4 per cent CBR. Choice of class S3 (5 - 7 per cent CBR) would be unjustified as the Figure indicates that between roughly 20 to 90 per cent of the sampled CBRs would be less than the class limits.

A good rule of thumb is to use the 10 per cent cumulative percentage (percentile) as a guide to the subgrade class, on the basis that only 10 per cent of the actual values would be expected to have a lower CBR than the indicated CBR. In this case, the 10 per cent rule indicates a CBR of approximately 4.5 per cent, thus confirming that the subgrade class of S2 is more appropriate than S3.

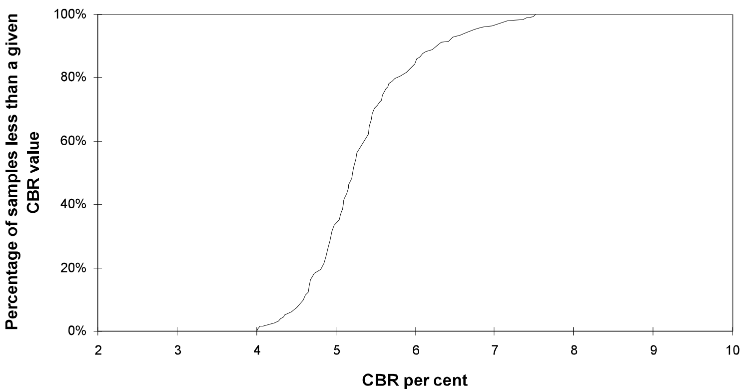


Figure 3.1: Illustration of CBR strength cumulative distribution

c) Control check on subgrade strength uniformity during construction
It is critical that the nominal subgrade strength is available to a reasonable depth in order that the pavement structure performs satisfactorily. A general rule is that the total thickness of new pavement layers (derived from the catalogue) plus the depth of subgrade which must be to the design subgrade strength should be 800 to 1 000 mm. Table 3.3 gives recommended depths for subgrade strength uniformity for the design subgrade classifications in Table 3.1.

Table 3.3: Recommended minimum subgrade depth meeting design strength

	Subgrade class designation					
	S1	**S2**	**S3**	**S4**	**S5**	**S6**
Minimum depth (mm)	250	250	350	450	550	650

It should be clearly understood that the minimum depths indicated in Table 3.3 are not depths to which recompaction and reworking would be anticipated. Rather, they are the depths to which the Engineer should confirm that the nominal subgrade strength is available. In general unnecessary working of the subgrade should be avoided and limited to rolling prior to constructing overlying layers.

For the stronger subgrades especially (class S4 and higher, CBR 8 - 14 per cent and more), the depth check is to ensure that there is no underlying weaker material which would lead to detrimental performance.

It is strongly recommended that the Dynamic Cone Penetrometer (DCP) be used during construction to monitor the uniformity of subgrade support to the recommended minimum depths given in Table 3.3 (see Section 5.8).

4. DEFINITION OF WET OR DRY CONDITIONS

4.1 Background

The design catalogue in this guide includes specific pavement structures for either nominally wet or nominally dry regions, in order to simplify the selection of appropriate pavements. While some consideration to prevailing conditions has already been given in the selection of appropriate subgrade classification, this section provides guidance on which set of structures to select.

Factors which will have an influence on the selection, apart from broad climatic considerations, also include drainage and maintenance regimes that are anticipated for the road. It is a basic fact that, for any road, the frequent ingress of water to the pavement layers will result in unwanted deterioration under trafficking. The rate and degree of such deterioration will also depend therefore on the level of trafficking.

While the underlying requirement for any road is the provision of good drainage and operation of an effective maintenance programme to ensure that water does not penetrate the pavement, real life conditions may not always match these needs.

Although it is implicitly assumed that suitable drainage and maintenance should be effected during the life of the road, and that lack of either of these will undoubtedly have a negative impact on long-term performance, the following guidance acknowledges that deficiencies occur. Such deficiencies should, nevertheless, be addressed in order to retain the investment made in the road. Appendix B provides some discussion on drainage.

4.2 Selecting the Appropriate Pavement Design Based on Moisture Influence

4.2.1 Predominantly Dry Regions

Selection of pavement structures from the catalogue for dry regions is appropriate where annual rainfall is less than 250 mm and there is no likelihood of moisture ingress due to factors such as significant flooding (in low-lying flood plain areas, or in tidal basins, for example), underground springs or wells, or any other detrimental conditions.

In regions of higher rainfall, where rainfall is evenly distributed throughout the year and no distinct rainy season conditions apply, the Engineer may deem the dry region catalogue to be appropriate. In such cases, it should be confirmed that there are not periods in which conditions will lead to significant possibility of moisture ingress to the pavement. It should be noted that long periods of light rain (or heavy fog), with heavy truck traffic, can cause serious damage of thin surfacings especially. It is unlikely that regions with rainfall more than 500 mm per annum would be regarded as dry regions for design purposes.

