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International pavement designs with cold recycled material

Deliverable D5

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Proposal of pavement design procedure including structure catalogue and identification of failure modes for MEPD

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1. Introduction

1.1. Background

Because the European Road Network is almost complete, the maintenance of the existing roads will cover an increasing rate of the future investments into road infrastructure. The vast majority of the roads are constructed with asphalt pavements, where the usual maintenance procedure is the removal of the entire asphalt structure or the upper layer(s) from it and the paving of new asphalt mixture on the remaining structure. As a consequence, the increasing age of the road network results in increasing amounts of reclaimed asphalt (RA).

Cold recycling (CR) of reclaimed asphalt RA is a procedure, by which high recycling rates of usually ≥ 75 % are reached with less sensitivity regarding RA properties (Mollenhauer & Simnofske, 2015) compared to other recycling options (i.e. in hot asphalt mixtures). CR has been successfully applied in numerous road structures within the secondary (Bocci et al. 2014) and the primary road network. Because the mixture is produced at ambient temperature, the main energy demanding process for asphalt production – the heating and drying of aggregates – is reduced significantly.

Despite the available practical experience with cold recycled materials (CRM), the failure modes are not yet fully understood. This can partly be explained by the frequent application within the secondary road network with comparably low traffic volume as well as with low expertise and need in maintenance and durability assessment. The mechanical properties of CRM are well-researched in laboratory-based projects. As a result, usually lower stiffness and strength properties are assessed compared to hot asphalt mixtures (HMA). Therefore, available pavement design procedures based on fatigue failure criteria usually applied on HMA result in thick required base layers and therefore are not practically applicable (Radenberg et al. 2015) and don't comply with the practical experience made with these pavement materials. Therefore, the practically applied pavement design procedures vary considerably and range between the same thickness estimations as for hot bituminous materials (e.g. in Switzerland) to thickness-increase factors of e. g. 50 % (Bocci et al. 2010) compared to standard HMA.

One reason for these caution-driven design estimations lies within the time-dependent change of material strength during curing which results in increasing bearing capacity of Cold Recycled Asphalt Bases (CRAB) structures over months or even years (Serfass, 2010, Godenzoni et al. 2018). International experiences show the applicability of CRM even in highly-trafficked highway structures (Wirtgen 2015). For these pavement designs, the failure criteria applied within pavement design are permanent deformation, shear strength and fatigue cracking (Asphalt Academy 2009, Liebenberg and Visser, 2004).

1.2. Aims and Methodology

The intention of this deliverable report as main output of CRABforOERE-work package (WP) 5 is to make a comparison of the different national existing empirical and empiric-mechanistic pavement design procedures. By including results from the durability assessment of several existing pavement structures (within WP2) the available procedures can be validated. From this synthesis, a proposal for design procedures for pavements including CRAB layer is drafted.

In the first section, this report summarises the pavement design procedures applied in five European countries (Sweden, UK, Germany, France, Italy), representing the European range of different climatic conditions.

These procedures are applied for the structural design of four model pavements with specific subbase bearing capacity and traffic loading conditions. Here, besides the pavement designs for pavements with CRAB layers, also the designs with traditional HMA pavements are assessed. By comparing the differences of pavement structures between CRAB and HMA bases, the different levels of expectations in the CRAB are evaluated.

The in-service durability and long-term performance of the assessed existing pavements with CRAB will be used for validating the identified design principles.

2. National pavement design procedures

The pavement design procedures within Europe differ considerably. In order to compare the structural design of CRAB as applied in various countries, firstly the commons of the design approaches are discussed. The design applied for standard asphalt base layers, as well as unbound base and cement stabilised base layers are used as reference for comparison of the structures with CRAB layers.

The generally applied pavement design procedure in Italy, Germany, Sweden, UK and France are individually translated to English and summarised.

2.1. Italian design approach

2.1.1. Pavement design approach for flexible pavements

In Italy, the only official reference document available on pavement is a pavement catalogue, "*CATALOGO DELLE PAVIMENTAZIONI STRADALI*", published in 1994 by the National Research Council (CNR) (CNR, 1995). The catalogue was developed using the design method described in the "AASHTO Guide for Design of Pavement Structures". In addition, the fatigue performance has been checked using multi-layered elastic models and suitable transfer functions. The properties of the materials and the climatic conditions are fixed, whereas different designs are developed considering:

- The type of road
- The traffic level and composition
- The subgrade (subground) load carrying capacity

Materials

The general structure of flexible pavements includes 4 layers:

1. An asphalt concrete surface layer (*conglomerato bituminoso per strato di usura*);
2. An asphalt concrete binder layer (*conglomerato bituminoso per strato di collegamento*);
3. An asphalt concrete base layer (*conglomerato bituminoso per strato di base*);
4. An unbound granular foundation layer (*misto granulare non legato*)

The asphalt concrete properties are defined based on the Marshall mix-design procedure, whereas granular material must have a California Bearing Ratio (CBR) value greater than 30%.

The general structure of semi-rigid pavements includes a cement treated layer (*misto cementato*) directly below the asphalt concrete base.

Climatic conditions

For flexible and semi-rigid pavements, the average climatic conditions of central Italy (altitude < 1000 m) are considered. The year is divided in four seasons (winter, spring, summer and autumn) with average temperatures of 4.5°C, 11.5°C, 22°C and 14°C.

Road Types

Eight road types are considered:

1. Rural Motorways (*Autostrade extraurbane*);
2. Urban Motorways (*Autostrade urbane*);

3. Rural Highways – heavy traffic (*Strade extraurbane principali e secondarie a forte traffico*);
4. Rural Highways – normal traffic (*Strade extraurbane secondarie ordinarie*);
5. Rural Highways – touristic traffic (*Strade extraurbane secondarie turistiche*);
6. Urban roads – heavy traffic (*Strade urbane di scorrimento*);
7. Urban and rural roads – light traffic (*Strade urbane di quartiere e locali*);
8. Urban roads – Specialized lanes (*Corsie preferenziali*)

Each road category is characterized by specific values of final Present Serviceability Index (PSI) value and reliability, according to the AASHTO Design Guide.

Traffic level and composition

Each road category has a specific traffic spectrum in terms of commercial vehicle type (axle number and weight) and proportion. A total of 6 traffic levels are defined in terms of total number of commercial vehicles passes, from 0,4 10^6 to 45,0 10^6.

The higher traffic levels are considered for higher-rank road types; lower traffic levels are considered for lower-rank road types.

The traffic load *n* is calculated in terms of equivalent single-axle load $ESAL_{80kN}$, using the commonly equation:

$$n = N \cdot 365 \cdot ADT \cdot p \cdot I \cdot CE \qquad \text{(eq. 1)}$$

With:

ADT= average daily traffic

p = proportion of heavy lorries

$I = \frac{(1+p)^N - 1}{p}$, where p is the traffic increase and N the service life time (assumed 30 years).

$CE = 4 \cdot \left(\frac{70\ kN}{80\ KN}\right)^4$, CE was calculated considering 4 axles per vehicle of 70 kN axle road.

Subgrade load bearing capacity

Three subgrade categories are considered based on the Resilient Modulus or the CBR index:

- Mr = 150 MPa, CBR = 15%
- Mr = 90 MPa, CBR = 9%
- Mr = 30 MPa, CBR = 3%

For road types 1, 2, 3 and 6, weak subgrades (Mr = 30 MPa, CBR = 3%) must be replaced or stabilized.

Example

Figure 1 and Figure 2 shows semi-rigid pavement structures for rural motorways, the main structural solution that has been adopted in the major Italian motorways. Nowadays, an unbound granular layer is constructed below the cement treated layer to avoid premature failures, modified binder is used for all asphalt layers and the surface layer is in porous asphalt.

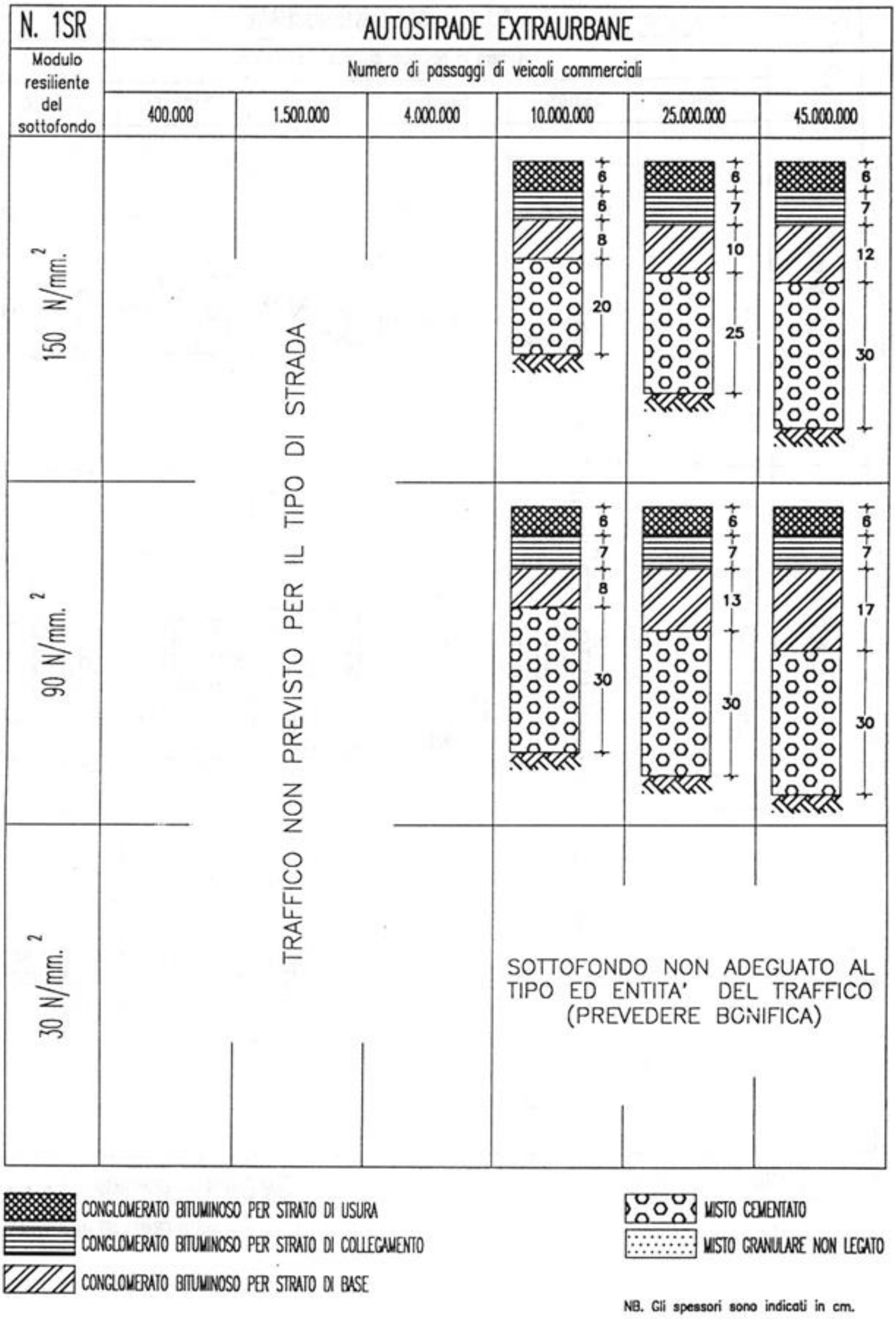


Figure 1: CNR Catalogue: semi-rigid pavement structures for Rural Motorways (Autostrade Extraurbane)

Figure 2: CNR Catalogue: flexible pavement structures for rural highways with normal traffic (Strade extraurbane secondarie ordinarie).

2.1.2. Pavement Design procedure for CRM pavements

Only conventional materials are included in the pavements designed according to the CNR catalogue. Therefore, the use of advanced and innovative materials like, for example, modified bitumen or CRM layers, must be supported with a mechanistic-empirical design.

For pavements including CRM layers, there are two main structural solutions:

1. CRM is used as sub-base (foundation) layer;
2. CRM is used as base layer (CRAB).

The use of CRM as foundation layer is normally associated with a full-depth reclamation where the existing asphalt layers are milled and mixed with the underlying granular foundation. CRAB require higher quality control on aggregate and thus are produced in-plant or with cold in-place recycling of existing asphalt layers. CRM are produced using both bituminous binder (emulsion or foam) and hydraulic binder (cement). CRM used for base layers is equivalent to a cold asphalt concrete and is produced only with emulsion of SBS-modified binder.

For structural design purpose, CRM used in foundation layers is considered a two-phase material [2]. The first phase is a fatigue phase similar to cemented materials, while the second phase is a permanent deformation phase, similar to unbound granular materials. On the other hand, CRM used in base layers is considered equivalent to a cold asphalt concrete. Fatigue and permanent deformation transfer functions normally adopted for asphalt concrete are applied and the final thickness is adjusted (normally with a 30% increase).

2.1.3. Structural design catalogue including CRM layers

Recently, the *Provincia Autonoma di Bolzano - Alto Adige* (Autonome Provinz Bozen - Südtirol) developed a structural design catalogue including CRM layers.

<u>Materials</u>

Beside traditional layers like unbound granular layers or cement treated layers, flexible and semi-rigid pavements include:

1. asphalt concrete for surface, binder or base layers (*usura, binder, base*) all manufactured with SBS modified binder;
2. CRAB layer with SBS modified bitumen emulsion (*base a freddo con emulsione di bitume modificato*) and cement
3. CRAB layer with foamed bitumen or bitumen emulsion (*base a freddo con bitumen schiumato o emulsion bituminosa*) and cement
4. Cement- or lime-stabilized foundation layers (*Fondazione stabilizzata a calce e/o cemento*) which are produced in-place.

All layers are designed according to individual technical specifications for materials and construction works.

<u>Climatic conditions</u>

The Alto Adige-Südtirol province is an alpine region, therefore, 4 climatic areas were defined based on the altitude. Each climatic area is characterized by different temperatures (Table 1).

Table 1: Climatic areas and temperature ranges

Climatic area	Altitude	Coldest monthly average temperature (°C)	Hottest monthly average temperature (°C)
1	< 500 m	0.9	23.1
2	500 to 1000 m	0.1	21.0
3	1000 to 1500 m	-0.1	17.9
4	> 1500 m	-1.5	15.8

Based on the climatic area and the frost penetration depth, a frost protection layer (*strato antigelo*) may be required.

Road types and traffic

Since the traffic composition in Alto Adige-Südtirol is extremely variable, the 80 kN ESAL is adopted in nine traffic levels, from 0.5 10^6 to 30.0 10^6 ESAL. The highest traffic level (from 24 to 30 10^6 ESAL) is considered only for climatic area 1.

Subgrade load bearing capacity

Three subgrade categories are considered based on the E_{V2} modulus measured with plate bearing test:

1. E_{V2} = 80 MPa;
2. E_{V2} = 120 MPa;
3. E_{V2} = 160 MPa.

The first two categories are considered for natural subgrades with medium and high bearing capacity. The third value identifies high strength cut areas (e.g. rock) or for lime or cement stabilized subgrades. Weaker subgrades must be replaced or stabilized.

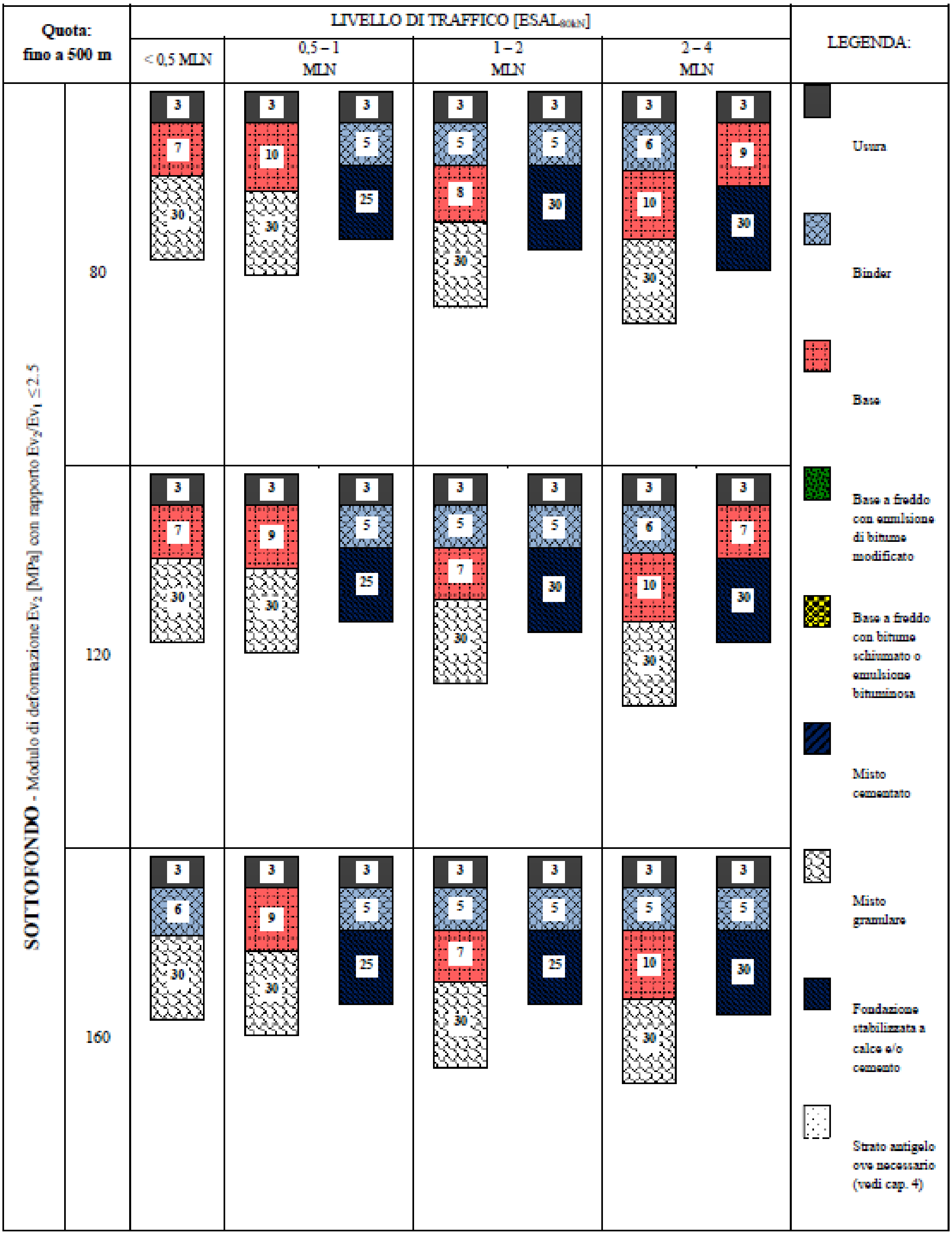
Example

Figure 3 and Figure 4 show pavement design catalogue for climatic area 1 (altitude< 500 m) with high and intermediate traffic levels (5 to 9). All pavements have asphalt concrete surface and binder layers. For each traffic/subgrade combination 3 pavement design solutions are presented.

For high traffic levels (7 to 9) the “conventional” design includes an asphalt concrete base layer plus a cement treated sub-base layer and a granular foundation. This is very similar to the design of the CNR catalogue.

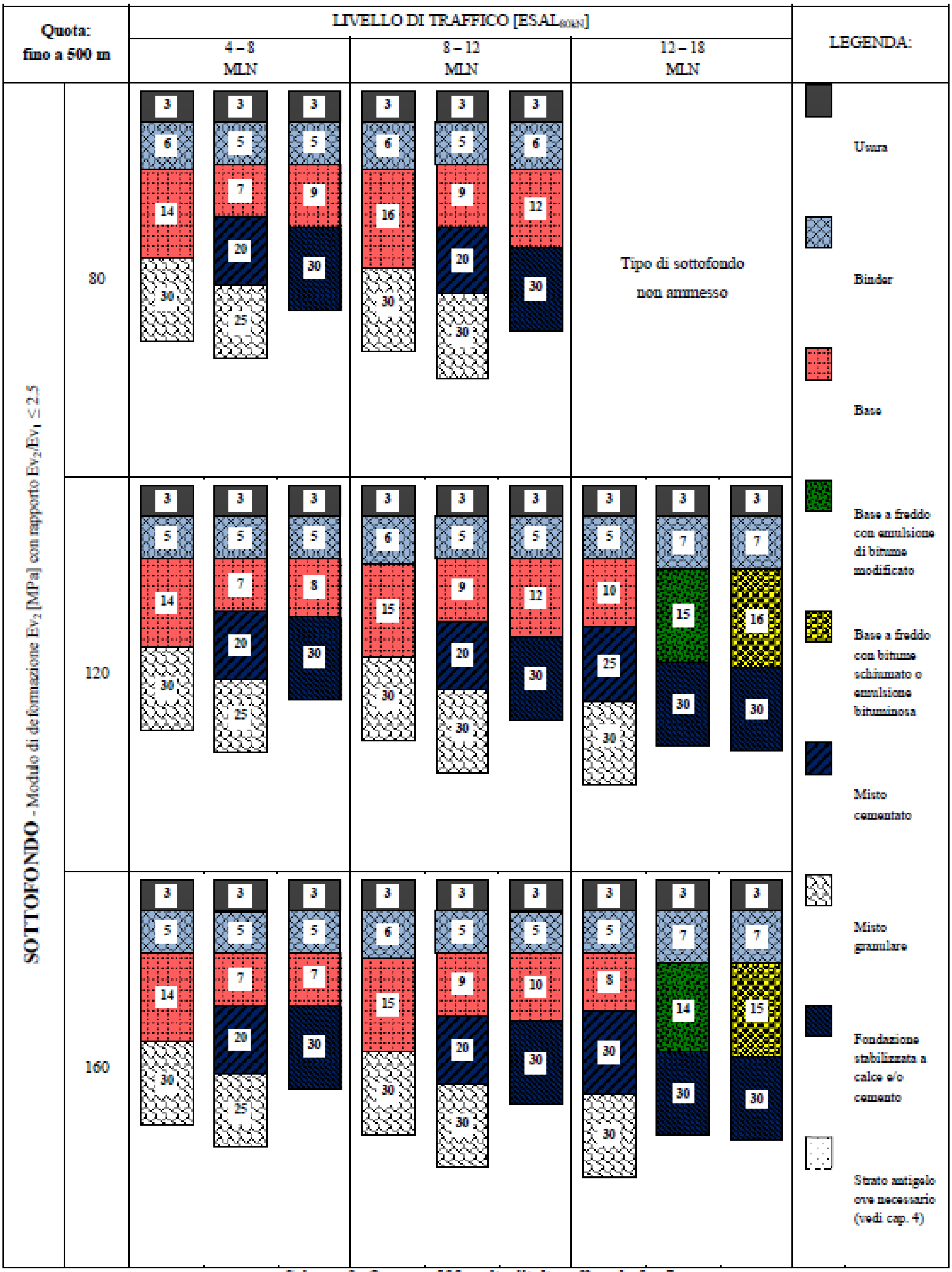
Then, there are two equivalent structures where the asphalt concrete base layer, the cement treated sub-base layer and the granular foundation are replaced with:

- a CRAB layer with SBS modified bitumen emulsion plus a cement- or lime-stabilised layer.
- a CRAB layer with foamed bitumen or bitumen emulsion plus a cement- or lime-stabilised layer.



Schema 1: Quota < 500 m, livelli di traffico da 1 a 4

Figure 3: Alto Adige-Südtirol catalogue – pavement design for climatic area 1 and low traffic levels



Schema 2: Quota < 500 m, livelli di traffico da 5 a 7

Figure 4: Alto Adige-Südtirol catalogue – pavement design for climatic area 1 and intermediate traffic levels (5 to 7)

The complete Italian pavement catalogue "*CATALOGO DELLE PAVIMENTAZIONI*" including CRM layers will be listed in the Annex A.

2.2. German design approach

2.2.1. Pavement design approach for flexible pavements

In Germany, the pavement design procedure is specified in RStO 2012 (FGSV, 2012). The procedure considers the mechanical properties of applied road materials, which themselves are specified in mix design standards, as well as environmental conditions.

The general structure of a pavement is shown in Figure 5, which also gives the German translation for the various layers applied. The asphalt blanket (*Asphaltdecke*), consisting of asphalt surface and asphalt binder course is laid on top of the asphalt base layer. Below, an additional base layer can be constructed, either unbound (crushed aggregates or gravel) or cement stabilized. Here also CRAB can be paved. Latter is not part of the general specification RSTO but described within a guidance document (FGSV, 2005).

In general, the structure is designed for a period of 30 years. The procedure is based on feasible drainage of the road surface as well as the surface of the subbase. All applied layers and construction materials are designed according to the individual technical specifications for materials and construction works. The resulting structures consider economic viable demand for road maintenance works.

The basis for the pavement structure is the bearing capacity of the subground surface. Before the pavement is constructed, the surface has to reach a modulus with plate bearing test (E_{v2}) of 45 MN/m². The surface of subbase layer shall have a modulus of at least 120 MN/m², unbound base layers 150 MN/m².

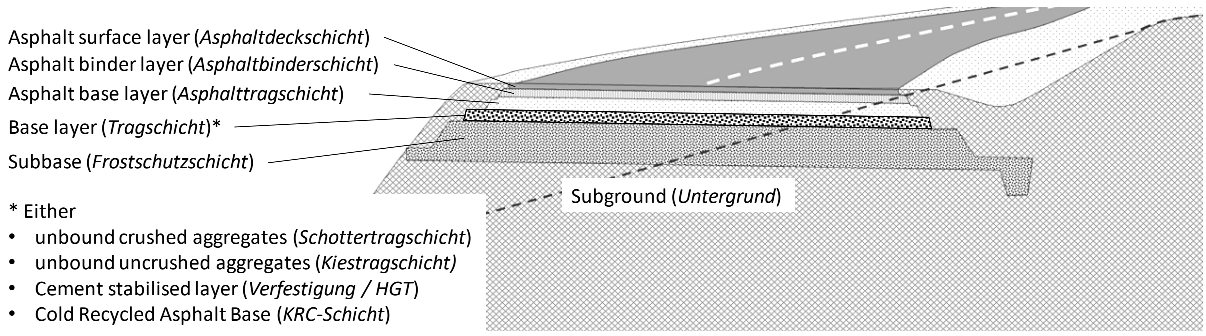


Figure 5: General road pavement structure and German translations of pavement layers

The pavement design procedure contains the following steps:

1. Calculation design traffic parameter “B”, which is the number of equivalent 10-t-axles loading the pavement during the design period (usually 30 a)
2. Determination of the pavement-thickness regarding frost resistance
3. Selection of a suitable configuration

<u>Step 1: Traffic loads</u>

$$B = N \cdot 365 \cdot DTV^{SV} \cdot f_A \cdot q_{Bm} \cdot f_1 \cdot f_2 \cdot f_3 \cdot f_z \qquad \text{(eq. 2)}$$

N: expected service life [a]

DTV^{SV}: Average daily traffic of heavy vehicles (≥ 3,5 t)

f_A: Average Number of axles per heavy vehicle

q_{BM}: mean axle load factor, considering the loading factor [k= $(\frac{La}{10t})^4$] for each axle

f_1, f_2, f_3: Factors to consider the number of lanes considered during the traffic count, the lane width and longitudinal slope of the road

$f_z = \frac{(1+p)^N - 1}{p \cdot N}$ Factor to consider the annual increase of heavy traffic (p)

Table 2: Factors considering the axle configuration of the heavy vehicle

Road type	*Factor for number of axle per vehicle* f_A	*mean axle load factor* q_{Bm}	*Annual increase of heavy traffic* p
Federal highway or local roads with $DTV^{SV} > 6$ %	*4,5*	*0,33*	*0,03*
Federal road or local roads with $DTV^{SV} \leq 6$ % *and* > 3 %	*4,0*	*0,25*	*0,02*
District road, or local roads with $DTV^{SV} \leq 3$ %	*3,3*	*0,23*	*0,01*

Table 3: Factors f1 considering daily traffic count procedure in regard to number of lanes

Number of lanes in section or direction	*Type of the* DTV^{SV} *registration* (f_1)	
	Both direction	*Single direction*
1	*-*	*1,00*
2	*0,50*	*0,90*
3	*0,50*	*0,80*
4	*0,45*	*0,80*
5	*0,45*	*0,80*
6 and more	*0,45*	*-*

Table 4: Factors f2 considering the lane width

Width of traffic lane [m]	*Factor* (f_2)
2,50 and less	*2,00*
2,50 to under 2,75	*1,80*
2,75 to under 3,25	*1,40*
3,25 to under 3,75	*1,10*
3,75 and more	*1,00*

Table 5: Factors f3 considering longitudinal slope

Maximum of longitudinal incline [%]	*Factor (f_3)*
less than 2	*1,00*
2 to under 4	*1,02*
4 to under 5	*1,05*
5 to under 6	*1,09*
6 to under 7	*1,14*
7 to under 8	*1,20*
8 to under 9	*1,27*
9 to under 10	*1,35*
10 and more	*1,45*

Table 6: Relevant Strain for dimensioning and the related load class

Relevant Strain for dimensioning B	*pavement class*
$\geq 32 \cdot 10^6$	*Bk100*
$10 \cdot 10^6$ *up to* $32 \cdot 10^6$	*Bk32*
$3{,}2 \cdot 10^6$ *up to* $10 \cdot 10^6$	*Bk10*
$1{,}8 \cdot 10^6$ *up to* $3{,}2 \cdot 10^6$	*Bk3,2*
$1{,}0 \cdot 10^6$ *up to* $1{,}8 \cdot 10^6$	*Bk1,8*
$0{,}3 \cdot 10^6$ *up to* $1{,}0 \cdot 10^6$	*Bk1,0*
Up to $0{,}3 \cdot 10^6$	*Bk0,3*

From the calculated design traffic parameter B, the pavement class (Belastungsklasse) is determined accordingTable 6.

<u>Step 2: climatic (frost) loads</u>

After determination of the load class, the required pavement-thickness regarding frost resistance is identified. Main parameters are the pavement class and the subground soil conditions. In general, the soil is classified according to the content of fine grains (≤ 0,063 mm).

- If content of fines is less than 5 %, the sub ground is considered as frost resistance (F1), and the subground can be used directly as a sub-base.
- If the content of fines is between 5 and 15 %, the soil is considered as frost sensitive (F2).
- If the content of fines is > 15 %, the soil is considered as very frost sensitive (F3).

For considering various climatic conditions, Germany is grouped in three frost zones depending on the frost intensity (see Figure 6). Additionally, surplus thickness is added for taking local frost conditions and ground moisture into account and the total pavement thickness is calculated by summing up the thickness values given in Table 7.

Table 7: Determination of the required thickness to get frost resistant pavement

1.	*Minimum thickness of the frost-resistant pavement structure* • *road pavement:* • *cycle path (with frost resistant sub-base):*	 *40 cm* *30 cm*
2.	*Frost-sensitivity class of the soil* • *frost-sensitivity class F2:* • *frost-sensitivity class F3:*	 *±0 cm* *+10 cm*
3.	*Pavement class* • *Bk100/Bk32/Bk10:* • *Bk3,2/Bk1,8/Bk1,0:* • *Bk0,3:*	 *+15 cm* *+10 cm* *±0 cm*
4.	*Frost zone* *To recording the different climate conditions in some regions it is necessary to increase the thickness to avoid freezing damage.* • *Frost zone I:* • *Frost zone II:* • *Frost zone III:*	 *±0 cm* *+5 cm* *+15 cm*
5.	*local climatic distinctions* • *unfavorable climate impacts (e. g. North slope/ridge):* • *favorable climate impacts (e. g. Lateral development):*	 *+5 cm* *-5 cm*
6.	*Water conditions in subground* • *No groundwater level up to 1,5 m below sub ground surface:* • *groundwater level higher than 1,5 m below sub ground surface:*	 *±0 cm* *+5 cm*
7.	*The position of the road in regard to the natural ground surface* • *cut/gate:* • *ground level up to dam ≤ 2,0 m:* • *dam > 2,0 m:*	 *+5 cm* *±0 cm* *-5 cm*
8.	*Drainage of the surface / design of the fringes* • *hollows, ditches or slopes:* • *channels, outlets, and pipes:*	 *±0 cm* *-5 cm*

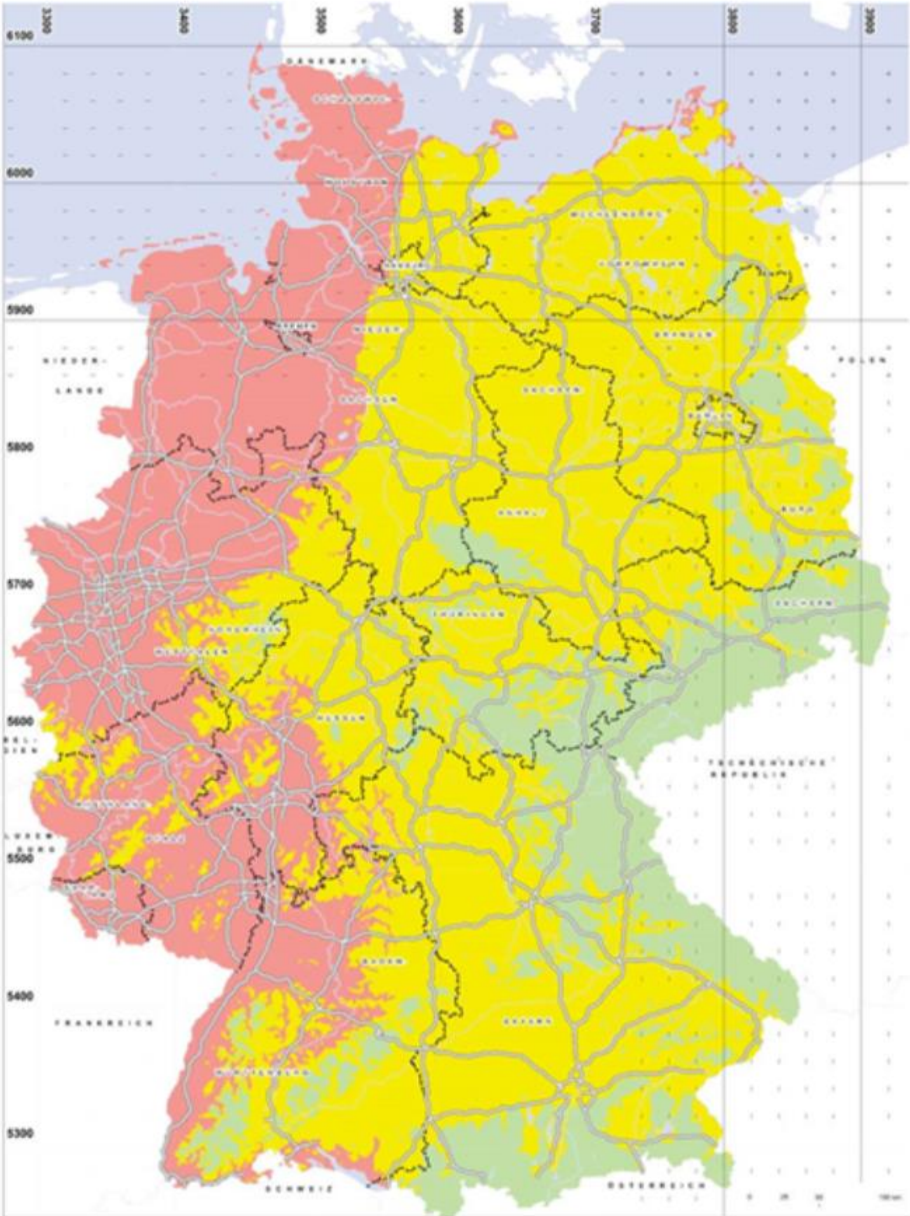


Figure 6: Map of frost zones in Germany (FGSV 2012)

For the resulting pavement class, a suitable pavement structure can be chosen from Figure 7. The structures given in the various lines are considered to be technically equivalent. The resulting thickness of the sub-base layer is calculated by subtracting the thickness of the pavement layers required for bearing capacity from the required total pavement thickness required by frost resistance.

