CEDR TRANSNATIONAL ROAD RESEARCH PROGRAMME

Call 2015

FALCON

Freight And Logistics in a Multimodal Context
WPC Fit for purpose road vehicles to influence modal choice
(performance based standards)
Task 3.2 Definition of Representative Road Network Library
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Abbreviations

AWF  Axle Wear Factor
B-WIM  Bridge Weigh-in-Motion
CA  Coefficient of fatigue aggressiveness (for bituminous pavements)
ESAL  Equivalent Single Axle Load
GVW  Gross Vehicle Weight
HV  Heavy Vehicle
LEF  Load Equivalence Factor
LM1  Load Model 1
PBS  Performance Based Standard
PER  Vehicle Road Wear Performance
TCF  Tyre configuration factor
VBI  Vehicle-Bridge Interaction
VRS  Vehicle Restraint Systems
VWF  Vehicle Wear Factor
WIM  Weigh-in-Motion
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Management Summary

In project FALCON (Freight And Logistics in a multimodal CONtext), work package 3, entitled “Fit for purpose road vehicles to influence modal choice (performance based standards)” focuses on compiling Smart Infrastructure Access Policy (SIAP) to selected segments of the infrastructure network, for current and future commercial vehicles that have multimodal use potential. Therefore the first step is to establish an extensive infrastructure catalogue, for which design criteria will be chosen and for which the Performance Based Standard will be adapted.

This extensive infrastructure catalogue is given in this deliverable D3.2: information is given for pavements, bridges, tunnels, road geometry elements, safety barriers and warehouses. Then, more details are given for pavements and bridges, as tunnel seem to be more of a service management issue.

For both infrastructure elements (pavement and bridges), damage mechanisms are explained and typical pavements/bridges are described. For each, infrastructure to be assessed for SIAP establishment is proposed: bituminous and concrete pavements as far as pavements are concerned and influence lines of various effects and for various bridges structures.

This deliverable is supposed to be the basis for task 3.5 for definition of Smart Infrastructure Access Policy.
1. Introduction

This report called “Catalogue of representative infrastructure components including roads, bridges and tunnels” is a list of infrastructure elements whose design depends on the characteristics of the traffic. More precisely, the choice and design of infrastructure depend on the traffic volume (number of vehicles) and loads.

This list is organized in categories of infrastructure elements that are commonly used in the assessment of heavy vehicle impact on the road network: development of pavement and bridge design codes (series of Eurocodes, from Eurocode 0 to Eurocode 9), European studies on longer and/or heavier trucks ([1], [2] or [3]), and European research projects on the development of new type of trucks ([4] or [5]). These categories mainly comprise pavements, bridges and tunnels, with geometrical and mechanical requirements, but also road equipment’s and services.

For each of these categories, an overview of the existing infrastructure is given. Then the relationships between the design of this infrastructure and the traffic characteristics (mainly heavy vehicles) are detailed. Finally, a methodology for assessment of pavements and bridges is proposed.

2. Scope – Assumptions

2.1. Design loads and design physical values

This report deals with the design of new infrastructure, and not with the assessment of existing infrastructure. This means that all the information available on the infrastructure is theoretical; indeed, there is no a-posteriori information, obtained on the infrastructure by monitoring or diagnosis. This is the case for the material properties (resistance), the dimensions and design of the infrastructure and for the characteristics of the traffic (volume, loads, etc.).

A corollary of this is that models of vehicles obtained from truck manufacturers are used, and not WIM data as this might have been done for existing infrastructure assessment against current or longer and heavier trucks.

2.2. Present design codes

A second assumption is that only infrastructure designed with existing standards and codes are considered. Indeed, the infrastructure stock is ageing but more and more infrastructure in service were designed with current codes. For example, European bridges built after 1995 were designed with the Eurocodes. Approximately 20% of the total number of bridges, and 20% of the total surface of bridge decks were designed after 1995 (see Figure 1 for French bridges).