4.2.2 Predominantly Wet Regions

Any regions which do not comply with (a) above must be regarded as being predominantly wet. In line with the earlier discussion, there are certain other factors which should have an influence on selection of appropriate designs, and Table 4.1 provides some guidance. Depending on the likely maintenance and drainage provisions, Table 4.1 indicates which set

of design catalogues might be appropriate. The Engineer must, however, review all the prevailing factors in finalising his selection.

Table 4.1: Guide to selecting design conditions for predominantly wet regions

<table>
<tr><th rowspan="2">Expected drainage provision</th><th colspan="4">Expected maintenance level</th></tr>
<tr><th colspan="2">Good, programmed, defects remedied timeously</th><th colspan="2">Deficient</th></tr>
<tr><td rowspan="3">Good, well planned, well constructed</td><td colspan="2" rowspan="3">D</td><td colspan="2">Trafficking levels</td></tr>
<tr><td>Low, class T1 or T2</td><td>High, class T3 and more</td></tr>
<tr><td>D</td><td>W</td></tr>
<tr><td rowspan="3">Deficient</td><td colspan="2">Trafficking levels</td><td colspan="2" rowspan="3">W</td></tr>
<tr><td>Low, class T1 or T2</td><td>High, class T3 and more</td></tr>
<tr><td>D</td><td>W</td></tr>
<tr><td colspan="5">Note: D and W indicate the Dry and Wet region designs in the catalogue</td></tr>
</table>

5. PRACTICAL CONSIDERATIONS

5.1 Background

The earlier sections have provided guidance to the designer in selecting the design parameters of traffic class, subgrade support classification and nominal conditions. These are the primary factors used in entering the design catalogue in Appendix C to determine appropriate structures.

Until now, however, no consideration has been given to other factors which will have a practical influence on finalising possible pavement structures. Most significant of these is the availability, in terms of both quantity and quality, of materials for road construction. Other factors include the general topography, and the use of established local methods for road layer construction. Each of these will affect the final selection of a pavement.

While general specification requirements should be met, and some of these are indicated in both Appendix A guidelines and Appendix C, there may be a need for the Engineer to review these in the light of specific local conditions. This section therefore aims to provide some guidance in that respect.

It should be noted, however, that it is implicitly assumed that suitable bituminous surfacing materials will be obtained, whether surface treatments (typically single and double seals, including variations such as Cape or slurry seals) or hot-mix asphalts. This is not, therefore, discussed here.

5.2 Materials Availability

The designs given in this guide are based on the nominal material strength classifications given in Table 5.1. For structural purposes, this provides a guide to the probable performance, assuming that no unexpected deterioration (for example, due to water ingress) takes place. The full specifications, given elsewhere, include a number of other indicatory properties to assure that such deterioration ought not take place during the life of the road.

For the granular materials, only a minimum strength requirement is specified since there are usually no disadvantages in attaining higher strengths, and long-term performance is likely to be better in such cases. In line with foregoing discussions, however, it should be noted that density achieved is critically important if deformation under subsequent trafficking is to be minimised.

In contrast to just a minimum strength requirement, distinct upper and lower strength limits are placed on cemented materials (here meaning use of a Portland cement binder), due to the propensity of strongly cemented materials to form wide, widely-spaced, cracks which can reflect through overlying layers and open the pavement to moisture ingress, as well as losing structural integrity. The strength bounds are intended to ensure that any detrimental effects from cracking of the layer, which is virtually unavoidable in this type of material, are minimised by ensuring closer-spaced narrow cracks.

Table 5.1: Nominal strength classification of materials in the design catalogue

Layer	Material	Nominal strength
Base	Granular	Soaked CBR>80% @ 98% mod. AASHTO density
	Cemented	7 day UCS*1.5 - 3.0 MPa @ 100% mod. AASHTO density (or 1.0 - 1.5 MPa @ 97% if modified test is followed)
	Bituminous	See specification
Subbase	Granular	Soaked CBR>30% @ 95% mod. AASHTO density
	Cemented	7 day UCS*0.75 - 1.5 MPA @ 100% mod. AASHTO density (or 0.5 - 0.75 MPa @ 97% if modified test is followed)
Capping/ selected	Granular	Soaked CBR>15% @ 93% mod. AASHTO density

* 7 day unconfined compressive strength

It should be recognised at the outset that the use of cemented layers will only normally be considered if there are not suitable granular materials available locally. The first consideration is therefore to determine what local materials could be feasiblely used, and how these could meet the nominal requirements in Table 5.1 without significant processing (such as crushing, screening and recombining, or mechanical or chemical stabilisation).

Bearing in mind that the cost of transport of materials becomes a major cost factor if materials must be brought in to the site from a distance, it is usually cost-effective to try to utilise the local materials even if this would then necessitate some form of processing. As indicated above this may take various forms, but the choice is, of course, ostensibly a matter of cost and economy and in most cases the pavement designer must select materials accordingly.

In the case of certain "problem" materials (requiring some form of processing to comply with nominal specification requirements, other than crushing or screening) the following techniques might be considered in order to improve their road-building potential. No specific details are given here, however, and the Engineer should determine the most appropriate method based on local experience, ad hoc trials and/or specialist advice.