Step 3: Design catalogue

A suitable pavement structure is selected from Figure 7.

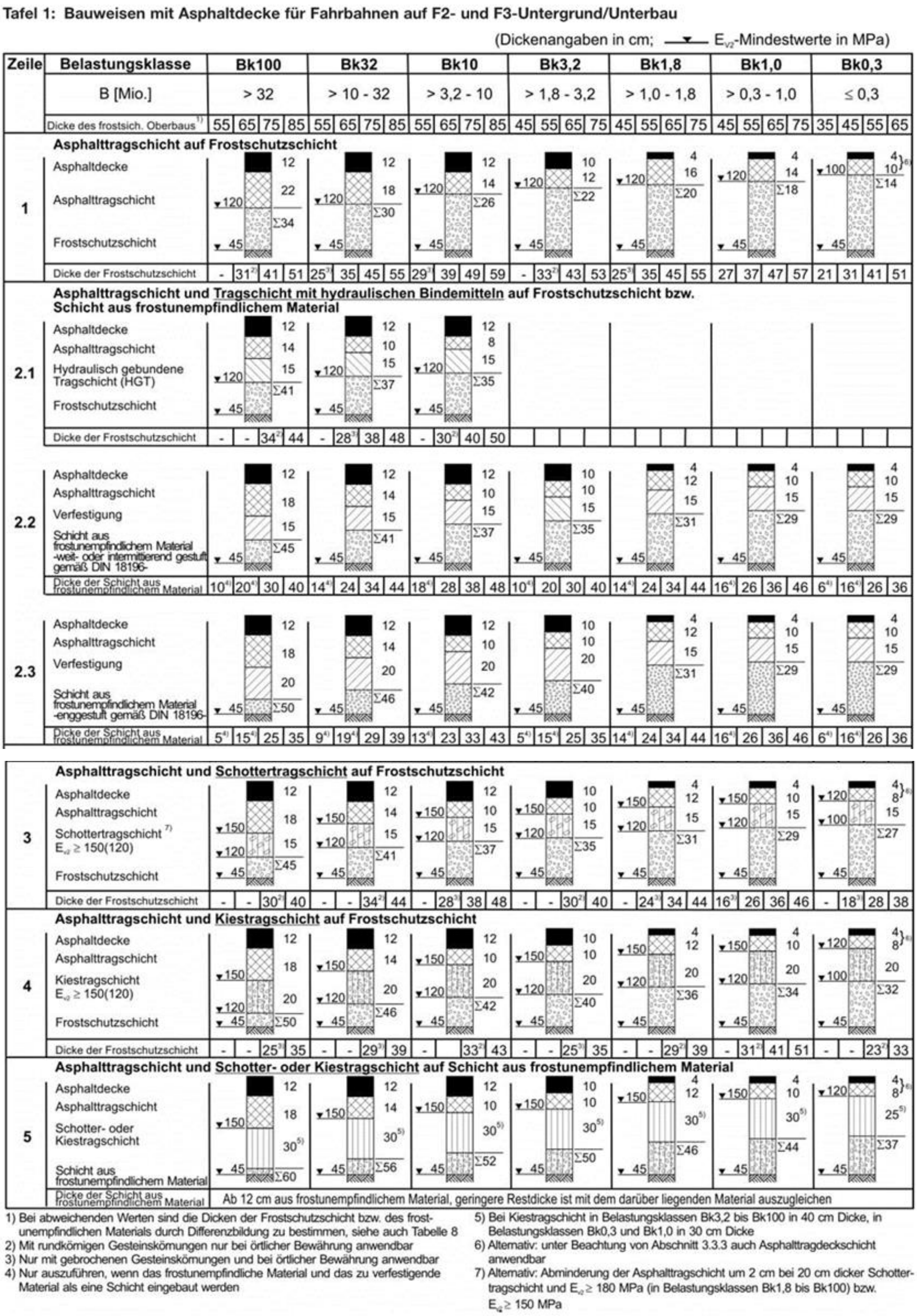


Figure 7: Asphalt Pavement structures for given traffic load classes (BK) (FGSV, 2012)

2.2.2. Pavement Design procedure for CRM pavements

The specifications for pavement design considering CRM are given in the guideline "M KRC" (FGSV, 2005). According to the pavement class and the bearing capacity of the sub-base/sub-ground, expressed by the modulus E_{V2} derived within loading plate test, Table 8 gives suitable thickness values for the pavement layers. If pavement structure is applied during a road rehabilitation, no frost resistance has to be specified if the pavement did not show any frost defects in the past.

Table 8: Layer thickness for asphalt pavements with CRAB:

Bearing capacity of subbase/ subground	*Layer*	*pavement class Bk3,2*	*pavement class Bk1,8*	*pavement class Bk1,0*	*pavement class Bk0,3*
		Layer thickness [cm]			
$E_{V2} \geq 30$ MN/m²	Asphalt surface course	Not applicable		4	10[1,2]
	Asphalt binder course			-	-
	Asphalt base course			10[1]	-
	CRM-layer[3]			**16**	**16**
	∑			*30*	*26*
$E_{V2} \geq 45$ MN/m²	Asphalt surface course	4	4	4	8[1,2]
	Asphalt binder course	6	-	-	-
	Asphalt base course	8[1]	10[1]	8[1]	-
	CRM-layer	**18**	**18**	**16**	**16**
	∑	*36*	*32*	*28*	*24*
$E_{V2} \geq 80$ MN/m²	Asphalt surface course	4	4	4	6[1,2]
	Asphalt binder course	4	-	-	-
	Asphalt base course	6[1]	8[1]	6[1]	-
	CRM-layer	**20**	**18**	**16**	**16**
	∑	*34*	*30*	*26*	*22*
$E_{V2} \geq 120$ MN/m²	Asphalt surface course	4	4	4	6[1,2]
	Asphalt binder course	4	-	-	-
	Asphalt base course	8[1]	6[1]	-	-
	Asphalt surface course	**20**	**18**	**16**	**14**
	∑	*32*	*28*	*24*	*20*

[1] Minimum layer thickness of asphalt base layer

[2] Combined asphalt base and surface layer

[3] Preferably hydraulic-dominant binding of the CRM-layer

2.3. Swedish design approach

2.3.1. Pavement design approach for flexible pavements

Flexible pavements constructed with bitumen bound layers shall be designed according to Figure 8 to Figure 10.

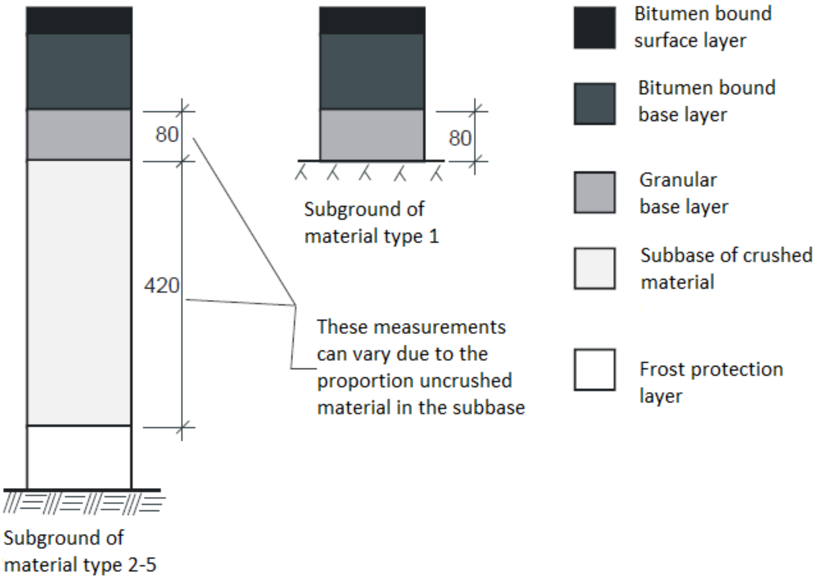


Figure 8: General flexible pavement design without binder layer.

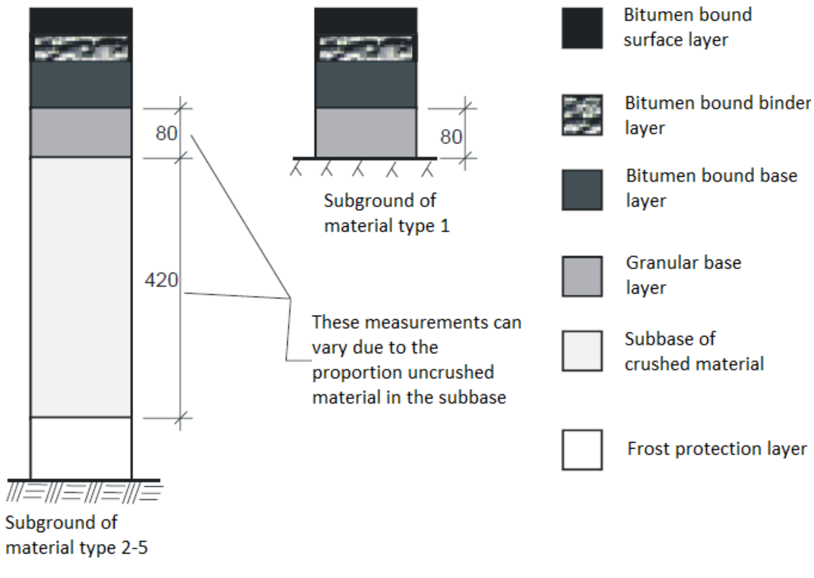


Figure 9: General flexible pavement design with binder layer.

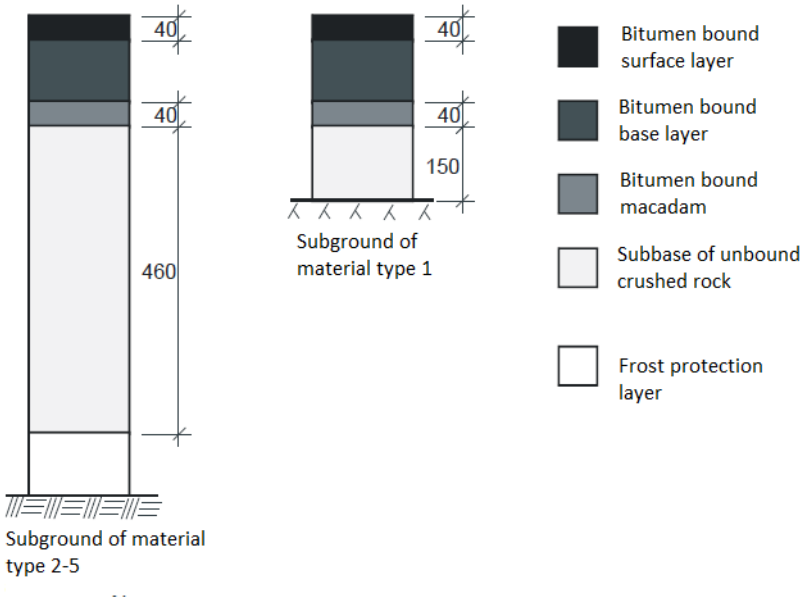


Figure 10: Flexible pavement of crushed rock

Regulations from the Swedish Transport Administration are specified in the TRVK Väg (Trafikverket, 2011) and state that pavement design should be made by limiting horizontal strain at the bottom surface of the bottom bituminous bound layer and vertical strain at the top surface of the subground. Additional requirements regarding minimum thicknesses for the different layers must also be fulfilled according to Figure 8 to Figure 10. Advice therefore is given in the TRVR Väg (Trafikverket, 2011). Almost every pavement design is made using the calculation program "PMS Objekt" provided by the Transport Administration.

The program calculates the equivalent number of 10-ton axles (N_{ekv}) using the equations given below with the parameters given in Table 9.

$$N_{ekv} = \text{Å}DT_k \cdot A \cdot B_{just} \cdot \sum_{j=1}^{n} \left(1 + \frac{k}{100}\right)^j =$$

$$= \begin{cases} \text{Å}DT_k \cdot 365 \cdot A \cdot B_{just} \cdot \left(1 + \frac{k}{100}\right)\left(\left(1 + \frac{k}{100}\right)^n - 1\right) \;\; if \;\; k \neq 0 \\ \text{Å}DT_k \cdot 365 \cdot A \cdot B_{just} \cdot n \;\; if \;\; k = 0 \end{cases} \quad \text{(eq. 3)}$$

$$B_{just} = B \cdot f_a \cdot f_b \cdot f_c \quad \text{(eq. 4)}$$

Table 9: Parameters used to calculate the equivalent number of 10-ton axles during the design life.

$ÅDT_k$	Annual average daily traffic in one direction
A	Percent heavy vehicles
B	Equivalent number of 10-ton axles per heavy vehicle
k	Annual increase of heavy vehicles
n	Expected service life
f_a	Lane width factor
f_b	Road type factor
f_c	Reference velocity factor
B_{just}	Adjusted *B*-factor

The B-factor is normally provided by the Transport Administration and is based on measurements or experience, expressing what kind of vehicles that normally traffic the road.

The next step is to look at how many standard (10-ton) axles the pavement in a specific climate zone corresponds to. The equations used for these calculations are given here below together with their parameters given in Table 10. Climate data are given in Table 11 and Table 12 together with Figure 11.

Vertical strain at the top surface of the subground

All pavements with at least one bitumen bound layer must be designed to limit the vertical strain at the top of the subground.

$$N_{till,te} \geq 2 \cdot N_{ekv} \qquad \text{(eq. 5)}$$

$$N_{till,te} = \frac{365}{\sum_{i=1}^{m} \frac{n_i}{N_{te,i}}} \quad where \quad N_{te,i} = \frac{8.06 \cdot 10^{-8}}{\varepsilon_{te,i}^4}$$

Horizontal strain at the bottom surface of the bottom bound layer (often bound base layer)

All pavements with at least one bitumen bound layer, >75 mm, must be designed to limit the horizontal strain at the top of the subground.

$$N_{till,b} \geq N_{ekv} \qquad \text{(eq. 6)}$$

$$N_{till,te} = \frac{365}{\sum_{i=1}^{m} \frac{n_i}{N_{bb,i}}} \quad where \quad N_{bb,i} = \frac{2.37 \cdot 10^{-12} \cdot 1.16^{(1.8T_i+32)}}{\varepsilon_{bb,i}^4}$$

Table 10: Parameters used to calculate strains in the pavement layers.

$N_{till,te}$	Allowed number of standard (10-ton) axles due to strain limit in the subground
$N_{till,bb}$	Allowed number of standard (10-ton) axles due to strain limit in the bitumen bound base layer
$N_{te,i}$	Allowed number of standard axles due to strain limit in the subground during climate period *i*
$N_{bb,i}$	Allowed number of standard (10-ton) axles due to strain limit in the bitumen bound base layer during climate period *i*
n_i	The number of days in climate period *i*
m	The number of climate periods
$\varepsilon_{te,i}$	Largest allowed compressive strain at the subground surface for climate period *i* when loading the pavement surface with a standard axle
$\varepsilon_{bb,i}$	Largest allowed tensile strain in the bitumen bound base layer for climate period *i* when loading the pavement surface with a standard axle
T_i	Temperature (°C) in the bitumen bound pavement for climate period *i*

Table 11: Length of climate periods (number of days during the year).

	Climate zone				
	1	2	3	4	5
Winter	49	80	121	151	166
Thawing winter	10	10			
Thaw	15	31	45	61	91
Late spring	46	15			
Summer	153	153	123	77	47
Autumn	92	76	76	76	61

Table 12: Temperature in bitumen bound layers (°C).

	Climate zone				
	1	2	3	4	5
Winter	-1.9	-1.9	-3.6	-5.1	-7
Thawing winter	1	1			
Thaw	1	2.3	4.5	6.5	7.5
Late spring	4	3			
Summer	19.8	18.1	17.2	18.1	16.4
Autumn	6.9	3.8	3.8	3.8	3.2

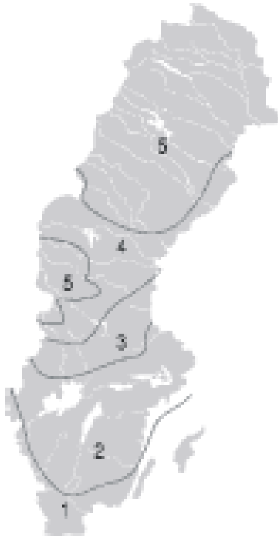


Figure 11: Climate zones in Sweden illustrated on a map.

2.3.2. Pavement Design procedure for CRM pavements

The Swedish Transport Administration is currently working on implementing the new European standard for Asphalt Concrete with Bitumen Emulsion (ACBE) (EN 13108-31) during 2020. The first procurement of base layers with ACBE for production and placement will thus take place during the season 2021 at earliest. However, pavements of base layers using CR with up to 80% RA are occasionally constructed according to the national standard. According to these guidelines, there are no special requirements when using cold recycled material. The goal when using CRM is that the CRAB layer should be equally good or better than the conventional asphalt base layer. Therefore, there is no difference in the design between the conventional HMA design and the CRM design within the same pavement number in this example. There are not any special requirements on mixes with bitumen emulsion and virgin aggregates. All requirements refer to the aggregates or the bitumen emulsion itself but there are no design requirements on that kind of mix, i.e. there are no requirements on layer thickness or stiffness modulus.

2.4. United Kingdom design approach

2.4.1. Pavement design approach for flexible pavements

Current pavement design guidance in UK is regulated by Highway Agency’s (HA’s) as Design Manual for Roads and Bridges (Highways Agency, 2006, 2009). It provides the procedure of standard design methods in detail (Thom, 2008).

The approach, which is based on empirical evidence and extended through analytical calculation, can be described as follows.

- The designer first must select a ‘foundation class’ (based on effective modulus), and then designs the lower layers of the pavement to achieve it, using the charts or equations provided.
- These foundation designs are a function of subgrade CBR and the thickness and moduli of one or more granular or hydraulically-bound layers.

- Upper pavement design is then derived from charts into which the design traffic (in equivalent standard axles), the selected road base materials and the selected foundation class are fed.

The general structure of a pavement is shown in Figure 12.

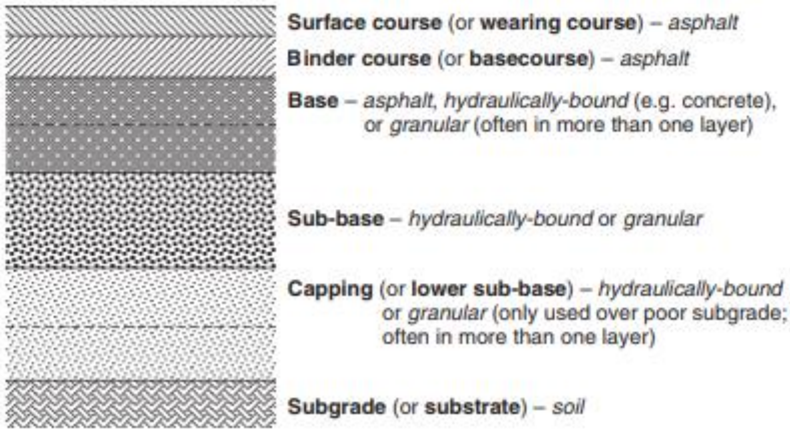


Figure 12: UK Pavement structure adapted from (Highways Agency, 2006)

<u>Thickness design</u>

The main purpose of the foundation is to distribute the applied vehicle loads to the underlying subgrade, without causing distress in the foundation layers or in the overlying layers. This is required both during construction and during the service life of the pavement. The four foundation classes are defined by the foundation surface modulus value used for design purposes, as follows (Highways Agency, 2009):

- Class 1 ≥ 50 MPa
- Class 2 ≥ 100 MPa
- Class 3 ≥ 200 MPa
- Class 4 ≥ 400 MPa

Pavement foundation design in the UK has been based on the principles of layered linear elastic modelling since the 1980s (Powell et al., 1984). This approach requires the elastic stiffness of each foundation layer to be defined, enabling critical stresses and strains to be predicted. These are subsequently assessed against empirically derived limits, in order to reduce the risk of premature pavement failure to an acceptable level.

The models have traditionally focused on a very restricted number of materials, with relatively well documented engineering properties. Assessing the engineering properties of individual materials for both the construction and in-service situations is a complicated and lengthy process. It is simpler and more cost-effective to develop a single proxy measure, which can be used in all situations with all types of material, to predict the likely overall performance of the foundation.

The use of a Performance Related Specification for assessing Foundation Surface Modulus is compatible with the current UK methodology for pavement design (Highways Agency, 2006)

This method requires a given level of Foundation Surface Modulus, referred to as a Foundation Class, to support various types of pavement construction and associated material thicknesses. Performance Design is a method that can be used to predict the likely Foundation Surface Modulus that will be achieved by certain combinations of foundation layers over different types of natural ground (the subgrade).

The process for designing, constructing and testing a Performance Related Foundation is summarized in Figure 13.

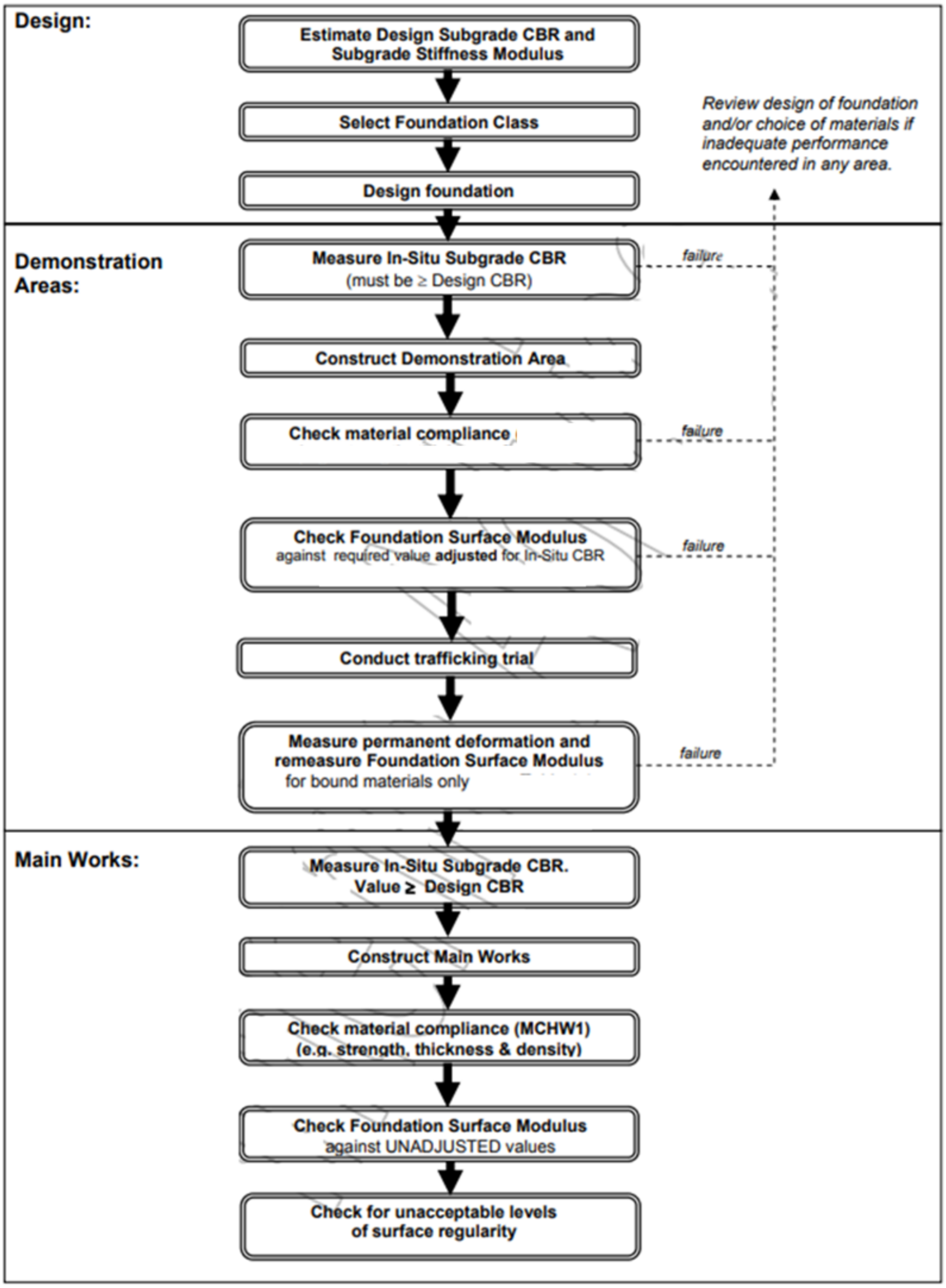


Figure 13: Summary Flowchart for Performance Related Foundations adapted from (Highways Agency, 2009)

Subgrade requirements

For design purposes, the Subgrade Surface Modulus must be estimated from CBR values.

Table 13 gives the unadjusted Mean Foundation Surface Modulus and Minimum Foundation Surface Modulus values, for each Foundation Class, and for different categories of materials, to be achieved or exceeded at the top of foundation level immediately prior to the construction of the overlying pavement layers.

Table 13: Top of Foundation Surface Modulus Requirements (Highways Agency, 2009)

		Surface Modulus (MPa)			
		Class 1	**Class 2**	**Class 3**	**Class 4**
Long-Term In-service Surface Modulus		**≥50**	**≥100**	**≥200**	**≥400**
Mean Foundation Surface Modulus	Unbound Mixture Types:	**40 ¤**	**80 #**	★	★
	Fast-setting Mixture Types:	**50 ¤**	**100**	**300**	**600**
	Slow-setting Mixture Types:	**40 ¤**	**80**	**150**	**300**
Minimum Foundation Surface Modulus	Unbound Mixture Types:	**25 ¤**	**50 #**	★	★
	Fast-setting Mixture Types:	**25 ¤**	**50**	**150**	**300**
	Slow-setting Mixture Types:	**25 ¤**	**50**	**75**	**150**

¤ Only permitted on trunk roads including motorways that are designed for not more than 20 msa

\# Not permitted for pavements designed for 80 msa or above (HD26 requirement)

★ Unbound materials are unlikely to achieve the requirements for Class 3 & 4

The practical minimum foundation thicknesses have been taken as 150 mm for all materials in Class 1 or 2 foundations, 175 mm for materials in a Class 3 Foundation and 200 mm for materials in a Class 4 Foundation. The increase in minimum thicknesses for Classes 3 and 4 relates to their proportional sensitivity to variations in thickness. Thin layers of stiffer bound materials are also more susceptible to cracking and it is important that these materials do not crack beyond the levels assumed in the design. Maximum permissible layer stiffness values have also been imposed for each Foundation Class to minimise the risk of selecting very thin, very stiff foundation layers at lower subgrade CBR values. The maximum permissible layer stiffness's to be used in the design are:

- Foundation Class 1: ≤ 100 MPa.
- Foundation Class 2: ≤ 350 MPa.
- Foundation Class 3: ≤ 1000 MPa.
- Foundation Class 4: ≤ 3500 MPa.

There are a large number of possible designs for the various combinations of Subgrade Surface Modulus and foundation material, in order to achieve the desired Foundation Class.

Figure 14 to Figure 17 show an example of design charts for foundation class 1 to 4.

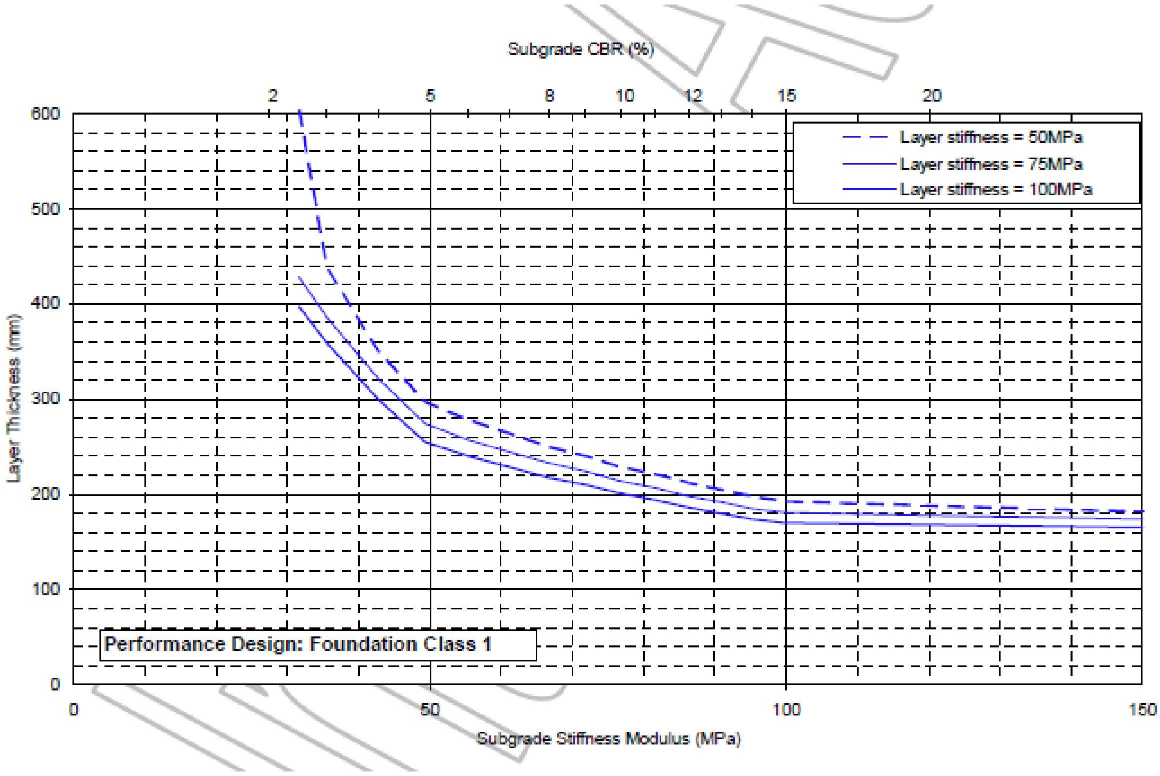


Figure 14: Class 1 Designs – Single Foundation Layer (Highways Agency, 2009)

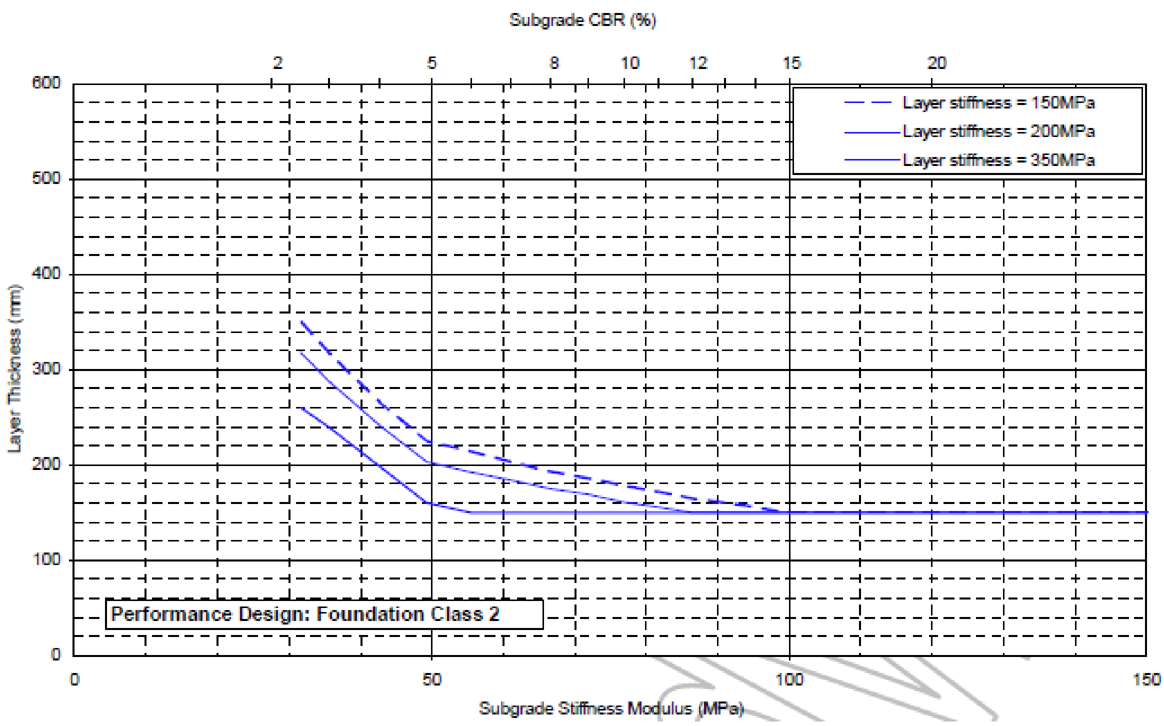


Figure 15: Class 2 Designs – Single Foundation Layer (Highways Agency, 2009)