Moreover, dealing with older design codes would also mean to take into ageing and the corresponding loss of structural capacity. This cannot be done accurately and in detail for a complete network of infrastructure elements.
2.3. Design structural behaviour

An assumption resulting from the previous one is that the considered infrastructure elements are in nominal shape. Therefore, mathematical equations and physical assumptions used in this report to describe their mechanical behaviour are valid. For example, the bridge bearing capacity and structural behaviour comply with the physical theories (Saint-Venant principle, material resistances, ...) that have been used to design them. Similarly, the pavements considered in this report are supposed not to have been exposed to harsh weather conditions, which would bring about a mechanical behaviour different from the design behaviour.

It is also assumed that infrastructure is correctly designed against the other actions (than traffic loads), and thus considering the traffic loads as the design loads is relevant. For example, for tunnels, if the smoke evacuation is not correctly designed, there might be a risk even with a light traffic.

So this report only considers structures designed according to the in-service rules and whose behaviour is still in accordance with the design principles.

3. Pavements and road structure characteristics

This section reviews typical road structure characteristics encountered in the countries of the project partners. It investigates their resistance to traffic loads, and their ability to carry different levels of traffic.

3.1. Terminology – functional description of pavements

Pavements are multi-layer structures built on top of the subgrade soil, and which function is to support the axle loads, and spread these loads on the subgrade (natural ground), to avoid deformations of the subgrade. A typical pavement structure consists of (see Figure 2):
Figure 2: Pavement structures - terminology.

1) A surface course, which can be divided into:
   - A **wearing course**, which is the top layer, on which the vehicles circulate, and which is also exposed to the effects of climate.
   - Possibly a **binder course**, placed between the wearing course, and the base layers.

The main functions of the surface course is to provide to the pavement good surface characteristics (evenness, skid resistance), to ensure a good riding quality and a good safety for road users, and to protect the road base from the wear due to traffic loads, and from the penetration of water (imperviousness).

2) **Base layers**, which are usually divided in two layers: the **road base** and the **subbase**. These layers consist of materials with a high mechanical resistance. They confer to the pavement the resistance to withstand the loads induced by traffic (fatigue resistance), and distribute the stresses on the pavement foundation.

3) A **pavement foundation**, which consists of the **subgrade** (upper part of the natural ground) often topped with a **capping layer**. The capping layer has two functions:
   - To protect the subgrade during the works phase, providing a surface of goods level and bearing capacity for construction equipment.
   - To improve the homogeneity and bearing capacity of the natural subgrade, and to protect it from frost.

3.2. Pavement deterioration mechanisms

Pavements generally deteriorate under the combined effects of repeated traffic loads and climate. The type of deteriorations depends on the nature of the layer, and on the material composing each layer. The most frequently encountered deteriorations are listed below.

**Wearing course**
- Wear due to the tangential forces imposed by traffic loads,
- Rutting due to creep of bituminous materials, under the combined effects of high temperatures and high traffic stresses, see Figure 3,
- Fatigue cracking, linked with poor bonding between the wearing course and the base layers (Figure 3),
• Cracking due to thermal variations, increased by aging of the bitumen,
• Reflective cracking, due to the propagation of cracks from the pavement base layers.

Figure 3: Damage phenomena in pavements, fatigue (left) and rutting (right).

Treated base layers
• For all materials, fatigue cracking caused by tensile stresses developing at the base of the treated layers, due to bending of the pavement structure under traffic loads,
• For cement-treated materials, cracking due to setting and thermal shrinkage of these materials (Figure 4),
• Pumping and shifting of slabs, in hydraulic or concrete layers with discontinuities (shrinkage cracks, joints), due to poor load transfer at these discontinuities, and to erodibility of the support, see Figure 4.

Figure 4: Other damage phenomena in pavements, like cracking in concrete pavements due to setting and shrinkage (left) and step appearing in pavement due to the discontinuity between concrete slabs (right).
Unbound granular base layers and subgrade:

- Permanent deformations (rutting) due to an accumulation of plastic strains, caused by traffic loads and increased by high moisture contents.

These deterioration mechanisms are detailed for each pavement type in Section 0.

### 3.3. Damage phenomena: fatigue and rutting

Two damage phenomena are commonly studied in pavement design: fatigue (for bituminous and concrete pavements) and rutting (for bituminous pavements).