5.2.1 Natural Gravel/soil Materials not Meeting CBR and/or PI Requirements

Techniques which have been found to be effective in certain cases include:

- Treatment with lime or any other cementitious material (typically 2 to 5 per cent by weight): normally effective for reducing high PIs; will normally enhance CBR. Carbonation can cause longer term reversion to the original properties, so some caution should be adopted when using such treatment.
- Treatment with both bitumen-emulsion (typically 0.7 to 1.5 per cent residual bitumen by weight) and cement (typically 1.0 per cent by weight): will normally enhance compactibility, strength/CBR.

5.2.2 Cohesionless Materials, Sands

Techniques which have been found to be effective in certain cases include:

- Treatment with bitumen }stability may be a problem unless
- Treatment with foamed bitumen }well confined

5.2.3 Dense Clays/expansive Materials

A technique which has been found to be effective in certain cases is:

- Treatment with lime: can increase Plastic Limit (PL) and make material friable/more stable; will normally enhance CBR.

5.2.4 "Collapsible" Sands and Soils

These are materials that are, in effect, uncompacted and in which the existing material skeleton is maintained by relatively weak bonds between particles (usually clayey bonds). In the first instance, they should be compacted to a depth of 1.0 metre to a minimum density of 85 per cent modified AASHTO MDD (test method T-180). The Engineer should then reassess their suitability as a subgrade, and determine the appropriate subgrade classification (Section 3).

5.3 Terrain

The performance of a road in otherwise similar conditions can be influenced by terrain, in that rolling or mountainous terrain (in which significant grades are encountered) tends to lead to significantly more traffic-related loading on surfacings and bases. This is fairly commonly observed on relatively heavily trafficked roads (say, class T5 and higher, carrying more than 3 million ESAs) where surface deterioration and rutting deformation occurs. Routes on which overloaded trucks are common (axle loads of 10 tonnes and more) are especially prone.

In such situations, it is imperative that compaction of layers is controlled extremely well and ideally to more than minimum standards. It is also advised that the surfacing layer is resistant to deformation and, of course, well-bonded to the base to avoid early failure due to debonding and traffic-induced slippage at the interface.

A bituminous base combined with a hot mix asphalt surfacing can be (and is often) used to provide a stable, relatively stiff, deformation resistant backbone, which can also mask possible compaction deficiencies in the underlying layers which may occur due to difficult working conditions. There is considerable merit in looking at the use of special bituminous binders which may help inhibit rutting due to heavy vehicles, and the guidance of the bitumen supplier should be sought in the first instance.

An alternative approach, not specifically covered in this guide, is to consider the possibility of a concrete base. This type of construction can be effective for these conditions, and can be laid by labour-enhanced methods where conventional large-scale construction equipment is unsuitable.

It is also commonly observed that moisture-induced problems, leading to possible local premature failures, occur in cuts and on sag curves (dips), emphasising the need for particular attention to drainage provision and maintenance in such locations.

5.4 Vehicle Overloading

Incidences of vehicle overloading can have a significant negative impact on the performance of a road, and the effects are observed especially by premature failures of surfacing layers (excessive rutting, bleeding, loss of surface texture, and ravelling being prevalent as early indicators). Naturally, every effort should be made to limit the amount of overloading (illegal loading) but it is recognised that current controls may not always be sufficient.

While the design process should account for the amount of heavy vehicle axle loads in determining the design traffic loading (Section 2), the specific effects of the very heavy abnormal axle loads on the pavement must be considered in finalising the design.

In situations where overloading is likely to occur, special attention must be given to the quality and strength of all the pavement layers during construction. Amongst other measures, there may be justification in increasing the specification CBR requirements for granular layers, in increasing the base and subbase layer thicknesses, and in specifying special bitumen binders and asphalt mixtures, such as stone mastic asphalts, which are more resistant to deformation.

The specific measures that the Engineer may deem necessary should, ideally, be based on either proven local practice or at least specialised advice/analysis in order to maintain a well balanced structure.

5.5 Subgrade CBR less than two per cent

In these cases, which must be treated according to the specific situation, some of the possible approaches include:

- In situ treatment with lime (for clayey materials).
- Removal and replacement with better quality material.
- Use of geofabrics.
- Construction of a pioneer layer (for highly expansive material and marshy areas) or rockfill.

These conditions are often encountered in low-lying, wet and swampy areas, and treatment should ideally be based on past proven practice for similar conditions. The use of geofabrics, usually in accordance with specialist advice from the manufacturer, can be extremely effective in situations where other approaches are inappropriate (for example, where better quality materials are either not readily available, or would tend to displace downwards).

When appropriately treated, the design for the overlying pavement can then be based on the re-evaluated subgrade support condition.

5.6 Design Trafficking Greater than 30 million ESAs

For such heavily trafficked roads, other established design methods must be used and the pavement designer is advised to look at UK, US and Australian practices in addition to the South African TRH4[2] guide.