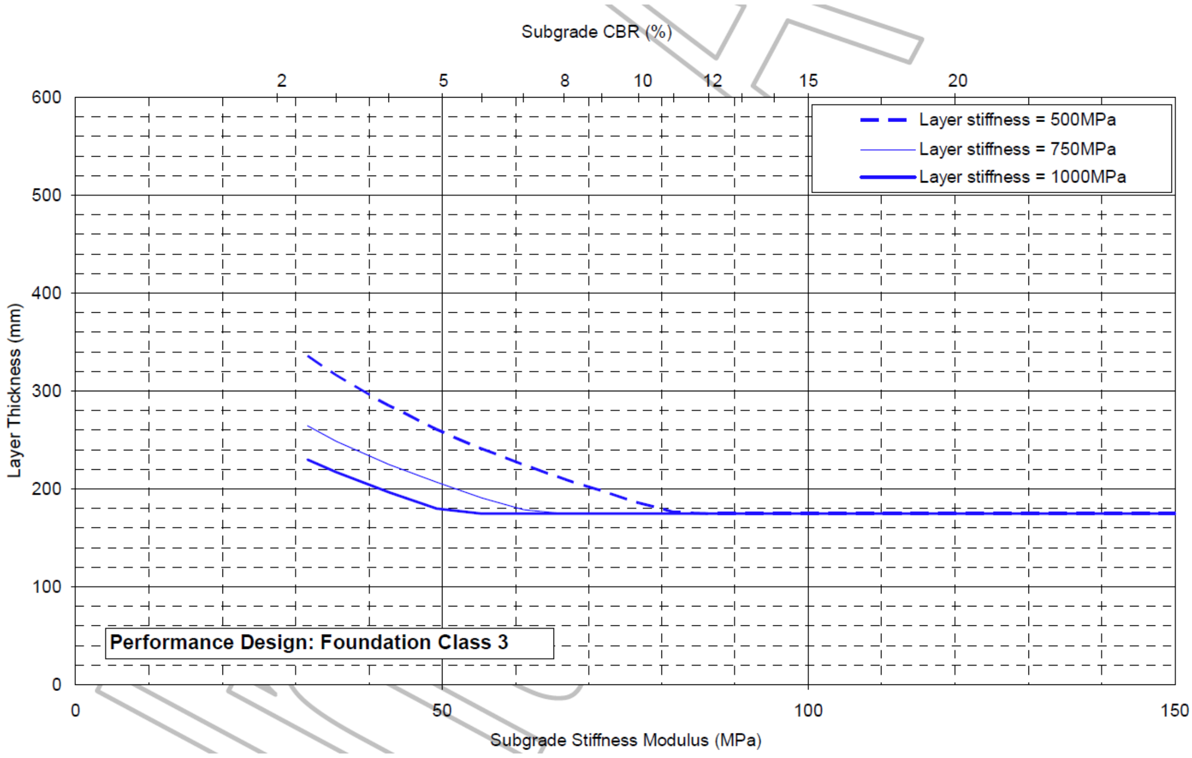


Figure 16: Class 3 Designs – Single Foundation Layer (Highways Agency, 2009)

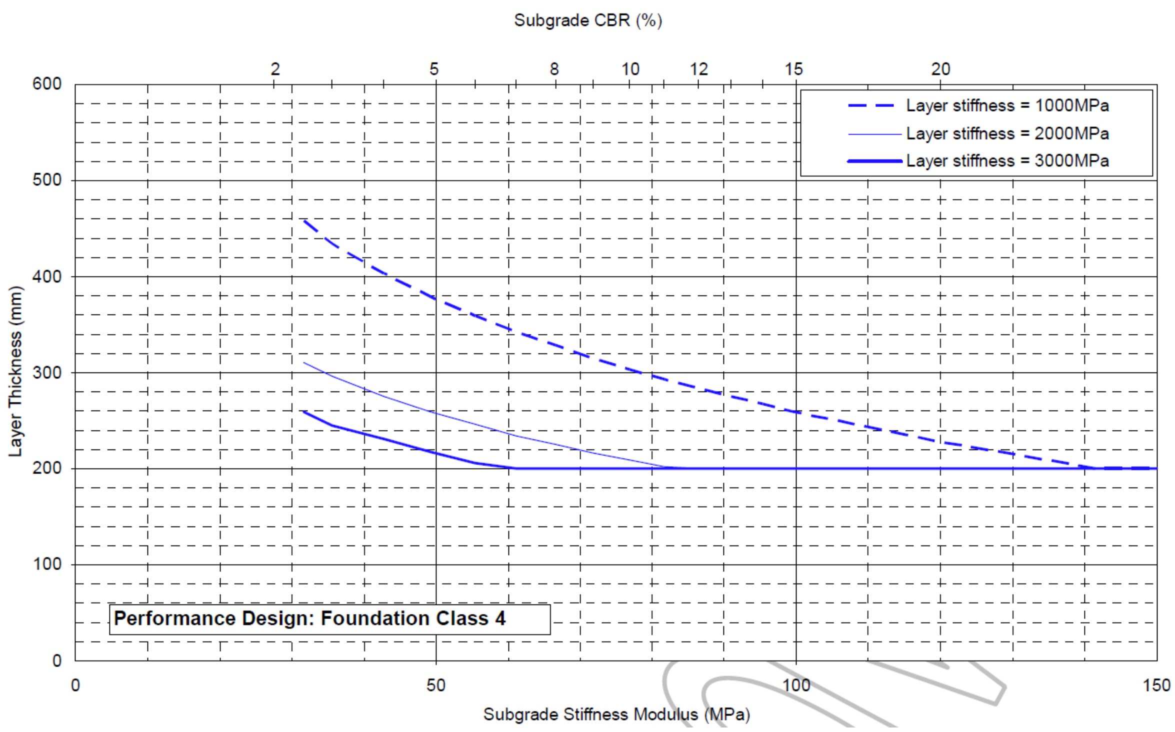


Figure 17: Class 4 Designs – Single Foundation Layer (Highways Agency, 2009)

Design Phase

In the UK, the primary material performance characteristic used in foundation design is stiffness modulus.

For subgrades, this property is difficult to measure reliably and consistently, so historically California Bearing Ratio (CBR) has been used as an indirect measure. Full access to the construction site is not always possible during the design phase so it can be difficult to carry out in-situ testing. Where a geotechnical investigation is carried out, representative samples should be taken of the subgrade materials likely to be encountered on site. Estimation of the likely long-term, short-term and hence, Design CBR should be derived using laboratory CBR tests in accordance with BS 1377 Pt 4 (1990).

Examples of designs with subbase on capping are only presented for Foundation Class 2. The structural contribution of capping materials with low Layer Stiffness values is limited when compared with the stiffness of subbase materials required to achieve Foundation Classes 3 and 4. Their inclusion in the design model does not, therefore, demonstrate a significant reduction in the thickness of subbase required but designers should consider the practical advantages of including capping materials in the foundation design.

The inclusion of a capping layer, however, should always be considered for the practical benefit they afford, enabling construction plant to lay the subbase and providing a good base for the necessary compaction to be achieved. The provision of a capping layer may be particularly appropriate for lower strength subgrades and can, if the material is suitable, also provide a drainage path below a layer of bound material.

Drainage and frost

It is of vital importance to keep water out of the subbase, capping and subgrade, both during construction and during the service life of the pavement. Wherever possible, the foundation drainage should be kept separate from pavement run-off drainage in all new construction and in reconstruction work. There should always be a down-slope route from the subbase to the drain (Highways Agency, 2004). A granular aggregate drainage blanket (Highways Agency, 2016) of thickness at least 150 mm and not more than 220 mm may be used to drain water that infiltrates through the pavement. In order to stop pore clogging by fines from other adjacent layers, geosynthetic separators may be used when those layers are constructed of fine soil or fine capping.

A drainage layer of this type may be particularly appropriate below a bound foundation layer. The drainage layers so formed may be treated as capping for structural design purposes. When the water table is high and especially when the subgrade is moisture sensitive with a Plasticity Index < 25, slot drains as detailed in the Highway Construction Details, can be beneficial. The drain is placed below the bottom of capping (or subbase if no capping is used), to drain any water that may permeate through these materials. Deeper drains can be beneficial in drying and strengthening these, and some other soil types.

If it is necessary to determine the permeability of the subbase or capping material, this must be done on the full grading, at the correct density under a low hydraulic head (Highways Agency, 1990). Drainage of the subbase may be omitted only if the underlying materials (capping, subgrade) are more permeable than the subbase, and the water table never approaches the underside of foundation closer than 300mm.

For routine cases all material within 450 mm of the road surface shall be non-frost susceptible as required (Highways Agency, 2016) and tested according to BS812: Part 124 (1989). This requirement can be over-severe in some places (e.g. coastal areas) and may be reduced to 350mm if the Mean Annual Frost Index (MAFI) of the site is less than 50. Advice on the frost index for any particular area may be obtained from the Met Office. The frost index is defined as the product of the number of days of continuous freezing and the average amount of frost (in degrees Celsius) on those days.

According to the UK designing standard, the traffic volume does not affect the foundation design, but it can be considered a crucial variable during the upper pavement layer design.

Table 14: Road type category in the UK

Road type category	*Traffic design standard (msa)*
0	Roads carrying over 30 to 80* msa
1	Roads carrying over 10 to 30 msa
2	Roads carrying over 2.5 to 10 msa
3	Roads carrying over 0.5 to 2.5 msa
4	Roads carrying up to 0.5 msa

Traffic assessment

This section describes the method of calculating design traffic for maintenance purposes.

The future cumulative flow, in terms of million standard axle (msa) for commercial vehicle (cv) class T_i can be determined according to the following equation (Liebenberg, 2004):

$$T_i = 365 \times F \times Y \times G \times W \times P \times 10^{-6} msa \qquad \text{(eq. 7)}$$

$$Design\ Traffic\ (T) = \sum T_i$$

The factors used to calculate the Design Traffic (T) are as follows:

- Commercial Vehicle Flow at opening (F);
- Design Period (Y); that represents the number of years over which traffic is to be assessed shall be selected. For past traffic, this will generally be the number of years since opening or last major structural maintenance. For future design traffic this shall generally be 40 years.
- Growth Factor (G); that The National Road Traffic Forecast (NRTF) published in eight-year intervals and predicts future traffic trends (Figure 19).
- Wear Factor (W), see Table 15.
- Percentage of vehicles in the heaviest loaded lane (P). All lanes are designed as for the heaviest loaded lane. For new and existing carriageways with 2 or more lanes in one direction, the proportion of vehicles in the most heavily loaded lane shall be estimated using Figure 18.

Finally, five road categories are identified based on the value of *msa* (Table 14).

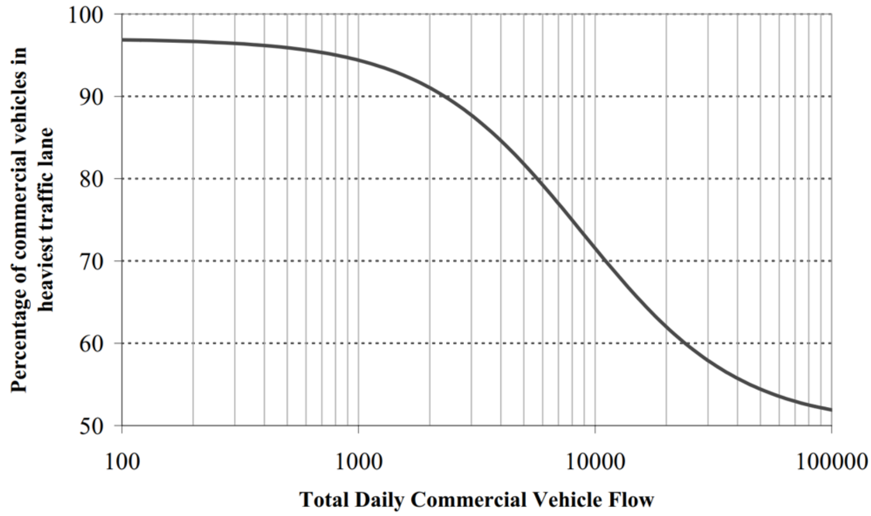
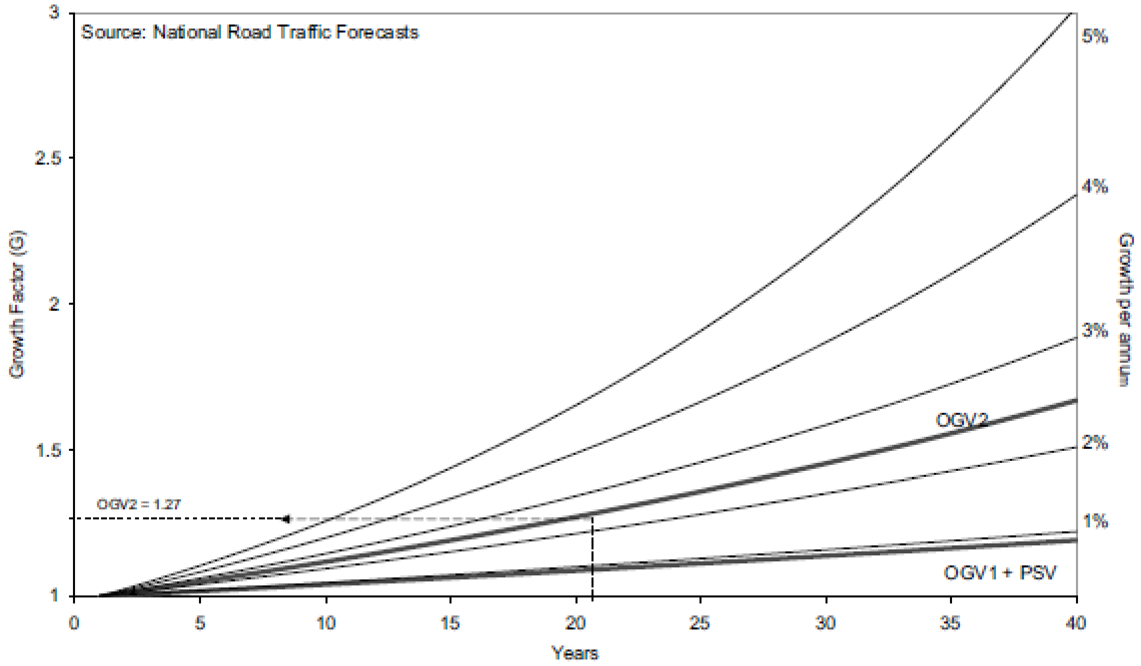


Figure 18: Percentage of Commercial Vehicles in Heaviest Loaded Lane (P) (Highways Agency, 2006)

Table 15: Wear Factors for cv Classes and Categories (Highways Agency, 2006)

Wear Factors	Maintenance W_M	New W_N
Buses and Coaches	2.6	3.9
2-axle rigid	0.4	0.6
3-axle rigid	2.3	3.4
4-axle rigid	3.0	4.6
3 and 4-axle articulated	1.7	2.5
5-axle articulated	2.9	4.4
6-axle articulated	3.7	5.6
OGV1 + PSV	0.6	1.0
OGV2	3.0	4.4

Commercial vehicle (cv)	cv class*	cv category
	Buses and Coaches	PSV
	2-axle rigid	OGV1
	3-axle rigid	
	3-axle articulated	OGV2
	4-axle rigid	
	4-axle articulated	
	5-axle articulated	
	6 (or more) -axle articulated	



Extracted Growth Factor (G) values assuming 1997 NRTF growth:

Design Period (Years)	5	10	15	20	25	30	35	40
OGV1 + PSV	1.02	1.04	1.06	1.09	1.11	1.14	1.17	1.19
OGV2	1.05	1.12	1.19	1.27	1.36	1.45	1.56	1.67

Figure 19: Derivation of Growth Factor (G) for Future Traffic (Highways Agency, 2006)

Upper pavement layer design

Flexible Pavements presents the upper layers of the pavement bounded in bitumen and the lower (base) layers either bounded in bitumen or with a hydraulic binder. Permitted binder and base layers are as follows (Highways Agency, 2006):

- Dense Bitumen Macadam (DBM50 or DBM125) or Heavy-Duty Macadam (HDM50);
- Stone Mastic Asphalt (SMA), for use as a binder course only, not as a base, except in Scotland where a 'Departure from Standard' must be obtained from the Overseeing Organisation;
- Hot Rolled Asphalt (HRA50), which must only be used in England, Wales and Northern Ireland with the approval of the Overseeing Organisation;
- EME2, but only on a Class 3 or 4 Foundation, subject to a Departure from Standard being granted by the Overseeing Organisation. (Also see Figure 20);
- Permitted hydraulically bound materials (HBM) for use in the base layers.

These may include:

- Cement Bound Granular Material (CBGM);
- Fly Ash Bound Material (FABM);
- Slag Bound Material (SBM).

Following the Flexible Pavement Construction steps:

1. Thickness to be rounded up to the next 10 mm.
2. Minimum allowable total asphalt material thickness is 200 mm for flexible construction with asphalt base. Minimum allowable HBM base thickness is 150 mm for flexible construction with HBM base, except in Scotland where the minimum thickness of HBM is 175 mm.
3. Asphalt surfacing thickness in mm (H) over HBM base is given by:

   $$H = -16.05 \times \left(Log(T)\right)^2 + 101 \times Log(N) + 45.8 \qquad \text{(eq. 8)}$$

   where:

   T = Design traffic (msa), up to 400 msa.
4. Calculated thickness (mm) to be rounded up to the next 10 mm; with a minimum thickness of 100 mm for < 4 msa, and a thickness of 180 mm for > 80 msa.
5. Where the asphalt design thickness is 300 mm or less, the material is to be laid with no negative tolerance
6. If 50 mm of Porous Asphalt (PA) surfacing is to be used, it must be modified with a polymer or fibre additive. Its contribution to the material design thickness is only 20 mm. A 60 mm dense binder course is required beneath PA surfacing.
7. A binder course must be provided beneath a thin surface course system (TSCS) but is optional beneath other materials such as HRA where this is permitted. If used, the binder course can be of any permitted material and be at least 50 mm thick (except for SMA binder courses, which should be at least 30 mm thick), and compacted so that the air voids are less than the maximum required.
8. DBM125 base and binder course must contain 100/150 penetration grade binder. HRA50, DBM50 and HDM50 base and binder course must contain 40/60 penetration grade binder. EME2 base and binder course must target a penetration of 15-20, which can be achieved using 10/20 or 15/25 penetration grade binder. In Scotland, where HMB35 might be used, the material must target a penetration of 30/45.
9. Where traffic exceeds 80 msa, binder course and base materials must contain crushed rock, or slag coarse aggregate, unless local experience exists of the successful use of gravel.
10. The thickness of asphalt layers for flexible construction with HBM base is applicable to all permitted base materials.
11. Where induced cracks are required in an HBM, these must be aligned (maximum 100 mm tolerance) with any induced cracks in the underlying construction.
12. EME2 must only be laid over a Class 3 or 4 foundations or a Class 2 foundation that has a surface stiffness modulus of at least 120 MPa at time of construction.

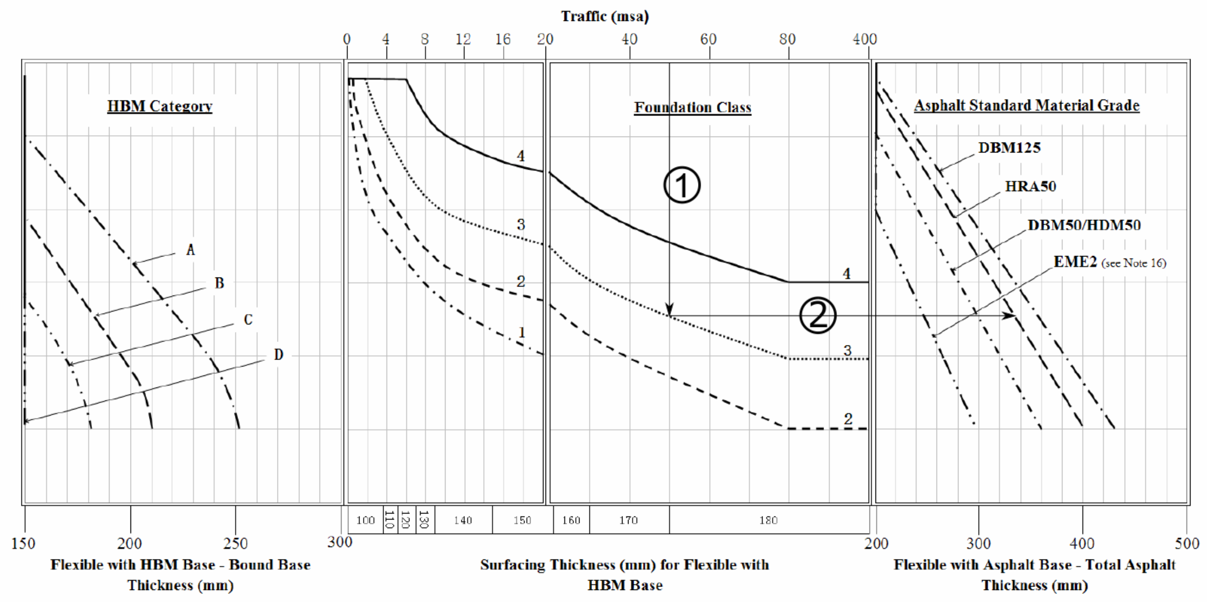


Figure 20: Design Thickness for Flexible Pavements adapted from (Highways Agency, 2006)

2.4.2. Pavement Design procedure for CRM pavements

Cold recycled materials can be utilised in a pavement structure in two ways (Merrill et al., 2004):

- The CRM forms the layer immediately above the foundation and is covered by a bituminous surfacing.
- Bitumen bound cold recycled material can be used as a substitute for conventional hot mix material for inlay treatments where a significant proportion of the existing pavement remains to form part of the rehabilitated pavement.

This report only including pavement design for full depth cold recycling. Figure 21 gives an illustrative view of the pavement design process for CRM. In general, the pavement design process has to differentiate in hydraulically bound and bitumen bound cold recycled structural course. According to the resulting traffic load one pavement design process is applied.

Hydraulically bound materials are classified into one of nine zones labelled H1 to H9. Once classified, the designer selects the thickness design chart for the appropriate foundation class; the thickness for the design traffic can be determined for the curve associated with the material zone. The combination of stiffness and strength is imperative for the design of a hydraulically bound structural course.

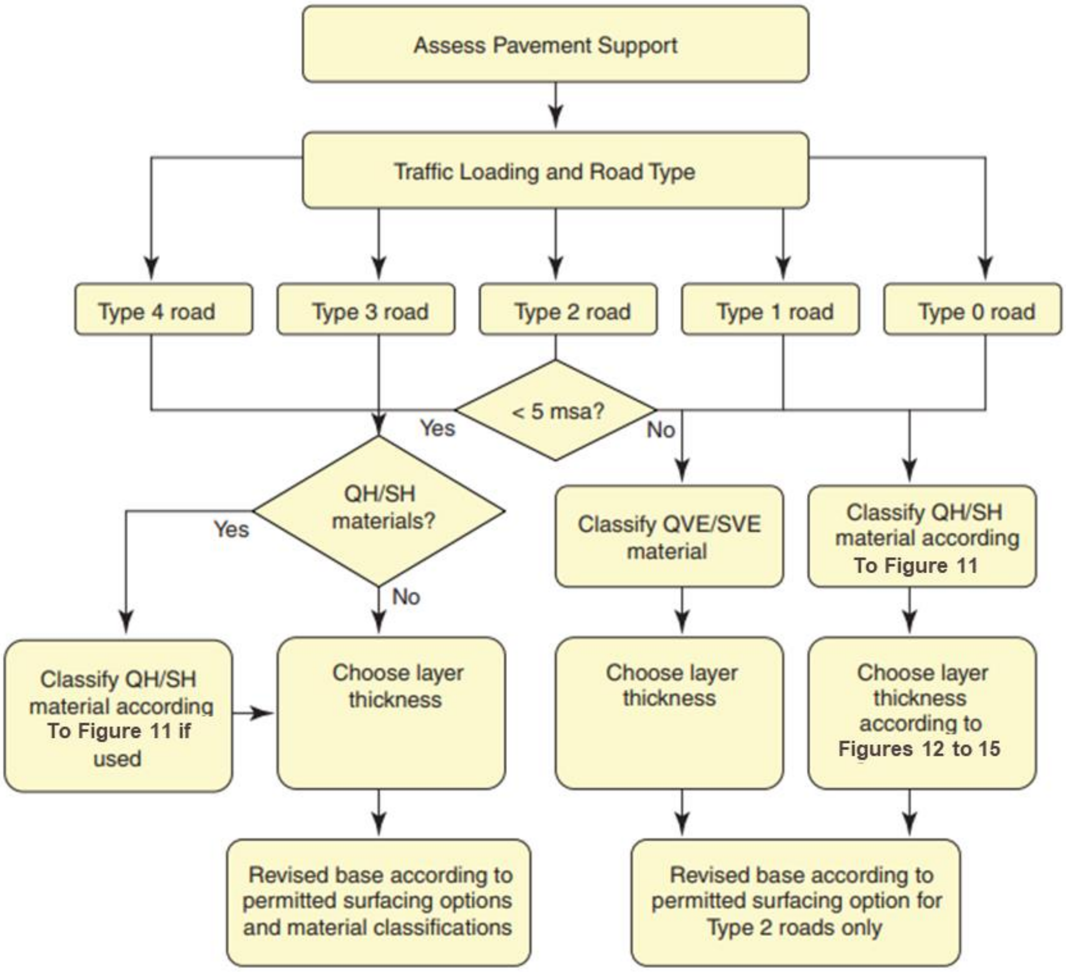


Figure 21: Full depth pavement design process for cold recycled materials (QH quick hydraulic, SH slow hydraulic, QVE quick visco-elastic and SVE slow visco-elastic) adapted from (Highways Agency, 2006)

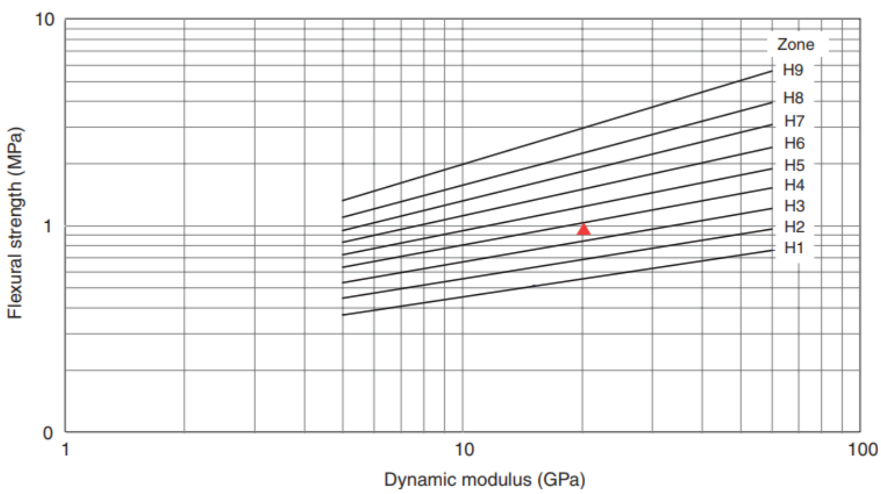


Figure 22: Performance classifications for hydraulically bound recycled materials adapted from (Merrill et al., 2004)

Two different hydraulically bound materials can have the same base thickness for a given level of traffic, provided their flexural strengths compensate for any differences in their levels of stiffness. If stiffness is increased, the traffic-induced tensile stresses in the structural course, which influence performance, also increase. Therefore, the strength would need to be higher to achieve the same performance.

Relationships between elastic stiffness modulus and flexural strength have been developed for equivalent performance and grouped into nine zones. These are shown for materials of stiffness between 5 GPa and 60 GPa in Figure 22.

The triangle illustrates that a material with elastic modulus of 20 GPa and a flexural strength of 0.9 MPa falls into material zone H3. Classification of materials according to Figure 22 requires that the dynamic modulus and flexural strength are known. When these values are not directly measurable, suitable alternative apparatus and transfer functions may be utilised as described below. Using compressive strength tests in accordance with BS EN13286-41 (BSI, 2003a) and using relationships from (Croney, 1997):

$$E_{dyn} = \frac{\log R_f + 0.733}{0.0301} \quad \text{(eq. 9)}$$

$$R_f = 0.11 R_c \quad \text{(eq. 10)}$$

Using the indirect tensile strength and static stiffness in accordance with BS EN13286-42 (BSI, 2003b).

$$E_{dym} = 8.4 + 0.93 E_s \quad \text{(eq. 11)}$$

$$R_f = 1.33 R_{it} \quad \text{(eq. 12)}$$

Where E_{dyn} is dynamic stiffness in GPa, E_S is the static stiffness in GPa, R_f is the flexural strength in MPa, R_c is the compressive strength in MPa, R_{it} is the indirect tensile strength in MPa.

These hydraulic classification zones shown in Figure 22 are used to fix the thickness of the structural course in Figure 23 to Figure 26. For road categories 3 and 4, permitted alternatives to these designs have been defined.

Figure 21 illustrates the use of the chart for a 20 msa design with a quick hydraulic (QH) or slow hydraulic (SH) type material in zone H3; this gives a 150 mm asphalt layer on a 300 mm thick layer of the QH or SH type material.

The asphalt thickness shown in Figure 23 to Figure 26 has been defined so that there is a minimal risk of reflection cracking. It is possible to reduce the thickness of asphalt cover with a corresponding increase in the thickness of hydraulically bound layer without compromising the bearing capacity of the structure; however, such action could increase the risk of reflection cracking occurring in the asphalt layer. Many slow-curing materials are thought to give a low risk of reflection cracking due to the diffuse nature in which naturally forming shrinkage cracks occur; for such materials, substituting asphalt for HBM will result in a minor change in the risk of reflection cracking. Materials that cure quickly are most likely to produce wide, naturally formed shrinkage cracks; for such materials, the substitution of asphalt for hydraulically bound material should be avoided.

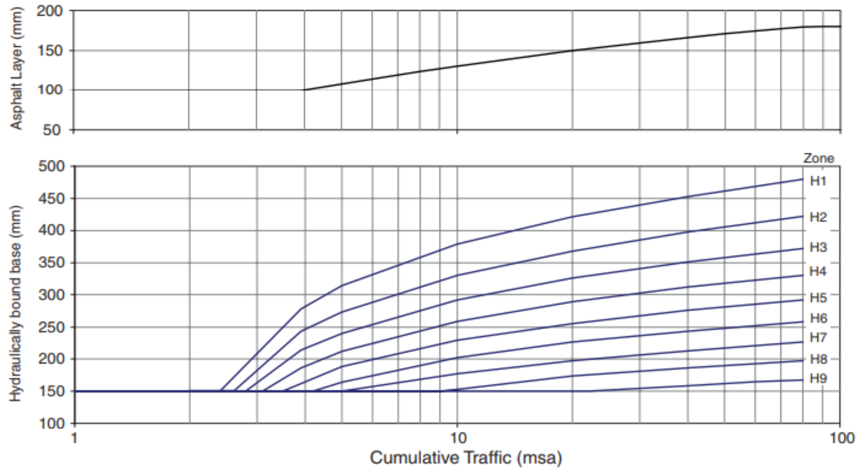


Figure 23: Design thicknesses for road Type 2 and superior roads with a Class 1 foundation (Merrill et al., 2004)

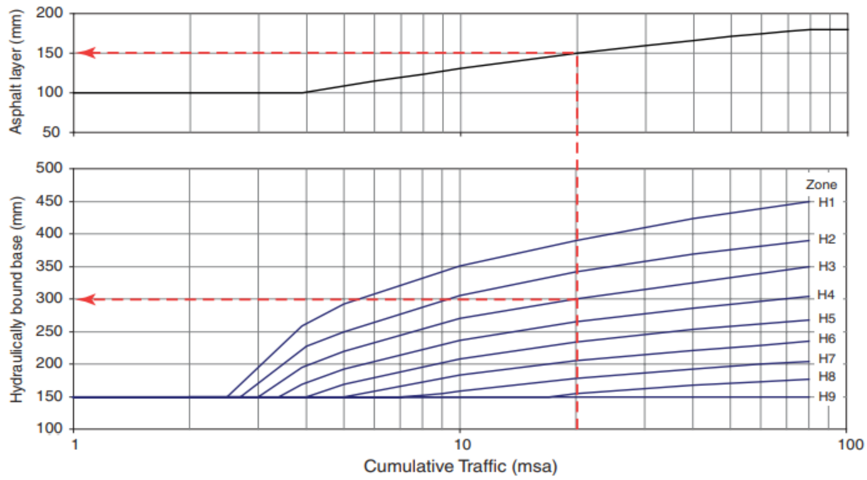


Figure 24: Design thicknesses for road Type 2 and superior roads with a Class 2 foundation (Merrill et al., 2004)

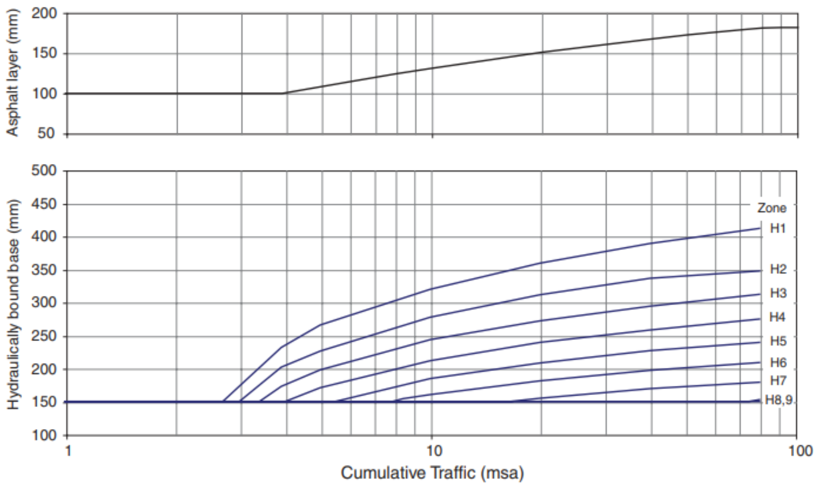


Figure 25: Design thicknesses for road Type 2 and superior roads with a Class 3 foundation (Merrill et al., 2004)

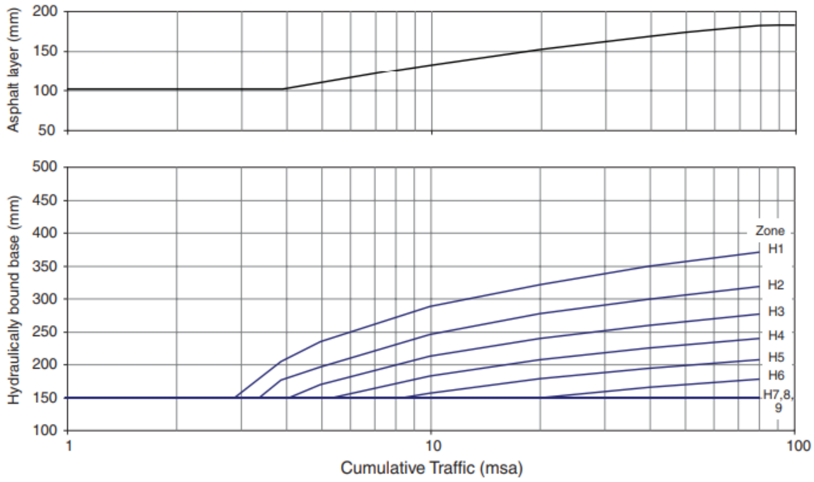


Figure 26: Design thicknesses for road Type 2 and superior roads with a Class 4 foundation (Merrill et al., 2004)

The standard pavement designs shown in Figure 23 to Figure 26 have a minimum recycled structural course layer thickness of 150 mm and a 100 mm thick bituminous surfacing. For low volume roads, these minimum thickness requirements may give excess structural capacity and overly low risk of failure.