#### 3.3.1. Fatigue

Fatigue is the phenomenon of damage induced by repeated applications of even-small actions. The expression of the fatigue law is:

\[
\varepsilon = \varepsilon_6 \left( \frac{N}{10^6} \right)^b
\]

(1)

where:
- \(\varepsilon\) is the tensile strain leading to failure for \(N\) load cycles in the laboratory fatigue test (in \(\mu\text{m/m}\)),
- \(\varepsilon_6\) is the tensile strain leading to failure for \(10^6\) cycles (in \(\mu\text{m/m}\)),
- \(b\) is the exponent (dimensionless) of the fatigue law (slope of the fatigue line in a log-log space).

For cement treated materials, the fatigue law is determined from laboratory 2 point bending fatigue tests with stress-controlled sinusoidal loading standard NF P 98-233-1). The expression of the fatigue law is:

\[
\sigma = \sigma_6 \left( \frac{N}{10^6} \right)^b
\]

(2)

where:
- \(\sigma\) is the tensile stress leading to failure after \(N\) load cycles in the laboratory fatigue test (in MPa),
- \(\sigma_6\) is the tensile stress leading to failure for \(10^6\) cycles (in MPa),
- \(b\) is the exponent (dimensionless) of the fatigue law.

The previous fatigue laws are used for design of bituminous pavements.

The number of loads \(N_i\) leading to fatigue failure of the pavement for a single load \(A_i\) is determined as follows:

- The response of the pavement to the load is calculated using the multilayer linear elastic model. The maximum tensile strain at the bottom of the bituminous base layer \(\varepsilon_i\) is determined from this calculation.
- Using the previous fatigue law, the number of loads \(N_i\) leading to failure (crack) is expressed by:
Number of load cycles leading to failure: \[ N_i = 10^6 \times \left( \frac{\varepsilon_i}{K\varepsilon_0} \right)^{1/b} \] (3)

where:

- \( K \) represents a combination of several correction factors, considering different design assumptions (risk of failure, subgrade quality, shift factor accounting for difference between laboratory and in situ performance).

In the case of the French pavement design method, the fatigue law for bituminous materials used is defined from 2-point bending fatigue tests on trapezoidal specimens, with strain controlled sinusoidal loading (standard EN 13286 – annex A). This experimental test and the calculation approach is illustrated in Figure 5.

![Mechanical modelling and Fatigue design criteria](image)

**Figure 5:** Conversion of real axle loads to reference axle loads and calculation of strains in the various layers of the road structure (left) and 2-point bending test for determination of the number of cycles leading to failure for a given strain (right).

The coefficient of fatigue aggressiveness \( CA_v \) of a vehicle \( v \) is defined by:

\[ CA_v = \frac{d_v}{d_{\text{ref}}} \] (4)

where:

- \( d_v \) is the fatigue damage produced by the heavy vehicle \( v \),
- \( d_{\text{ref}} \) is the fatigue damage due to a reference load (for example a 5-axle 40-ton standard vehicle) used as reference for comparison with new vehicle concepts developed in the project.
3.3.2. Rutting (for bituminous pavements)

The approach explained below is used only for bituminous pavements, and no rutting is assumed to occur on concrete roads.

The most comprehensive study in recent years concerning the influence of tyres and axle loads in relation to pavement damage (rutting of asphalt roads in the primary road network) was the so-called COST 334 Study “Effects of Wide Single Tyres and Dual Tyres”, published in 2001 [7]. From extensive rutting tests performed in different European countries with different tyres, tyre configurations, axle loads, inflation pressures, etc. a so-called tyre configuration factor (TCF) was defined. The TCF value relates the pavement wear of a given tyre to the pavement wear of a reference tyre. Within different axle categories (steered, driven or towed axle), there is a wide range of TCF values which reflects the fact that there are more and less pavement damaging tyres and tyre configurations as options possible.

Tyre assembly (single/dual), tyre width and tyre diameter are the most important factors which influences the TCF.