Therefore, an alternative design approach for these roads is provided for roads with traffic less than 5 msa that will take advantage of a reduced requirement for structural capacity and improve the economic viability of cold recycling. Potter proposed designs for pavements containing cold in situ recycled materials for Type 2, 3 and 4 roads (see Table 16) (Potter, 1996). These low volume roads can be designed from the formation level with the structural contribution of the subbase layer incorporated into an increased thickness of recycled structural course.

There is no evidence to suggest that these designs have not performed well and, therefore, they have been maintained for all cold recycled materials covered within this guide. These designs cover a range of surfacing options from surface dressing up to a cover of 100 mm of asphalt surfacing. Table 16 shows the thickness design of cold recycled structural courses (incorporating the foundation platform).

Table 16: Thickness of pavements using cold recycled materials as the combined structural course and foundation platform in roads up to 5 msa (Merrill et al., 2004)

Binder type	*Bitumen bound material*								
	Type 2 road			*Type 3 road*			*Type 4 road*		
Surfacing thickness (mm)	*Surface dressing*	*40*	*100*	*Surface dressing*	*40*	*100*	*Surface dressing*	*40*	*100*
Subgrade CBR (%)									
<2	n/r	n/r	n/r	n/r	n/r	n/r	n/r	n/r	n/r
2-4	n/r	n/r	n/r	n/r	310(n/r)	250	320(n/r)	280	195
5-7	n/r	n/r	n/r	330	290	230	300	260	185
8-14	n/r	n/r	300	315	275	215	285	245	160
>15	n/r	n/r	270	285	245	185	255	215	150

n/r: not recommended

For hydraulically bound materials classed as H1, H2, H3 or H4 the adjustments in Table 17 can be applied to the thickness of the materials in Table 16. All permitted alternatives are subject to a 150 mm minimum layer thickness of cold recycled material; a maximum thickness of 300 mm is also recommended. Designs are not provided for weak subgrade conditions that may give inadequate resistance to damage by construction traffic or may not provide the necessary support for adequate compaction of the cold recycled layer.

Table 17: Adjustments for hydraulically bound materials H1 to H4 in roads up to 5 msa (Merrill et al., 2004)

Material	*Thickness adjustment*
H1	1.66
H2	1.45
H3	1.28
H4	1.13
H5 or superior	1.0

The design method for pavements comprising bitumen bound structural courses has two stages: material classification and thickness design. Bitumen bound materials are classified into one of three zones labelled B1, B2 and B3. Once classified, the designer selects the appropriate thickness design chart for the foundation class; the thickness for the design traffic can be determined for the curve associated with the material zone. Bituminous bound recycled structural courses cover a wide range of material compositions. They may contain cement in the case of quick visco-elastic (QVE) type materials or other binders.

As a result of the similarity between conventional HMA and cold mix bituminous material, the entire family of bituminous bound structural courses is treated in a similar fashion to conventional bituminous materials. Therefore, the material performance requirements are given in terms of indirect tensile stiffness modulus as shown in Table 18. The pavement structural courses can be designed according to Figure 27 to Figure 30, depending on the material and foundation classification. It is unlikely that in situ stabilized CRM will satisfy the premium category in Table 18, Zone B3, due to inherent variations in the production process. For road categories Type 2 and below, the bitumen bound structural course is supported by an adequate foundation and is generally surfaced with 100 mm of HMA comprising binder course and surfacing. For road categories Type 3 and 4, permitted alternatives to these designs have been defined. The minimum thickness of surfacing is dependent on the traffic category as given in *Table 19*.

Table 18: Bitumen bound cold recycled material classification (Merrill et al., 2004)

Bitumen bound cold recycled zone	*Minimum long-term stiffness*
B1	1900 MPa
B2	2500 MPa
B3	3100 MPa

Table 19: Requirements for surfacing thickness (Merrill et al., 2004)

Road type category	Minimum thickness of surfacing (mm)
0	100
1	70
2	50

For Type 1 and 2 roads, the thickness of the surfacing placed on top the bitumen bound cold recycled material described in Figure 27 to Figure 30 can be reduced to a minimum of 50 mm with a compensating increase in the thickness of cold recycled structural course. For Class B1 and B2 materials the compensation of the structural course can be determined using the equivalence relationship given in equation 13.

$$\Delta H_{RBase} = 1.3x\Delta H_{Surfacing} \qquad \text{(eq. 13)}$$

With:

$\Delta H_{Surfacing}$ = change in the thickness of bituminous surfacing.

ΔH_{RBase} = change in the thickness of bitumen bound cold recycled base.

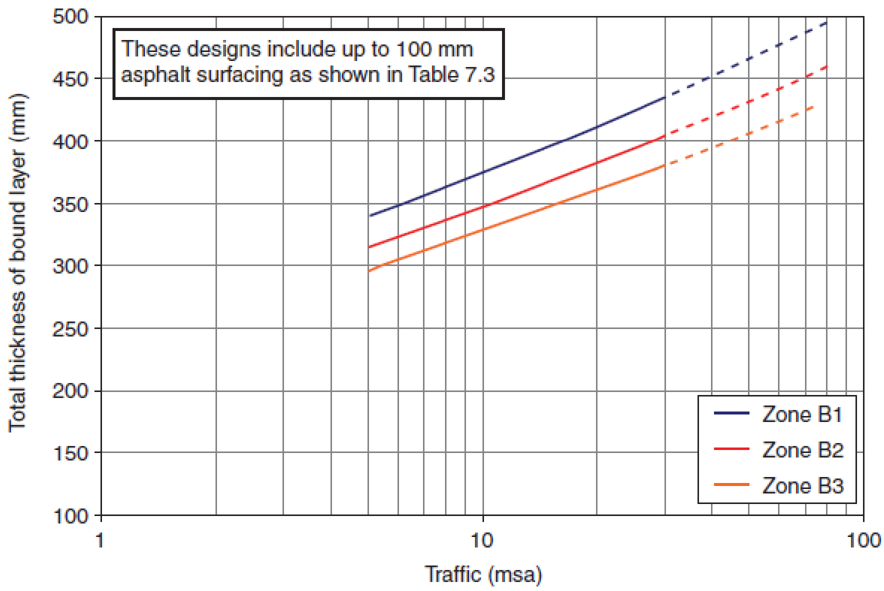


Figure 27: Design curves for bitumen bound cold recycled material (Foundation Class 1) (Merrill et al., 2004)

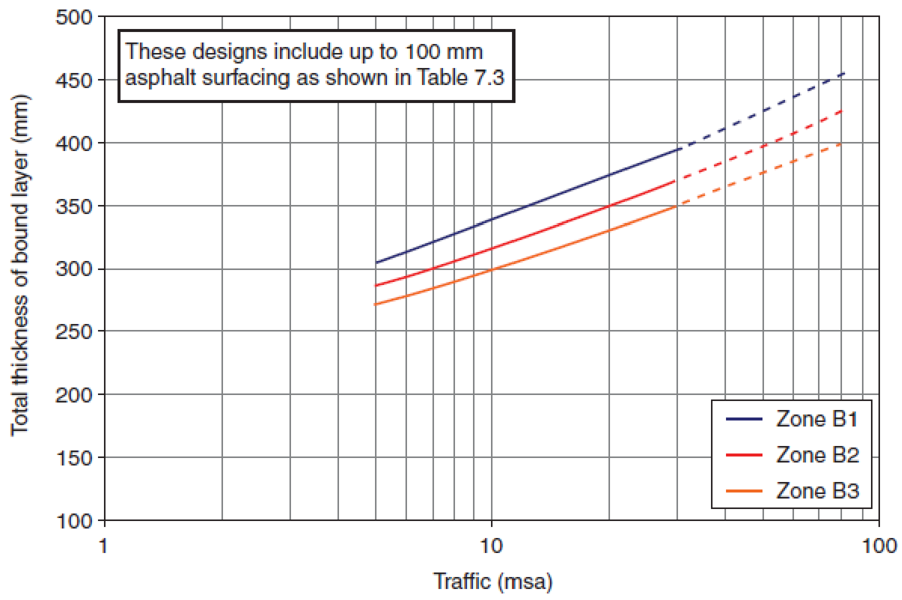


Figure 28: Design curves for bitumen bound cold recycled material (Foundation Class 2) (Merrill et al., 2004)

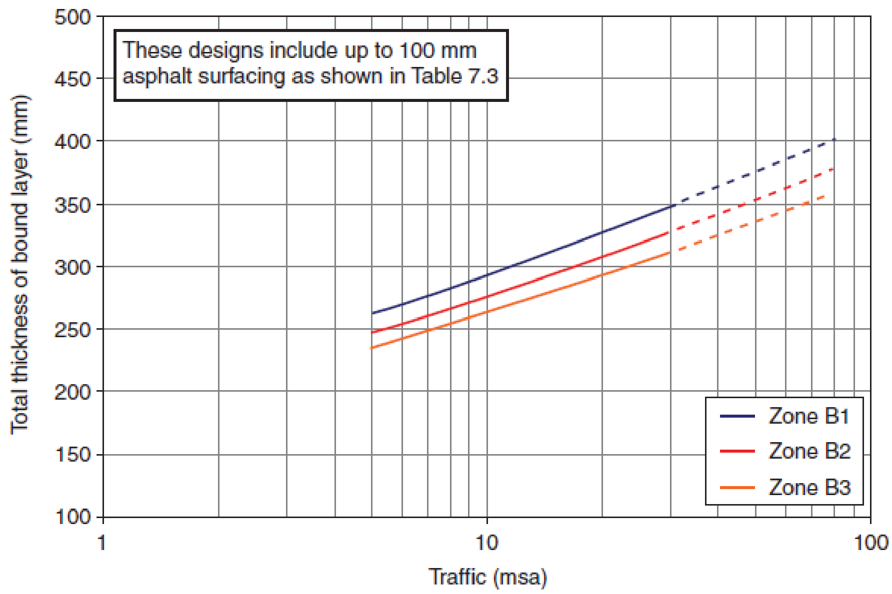


Figure 29: Design curves for bitumen bound cold recycled material (Foundation Class 3) (Merrill et al., 2004)

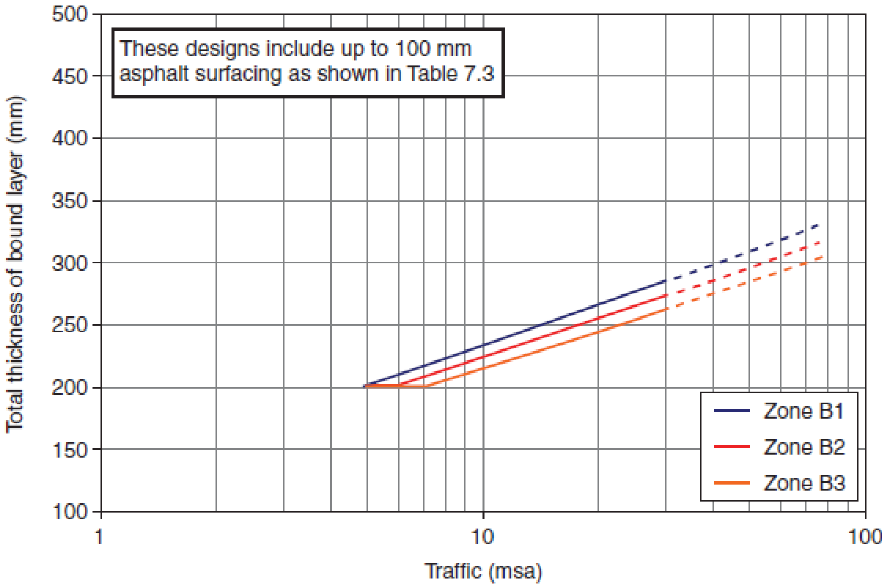


Figure 30: Design curves for bitumen bound cold recycled material (Foundation Class 4) (Merrill et al., 2004)

2.5. French design approach

2.5.1. Pavement design approach for flexible pavements

The French pavement design method for road pavements is described in the French standard NF-P 98-086. It is a mechanistic-empirical design method, based on the following general principles:

- Pavement calculations are performed using a multi-layer linear elastic pavement model (ALIZE Software).
- Pavement properties are considered constant throughout the life of the pavement. For asphalt layer moduli, a constant temperature (called the equivalent temperature) and a constant frequency of 10 Hz are considered.
- The design traffic is converted into an equivalent number of Standard Axle Loads (ESALS), which are defined as dual wheel axles, loaded at 130 kN. Design is generally performed for a period of 30 years for main roads and 20 years for secondary roads.
- The stresses and strains calculated in the pavement layers are compared with acceptable design values, which are mainly function of the number of ESALS and pavement layer properties (in particular fatigue resistance).

The design procedure consists in defining the pavement structure, the design traffic NE, expressed in number of equivalent axle loads, and also the risk of failure r, expressed in percent, which is defined as the probability of failure of the pavement. In practice, when analysing an existing pavement, the risk of failure is assimilated to the percentage of the pavement surface which is damaged. The risk of failure is defined by the road owner, depending on the expected level of service of the road.

With these assumptions, the critical stresses and strains in the pavement structure are calculated and compared with the limit design values. An optimal design is obtained when the critical stresses and strains are slightly lower than the limit values. The design calculations can be used in different ways:

- Knowing the design traffic and choosing a risk of failure, it is possible to calculate the design thickness of the pavement layers.
- Alternatively, for an existing pavement, knowing the layer thicknesses, and choosing the risk of failure, it is possible to calculate the design life (number of ESALS leading to failure NE),
- Finally, knowing the pavement characteristics, and the traffic, it is possible to calculate the expected risk of failure of the pavement.

Types of pavement structures

The French pavement design method considers 6 main types of pavement structures, which main characteristics are described in Figure 31.

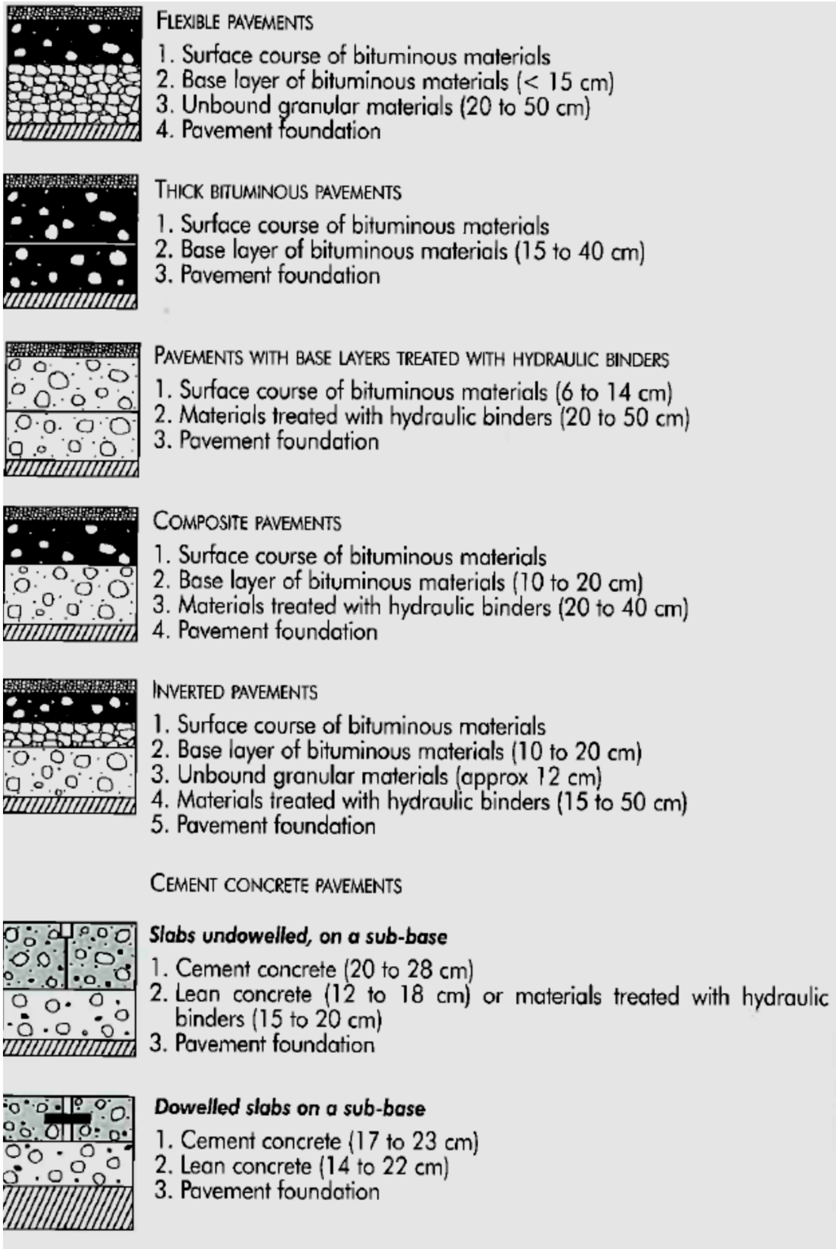


Figure 31: Main types of pavement structures used in France

Description of traffic

Pavements are generally designed for a period of 30 years for main roads and 20 years for secondary roads.

The traffic of heavy goods vehicles (HGV) (corresponding to vehicles with a total weight > 3.5 tons) expected on the pavement during its design period, expressed by the cumulative number of HGV, N_{PL}, is converted into an equivalent number NE of ESALS, to be used for the design calculations (single axle with dual wheels, loaded at 130 kN). This equivalent number NE is determined by the following equation:

$$NE = N_{PL} \times CAM \quad \text{(eq. 14)}$$

With:

CAM = the coefficient of average aggressiveness of traffic

The value of CAM depends on the type of pavement structure, the type of material and the precise composition of the HGV traffic (types of vehicles, axle loads). Two approaches can be used to determine the value of CAM for a given project:

- If vehicle type and axle load data are available (for example if data from a weight in motion system is available), an incremental damage calculation, taking into account the load characteristics of all the vehicles, can be done with the pavement design software, to calculate the exact value of CAM. The calculation of CAM is detailed in the standard NF P 98 160. The value of CAM must be calculated in particular for HGV which do not respect the French Highway Code or European Directive 96/53/EC, or in areas outside of the usual context: Activity Zones (ZAC), access routes to Industrial Areas (ZI) or port areas.
- When load information for calculating the CAM coefficient is not available, empirical values of CAM, provided in the NF P 98-086 standard can be used. The values of CAM depend on the type of road, type of material and level of traffic, as detailed below.

Traffic classes

In addition to the cumulative number of heavy vehicles (N_{PL}) the design method also defines traffic classes, based on the average annual daily traffic (AADT) of HGV. The traffic classes, defined in Table 20, are used for the calculation of the CAM, and also for the selection of types of materials and pavement structures, according to traffic level. For example, structures with a single bituminous wearing course and a granular base are allowed only for traffic levels up to T3.

Table 20: Definition of traffic classes Ti, based on the heavy goods vehicle traffic used for design, AADT (NPL/day)

Class	T5	T4	T3-	T3+	T2-	T2+	T1-	T1+	T0-	T0+	TS-	TS+	TEX
AADT for design	1–25	25–50	50–85	85–150	150–200	200–300	300–500	500–750	750–1200	1200–2000	2000–3000	3000–5000	>5000
Geometric average for each class	5	35	65	115	175	245	390	615	950	1.550	2.450	3.875	5.920

Values of the average coefficient of aggressiveness of traffic (CAM)

Main rural roads

These are generally inter-urban type pavements (like motorways or 2x2-lane roads), carrying long distance traffic, with a significant percentage of HGV. The CAM values for these roads are given in Table 21.

Table 21: CAM according to type of material for main rural road pavements

Type of material	CAM value
Bituminous materials	0.8
Hydraulically bound materials and cement concretes	1.3
Formation level, UGM	1

Secondary roads

These pavements correspond to a local road network: suburban roads, roads connecting cities, open countryside, tourist routes, etc. The percentage of Heavy vehicles, and their average load are lower. The CAM values for secondary roads depend both on the class of traffic and type of material (Table 22).

Table 22: CAM according to type of material and traffic level for secondary road pavements

Type of material	T5	T4	T3-	T3+	T2, T1, T0
Bituminous materials	0,3	0,3	0,4	0,5	0,5
Hydraulically bound graded materials and cement concretes	0,4	0,5	0,6	0,6	0,8
Treated soils	0,4	0,5	0,7	0,7	0,8
Formation level, UGM	0,4	0,5	0,6	0,75	1

Urban pavements

These pavements correspond to the urban road network. The corresponding CAM values are given in Table 23.

Table 23: CAM according to type of road and type of material for pavements in an urban environment

Type of material	Residential areas	Urban avenues or boulevards	Main roads with heavy traffic
Bituminous materials	0,1	0,1	Refer to CAM for secondary road pavements
Hydraulically bound materials and cement concretes	0,1	0,2	
Formation level, GNT	0,1	0,2	

Remark: for roundabouts, it is considered that heavy vehicle loads are more aggressive, and an increase of the thickness of the pavement foundation is applied.

Subgrade characteristics

The French pavement design method considers 5 classes of subgrades, (PF1 to PF4). The long-term bearing capacity (or elastic modulus) values corresponding to each class, used in structural pavement design calculations are indicated in Table 24.

Table 24: Long-term bearing capacity class of the subgrade

Modulus	20 MPa ≤ E < 50 MPa	50 MPa ≤ E < 80 MPa	80 MPa ≤ E < 120 MPa	120 MPa ≤ E < 200 MPa	E ≥ 200 MPa
Class of subgrade	PF1	PF2	PF2qs	PF3	PF4

Subgrade correction coefficient considered in the design

If the bound pavement layers rest directly on the subgrade, a correction coefficient k_s, taking into account the quality of the subgrade, is introduced in the design (as explained below. The values of k_s depend on the modulus of the subgrade and are given in Table 25.

Table 25: Values of ks taken into consideration as a function of the bearing capacity of the subgrade

Bearing capacity or Modulus	$E < 50$ MPa	$50 \text{ MPa} \leq E < 80$ MPa	$80 \text{ MPa} \leq E < 120$ MPa	$E \geq 120$ MPa
k_s	1/1,2	1/1,1	1/1,065	1

Material characteristics

The mechanical characteristics (Elastic modulus, fatigue behaviour) of pavement materials required for design may be defined by two methods:

i. By adopting reference values of design parameters. For each class of materials, the design standard NF P 98-086 defines reference values of the different material parameters (modulus, fatigue parameters, etc..). These values shall then be verified by tests on the site materials.
ii. By using values resulting from laboratory tests on the representative projected site materials, prepared with the required void percentage.

Bituminous materials

The design standard distinguishes 3 main types of bituminous materials:

- Base course bituminous mixes (Grave-bitume or GB),
- High modulus mixes (Enrobés à module élevé or EME),
- Different types of materials for wearing courses.

Base course bituminous mixes (GB)

These bituminous mixes are divided into three performance classes (GB2, GB3 and GB4). For each class, the mechanical parameters to use for design are defined in Table 26. The significance of the different fatigue parameters is defined below.

Table 26: Minimum and maximum mechanical characteristics of EB-GB

	Class	2	3	4
Minimum values	Modulus at 15°C – 10 Hz or 0.02 s (MPa)	9000	9000	11.000
	ε_6 (µdef)	80	90	100
Maximum values	Modulus at 15 °C – 10 Hz or 0.02 s (MPa)	11.000	11.000	14.000
	ε_6 (µdef)	90	100	115
Fixed values to be applied	- 1/b	5	5	5
	S_N	0,3	0,3	0,3
	k_c	1,3	1,3	1,3

High modulus bituminous mixes (EME)

High-modulus mixes are divided into two performance classes (EME1 and EME2). For each class, the mechanical parameters to use for design are defined in Table 27. The significance of the different fatigue parameters is defined in below.

Table 27: Minimum and maximum mechanical characteristics of EB-EME

	Class	1	2
Minimum values	Modulus at 15°C – 10 Hz or 0.02 s (MPa)	14.000	14.000
	ε_6 (µdef)	100	130
Maximum values	Modulus at 15°C – 10 Hz or 0.02 s (MPa)	17.000	17.000
	ε_6 (µdef)	115	145
Fixed values to be applied	– 1/b	5	5
	S_N	0,3	0,25
	k_c	1	1

Bituminous materials for thick binder and wearing courses EB-BBSG, EB-BBME and SMA

EB-BBSG and EB-BBMEs are divided into three performance classes. For each class, the mechanical parameters to use for design are defined in Table 28. The design method does not include fatigue verification of the wearing and binder courses.

Table 28: Minimum and maximum mechanical characteristics of EB-BBSG, EB-BBME and SMA

		EB-BBSG		EB-BBME		SMA
	Class	1	2 and 3	1	2 and 3	
Minimum values	Modulus at 15°C – 10 Hz (MPa)	5.500	7.000	9.000	11.000	3.500
	ε_6 (µdef)	100	100	100	100	100
Maximum values	Modulus at 15°C – 10 Hz (MPa)	9.000	11.000	11.000	14.000	6.000
	ε_6 (µdef)	115	130	115	130	130
Fixed values to be applied	– 1/b	5	5	5	5	5
	S_N	0,25	0,25	0,25	0,25	0,25
	k_c	1,1	1,1	1,1	1,1	1,1

Bituminous materials for thin binder and wearing courses BBM, BBTM, BBDr and ACR

For each type of material, the mechanical parameters to use for design are defined in Table 29. For these thin layers, the design method does not require to perform modulus and fatigue tests. Only fixed modulus values are taken into account in design for these materials.

Table 29: Mechanical characteristics for thin binder and wearing courses

	Thin BB (BBM)	Very thin BB (BBTM)	Porous BB (BBDr)	Road mastic asphalts (ACR)
Modulus at 15°C (MPa)	5.500	3.000	3.000	5.500

Hydraulically bound materials

The design standard distinguishes 3 main types of hydraulically bound materials:

- Hydraulically bound aggregates,
- Hydraulically bound sands,
- Hydraulically bound soils

According to the standards in the series NF EN 14427 (1 to 5 and 15), these materials are divided into 5 mechanical performance classes, noted T1 to T5, depending on their average Young's modulus values and direct tensile strength, evaluated at 360 days.

The design parameter values (Young's modulus E, fatigue parameters) of these materials can be established according to 3 procedures:

i. By adopting the reference values provided in the pavement design standard, according to the type and class of material,
ii. Using modulus and direct tensile strength Rt measurements, determined by laboratory tests,
iii. Using direct tensile modulus and fatigue strength σ_6 measurements.

In case i), the values considered must be verified before the start of works.

Hydraulically bound aggregates and compacted road concretes

For each class of material, reference parameters to be used for design are defined in Table 30.

Table 30: Reference design parameter values for hydraulically and pozzolan bound aggregates and compacted road concretes compliant with standards

<table>
<tr><th>Material</th><th>E (MPa)</th><th>σ_6 (MPa)</th><th>– 1/b</th><th>S_N</th><th>$S_{h\ (m)}$</th><th>R_t min (MPa)</th></tr>
<tr><td>Cement-bound graded aggregate (T3)
Hydraulically bound road aggregate (T3)
Hydraulic fly ash bound aggregate (T3)</td><td>23.000</td><td>0,75</td><td>15</td><td>1</td><td rowspan="9">0,03</td><td>1,15</td></tr>
<tr><td>Cement-bound graded aggregate (T4)
Hydraulically bound road aggregate (T4)</td><td>25.000</td><td>1,20</td><td>15</td><td>1</td><td>1,80</td></tr>
<tr><td>Slag-bound graded aggregate (T2)
Pre-crushed slag-bound aggregate (lime dope) (T2)</td><td>15.000</td><td>0,60</td><td>12</td><td>1</td><td>0,90</td></tr>
<tr><td>Pre-crushed slag-bound aggregate (T3)
(sulphate or calcium dope)</td><td>20.000</td><td>0,70</td><td>14</td><td>1</td><td>1,05</td></tr>
<tr><td>Slag-fly ash-bound aggregate - Lime (T3)</td><td>22.000</td><td>0,80</td><td>13</td><td>1</td><td>1,2</td></tr>
<tr><td>Silico-aluminate fly ash bound aggregate - Lime (T4)</td><td>30.000</td><td>1,40</td><td>16</td><td>1</td><td>2,10</td></tr>
<tr><td>Pozzolanic-lime aggregate (T2)</td><td>15.000</td><td>0,6</td><td>12</td><td>1</td><td>0,9</td></tr>
<tr><td>Compacted road concrete (T4)</td><td>25.000</td><td>1,20</td><td>15</td><td>1</td><td>1,80</td></tr>
<tr><td>Compacted road concrete (T5)</td><td>28.000</td><td>1,85</td><td>15</td><td>1</td><td>2,80</td></tr>
</table>

Hydraulically bound sands

For each class of material, reference parameters to be used for design are defined in Table 31.

Table 31: Reference design parameter values for hydraulically bound sands compliant with the standards

<table>
<tr><th>Material</th><th>Class Ti</th><th>E (10^3 MPa)</th><th>σ_6 (MPa)</th><th>– 1/b</th><th>S_N</th><th>S_h</th></tr>
<tr><td rowspan="3">Slag bound sand

Pozzolanic lime sand</td><td>T3</td><td>12.500</td><td>0,65</td><td rowspan="3">10</td><td rowspan="3">0,8</td><td rowspan="3">0,025</td></tr>
<tr><td>T2</td><td>8.500</td><td>0,43</td></tr>
<tr><td>T1</td><td>3.700</td><td>0,18</td></tr>
<tr><td rowspan="3">Cement-bound sand
Silico-aluminate-fly ash-lime-bound sand
Hydraulically bound road sand</td><td>T3</td><td>17.200</td><td>0,75</td><td rowspan="3">12</td><td rowspan="3">0,8</td><td rowspan="3">0,025</td></tr>
<tr><td>T2</td><td>12.000</td><td>0,50</td></tr>
<tr><td>T1</td><td>5.000</td><td>0,21</td></tr>
<tr><td>Slag fly ash lime bound sand</td><td>T2</td><td>8.500</td><td>0,43</td><td>10</td><td>0,8</td><td>0,025</td></tr>
<tr><td>Hydraulic fly ash bound sand</td><td>T1</td><td>5.000</td><td>0,21</td><td>12</td><td>0,8</td><td>0,025</td></tr>
</table>

Hydraulically bound soils

Given the diverse origins of soils and the variety in binder composition and dosing due to considerations of immediate stability and long-term mechanical performance, a specific laboratory study must be carried out each time on hydraulically bound soils, in accordance with standard NF P 98-114-3.

Contrary to hydraulically bound aggregates and sand, there are no reference values for E and σ_6 of soil. These parameters must therefore be determined through laboratory testing.

Table 32 gives the values of the other calculation parameters to be used for treated soils.

Table 32: Reference design parameter values for hydraulically bound soils, compliant with the standards

Material	– 1/b	S_N	S_h	$R_{t\ min}$ (MPa)
Fine soil (T1, T2, T3)	11	0,8	0,04 if treatment on site 0,025 if treatment at plant	0,2
Sand type soil (T1, T2, T3, T4)				
Gravel type soil (T1, T2, T3, T4)		1	0,05 if treatment on site 0,03 if treatment at plant	

B, S_N and S_h are fatigue parameters, defined above.

<u>Unbound granular materials</u>

The mechanical properties of unbound granular materials (UGM) to be considered in the design of structures depend on:

- their classification into 3 categories of quality, CG1, CG2 and CG3. These categories depend on grading and aggregate characteristics;
- the type of pavement structure;
- the pavement layer (base or subbase).

The Poisson's ratio of UGMs is taken equal to 0.35.

The Young's modulus E_{UGM} to be used for design is determined according to Table 33. It depends on the parameters k and E_{max}, defined in Table 34, according to the category of UGM and the type of structure concerned.

Table 33: Determination of the elastic modulus of unbound granular layers for design calculations

Type of course	Young's modulus
Base course of flexible pavement Or UGM layer of inverted pavement	$E_{GNT} = E_{max}$
Subbase course of flexible pavement Or Subbase course of bituminous pavement	- Subdivision of the UGM layer into 25 cm thick sublayers, from the bottom (last sub-layer may be thinner). - Calculation of the modulus of each sublayer indexed from bottom to top, according to the relationship: For i=1; E_{UGM} (1) = Min (k x $E_{formation\ level}$; E_{max}) For i>1; E_{UGM} (i) = Min (k x E_{GNT} (i-1); E_{max})

Table 34: Determination of parameters k and E_{max} used to define the elastic modulus of UGM layers

Category	CG1	CG2	CG3
Flexible pavements			
k	3	2,5	2
E_{max} (MPa)	600	400	200
Bituminous pavements with UGM subbase			
k	3		
E_{max} (MPa)	360		
Inverted pavements			
E_{max} (MPa)	480	Not suitable	Not suitable

Design approach

The pavement design approach for pavement structures identifies three damage mechanisms:

- Damage due to fatigue of the layers of bituminous materials by repeated bending and tension at their base (deformation criterion).

- Damage due to fatigue of the layers of hydraulically bound materials or cement concrete, by repeated bending and tension at their base (stress criterion).

- The accumulation of permanent deformations within the layers of unbound material under the effect of repeated vertical compression (vertical deformation criterion)

Each of these mechanisms leads to calculations of internal stresses and strains in the pavement layers, and their comparison with acceptable design values, function of the design traffic and material properties.

The acceptable design values are defined as follows:

- The acceptable damage of bituminous materials by fatigue under repeated loads is evaluated using the amplitude of the acceptable extension deformation, $\varepsilon_{t\ ad}$.
- The acceptable damage of hydraulically bound materials and cement concretes by fatigue under repeated loads is evaluated using the amplitude of the acceptable tensile stress, $\sigma_{t\ ad}$.
- The accumulation of acceptable permanent deformation of unbound materials (including that of the formation level) under repeated loads is evaluated using the amplitude of the acceptable vertical contraction deformation, $\varepsilon_{z\ ad}$.

Acceptable strain for bituminous materials, ε_{t_ad}

The limit tensile strain $\varepsilon_{t,ad}$ at the base of the bituminous layers is defined by equation 15, for an equivalent design temperature θ_{eq}:

$$\varepsilon_{t\ ad} = \varepsilon_6(10°C, 25Hz)(\frac{NE}{10^6})^b k_r k_s k_c k_\theta \qquad \text{(eq. 15)}$$

With:

$\varepsilon_6(10°C, 25Hz) =$ the tensile strain leading to fatigue failure for 1 million load cycles, determined from laboratory two-point bending fatigue tests

NE = number of equivalent axle loads

k_c = calibration factor, function of the type of asphalt material

k_s = correction factor for subgrades of low bearing capacity. $1/k_s$ = 1,065 for E_{Soil} = 80 MPa; $1/k_s$ = 1,1 for E_{soil} = 50 MPa.

k_r = coefficient which adjusts the working strain value to a design risk r. k_r takes into account the variability of the pavement layer thickness (standard deviation Sh) and the variability of the fatigue properties (standard deviation SN, determined from the laboratory fatigue tests).

$$k_r = 10^{-ub\delta} \qquad \text{(eq. 16)}$$

With:

u = reduced centered variable associated with the risk of failure r (expressed in percent).

b = slope of the material fatigue law (bi-logarithmic law)

δ = standard deviation of the distribution of logN at failure.

$$\delta = [SN^2 + \frac{c^2}{b^2} Sh^2]^{0.5} \qquad \text{(eq. 17)}$$

With:

c = coefficient linking the variation in strain to the random variation of the pavement thickness. For usual structures, c is taken equal to 0.02cm^{-1}.

Sh = 0,01 m for usual construction conditions.

$K\theta$ = coefficient taking into account the variation of fatigue properties with temperature, defined by:

$$k_\theta = \sqrt{\frac{E\ (10°c, 10Hz)}{E\ (\theta_{eq}, 10\ Hz)}} \qquad \text{(eq. 18)}$$

With:

E (10°c, 10 Hz) the elastic modulus of the material determined at 10 °c and 10 Hz and E (θ_{eq}, 10Hz) the elastic modulus of the material determined at the design temperature θ_{eq} and 10 Hz.

Acceptable stress for hydraulically bound materials and cement concretes, σ_{t_ad}

The acceptable stress for hydraulically bound materials and cement concretes is calculated using equation:

$$\sigma_{t\ ad} = \sigma_6 . (\frac{NE}{10^6})^b k_r k_s k_c k_d \quad \text{(eq. 19)}$$

With:

σ_6 = tensile stress value leading to fatigue failure for 10^6 cycles, determined by a two-point bending fatigue test; Rules also exist to estimate this quantity using direct or indirect tensile tests.

b = slope of the fatigue law of the material, determined using the same fatigue test, by log-log linearisation between 10^5 and 10^7 cycles ($-1 < b < 0$)

NE = number of cycles to failure, considered equal to the reference number of equivalent axle loads.

k_c = calibration factor, function of the type of hydraulic material.

k_s = correction factor for subgrades of low bearing capacity. $1/k_s$ = 1,065 for E_{Soil} = 80 MPa; $1/k_s$ = 1,1 for E_{soil} = 50 MPa.

k_d = coefficient of stress concentration, introduced to take into account the discontinuous nature of pavements with layers made of concrete or hydraulically bound materials (discontinuities due to joints between slabs or shrinkage cracks). This coefficient only applies to materials with a higher modulus: cement concretes, compacted road concretes and class T4 and T5 hydraulic materials. For other materials, the increase is negligible and k_d =1.

Acceptable deformation criterion for unbound materials and pavement subgrade, $\varepsilon_{z\ ad}$

The acceptable vertical deformation for unbound material layers and pavement subgrade is calculated using equation 20:

$$\varepsilon_{z\ ad} = A \cdot NE^b \quad \text{(eq. 20)}$$

With:

A, b = parameters dependent on the level of traffic, type of material and structure, ($-1 < b < 0$)

NE = number of reference axle loads.

Risk coefficient k_r

As explained previously, an important parameter of design is the risk coefficient k_r, which is a probabilistic coefficient, defining the probability of failure of the pavement at the end of the design; This coefficient is a kind of safety factor, taking into account the variability of pavement layer thickness, and of fatigue properties. In the design method, values of k_r are defined depending on the type of pavement structure, and level of traffic. Values of k_r to be used for interurban roads are defined in Table 35. It can be seen that the higher the level of traffic, the lower is the specified level of risk.

Table 35: Common risk values (in %) according to traffic and structure, for interurban roads

<table>
<tr><th colspan="2">Type of structure</th><th>TEX</th><th>TS</th><th>T0</th><th>T1</th><th>T2</th><th>T3</th><th>T4</th><th>T5</th></tr>
<tr><td rowspan="2">Semi-rigid bituminous structures</td><td>MB</td><td>1,0</td><td>1,0</td><td>2,0</td><td>5,0</td><td>12,0</td><td>25,0</td><td>30,0</td><td>30,0</td></tr>
<tr><td>MTHB</td><td>1,0</td><td>1,0</td><td>2,5</td><td>5,0</td><td>7,5</td><td>12,0</td><td>25,0</td><td>25,0</td></tr>
<tr><td rowspan="2">Inverted structures</td><td>MB</td><td>1,0</td><td>1,0</td><td>2,0</td><td>5,0</td><td>12,0</td><td>25,0</td><td>30,0</td><td>30,0</td></tr>
<tr><td>MTHB</td><td>1,0</td><td>2,0</td><td>5,0</td><td>10,0</td><td>15,0</td><td>24,0</td><td>25,0</td><td>25,0</td></tr>
<tr><td rowspan="2">Mixed structures</td><td>MB</td><td>1,0</td><td>1,0</td><td>2,0</td><td>5,0</td><td>12,0</td><td>25,0</td><td>30,0</td><td>30,0</td></tr>
<tr><td>MTHB</td><td>1,0</td><td>2,0</td><td>3,0</td><td>10,0</td><td>20,0</td><td>35,0</td><td>50,0</td><td>50,0</td></tr>
<tr><td rowspan="3">Concrete structures</td><td>Base / wearing course</td><td>1,0</td><td>1,0</td><td>2,8</td><td>5,0</td><td>7,5</td><td>15,0</td><td>25,0</td><td>25,0</td></tr>
<tr><td>Foundation except BAC and BCg</td><td>2,0</td><td>2,0</td><td>5,6</td><td>10,0</td><td>15,0</td><td>25,0</td><td>25,0</td><td>25,0</td></tr>
<tr><td>Foundation for BAC and BCg</td><td>50,0</td><td>50,0</td><td>50,0</td><td>50,0</td><td>50,0</td><td>50,0</td><td>50,0</td><td>50,0</td></tr>
</table>

Additional values of k_r for urban pavements are given in Table 36.

Table 36: Common risk values (in %) in an urban environment

	Residential areas	Urban avenues or boulevards	Main roads with heavy traffic
Risk	25%	15 to 20%	5%

<u>General Design procedure</u>

The general approach for the design of a pavement can be divided in 6 steps:

Step 1: Preliminary design

A first initial pavement structure is defined, depending on the type of materials, and level of traffic, by experience, or by reference to similar situations

Step 2: Structural calculation

The stresses and strains in the pavement structure defined in stage 1 are calculated using the multilayer linear elastic pavement model (ALIZE software), under the reference 130 kN axle load.

Step 3: Comparison of calculated stresses and strains with the acceptable design values

This step consists in comparing the calculated stresses or strains with the acceptable design values, which depend on the type of material (defined previously). The objective is to obtain calculated values which are just slightly lower than the acceptable values. If this is not the case, the layer thicknesses are adjusted, and a new calculation is performed (Step 2). This process is repeated until a satisfactory design is obtained

Step 4: Choice of the final layer thicknesses

After completing step 3, it is verified that the layer thicknesses satisfy:

- Technological restrictions concerning the minimum and maximum thickness defined for each type of pavement material (needed in particular to achieve good compaction and evenness)
- Minimum thicknesses to ensure good protection of hydraulically bound materials (in particular to avoid reflective cracking).

These minimum and maximum thicknesses are specified in standard NF P 98-086, for the different types of materials. If necessary, layer thicknesses can be adjusted to satisfy these criteria.

Step 5: verification of frost-thaw resistance

A separate calculation is performed to verify the frost-thaw resistance of the pavement. It is not presented in detail here, but the principle consists in comparing:

- The atmospheric frost index chosen as reference (IR) for the design site.
- The acceptable frost index that the pavement can withstand (IA)

The acceptable frost index IA depends on:

- The class of frost susceptibility of the subgrade, which defines the quantity of frost allowed on the surface of the subgrade
- A 1D thermal calculation (or a simplified equation), which is used to define the quantity of frost transmitted to the subgrade. This quantity depends on the thickness and thermal properties of the frost-resistant materials above the subgrade

The design is validated if IR is lower than the IA. If this condition is not satisfied, the thickness of frost-resistant materials is increased, until the condition is satisfied.

Step 6: Definition of the pavement cross-section

The total pavement thickness obtained by the design corresponds to the nominal thickness, which is the thickness at the right edge of the most heavily trafficked lane. Transversal variations in layer thickness can be defined, according to the traffic per lane, the geometry of the road, compensations for cross fall between the subgrade and the surface layer.

Design of Low traffic pavements

The French design method defines a specific approach for the design of low traffic pavements (which are defined as pavements with a design traffic lower than 250 000 ESALS), and other pavements, for heavier traffics

For low traffic pavements, which generally consist of a thin bituminous wearing course over a granular base, the only design criterion which is considered is the acceptable vertical deformation criterion for unbound materials and subgrade: $\varepsilon_{z\,ad} = A.NE^{b}$. For theses pavements, the fatigue criterion is not considered appropriate, and therefore, the thickness of the wearing course is determined by an empirical approach, using the design chart of Figure 32, in function of the level of traffic.

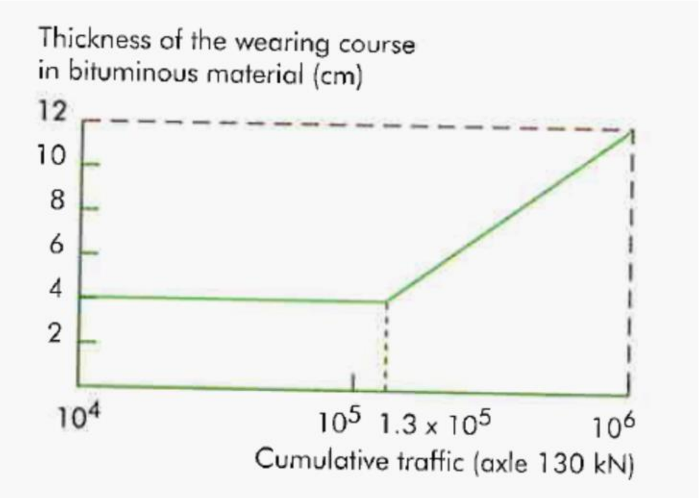


Figure 32: Design chart for defining the thickness of the bituminous wearing course

2.5.2. Pavement Design procedure for CRM pavements

The French pavement design method does not define a specific design procedure for pavements with CRM layers. The method only proposes specific mechanical performance classes for cold bituminous mixes (called Grave-Emulsions or GE). Three types are considered, depending on the content of bituminous binder (see Table 37):

- Type 1 is used for reprofiling works. Their maximum grain size can be 10 or 14 mm
- Types 2 and 3 are used for base layers. Their maximum grain size can be 10, 14 or 20 mm

Minimum binder contents for each cold mix type are defined in Table 37.

Table 37: Minimum binder content of cold bituminous mixes (GE)

GE type	Type 1	Type 2	Type 3
Minimum binder content (%)	4,2	3,2	2,8

<u>Mechanical properties for design</u>

Presently, there are few results on fatigue behavior of emulsion bound mixes, and the current design approach for pavements with cold mix layers consists in considering only the design criterion limiting the vertical strain at the top of the subgrade, and no fatigue criterion for the cold mix layers. Thus, only the elastic modulus of the mixes is required for design calculations.

Reference elastic modulus values for cold bituminous mixes to be used for design are defined in Table 38.

Table 38: Reference elastic moduli of cold bituminous mixes (GE)

GE type	Type 1	Type 2	Type 3
Reference Elastic modulus (MPa)	3.000	2.000	-

3. Comparison of the national approaches including CRM

As the mechanical design approaches are not yet fully accepted and failure modes for CRM are not implemented within the procedures, existing empirical pavement design methodologies in several countries were compared. As already the standard design procedures applied for conventional pavement materials (especially HMA) vary considerably from one country to another these approaches are compared in a first step. Afterwards, special approaches for CRM are introduced. For comparing the design procedures both for pavements with base layers from HMA and CRM, the results from the previous chapter are assessed.

3.1. General structure of flexible pavements

Within Europe almost each country has an individual approach for pavement design procedure. However, despite varying tools for considering traffic loading and subground conditions, the general pavement designs are similar with stiffness, bearing capacity and resistance against cracking and/or permanent deformation increasing from the bottom to the top of the pavement. Table 39 shows the general pavement structures and applied technical terms from the five CRABforOERE partner countries. These cover the range of climatic conditions within Europe.

On top of the existing subgrade of low bearing capacity (e.g. clay), a first subbase layer (1) is constructed either by applying a granular base layer or hydraulically bound layer, later often mixed in place. The subbase layer (2) is applied in order to provide increasing bearing capacity as well as to allow for frost-resistant and water-draining pavement bases.

Table 39: National terms and applied materials for pavement layers according to national pavement design guides

Layer type	UK[1]	SWE[2]	ITA[3]	GE[4]	FR[5]
Surface layer	"Surface course" (asphalt)	"Slitlager" (asphalt)	"Tappeto di usura" (asphalt)	"Deckschicht" (asphalt)	"Couche de roulement"(asphalt)
Binder layer	"Binder course" (asphalt)	Bindlager (asphalt)	"Strato di collegamento" (asphalt)	"Binderschicht" (asphalt)	"Couche de liaison" (asphalt)
Base layer 2	"Base" (asphalt, hydraulically-bound or granular)	"Bundet bärlager" (asphalt)	"Strato di Base" (asphalt)	"Tragschicht" (asphalt)	"Couche de base" (asphalt, hydraulically bound or granular)
Base layer 1	-	"Obundet bärlager" (granular)	"Misto cementato" (hydraulically-bound)	"Tragschicht" (hydraulically bound, granular)	-
Subbase layer 2	"Sub-base" (hydraulically-bound, granular)	"Förstärkningslager" (granular)	"Misto granulare" (granular)	"Frostschutzschicht" (granular)	"Couche de foundation" (asphalt, hydraulically bound or granular)
Subbase layer 1	"Capping" (hydraulically-bound, granular)	"Skyddslager" – (granular)	"Strato antigelo" (granular)	"Bodenverfestigung" In-place hydraulically bound	"Couche de forme" (hydraulically bound, granular)
Subgrade					

[1] (Highways Agency, 2006), [2] (Trafikverket, 2011), [3](Consiglio Nazionale Delle Ricerche, 1995), [4] (FGSV, 2012)., [5] (NF-P 98-086, 2011)

The base layers provide the required bearing capacity and can be composed of granular, hydraulically bound or asphalt concrete layers. The asphalt surfacing is composed of an asphalt binder layer topped by a surface course, providing high resistance against permanent deformation by high vertical and horizontal stress below the wheel paths as well as suitable road surface characteristics.

3.2. Design parameters

3.2.1. Traffic loading

The calculation procedures for considering traffic loads are similar for the examined European design methods. Table 40 gives an overview about various input parameters, which are applied within the national guidelines. The basic value considering traffic loading is the number of standard axle loads during aspired service life of the pavement. As standard axle loads 8,2 t (UK, IT), 10 t (SWE GE) or 13 t (FR) are applied and design lives are 30 years (SWE, IT, GE, FR) or 40 years (UK). All calculations take the average daily traffic volume (ADT), the proportion of heavy lorries and the road type into account.

Table 40: Compilation of input parameters regarding traffic load

	UK	SWE	ITA	GE	FR
Standard axle load [t]	8,2	10	8,2	10	13
Standard axle load [kN]	82	100	82	100	130
ADT	✓	✓	✓	✓	✓
Proportion of heavy lorries	✓	✓	✓	✓	✓
Design period [a]	40	30	30	30	20 / 30
Traffic increase	✓	✓	✓	✓	✓
Road type factor	✓	✓	✓	✓	✓
Lane width	x	✓	✓	✓	✓
Percentage of vehicles in the heaviest loaded lane	✓	x	x	✓	✓
Reference velocity factor	x	✓	x	x	x
Longitudinal slope	x	x	x	✓	x

3.2.2. Subground conditions

The procedures for considering the individual subground bearing capacity vary within the pavement design specifications. In Germany, a minimum value for the subground bearing capacity is defined in terms of a deformation modulus $E_{V2} \geq 45$ MN/m². In case the soil conditions don't suit this requirement, the subground is stabilized by cement to form a first subbase layer.

Other design approaches have a higher flexibility regarding varying bearing capacities of the natural ground and provide foundation classes according the available bearing capacity, in terms of resilient modulus Mr, CBR or surface modulus, see Figure 33. In case of low bearing capacity of the natural soil, cement or lime treatment can be used to improve the soil properties.

In addition, the frost sensitivity of the subground is considered according to the German, Swedish and French design guides. According to the soil parameters (especially grading), the soil is classified in three (GER, FRA) or four (SWE) frost sensitivity classes according to the danger of frost heave in winter conditions. The frost sensitivity class of the soil determines an additional thickness parameter for the pavement (usually a granular frost-protection layer).

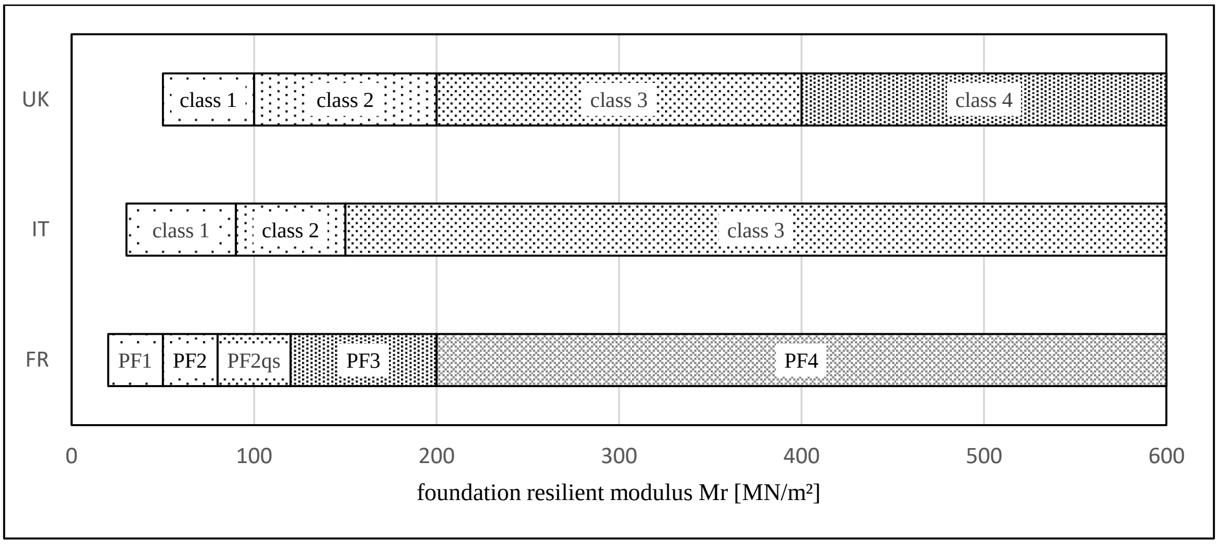


Figure 33: Applied foundation bearing capacity classes in Italy (CNR-Catalogue), UK and France according to resilient modulus

3.2.3. Climatic conditions

The climate condition is considered differently from country to country. But in most of the guidelines a map indicating different climate zones (frost zones) is applied. The German and Swedish guidelines have landscape zones which define three (GER) or five (SWE) climate zones for considering the frost depth during wintertime, compare Figure 34. The Italian pavement catalogue "*CATALOGO DELLE PAVIMENTAZIONI STRADALI*" explicitly refers to climatic conditions typical of Central Italy, whereas according to the Alto Adige-Südtirol design guide the altitude above the mean see level is considered, see Table 41. Within the UK and France, the mean annual frost index is the basis for assessing the frost depth.

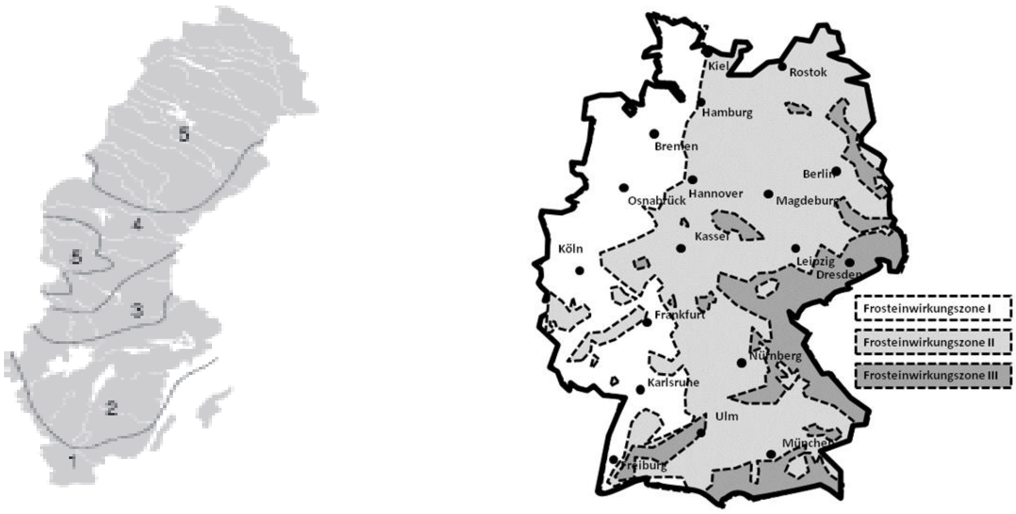


Figure 34: Climatic zones for considering frost depth in Sweden (left) and Germany (right)

Table 41: Italian climate areas according to Alto Adige-Südtirol catalogue (Provincia Autonoma di Bolzano - Alto Adige, 2016)

Climate area	Altitude [m]	Hottest monthly average temperature [°C]	Coldest monthly average temperature [°C]
1	up to 500	23,1	0,9
2	500 to 1000	21,0	0,1
3	1000 to1500	17,9	- 0,1
4	over 1500	15,8	- 1,5

3.2.4. Design procedures

The study of the different pavement design procedures identified three different fundamental approaches to dimension a suitable pavement. In the United Kingdom nomograms with deposited functions to design the total asphalt thickness are used. The layers below will be defined by using nomograms depending on the foundation class of the natural ground. The guidelines from Italy and Germany use a systematic catalogue to design pavements. In Sweden and France, a mechanistic-empirical design procedure is applied based on multi-layer theory, considering the resistance against fatigue cracking (strain at the bottom of the bound layers) and against rutting (permanent vertical deformation at the top of the subgrade).

As a common for all procedures, the total thickness of the pavement in order to prevent frost heave is calculated considering climatic parameters as well as soil composition. Whereas in Sweden and Germany, the required thickness can sum up to 80 cm, in UK, a thickness of 45 cm (35 cm in coastal regions) is applied.

3.2.4.1. Catalogue systems

The empirical pavement design procedures applied in Italy and Germany are based on a design catalogue. Suitable pavement structures are defined for given traffic loading classes, subground bearing capacities as well as applied base layer materials. In France, a catalogue also exists in addition to the mechanistic-empirical design procedure (SETRA-LCPC, 1998), and provides pre-calculated design solutions for some standard types of pavement structures, for different traffic classes and subgrade bearing capacity levels.

3.2.4.2. Nomogram system

In contrast to the catalogue system, the guidelines of the United Kingdom apply nomograms with deposited functions to identify the total asphalt thickness of the pavement. For this procedure the number of equivalent standard axle loads on the pavement during the design period as well as the present foundation class are used as input parameters. The thickness design of flexible pavements is done by using a nomogram solution (compare Figure 20). After calculating the traffic load, the relevant foundation class is linked (1). The required total thickness of the asphalt layer depends on the used asphalt base layer material (2). Besides, the design procedure allows to replace the asphalt base by an HBM base. The thickness of asphalt surfacing is depending on the traffic loads only.

3.2.4.3. Mechanistic-empirical design procedure

Within Swedish and French design procedures, a software, based on elastic multilayer theory, is used to calculate the resulting stresses and strains within the pavement. The stiffness parameters for the road layers are chosen according to the relevant pavement material. Subground stiffness is varied according to the temperature during the year and the frost susceptibility conditions (in Sweden) or rather according to the long-term bearing capacity class of the subgrade (in France). Within the pavement design procedure, the layer thicknesses are set to limit the vertical stress on top of the subground surface in order to withstand the actual number of loading cycles during the pavement design life. Further, the horizontal strain at the bottom of the asphalt base layer, or the horizontal stress at the bottom of the hydraulically bound base layer, is limited in order to avoid fatigue cracking.

3.3. Design of CRM pavements

In Sweden CRAB layers are considered as a standard pavement material and they are incorporated within the pavement design procedures. In France, Germany, Italy and UK, the CRM materials are not included in the original design guide. However, standard documents as guidelines exists and are applied in practice.

In Sweden, the CRM layers are equalised with HMA layers. Here, the mechanical requirements on CRM and HMA layers are the same, and therefore, they are considered in pavement design in the same way.

In France, the cold-mix base layers with emulsion mixtures (grave emulsion, GE) are described with an elastic modulus of 2.000 MPa (compare Table 38). This value is used for the mechanistic-empirical pavement design against permanent deformations of the subbase. So far, no failure criteria for GE itself are applied (fatigue or rutting). The stiffness value applied for GE bases is considerably lower compared to the minimum stiffness value of standard asphalt base layers, which are prepared with HMA (minimum modulus = 9.000 MPa, see Table 26).

In Germany, a guideline document contains model pavement structures for CRAB layers are applied. When comparing the pavement thickness of these structures with standard asphalt pavement thicknesses with HMA base layer, it can be observed, that independent of the traffic loading, a thickness surplus of 10 cm is applied, compare Figure 35. This constant thickness increase results in increasing “transfer” factors with decreasing traffic loading.

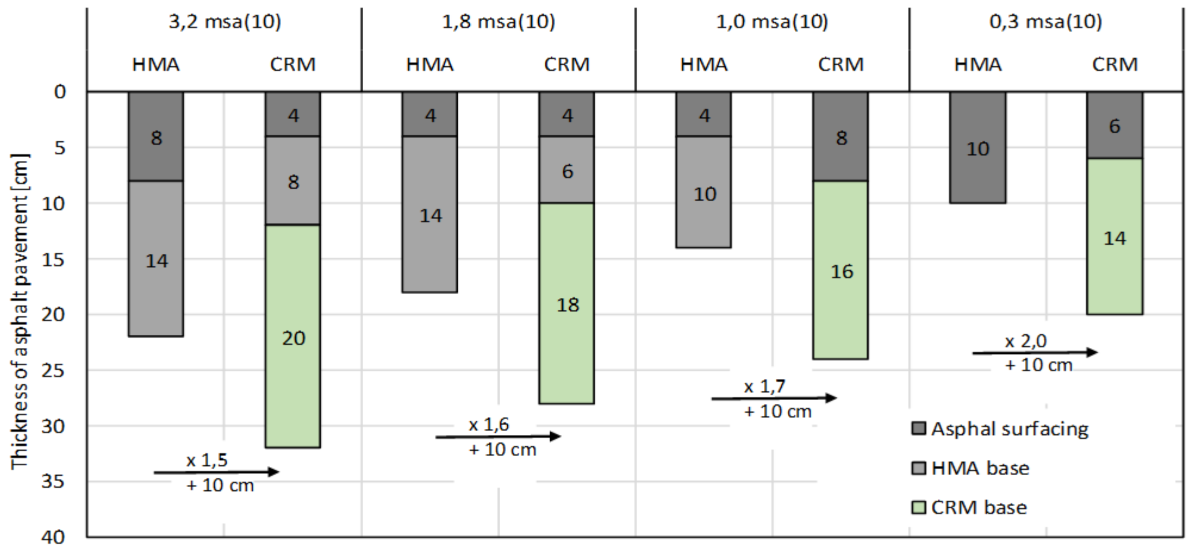


Figure 35: Design of flexible pavements with HMA and CRAB layer for a gravel subbase (E_{V2} = 120 MN/m²) in Germany

In Italy, CRM are part of the design catalogues as an optional base layer material. If the total thicknesses of asphalt layers within the design catalogues are compared to the thickness of equivalent structures including CRAB layers, the thickness is about 1,65 times higher compared to HMA structures (see Table 42).

Table 42: Asphalt layer thickness for pavements with HMA and CRAB layer in Italy

Subground condition	*Traffic load [msa (10 t)]*	*Total thickness of asphalt layers [cm]*		*factor*	*Difference [cm]*
		HMA	*CRAB*		
120 MPa	*12-18*	*18*	*26*	*1,44*	*8*
	18-24	*18*	*31*	*1,72*	*13*
	24-30	*20*	*36*	*1,80*	*16*
160 MPa	*12-18*	*16*	*25*	*1,56*	*9*
	18-24	*18*	*30*	*1,67*	*12*
	24-30	*20*	*35*	*1,75*	*15*

In the UK, the design of CRM pavements is described in an additional guideline. Both, for flexible pavements with CRAB and HMA base layers, the thickness design is derived by nomogram solutions (figures 20 and 23 - 26). When these are applied for different subground conditions (here foundations classes 1 and 2) and traffic loading (5 msa, 20 msa), the total asphalt layer thickness of the pavement with CRA is about 20 % higher compared to the thickness of the pavement with HMA base layer (here: DBM50).

Table 43: Asphalt layer thickness for pavements with HMA base and CRAB layer in the United Kingdom

Foundation class	Traffic load [msa (8,2 t)]	Total thickness of asphalt layers [cm]		factor	Difference [cm]
		HMA (DBM50)	CRAB		
1	5	26	31	1,19	5
	20	36	38	1,06	2
2	5	24	28	1,17	4
	20	29	35	1,21	6

3.4. Comparative structural design for model pavements

In section 3.3 it could be shown, that the analyzed national pavement design procedures result in various approaches to adjust the pavement thickness to the relatively new pavement base layers CRM. In order to analyse additional differences between the resulting design pavement thicknesses, model pavements based on the same input parameters (bearing capacity of subground, traffic loading) are assessed in this section.

3.4.1. Definition of model traffic and subground conditions

The pavement design procedures summarised in section 2 are applied on example structures, which are defined by common traffic loads and subground conditions. For each structure, pavement designs are compared for a standard pavement structure with a conventional asphalt base layer as well as a structure with CRAB.

The conditions for the four example pavements are given in Table 44. To allow common approach for the design, the traffic loading is defined by the number of average daily traffic and proportion of heavy vehicles as well as an annual traffic increase. Here two model traffic loads are considered, representing medium and low traffic conditions on rural roads. Higher traffic volumes could not be considered because design principles for CRM pavements are not available for all considered countries.

Because of different approaches for considering the subground bearing capacity, two variations were defined representing gravely soil conditions of high bearing capacity and subsoil conditions with medium bearing capacity for soils with higher content of fines. The subground conditions are defined by CBR or deformation modulus values.

Table 44: Traffic loads and subground conditions for example pavements

<table>
<tr><th></th><th>Traffic loading</th><th>Subground condition</th></tr>
<tr><td>Pavement 1</td><td rowspan="2">Medium traffic:
• ADT on 1 lane:
4500 vehicles/day
• proportion of heavy lorries: 3 %
• lane width: 3,0 m
• Traffic increase: 2 %/year</td><td>High bearing capacity:
• E_{V2} = 120 MN/m²
• CBR: > 15 %</td></tr>
<tr><td>Pavement 2</td><td>Medium bearing capacity:
• E_{V2} = 50 MN/m²
• CBR: 10 %</td></tr>
<tr><td>Pavement 3</td><td rowspan="2">Low traffic:
• ADT on 1 lane:
450 vehicles/day
• proportion of heavy lorries: 3 %
• lane width: 2,6 m
• Traffic increase: 1 %/year</td><td>High bearing capacity:
• E_{V2} = 120 MN/m²
• CBR: > 15 %</td></tr>
<tr><td>Pavement 4</td><td>Medium bearing capacity:
• E_{V2} = 50 MN/m²
• CBR: 10 %</td></tr>
</table>

3.4.2. Resulting model pavement structures

In Table 45 to Table 48 the resulting layer thickness of each national design procedure for standard pavement with a conventional base layer (Hot Mixed Asphalt - HMA) as well as a structure with CRAB (Cold Recycled Material - CRM) are given.

Table 45: Layer thickness for standard pavement structure within HMA and CRM for Pavement 1

	GER		**ITA**		**UK**		**SWE**		**FR**	
	Thickness [cm]									
Layer/Material	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM
Surface	4	4	3	3	4	4	4	4	3	3
Binder	6	0	5	7	8	6	0	0	3	3
Base	12	8	14	0	16	0	10	0	11	0
CRAB	-	20	-	16	-	19	-	10	-	14
Unbound	0	0	30	0	0	0	8	8	0	0
Hydraulic	0	0	0	30	0	0	0	0	0	0
Subbase	0	0	0	0	0	0	42	42	0	0

Table 46: Layer thickness for standard pavement structure within HMA and CRM for Pavement 2

	GER		ITA		UK		SWE		FR	
	Thickness [cm]									
Layer/Material	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM
Surface	4	4	3	Not used!	4	4	4	4	3	3
Binder	6	6	6		8	6	0	0	3	3
Base	12	8	14		18	0	10	0	16	0
CRAB	-	18	-		-	23	-	10	-	15
Unbound	30	30	30		0	0	8	8	0	20
Subbase	0	0	0		0	0	42	42	0	0

Table 47: Layer thickness for standard pavement structure within HMA and CRM for Pavement 3

	GER		ITA		UK		SWE		FR	
	Thickness [cm]									
Layer/Material	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM
Surface	4	4	3	Not used!	4	4	4,5	Not used!	1,5	1,5
Binder	0	4	0		6	0	0		0	0
Base	10	0	7		10	0	0		0	0
CRAB	-	16	-		-	24,5	-		-	12
Unbound	0	0	30		0	0	8		20	0
Subbase	0	0	0		0	0	42		0	0

Table 48: Layer thickness for standard pavement structure within HMA and CRM for Pavement 4

	GER		ITA		UK		SWE		FR	
	Thickness [cm]									
Layer/Material	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM
Surface	4	4	3	Not used!	4	4	4,5	Not used!	1,5	1,5
Binder	0	0	0		6	0	0		0	0
Base	10	8	7		13	0	0		0	0
CRAB	-	16	-		-	27,5	-		-	12
Unbound	30	30	30		0	0	8		15	0
Subbase	0	0	0		0	0	42		27	14

As a result, Figure 36 shows the resulting pavement structures according to the different pavement design procedures, for model pavement 1. The structures vary considerably in total thickness. The resulting pavement structures for model pavement 2 to 4 are shown in Annex B.

For Germany, considering the high bearing capacity of the subground, no granular subbase layer is required for frost protection or increasing bearing capacity. The total HMA thickness sums up to 22 cm. When CRM is used as an additional base layer below the HMA base layer, a total thickness of the bituminous structure of 32 cm is required, composed of 12 cm HMA and 20 cm CRM.

As result from the Italian design, total asphalt thickness sums up to 22 cm, including a base asphalt layer of 14 cm. Furthermore, a granular base layer is required. When a base layer of CRM is used instead of HMA, the base layer thickness is increased to 16 cm, and the asphalt surfacing from 8 to 10 cm. In addition, the unbound base layer has to be replaced by a hydraulically bound foundation layer to ensure a sufficient bearing capacity.

In Sweden a mechanistic-empirical design is applied considering the actual stiffness properties of the pavement materials. As CRM is not specifically described, the same properties as for HMA base layers were applied which results in the same structural thickness of the pavement layers. In total the asphalt layers sum up to 14 cm and are based on 8 cm granular base and 42 cm granular subbase layer applied below all flexible road structures.

The UK design approach results in a total asphalt thickness of 28 cm containing an HMA base layer with a thickness of 16 cm. When CRM as alternative to HMA is used as bituminous base layer, a thickness of 29 cm is required.

The design according the French procedure results in a total thickness of 17 cm containing an HMA surface layer with a thickness of 6 cm and a base layer with a thickness of 11 cm. If the pavement contains CRM, the thickness of the base layer with Grave emulsion increases up to 14 cm.

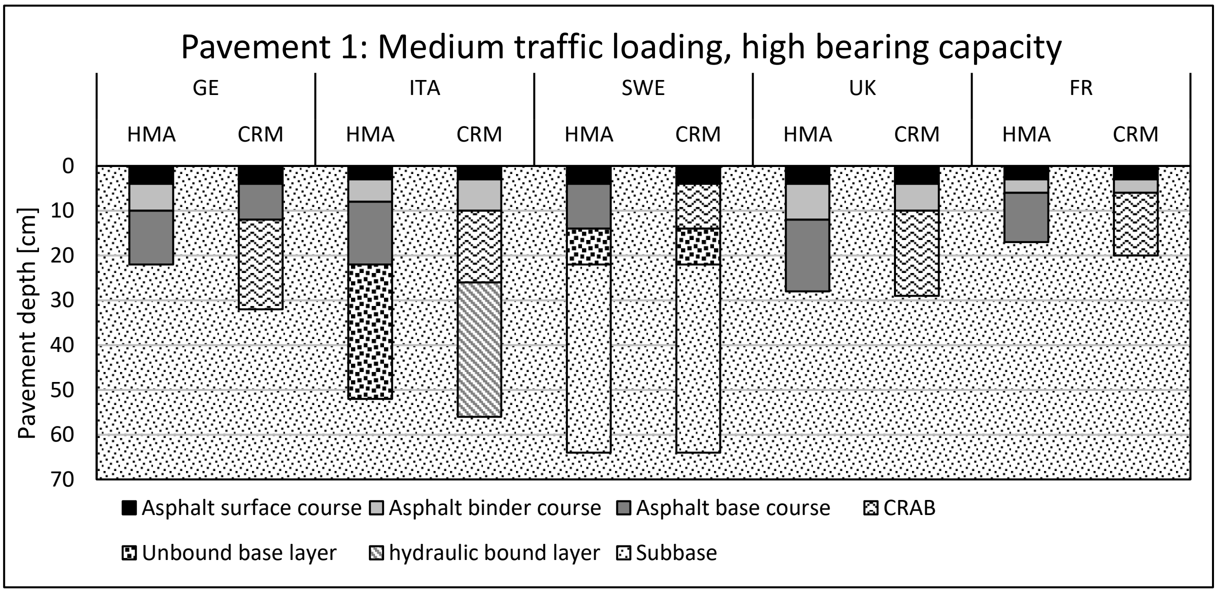


Figure 36: Resulting designs for model pavement 1 (medium traffic loads, high bearing capacity)

For the other model pavement structures described in Table 44, the resulting total bituminous layer thicknesses are summarised in Table 49. Firstly, it can be observed, that the HMA pavement structures according to German and Italian pavement design result in similar HMA layer thickness. The pavements according to UK design have higher asphalt thickness, whereas Sweden and France pavements are designed with a thinner asphalt thickness.

Regarding the difference between pavements with a bituminous base layer of CRM and HMA, it can be observed, that according to German empirical design, the asphalt layer thickness is increased for 45 % for high bearing capacity subground conditions and up to 100 % for low bearing capacity conditions. In UK, the asphalt layer thickness is increased by 3 % to 43 %

when CRM is applied instead of HMA as asphalt base layer. Again, with decreasing traffic loading and bearing capacity of the subground, the surplus thickness for use of CRM is increasing. The French designs result in low-thickness pavements for the low traffic loadings where only a surface coating and unbound base material is required. Therefore, the resulting comparison is not valid. Similarly, only pavement 1 can be designed according to Italian procedures. Here, a surplus of asphalt layer thickness of 18 % is observed when CRM is applied instead of an asphalt base layer.

The resulting pavement designs indicate, that for traditional pavement materials, the various design strategies result in more or less comparable structures. Obviously, the different calculation approaches were originally designed on a common basis.

Considering a novel pavement material (cold recycled material), different safety levels were applied for introducing them into the pavement structures. Further, the pavement materials, here stated as CRM as simplifications vary considerably in mix design regarding applied binder content and binder types which also can be an explanation of the large variety.

Table 49: Structural bituminous bound layer thickness for model design pavements and differences between CRM and HMA

Pave-ment	Asphalt layer thickness	GER		ITA		SWE		UK		FR	
		HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM	HMA	CRM
1	Thickness	22	32	22	26	14	14(4)	28	29	17	20
	Δ [cm]	+10 cm		+4 cm		+0 cm		+1 cm		+3 cm	
	Δ [%]	+45 %		+18 %		-		+3 %		+17 %	
2	Thickness	22	36	23	(2)	14(3)	14(4)	30	33	22	22
	Δ [cm]	+14 cm		-		+0 cm		+3 cm		+0 cm	(+ granular subbase)
	Δ [%]	+64 %		-		-		+10 %		-	
3	Thickness	14	24	10	(2)	4,5	(4)	20	28,5	1,5(1)	13,5
	Δ [cm]	+10		-		-		+8,5 cm		+12 cm	
	Δ [%]	+43 %		-		-		+ 43 %		-	
4	Thickness	14	28	10	(2)	4,5(3)	(4)	23	31,5	1,5(1)	13,5
	Δ [cm]	+14		-		-		+8,5		+12 cm	
	Δ [%]	+100 %		-		-		+37 %		-	

(1) For this very low traffic level, the French method allows to use structures with only a bituminous surface dressing, and granular base and subbase as structural layers. This explains the very low bound layer thickness.
(2) Italian pavement design is not applicable because of low bearing capacity and low traffic loading.
(3) Pavement 1 and 2 respectively 3 and 4 becomes identical because of minimum thickness for asphalt-layers according to Swedish standards.
(4) Swedish requirements are identical for cold recycled materials and Hot Mix Asphalt, which results in identical layer-thicknesses

4. Pavement design of the existing structures

Within WP2 each project partner provided data for at least two road sections including cold recycling material or cold asphalt. For all sections the structural design as well as the structural performance are available and summarised in deliverable report D2 “Compendium of CR performance in different climatic zones and critical review of the impact of mixture composition on performance”.

4.1. Structural design

Structural designs for the road sections included in this project are presented from the surface layer down to the deepest bitumen/cement bound layer. This is the CRAB layer for all road sections except for two of the UK sections and one Italian section where the CRAB layers rest on cement bound foundations. All these layer thicknesses are presented in

Table 50 and graphically in Figure 37. The last column in

Table 50 indicate what is believed to lie beneath the bottom bound layer. Information on these underlying was not always available, and the right column can thus be seen as an engineering judgement.

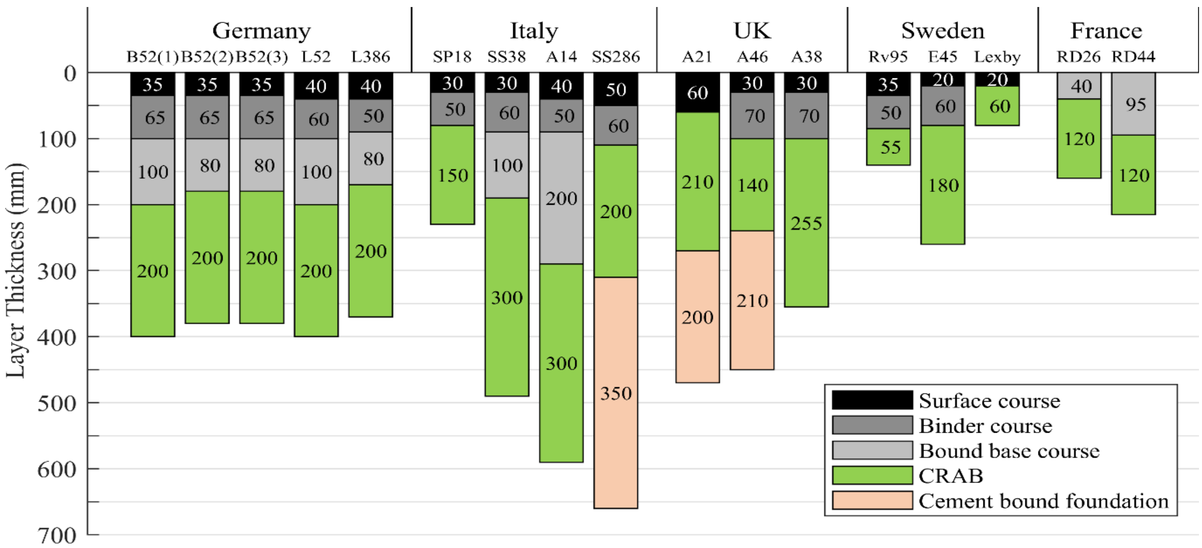


Figure 37. Layer thicknesses for the 17 studied road sections illustrated from the surface down to the deepest bound layer.

In total 17 road structures in five countries were assessed for this study. All sections contained a base layer constructed using cold asphalt and most of CRM. The structures are demonstrating a large variation in traffic loading and climatic conditions (location) as well as design approach.

The total thickness of all bituminous bound layers is plotted against the ADT volumes for HCV for all road sections where traffic load information was available in Figure 38. The total thickness of the bound layers for the pavements indicate a common trend for the structures from France, Italy and Sweden. This trend is shown in a regression function. It should be emphasised that the trend seems to be very consistent considering that the experiences are collected from southern, central and northern Europe.

Table 50: Layer thicknesses [mm] of the studied road sections.

Country	Road section	Surface	Binder	Asphalt base	CRAB	Cement bound foundation	Underlying layers
Germany	B52(1)	35	65	100	200	-	Unbound base layer
	*B52(2)**	*35*	*65*	*80*	*200*	-	
	*B52(3)**	*35*	*65*	*80*	*200*	-	
	L52	40	60	100	200	-	30 cm unbound granular material
	L386	40	50	80	200	-	Unbound granular material
Italy	SP18	30	50	-	150	-	18 cm of granular foundation (existing foundation prior to FDR)
	SS38	30	60	100	300	-	Natural subgrade poorly graded gravel, GP (ASTM-D2488)
	A14	40	50	200	300	-	>20 cm granular foundation (existing foundation prior to FDR)
	SS268	50	60	-	200	350	Natural subgrade(E=64 MPa, from FWD on distressed pavement)
UK	A21	60	-	-	210	200	Old existing pavement. Two layers of HMA.
	A46	30	70	-	140	210	unknown
	A38	30	70	-	255	-	Granular base layer
Sweden	Rv95	35	50	-	55	-	Old penetration macadam
	E45	20	60	-	180	-	70 cm sandy gravel, silty sand subgrade
	Lexby	20	-	-	60	-	Unbound material, subground
France	RD26	-	-	40	120	-	No data available
	RD44	-	-	95	120	-	~10 cm old asphalt pavement on ~45 m unbound base layer

* B52 (2) and (3) are sub-sections with variation in CRAB material and thickness. For representing the general performance of the pavement structure, only B52(1) is further considered in the following sections.

Regarding the sections from Germany, the thickness of the bound layers doesn't show an influence of the actual traffic loading. A reason for this can be, that the CR procedure in Germany is applied only for recycling of tar-contaminated pavements in new road structures. Here, sometimes the aim is to recycle a high amount of reclaimed road materials even if this is not necessarily required according to the loading conditions. The pavement structure of road B52 which shows the highest traffic loading fits well to the "common" regression.

For the UK pavements, only the structure of A38 fits to the "common" pavement thicknesses function. The other structure of A46 with similar traffic load as well as A21 have a cement stabilised base layer below the CRAB layer, therefore less bituminous layer thickness is required.

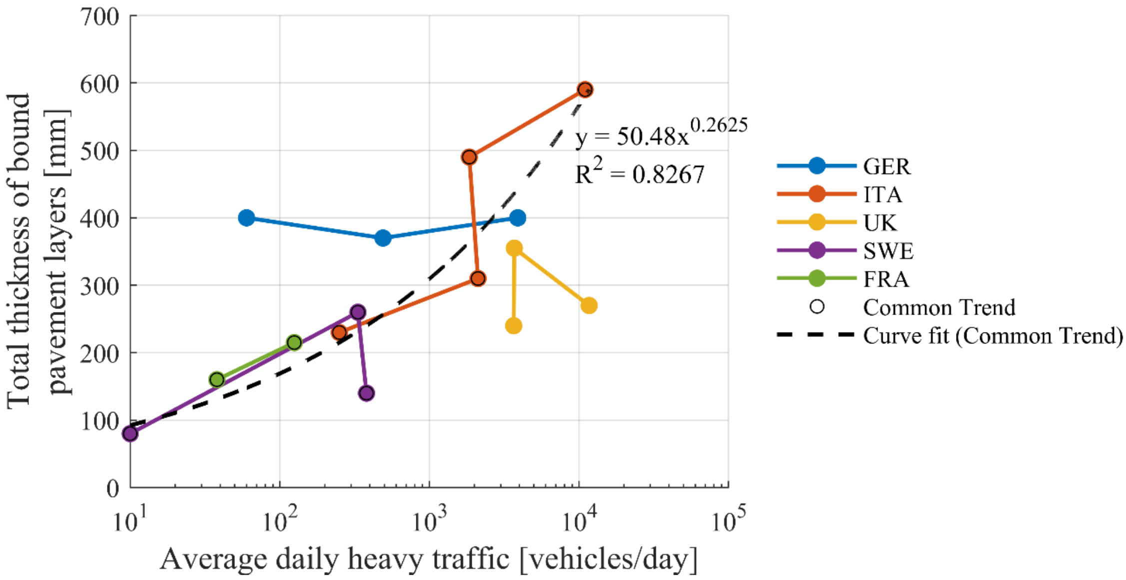


Figure 38. Total thickness of all bound layers plotted against the average number of daily heavy vehicles for all studied sections.

4.2. Structural performance

For many of the assessed roads, data from regularly applied performance monitoring was available. Here, it was learnt, that the internationally applied procedures for monitoring differ considerably. For allowing a comparison of the structural design success, a common procedure for assessment of the pavements condition is required. Therefore, the German procedure for monitoring and evaluation of road surface conditions of federal highways (FGSV, 2006) was applied to all pavements. The measured surface condition is transferred to condition quality marks as followed:

- < 1,5: very good / new road structure
- 1,5 – 2,49: good
- 2,5 – 3,49: satisfactory
- 3,5 – 4,49: sufficient, but maintenance methods should be prepared
- ≥ 4,5: defective, maintenance is required shortly

The transfer functions from the measured condition value to the quality mark is calculated linearly for the transfer parameters according to Table 51.

The permanent deformation in terms of rut depth measurement in millimetres was available for the pavements in Germany, Sweden and UK. The proportion of cracking in the road surface was available for pavements from Germany only. In order to include this assessment for the other pavements, the proportion of cracked surfaces was estimated by analysing photographs taken from the road pavement. Here, Google Street View photos (https://www.google.com/maps) were used, which allowed the rough assessment of cracks.

As the sections were of various lengths, and more than one assessment value was available for each section, the arithmetic mean quality mark was calculated as well as the standard deviation. From these, the 95 % quantile quality mark was calculated in order to assess the variability of the pavement condition.

Table 51: Surface condition transfer parameters

Surface condition quality mark	1,5	2,5	3,5	4,5
Rut depth [mm]	4	7	10	20
Proportion of cracked surface [%]	1	7	15	25

The results of the surface condition assessment are listed in Table 52. For each section the year of the implementation of the CRA pavement is given (year of rehabilitation). Further, the newest result for the surface monitoring assessment is given as well as the average daily traffic of heavy vehicles. For the rutting depth and the cracking, the mean quality values are given (in bold) as well as the 95% quantile (in italic).

In general, the mean surface conditions of most of the assessed sections is very good or good. The Italian section SP18 is the only one which results in satisfactory condition with mean quality higher than 2,5.

Table 52: Pavement condition according to the German standard ZTV ZEB-StB 2007 of each section since rehabilitation.

Country	Section	Year of Rehabilitation	Last year of Measurement	Daily heavy traffic		**quality value***	
						rutting	cracking
GER	B52	2009	2015	3900	**Average / 95%-Quantil**	**1,88**	**1,04**
						2,18	*1,20*
GER	L52	2011	2017	60		**1,28**	**1,00**
						2,00	*1,00*
GER	L386	2007	2017	365		**1,50**	**1,36**
						1,79	*3,45*
SWE	Rv95	2014	2019	380		**2,10**	**1,35**
						2,25	*2,75*
SWE	E45	2012	2019	333		**1,74**	**1,54**
						1,81	*2,79*
FR	RD44	2008	2019	125			**1,21**
							2,07
FR	RD26	2011	-	38			
ITA	SS38	2007	2019	1850			**1,00**
							1,00
ITA	SS268	2016	2017	2115			**1,00**
							1,00
ITA	SP18	2008	2019	250			**2,74**
							4,50
ITA	A14	2007	2019	11000			**1,18**
							1,73
UK	A46	2006	2018	3664		**1,64**	**1,36**
						1,84	*1,82*
UK	A21	2002	2018	11700		**1,56**	**1,00**
						2,01	*1,00*

**status value written in bold and a 95-% quantile written in italic*

However, when considering the variation of the surface condition, additional sections can be identified with problematic performance on a part of the total length. This is indicated by higher 95%-quantile values on sections L386 (GER) and Rv95, E45 (SWE). The section L386 (GER) is therefore of special interest, because shortly after the original maintenance with CRAB, transversal cracking occurred. Furthermore, the original 4 cm surfacing didn't meet the specifications and showed excessive short-term ageing. It was removed by milling on the total length of the section. Additionally, the asphalt base layer was removed in a length of 5 m before and after the transversal cracks after addition of a stress absorbing membrane interlayer (SAMI) prepared by 5 cm unbound gravel and on top additional asphalt for levelling, a new surfacing of 9 cm (asphalt binder and asphalt surface course) was paved.

5. Validation of national empirical pavement design approaches

To validate the national empirical pavement design procedures from Germany, Italy and UK, the observations regarding structural design, loading parameters and pavement conditions for the existing CRAB pavement sections made in WP2 are assessed. The validation is split in 3 phases.

Firstly, the individually applied traffic load parameter is calculated for all pavement structures. These are the number of equivalent standard axle loads, which are based on the composition and volume of heavy traffic as well as the aimed service lifetime.

In a second step, each empirical design procedure is applied for each of the CRAB pavement sections. This will allow the direct comparison of the three design procedures.

At last, the structural performance is estimated by taking the actual service lifetime into account. By these means, a theoretical residual service life parameter is estimated for each section which can be compared with the actually observed pavement condition.

5.1. Comparative evaluation of the traffic loads

A common design input is the traffic load. In all design procedures, the average daily traffic of heavy vehicles and the aimed service lifetime is used for calculating the number of equivalent standard axle loads (ESAL). Here, the three design procedures apply different load values for the standard axle load (IT, UK: 8,2 t; GER, SWE: 10 t). Additionally, different aimed service lifetimes are considered: GER, IT: 30 a; UK: 40 a. In order to base the different pavement design on the same traffic load estimations, the individual number of equivalent standard axles of the considered country, where the structure is located is used as an input value. The other two traffic volume parameters are then calculated, by application of the “power 4-law” (compare section 2.1.1 and 2.2.1 and considering the different aimed service lifetime). By these means, the different approaches for traffic assessment and estimation of the equivalent load parameters are biased.

The resulting numbers of ESAL are summarised in Table 53. The basis traffic parameter is printed in bold and is the basis value for calculating the other two parameters.

Table 53: Traffic load parameters of CRAB validation structures (Millions of equivalent standard axle loads (ESAL: GER, SWE, ITA / msa: UK)

Section	ADT [veh/d]	Proportion of heavy veh. [%]	Service life [a]	GER	ITA	UK
				ESAL10, 30 a	ESAL8.2, 30 a	msa (8,2 t; 40 a)
B52	26000	15	11	**47,5**	105,1	131,4
L52	1500	4	9	**0,5**	1,0	1,3
L386	7000	5	13	**2,8**	6,3	7,8
SP18	5000	5	12	2,7	**6,0**	7,5
SS38	30000	6	13	13,6	**30,0**	37,5
A14	44000	25	13	67,8	**150,0**	187,5
SS268	19661	11	4	10,9	**24,0**	30,0
A21	47714	25	18	8,7	19,2	**25,6**
A46	19.92	19	14	5,8	12,8	**17,0**
A38	37000	10	14	11,9	26,3	**35,0**
RV95	3136	12	6	**10,2**	22,6	29,3
E45	1233	27	8	**9,2**	20,3	26,5

5.2. Re-design of the existing pavements

In this section, the existing pavements were re-designed according to the national standards of Germany, UK and Italy. For the design procedures of Germany and Italy, the deformation modulus E_{V2} is input parameter. For the UK, the design depends on the foundation class Therefore, the required input data was estimated for each section according to the data given in Table 50. For deformation moduli between 100 and 200 MPa, representing the foundation class 2 and 3, the layer thickness according to UK design was estimated for both foundation classes and afterwards linearly interpolated.

The design procedures define different bearing capacity classes. For the catalogue systems (GER, IT), this results in different modulus classes applied.

The resulting estimations for the subground / subbase deformation moduli are summarised in Table 54. For the reference pavements in France, relevant data was not available, and the re-design was not possible.

Table 55 shows an example for the re-design of the Italian pavement of SS268. The actual pavement structure has a total thickness of bound layers of 31 cm. The re-design results in higher thickness values: GER: 36 cm, UK: 35,5 cm, IT: 30 cm (with additional stabilised subbase). Obviously, the actual thickness of the pavement is lower than the designed values according to German and UK pavement design.

The surface condition values (mean / 95%-quantile) for this section are 2,74 / 4,5, compare Table 52. The observed cracking in the pavement correlates to the undersized design.

Table 54: Estimation of the subbase deformation modulus for each project section

	Design deformation modulus E_{V2} [MN/m²]			
Section	Actual modulus [MPa]	GER $E_{V2,max} = 120$	IT $E_{V2,max} = 120$	UK
B52	150	150	160	150
L52	120	120	120	120
L386	120	120	120	120
SP18	120	120	120	120
SS38	120	120	120	120
A14	160	150	160	160
SS268	160	120	160	160
A21	200	120	160	200
A46	200	120	160	200
A38	120	120	120	120
RV95	120	120	120	120
E45	120	120	120	120

Table 55: Results for redesigned pavement of the Italian section SS268

SS268	Current state	Redesigned pavement		
		ESAL10	msa8.2	ESAL8.2
ESAL/msa	0,8	10,9[1)]	30,0	24,0
deformation modulus subbase	160[2)]	150	160	160
surface	5	4	4	3
binder	6	0	6	7
base	0	8	0	0
CRAB	20	24	25,5	20
stabilized base layer	35			30
Total asphalt thickness	31	36	35,5	30

[1)] Extrapolation within the design procedure

The summary of this data for all assessed pavement structures is synthesized in Annex C. The resulting total thickness values of the bituminous layers are summarised in Table 56. For the re-designed thickness results, the thickness-deviation of the actual pavements is given and highlighted by colouring:

- Green: actual thickness is higher compared to re-designed thickness
- Grey: actual thickness is the same (± 2 cm) as the re-designed thickness
- Red: actual thickness is lower than the re-designed thickness.

The three assessed empirical design procedures identify over- and under-designed pavement structures in the assessed roads similarly. Despite of differences in re-design thickness of up to 7 cm (section E45), the German and UK design procedure are capable to identify the same sections as under- (red) or overdesigned. However, the pavements designed according to the

UK specifications are generally thinner compared to the German design thickness, with B52 as an exception.

For the Italian re-design results, the thickness of all structures is lower compared to English or German designs. However, these CRAB structures are based on top of a cement stabilised layer, which gives higher bearing capacity. Generally, the same differences between the actual pavement structure thickness and the designed ones are identified also with the Italian method with the B52 as an exception. In three cases (SP18, SS268, A46) the difference between pavement thickness and re-designed thickness is lower than ± 2 cm.

Four of the pavements (L52, L386, SS38, A14) seem to be over-designed according to all design procedures applied. Three of these structures are identified by a very low crack condition value and nearly show no crack damage at all. The exception for this conclusion is section L286, where cracking could be observed in some specific spots and which originally showed transversal cracking shortly after construction and which was maintained.

Table 56: Bound layer Thickness according to national standards

Section	Actual thickness [cm]	Thickness according to national standards [cm]			Crack condition value: (mean 95%-quantile)
		GER	UK	IT	
B52	38	38	42	35	1,04
	Δ [cm]	0	4	-3	1,2
L52	40	28	28,5	21	1,0
	Δ [cm]	-12	-12	-19	1,0
L386	37	32	27,5	21	**1,36**
	Δ [cm]	-5	-10	-16	**3,45**
SP18	23	32	37	21	**2,74**
	Δ [cm]	9	14	-2	**4,5**
SS38	49	40	34,5	36	1,0
	Δ [cm]	-9	-15	-13	1,0
A14	59	38	44	35	1,18
	Δ [cm]	-21	-15	-24	1,73
SS268	31	36	35,5	30	1,0
	Δ [cm]	5	5	-1	1,0
A21	27	34	32	31	1,0
	Δ [cm]	7	5	4	1,0
A46	24	34	30	25	1,36
	Δ [cm]	10	6	1	1,82
A38	35,5	36	34	36	-
	Δ [cm]	+0,5	-1,5	+0,5	-
Rv95	14	40	33,5	31	**1,35**
	Δ [cm]	26	20	17	**2,75**
E45	26	40	33	31	**1,54**
	Δ [cm]	14	7	5	**2,79**

Of the six sections, which structures are identified as under-designed (SP18, SS286, A21, A46, Rv95, E45), three actually show some (Rv95, E45) or considerable (SP 18) crack damages.

This general correlation between difference in design thickness and crack condition is plotted in Figure 39. The observed trends between design thickness difference and crack condition are very rough but generally plausible. Here, the actual age of the pavement sections as well as the expectations in their performance is not considered. Even if no clear answer about the success of the assessed pavements can be given, the result shows, that generally the observed cracks in the pavements can have an origin in the pavement design.

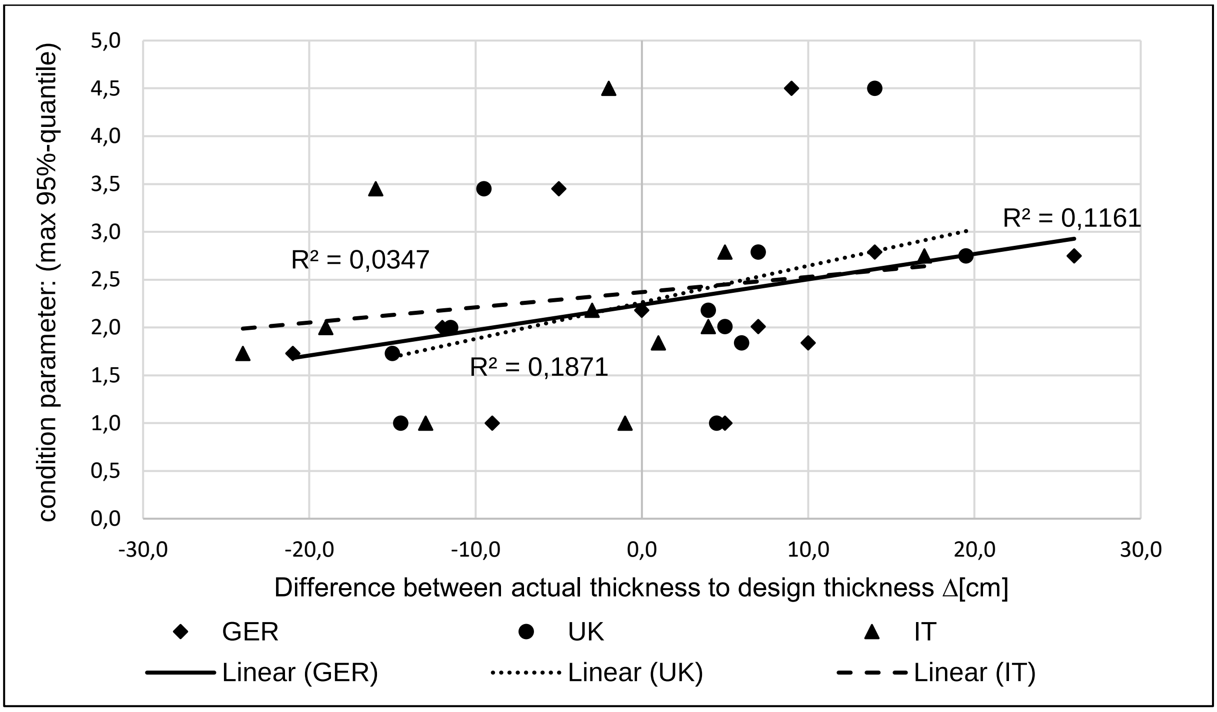


Figure 39: Maximum 95%-quantile plotted against the deviation of bounded layer thickness

5.3. Calculation of structural condition parameter

As can be observed in Figure 39, there is only a low correlation between the thickness differences between design thickness and actual thickness and the observed cracking in the pavements. One reason is, that the design life is considerably higher for all of the assessed structures than the actual service lifetime (compare Table 53). In order to consider the actual received traffic loading on the assessed pavements, a theoretical structural condition parameter is calculated according to a German guideline document (FGSV, 2019).

Therefore, the required thickness ($DI_{erf.}$) of the bound layers are calculated by considering the traffic load class and the bearing capacity of the unbound subbase. This thickness is compared to the “active” thickness ($DI_{vorh.}$) of the assessed pavement structure, where the actual thicknesses of the structural layers are reduced according to the age of the layer.

The required thickness can be calculated with the equations 21 to 27. These are based on the German design catalogue structures and have input parameters linked to the number of equivalent axle loads (10 t, 30 a) and the subbase bearing capacity in terms of deformation modulus E_{V2}. The resulting loading class (Bk) refer to the upper limit of million 10-t-ESAL for each design catalogue column and regression equation, compare section 2.2.1.

Bk100: $DI_{erf.} = 54{,}57 - 0{,}2019 * E_{v2} + 2{,}54 * 10^{-4} * {E_{v2}}^2$ (eq. 21)

Bk32: $DI_{erf.} = 50{,}57 - 0{,}2019 * E_{v2} + 2{,}54 * 10^{-4} * {E_{v2}}^2$ (eq. 22)

Bk10: $DI_{erf.} = 46{,}57 - 0{,}2019 * E_{v2} + 2{,}54 * 10^{-4} * {E_{v2}}^2$ (eq. 23)

Bk3,2: $DI_{erf.} = 45{,}15 - 0{,}2244 * E_{v2} + 3{,}17 * 10^{-4} * {E_{v2}}^2$ when $E_{v2} \leq 200$ MPa

$DI_{erf.} = 12$ when $E_{v2} > 200$ MPa*(eq. 24)

Bk1,8: $DI_{erf.} = 44{,}02 - 0{,}2389 * E_{v2} + 3{,}38 * 10^{-4} * {E_{v2}}^2$ when $E_{v2} \leq 180$ MPa

$DI_{erf.} = 12$ when $E_{v2} > 180$ MPa*(eq. 25)

Bk1,0: $DI_{erf.} = 38{,}68 - 0{,}2049 * E_{v2} + 2{,}70 * 10^{-4} * {E_{v2}}^2$ when $E_{v2} \leq 170$ MPa

$DI_{erf.} = 12$ when $E_{v2} > 170$ MPa* (eq. 26)

Bk0,3: $DI_{erf.} = 37{,}58 - 0{,}2858 * E_{v2} + 5{,}67 * 10^{-4} * {E_{v2}}^2$ when $E_{v2} \leq 120$ MPa

$DI_{erf.} = 12$ when $E_{v2} > 120$ MPa*

$DI_{erf.} = 10$ when $E_{v2} > 150$ MPa* (eq. 27)

* minimum thickness of the bounded pavement

The active thickness of the existing pavement DI_{vorh} is given in the following equation:

$$DI_{vorh.} = \sum_i (D_i * Aq_{i\,t}) \quad \text{(eq. 28)}$$

where:

D_i = Thickness of the bound layer i [cm]

$Aq_{i\,t}$ = age-related factor of thickness equivalation of the layer i (see Table 57)

$Aq_{i\,t}$ = MIN ($Aq_{i\,max}$; MAX ($Aq_{i\,min}$; Equation according to Table 57))

Table 57: Age-related factor of thickness equivalation for different types of layers

Layer type	Age-related factor of thickness equivalency $Aq_{i\,t}$ with t = age of layer [a]
Surface asphalt	0,35 < 1,0392 – t * 0,0392 < 1
Mastic asphalt	0,4 < 1,0192 – t * 0,0192 < 1
Asphalt Binder	0,4 < 1,0400 – t * 0,0200 < 1
Asphalt Base	0,5 < 1,0200 – t * 0,0100 < 1
Hydraulic bound	0,33 < 0,5540 – t * 0,0070 < 0,54

The ratio of these values (called “thickness comparison number DVZ”) is used as an indicator for the structural thickness reserve, which is already reduced by traffic loading (eq. 29).

$$DVZ = \frac{DI_{vorh.}}{DI_{erf.}} \quad \text{(eq. 29)}$$

This number is the basis for assessment of a structural condition indicator, where a DVZ of 0,9 (thickness reserve is reduced by 10%) is considered as “very good” and a DVZ of 50% (structural thickness is reduced to half of its initial value) is considered as defective. By this estimation, a structural condition parameter SW is obtained (compare Figure 40), which can be directly compared with surface condition parameters.

This comparison is shown in Figure 41, where the observed surface parameter (maximum of the 95 %-quantile of cracking or rutting) is plotted against the structural condition parameter SW_B. There is a surprisingly good correlation between these two parameters, representing all assessed CRAB pavements. This further indicates, that the observed cracking is linked to the structural design of these pavements and can be used as a parameter to evaluate the success of an applied pavement design.

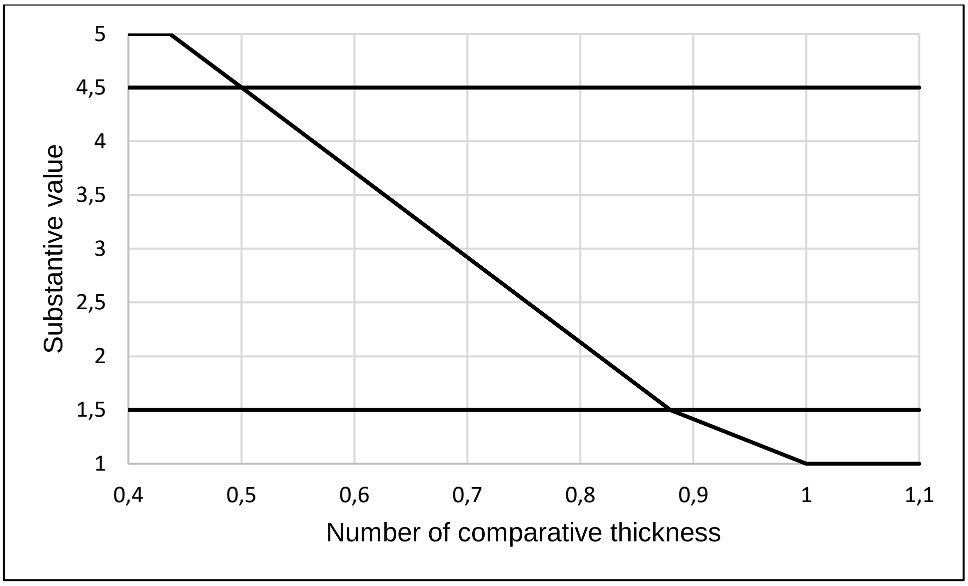


Figure 40: Nomogram for the number of comparative thickness

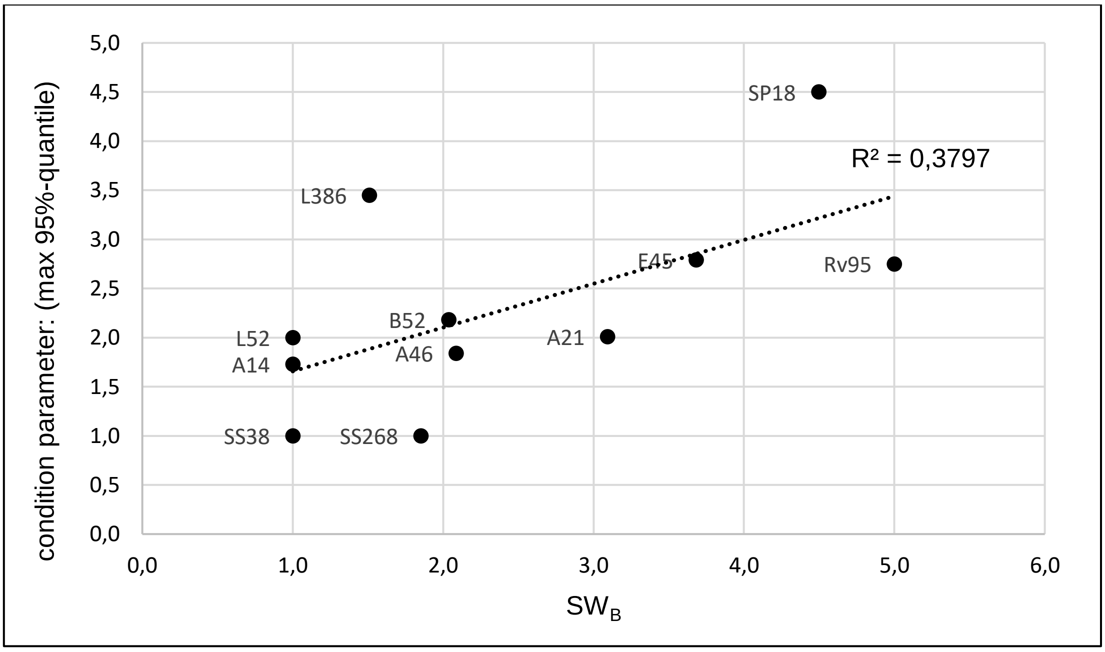


Figure 41. Comparison of structural condition parameter SW_B with surface cracking conditions for the assessed CRAB pavements

The structural condition parameter can further be used for calculating the theoretical structural service life of an existing pavement structure. Therefore, the structural condition parameter is divided by the actual pavement age in order to obtain the annual structural reduction. Then it is calculated, how many years are required until the structural condition is defective and reaches a condition parameter of 4,5, compare eq. 30.

$$t_{sl} = \frac{(4{,}5-1)}{\frac{SW_B}{a}} \qquad \text{(eq. 30)}$$

where:

t_{sl} = theoretical structural life [a]
4,5 and 1 = (parameter values indicating defective and very good performance)
SW_B = Stuctural performance parameter
a = age of the pavement structure [a]

This procedure is applied for the assessed structures in Figure 42. The assessment shows, that the pavements which show lower thickness compared to the German design procedures would result in structural life lower than 25 a, which is close to the aimed service life of 30 a. By this plot, the effect of thickness deviations from the actual design can be estimated. It has to be stated, that it is a purely theoretical calculation based on the German design principles. However, as plotted in Figure 43, there is a feasible correlation between theoretically remaining structural life and the pavements condition parameter, obtained from the surface defects (ruts, cracks). Only two pavement structures don't fit to the general trend: The German structure L386 shows a worse surface condition, due to cracked areas at closely located intersections. On the other hand, Italian structure of SS268 shows a better condition estimated according to the theoretical assessment.

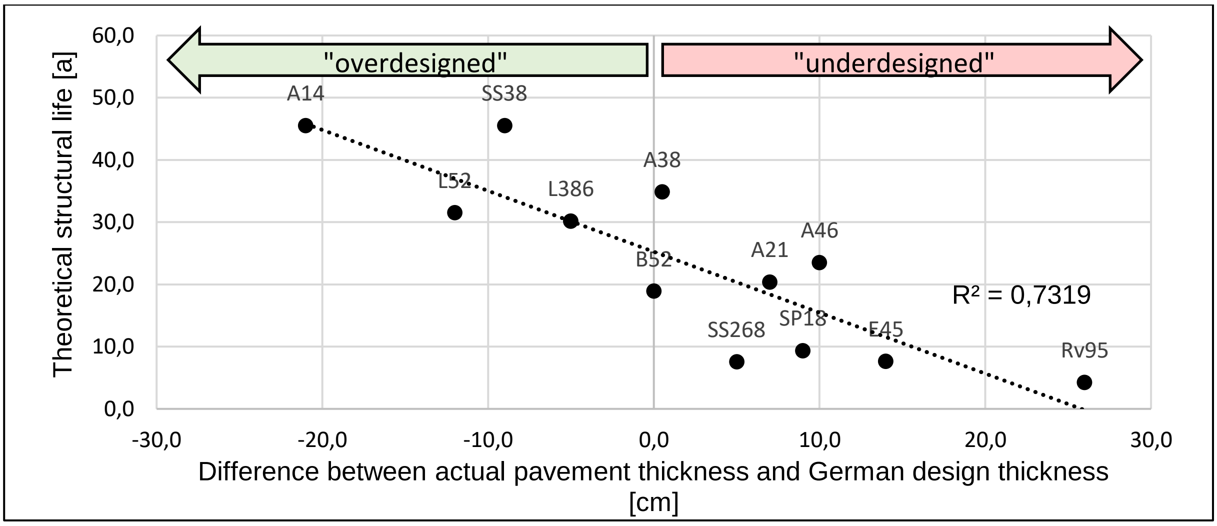


Figure 42. Effect of over- and under-design according to German design principles to the theoretical structural service life for the assessed CRAB pavements

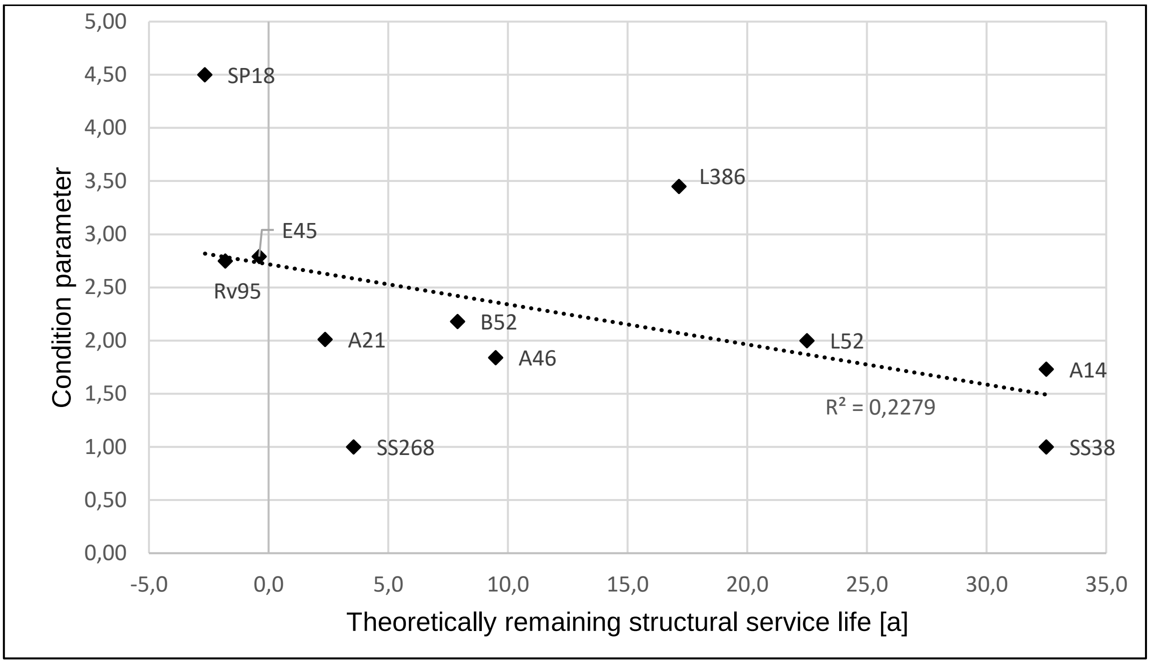


Figure 43: Correlation between surface condition parameter (95 % quantile) and theoretically remaining structural life for the assessed CRAB pavements

6. Discussion

From the general assessment of the pavement design procedures for standard flexible pavements in Germany, Italy and UK (empirical) and France and Sweden (empiric-mechanistic), compare section 2, it could be learnt, that different approaches for considering traffic loads and subground bearing capacity are applied. However, all procedures seem to be based on similar principles, which indicate a common origin. However, the design procedures cannot be compared to each other directly.

Therefore, four model pavements were defined with exemplarily given traffic volume and subground conditions (section 3). The result of these designs for model pavements indicate differences in the resulting thickness of standardly applied asphalt layers (hot mixed) as well as differences in applications and expectations in granular subbase below the asphalt pavements, compare Table 58.

Especially the pavement design applied in Sweden results in comparably low asphalt layer thickness. On the other side, the UK pavements show comparatively high asphalt layer thickness, however this can be partly explained by the lack of granular base or subbase layers in these model designs. For model pavement 2, the designs resulting from French, Italian and German design procedure is very similar. The highest difference can be identified for the model pavements 3 and 4 with low traffic loading. Here the empiric-mechanistic design of France and Sweden results in only a thin surface asphalt layer (SWE) or even a surface coating (FRA), whereas the purely empirical procedures result in asphalt surfacing of at least two asphalt layers.

Table 58: Design thickness for comparative model pavements (see section 3.4)

Model pavement	Thickness of asphalt layers and *granular (sub-)base layers* [cm]				
	SWE	FRA	ITA	GER	UK
1: medium traffic, high bearing cap.	14 / *50*	17 / -	22 / *30*	22 / -	28 / -
2: medium traffic, low bearing cap.	14 / *50*	22 / -	23 / *30*	22 / *30*	30 / -
3: low traffic, high bearing cap.	4,5 / *50*	1,5 / *20*	10 / *30*	14 / -	20 / -
4: low traffic, low bearing cap.	4,5 / *50*	1,5 / *42*	10 / *30*	14 / *30*	23 / -

These differences in the standard design procedures indicate diverting safety considerations and expectations to new pavements, different climate conditions, especially in regrd to frost protective unbound base layers, as well as presumably different pavement material properties. Therefore, the national design principles for CRAB pavements result in varying structures as well, as shown in section 3.4.

Therefore, no common empirical design proposal can be given which would be applicable in all European countries. However, the comparison between the traditional HMA structures to CRAB structures give an indication for the expectation in cold recycled asphalt mixtures as base layer material. In section 3.3 transfer factors could be obtained which indicate the thickness increase applied for CRAB layers compared to HMA asphalt layers. These are verified by the result of the design of the model pavements (section 3.4):

- Sweden:
  Currently, for base layers prepared with bitumen emulsion, the same requirements regarding mechanical properties are applied as for HMA base layers. Therefore, the layer thickness applied in pavement design is the same for both types of materials.
  This also is shown by the lack of design difference of the model pavements (section 3.4).
- France:
  Within the empiric-mechanistic design procedure applied, the stiffness modulus for CRAB (here: GE) is 2.000 MPa and considerably lower than the value usually applied for HMA base layers of 9.000 MPa. According to Odemark's Equivalent Thickness Method (Odemark ,1949) this stiffness difference result in a required thickness increase of 1,65[1].
  For the actually designed model pavements, the thickness is increased by the factor 1,2.
- UK:
  The empiric design procedures result in a thickness increase factor for the total asphalt pavement between 1,1 and 1,2 when CRAB is applied instead of HMA base (here: "DBM50"). Further, a factor of 1,2 is applied to increase the layer thickness, if a part of the HMA surfacing is replaced by the CRAB layer.
  For the assessed model pavements, the resulting thickness increase is between 1,0 and 1,4.
- Italy:
  The comparison of design catalogues result in a thickness increase factor of between 1,4 and 1,8 which CRAB is applied instead of HMA base layer.
  The actual thickness increase for the model pavements is 1,2 only. However, for these pavements, a lime or cement stabilised subbase (produced in-situ, with 100% recycled aggregates) was added to the pavement structure.
- Germany:
  A constant surplus thickness of +10 cm is applied for the total asphalt layer thickness, when a CRAB layer is applied, independently of the actual traffic loading and the general pavement thickness. This results in thickness increase factors of 1,5 to 2,0.
  The same range of thickness increase is observed for the model pavements calculated in section 3.4.

A summary of these values is given in Table 59. From these observations in the theoretical pavement design procedures, it can be concluded that CRAB base layers can be generally applied in flexible pavements instead of HMA base layers. Because of the usually reduced stiffness of these materials, the asphalt base layer thickness should be increased by a specific factor. When comparing the individual approaches, a factor of 1,5 seems to be feasible.

If a subbase layer is added, e. g. by a cement stabilisation of existing granular material or by a lime stabilization of existing clayey subground, an asphalt layer thickness increase of 20 % is proposed.

[1] Thickness increase = $\sqrt[3]{\frac{E1}{E2}} = \sqrt[3]{\frac{9000}{2000}} = 1{,}65$

Table 59: Applied and proposed thickness adjustment factors for the total asphalt layers for exchange of HMA base layer by a CRAB layer

	SWE	FRA	UK	ITA	GER
According to Design guides	1,0	1,65	1,1 – 1,2	1,4 – 1,8	1,5 – 2,0
According to model pavements	1,0	1,2	1,0 – 1,4	1,2	1,5 – 2,0
Proposal	1,5 (1,2, when subbase is added)				

In section 4 and 5 the structural performance of existing CRAB pavement structures was assessed in detail. Regarding the surface condition performance, ten of these 15 structures showed none or only very little rutting or cracking, two showed some cracking which can be considered as moderate and only for two sections showed defective surface cracking on some parts of the pavement area. This observed surface performance could be correlated against the difference between actually applied structural thickness and the required thickness (according to German, Italian or UK design) only roughly. However, when the actual service life is considered within this comparison, a close correlation between the surface condition and the theoretical design life could be identified. In this way it could be observed, that the structures of the Swedish pavements as well as an Italian pavement, have a shorter remaining expected service life. It has to be stated, that here a German procedure was applied for estimating the theoretical service life, which is based on German pavement design thickness results.

However, the good surface conditions of most assessed sections clearly show, that the construction of durable pavements with CRAB is possible. For three of the four sections with only moderate or even defective condition, the thickness of the pavement structures could be identified as not sufficient for the traffic loading conditions. Therefore, it can be concluded, that the general applied empirical design approach, in which the CRAB layer thickness is increased by 50 % compared to a HMA base will result in a feasible pavement structure.

Regarding mechanistic design procedures, there is a lack of knowledge about the applicable and relevant failure modes to be considered. Within the empirical-mechanical design procedures assessed within this project, it can be summarised, that these doesn't seem to be optimised for CRAB already. In the Swedish method, there is a theoretical estimation, that cold asphalt base layers show the same mechanical behaviour as HMA bases. However, in France a significant lower stiffness for base layers prepared with bitumen emulsion is applied and, additionally, no fatigue or other cracking criterium is applied, so far. Though, the assessment of existing structures indicated that cracking was more severe compared to rutting in several of the assessed sections. This cracking also could be generally linked to the fatigue-related combination of traffic volume and service age of the pavements.

For the assessment of cracking phenomena in pavements with CRAB layers, additional research is required in order to identify the origin of the cracks. It has to be identified if the crack result from traditional bottom-up cracks in the CRAB layer or rather in the asphalt surfacing as a result of low thickness and low bearing capacity of the CRAB layer. This may be related to the CRAB composition (e.g. cement content) but could not be analysed within this project.

7. Conclusions

Following conclusions can be drawn from the assessment discussed within this report:

1. It is possible to design pavements with CRAB which will reach the usually aimed service life.
2. The observed surface conditions in CRAB pavements can be linked to their structural design properties (structural thickness, subbase layers) and time from construction.
3. The general pavement design procedures are all based on a common origin and consider similar design aims, however the individual approaches for calculation design input values for traffic loads and subground bearing capacity vary considerably. Therefore, no general empiric design for the application is recommended in order to facilitate the national applicability of CRAB layers.
4. For empirical pavement design, the generally applied HMA base layer can be changed to a CRAB layer. In this case the total thickness of the asphalt layers shall be increased by 50%. When applying a CRAB layer, an HMA surfacing with a thickness of at least 10 cm is recommended.
   Here, existing catalogue systems or nomogram solutions easily can be adopted.
5. So far, cracking phenomena in CRAB pavements are not considered in existing mechanic-empirical design procedures. However, in existing CRAB pavements, cracking could be identified as relevant failure mode which limits the service life of these pavements. Here an additional assessment of the actual cracking phenomena is required, which could not be included in this project. The phenomena are not only linked to the application of hydraulic binders within the CRAB material, as the Swedish sections indicate. Additional reason for cracks could be an unbalanced pavement, where the stiffness of the asphalt surfacing might be too high for the comparatively low stiffness of the CRAB layer.

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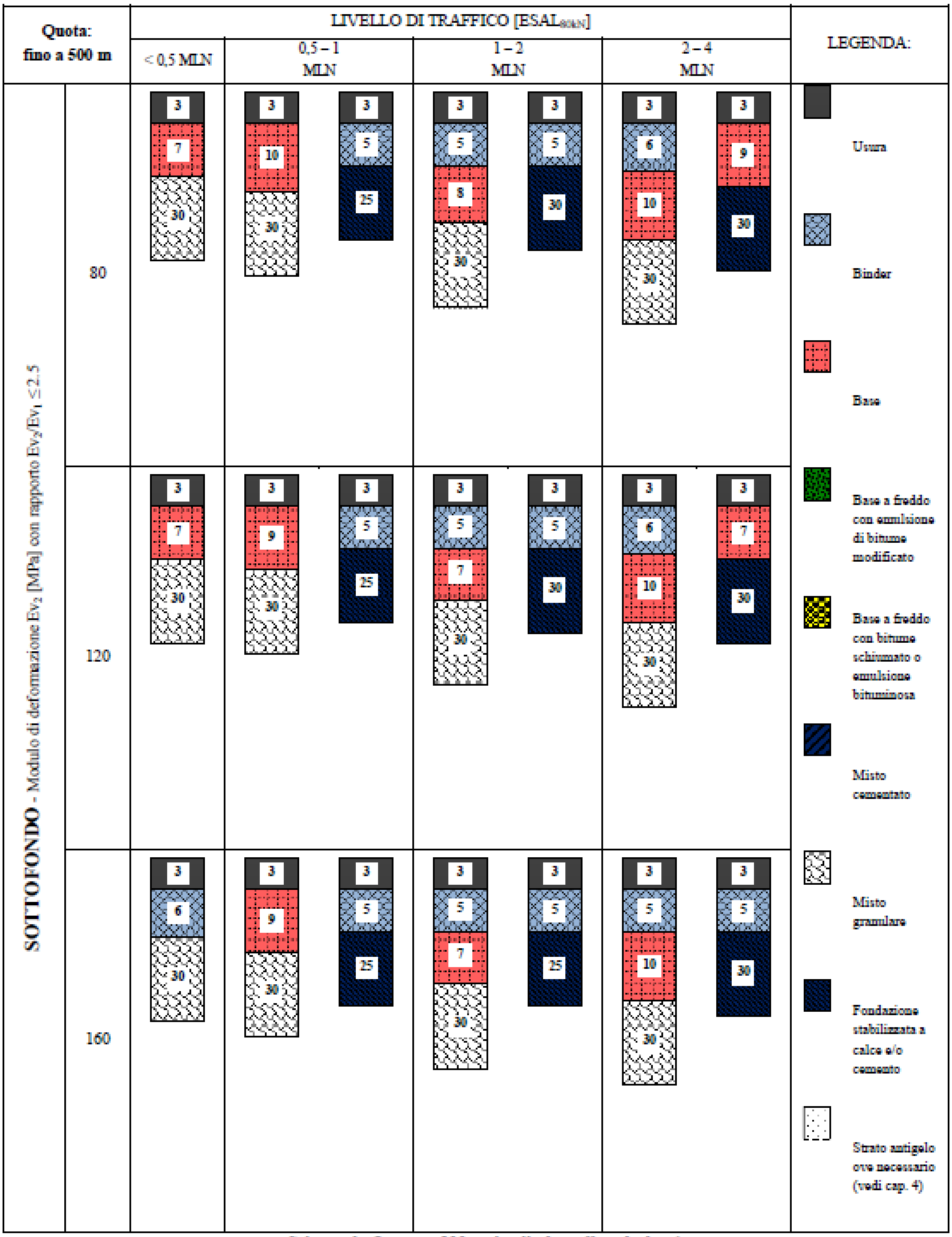
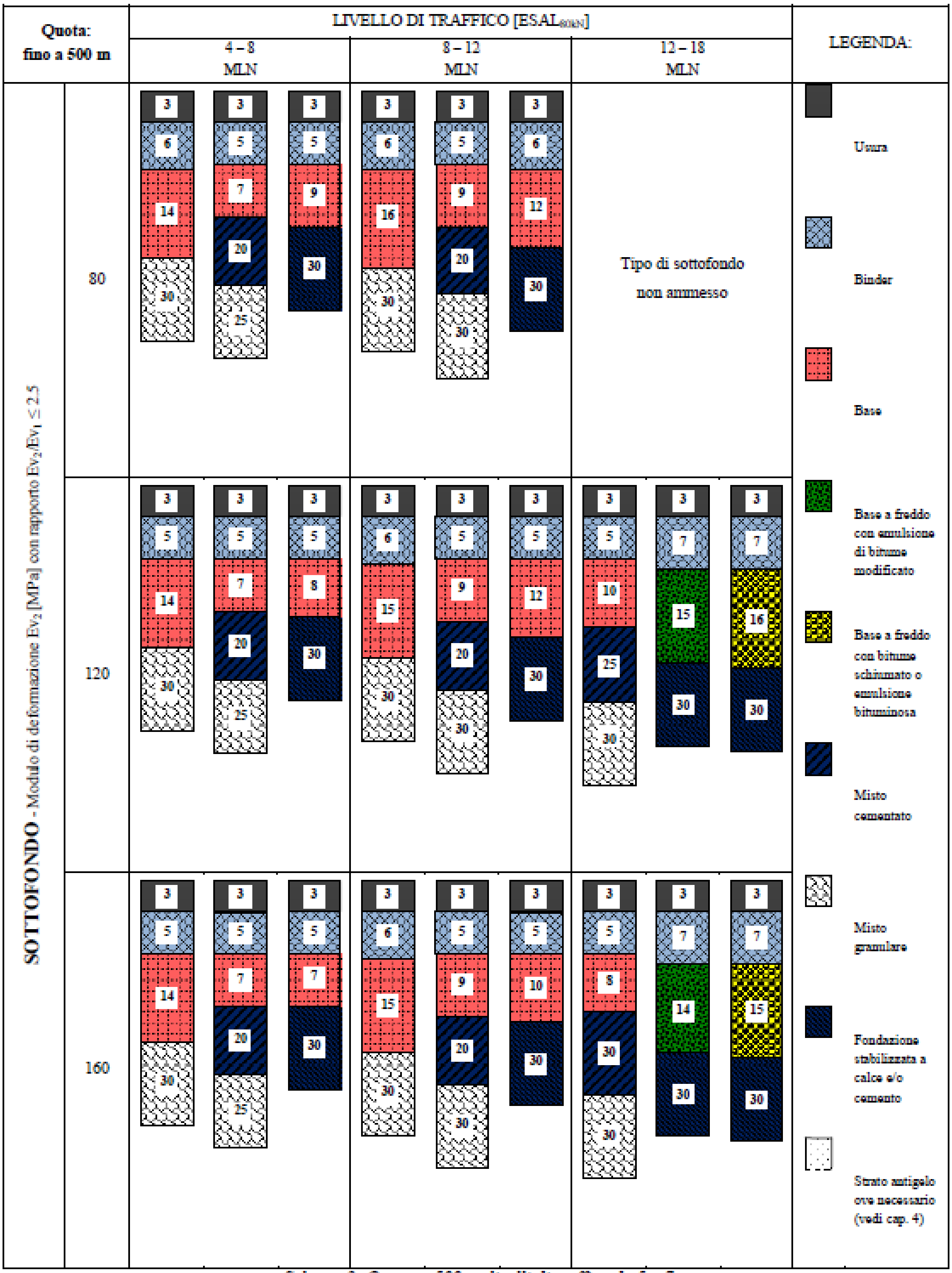
List of Annexes

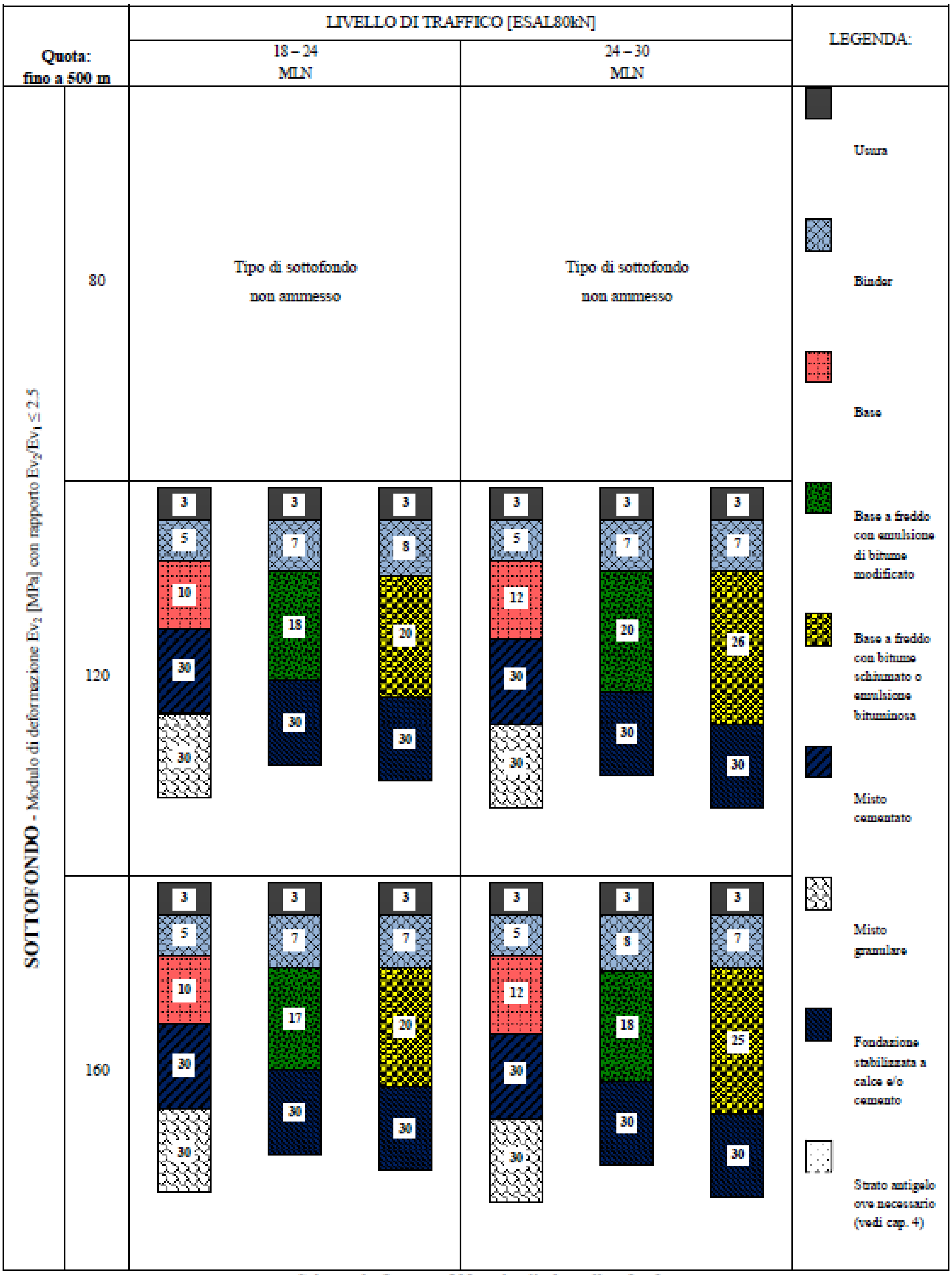
A. Italian structure catalogue including CRM layers



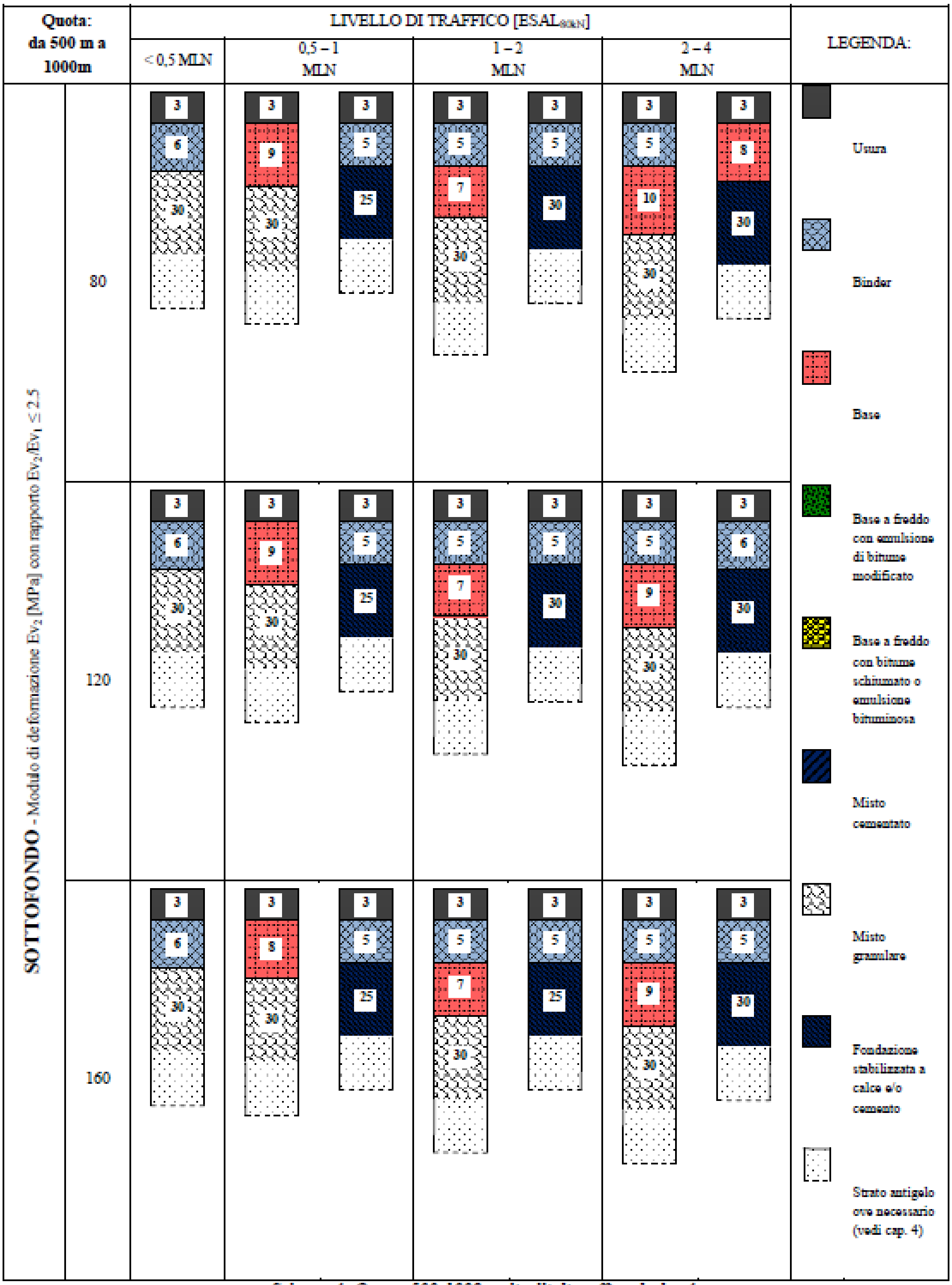
Schema 1: Quota < 500 m, livelli di traffico da 1 a 4



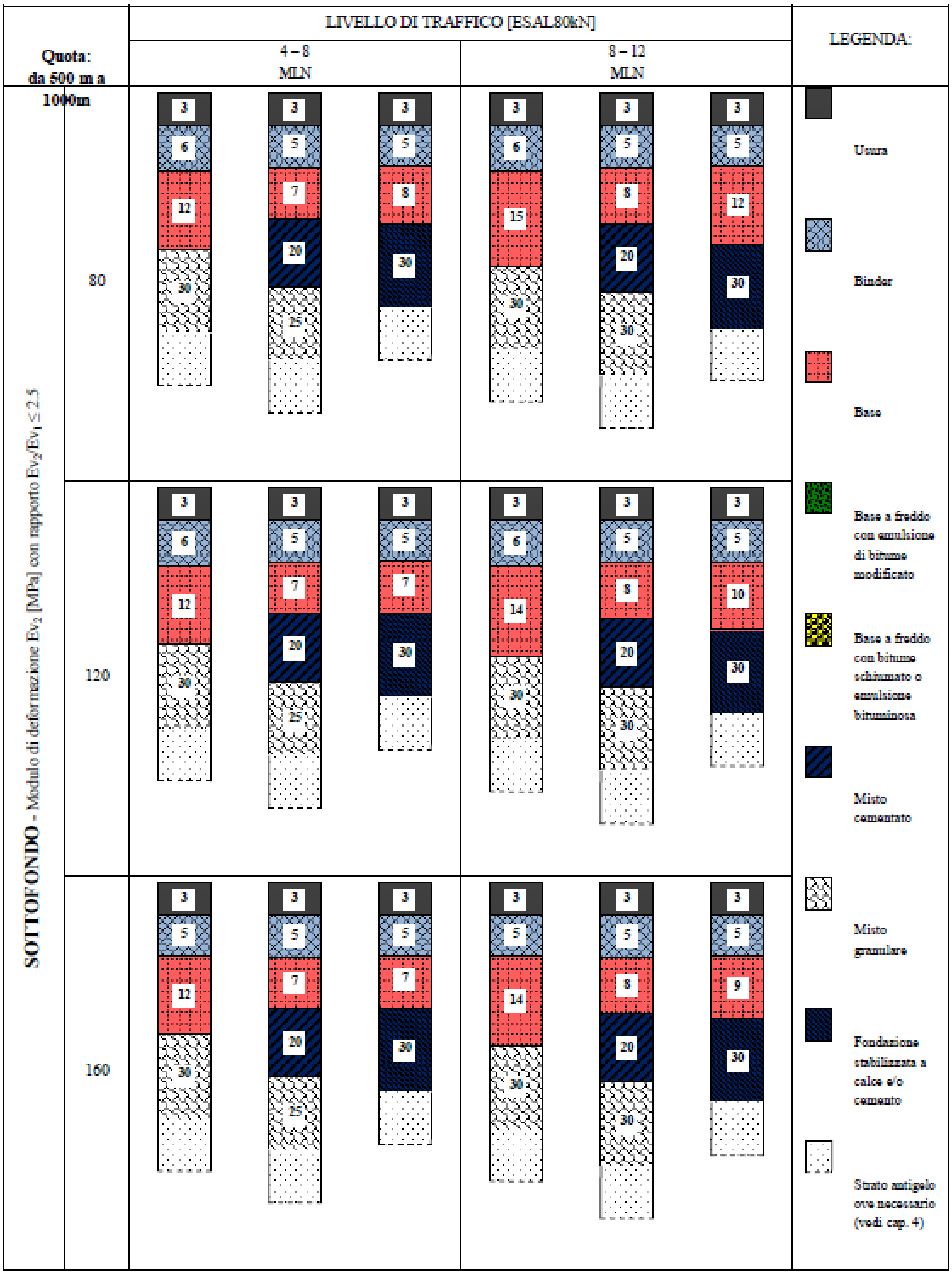
Schema 2: Quota < 500 m, livelli di traffico da 5 a 7



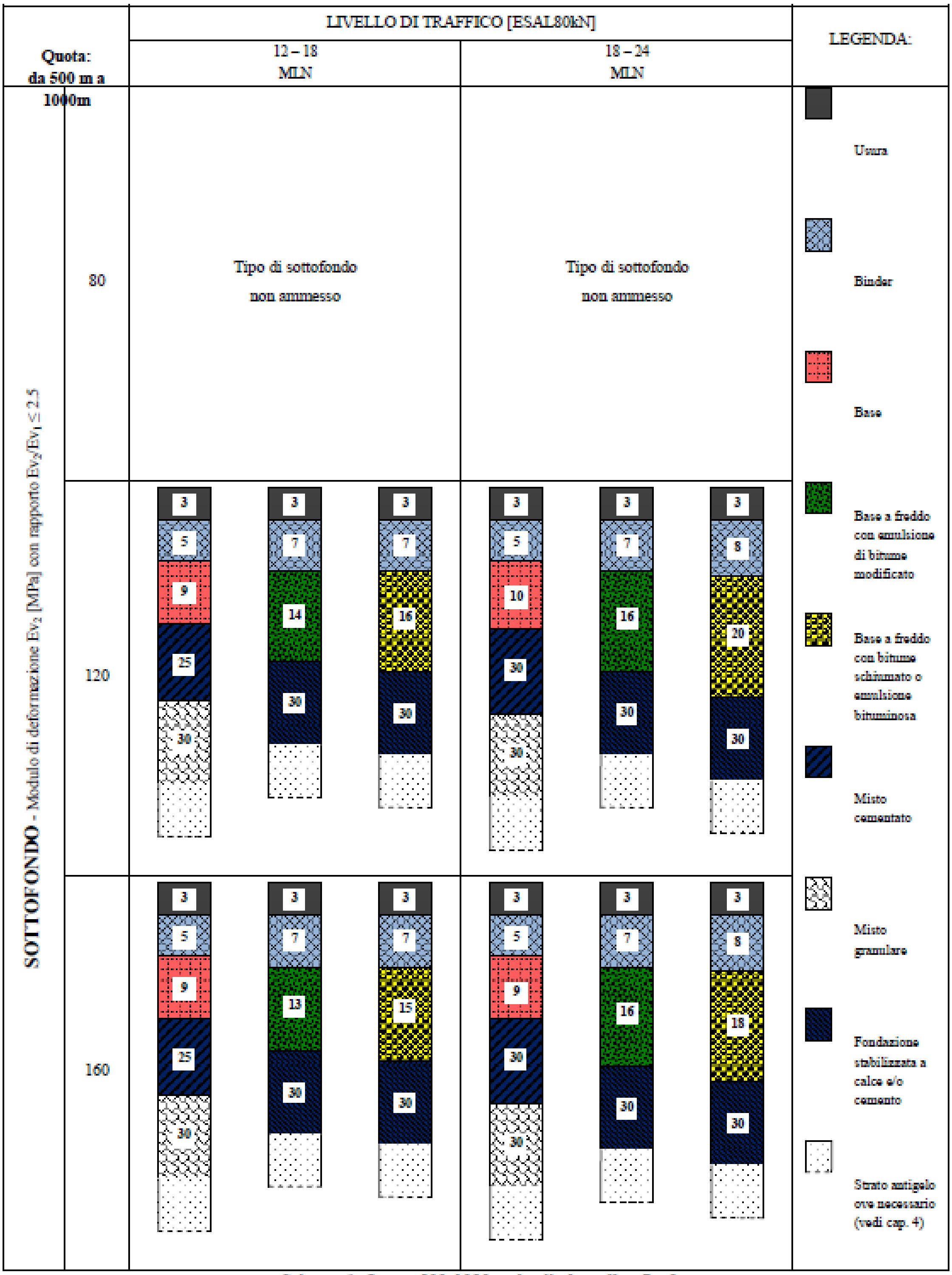
Schema 3: Quota < 500 m, livelli di traffico 8 e 9



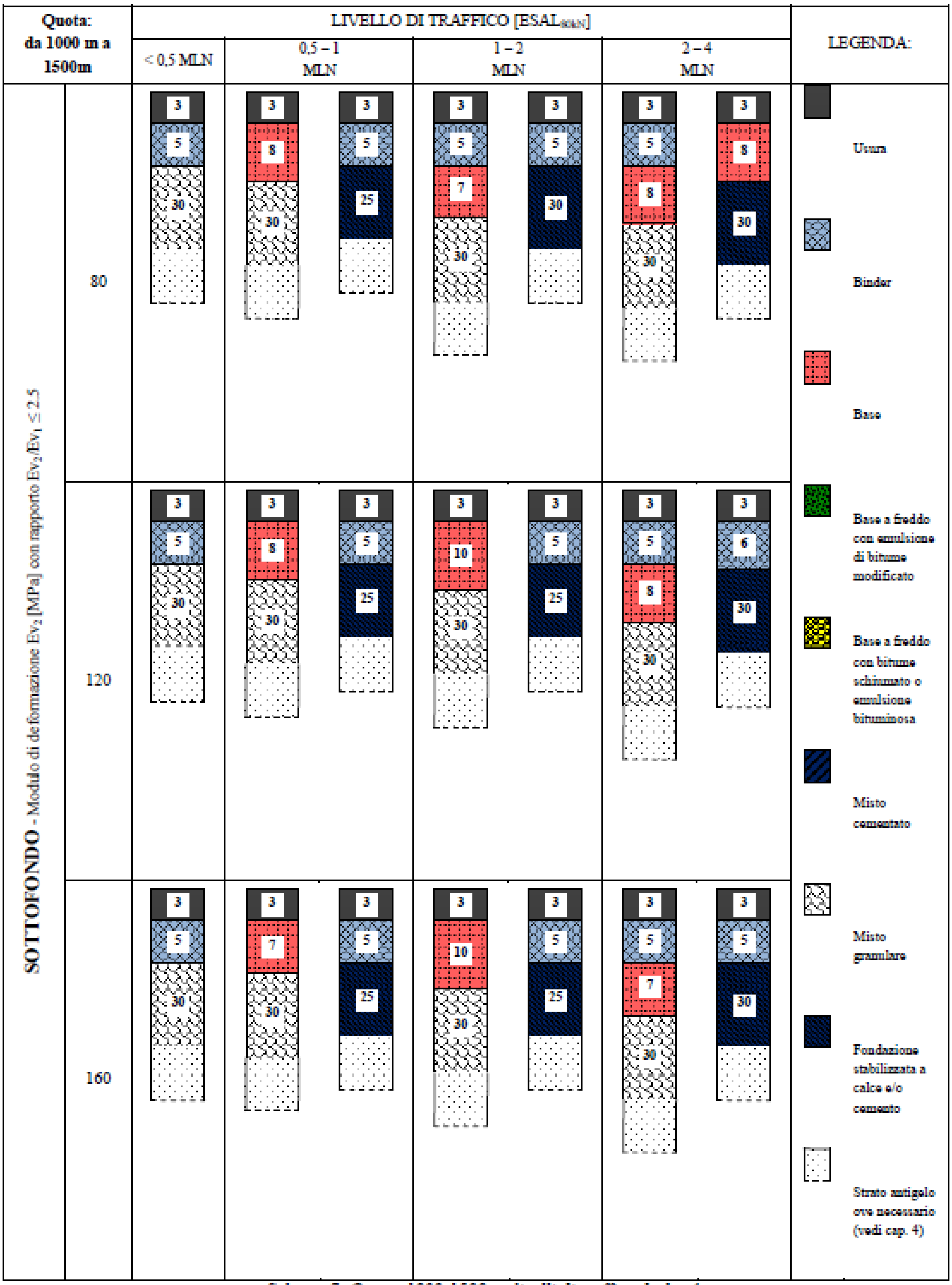
Schema 4: Quota 500-1000 m, livelli di traffico da 1 a 4



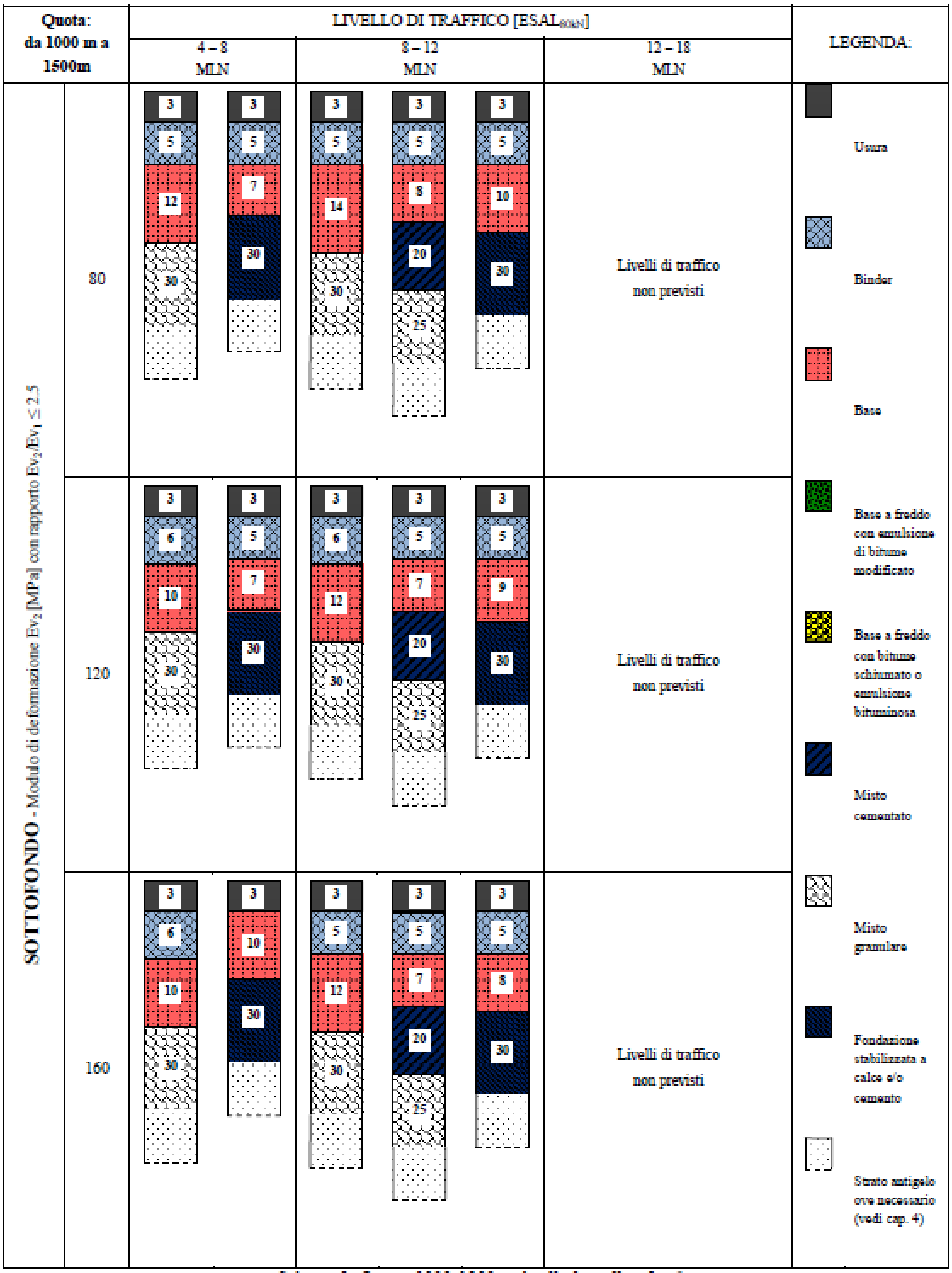
Schema 5: Quota 500-1000 m, livelli di traffico 6 e 7



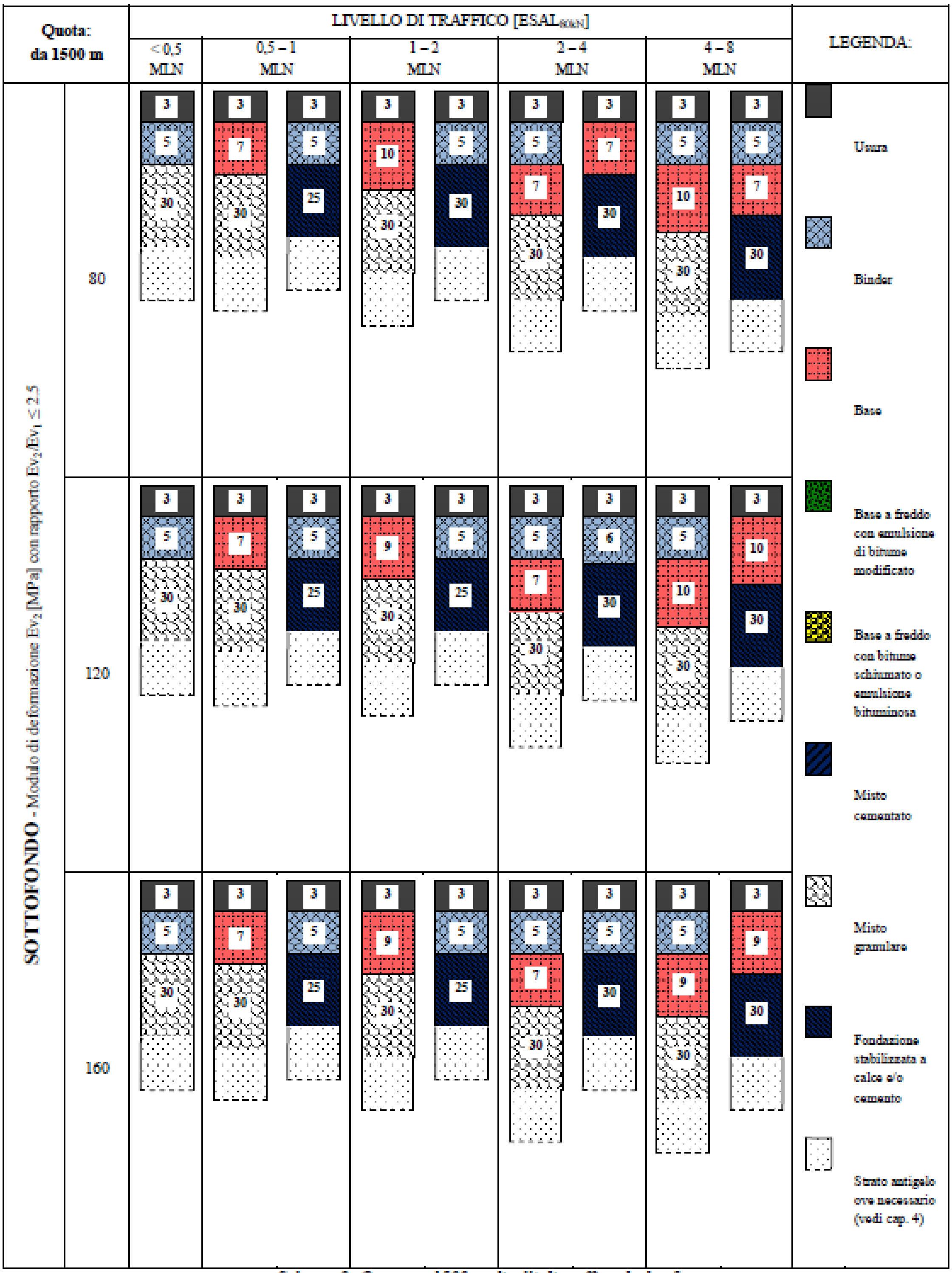
Schema 6: Quota 500-1000 m, livelli di traffico 7 e 8



Schema 7: Quota 1000-1500 m, livelli di traffico da 1 a 4

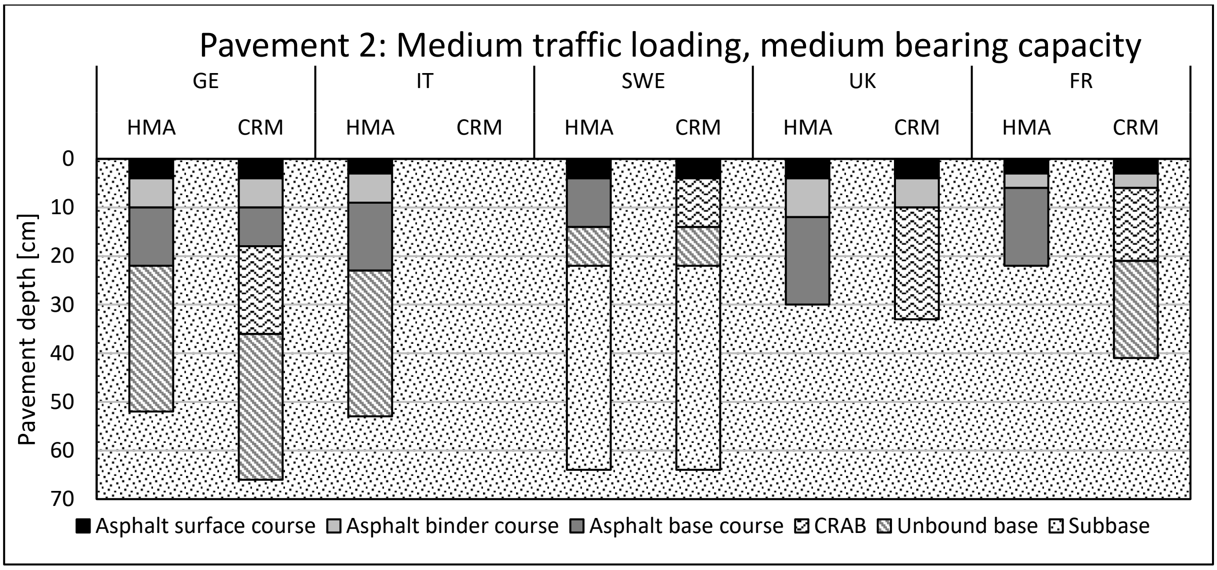


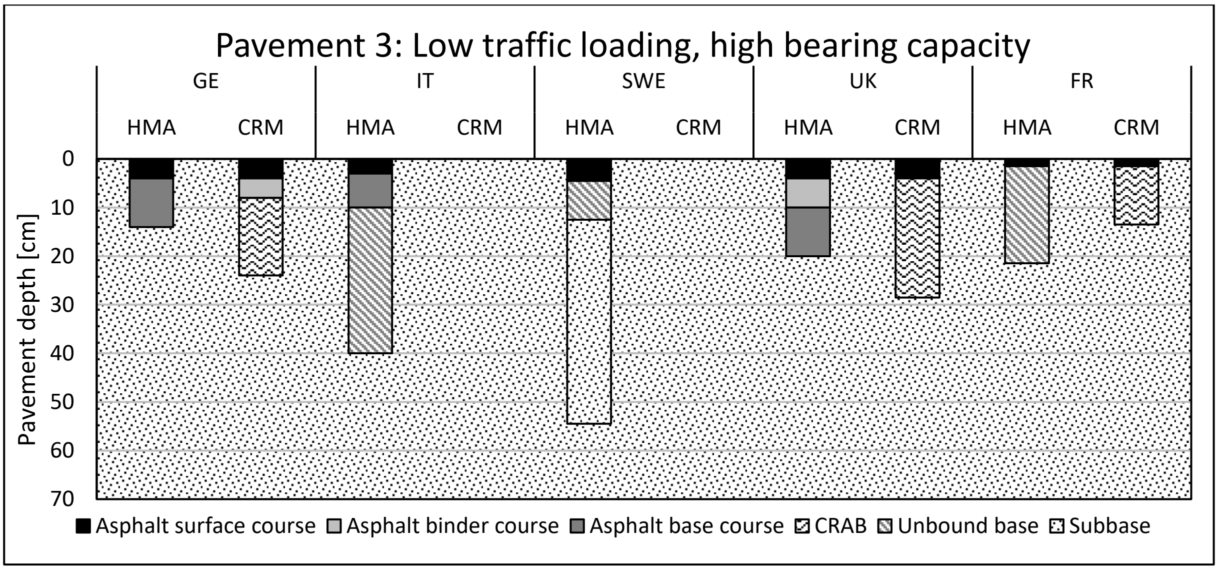
Schema 8: Quota 1000-1500 m, livelli di traffico 5 e 6

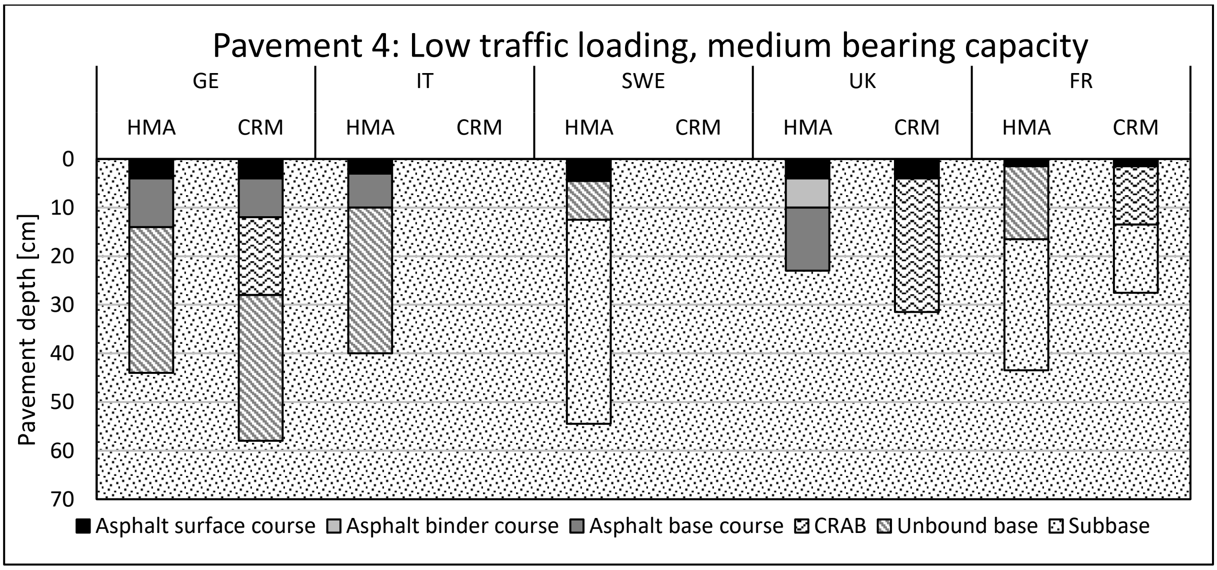


Schema 9: Quota > 1500 m, livelli di traffico da 1 a 5

B. Resulting structures for model pavement 2 to 4







C. Results of all redesigned pavements

[1)] Extrapolation within the design procedure

[2)] Estimation because of missing data

B52	Current state	Redesigned pavement		
		ESAL10	msa8.2	ESAL8.2
ESAL/msa	47,5	47,5	131,4	105,1
deformation modulus subbase	150	150	150	120
surface	3,5	3,5	4	Not applicable
binder	6,5	6,5	6	
base	8	8	0	
CRAB	20	20	32	
total	38	38	42	-

L52	Current state	Redesigned pavement		
		ESAL10	msa8.2	ESAL8.2
ESAL/msa	0,5	0,5	1,3	1,0
deformation modulus subbase	120	120	120	120
surface	4	4	4	Not applicable
binder	6	0	6	
base	10	6	0	
CRAB	20	18	18,5	
total	40	28	28,5	-

L386	Current state	Redesigned pavement		
		ESAL10	msa8.2	ESAL8.2[1)]
ESAL/msa	2,8	2,8	7,8	6,3
deformation modulus subbase	120	120	120	120
surface	4	4	4	3
binder	5	0	6	6
base	8	8	0	0
CRAB	20	20	17,5	12
stabilized base layer				30
total	37	32	27,5	21

SP18	Current state	Redesigned pavement		
		ESAL10	msa8.2	ESAL8.2[1)]
ESAL/msa	2,8	2,8	7,8	6,3
deformation modulus subbase	120	120	120	120
surface	4	4	4	3
binder	5	0	6	6
base	8	8	0	0
CRAB	20	20	17,5	12
stabilized base layer				30
total	37	32	27,5	21

SS38	Current state	Redesigned pavement		
		ESAL10[1)]	msa8.2	ESAL8.2[1)]
ESAL/msa	30	13,6	37,5	30,0
deformation modulus subbase	120[2)]	120	120	120
surface	3	4	4	3
binder	6	4	6	7
base	10	8	0	0
CRAB	30	24	24,5	26
stabilized base layer				30
total	49	40	34,5	36

A14	Current state	Redesigned pavement		
		ESAL10[1)]	msa8.2[1)]	ESAL8.2[1)]
ESAL/msa	150	67,8	187,5	150,0
deformation modulus subbase	160[2)]	150	160	160
surface	4	3,5	4	Not applicable
binder	5	6,5	6	
base	20	8	0	
CRAB	30	20	34	
stabilized base layer				
total	59	38	44	-

SS268	Current state	Redesigned pavement		
		ESAL10	msa8.2	ESAL8.2
ESAL/msa	0,8	10,9 1)	30,0	24,0
deformation modulus subbase	160 2)	120	160	160
surface	5	4	4	3
binder	6	0	6	7
base	0	8	0	0
CRAB	20	24	25,5	20
stabilized base layer	35			30
total	31	36	35,5	30

A21	Current state	Redesigned pavement		
		ESAL10 1)	msa8.2	ESAL8.2
ESAL/msa	25,6	8,7	25,6	19,2
deformation modulus subbase	200 2)	120	200	120
surface	6	4	4	3
binder	0	0	6	8
base	0	8	0	0
CRAB	21	22	22	20
stabilized base layer	20			30
total	27	34	32	31

A46	Current state	Redesigned pavement		
		ESAL10 1)	msa8.2	ESAL8.2
ESAL/msa	17	5,8	17,0	12,8
deformation modulus subbase	200 2)	120	200	160
surface	3	4	4	3
binder	7	0	6	7
base	0	8	0	0
CRAB	14	22	20	15
stabilized base layer	21			30
total	24	34	30	25

A38	Current state	Redesigned pavement		
		ESAL10[1)]	msa8.2	ESAL8.2
ESAL/msa	35	11,9	35,0	26,3
deformation modulus subbase	120[2)]	120	120	120
surface	3	4	4	3
binder	7	0	6	7
base	0	8	0	0
CRAB	25,5	24	24	26
stabilized base layer				30
total	35,5	36	34	36

Rv95	Current state	Redesigned pavement		
		ESAL10[1)]	msa8.2	ESAL8.2
ESAL/msa	10,2	10,2	29,3	22,6
deformation modulus subbase	120[2)]	120	120	120
surface	3,5	4	4	3
binder	5	4	6	8
base	0	8	0	0
CRAB	5,5	24	23,5	20
stabilized base layer				30
total	14	40	33,5	31

E45	Current state	Redesigned pavement		
		ESAL10[1)]	msa8.2	ESAL8.2
ESAL/msa	9,2	9,2	26,5	20,3
deformation modulus subbase	120[2)]	120	120	120
surface	2	4	4	3
binder	6	4	6	8
base	0	8	0	0
CRAB	18	24	23	20
stabilized base layer				30
total	26	40	33	31