The damage contribution of a single passage of an axle is expressed by the so-called axle wear factor (AWF). This AWF is a dimensionless factor relating the damage contribution of a specific tyre at a given axle load to the damage contribution of a single passage of the reference tyre(s) with a reference axle load. Reference for the AWF means a passage of a 10-t axle equipped with 295/80R22,5 tyres mounted as twin assembly. To adjust the axle load effect on pavement damage a load equivalency factor (LEF) was introduced in the COST 334 formulas. If only asphalt roads in the primary road network are considered and only primary rutting as damage cause is taken into account, the pavement damage increases with the power of 2 by axle load.

The sum of all axle wear factors of a truck combination are called vehicle wear factor (VWF). For equal TCF and LEF the higher the number of axles the higher is the vehicle wear factor, but on the other hand the higher the payload can be.

For the same gross vehicle weight, the higher the number of axles the lower is the axle wear factor for each axle and also the vehicle wear factor as sum of all axles.

The performance of a vehicle regarding pavement wear can be calculated by relating the payload to the vehicle wear factor. This performance indicator: VWF / Payload is abbreviated in the following as PER (vehicle road wear performance). It can be used for relative comparisons of aggressiveness of different vehicles.

The following formulas are generally used for calculation:

- **Load Equivalence Factor (dimensionless):**
  \[ \text{LEF} = \left( \frac{\text{axle load}}{10} \right)^2 \] (5)

- **Tyre Configuration Factor (dimensionless):**
  \[ \text{TCF} = \frac{\text{tyre width}}{470} - 1,65 \times \frac{\text{tyre diameter}}{1059} - 1,12 \] (6)

- **Axle Wear Factor (dimensionless):**
  \[ \text{AWF} = \text{TCF} \times \text{LEF} \] (7)

- **Vehicle Wear Factor (dimensionless):**
  \[ \text{VWF} = \sum (\text{AWF}) \] (8)
Vehicle Road Wear Performance (dimensionless): 
PER: VWF / Payload 

As an example, the impact on road pavement by a 5-axle, 40-ton semitrailer (26-ton payload) is given in Table 1.

Table 1: Impact on road pavement by 5-axle, 40-ton semitrailer (26-ton payload).

<table>
<thead>
<tr>
<th>Tire specification</th>
<th>295/80R22.5</th>
<th>295/80R22.5</th>
<th>385/65R22.5</th>
<th>385/65R22.5</th>
<th>385/65R22.5</th>
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</thead>
<tbody>
<tr>
<td>Twin/single tire</td>
<td>Single</td>
<td>Twin</td>
<td>Single</td>
<td>Single</td>
<td>Single</td>
</tr>
<tr>
<td>Axle load (tons)</td>
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<td>11.5</td>
<td>7.33</td>
<td>7.33</td>
<td>7.33</td>
</tr>
<tr>
<td>TCF</td>
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<td>1.00</td>
<td>2.25</td>
<td>2.25</td>
<td>2.25</td>
</tr>
<tr>
<td>AWF</td>
<td>1.33</td>
<td>1.32</td>
<td>1.21</td>
<td>1.21</td>
<td>1.21</td>
</tr>
<tr>
<td>VWF</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PER</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.24 pour 26-ton payload</td>
</tr>
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3.4. Main types of pavement structures
Pavement structures can be classified five main groups (see Figure 6 and Figure 7), depending on the materials they are composed of:

- Flexible pavements, with unbound base layers,
- Thick bituminous pavements,
- Pavements with base layers consisting of hydraulic materials (also called semi-rigid pavements),
- Composite pavements,
- Cement concrete pavements.

3.4.1. Flexible pavements
These structures consist of a relatively thin bituminous surface course (which can even be reduced to a surface dressing for very low traffic pavements), resting on one or more layers of unbound granular materials. Typically, the thickness of these pavements is between 30 and 60 cm.

These structures are the most economical, but due to the low stiffness of unbound granular base layers, they are suitable only for low to medium traffic levels. For instance, in France, the use of such structures is limited to a maximum traffic of 150 heavy vehicles (HV, GVW > 3.5 tons) per day. They also generally offer a lower level of service than other pavement types, and are generally designed for 10 to 20-year design life.
Due to their low stiffness, the main mechanisms of damage of these pavements are permanent deformations, originating from the subgrade or unbound base layers, appearing in the form of wide radius rutting, or subsidence. Due to the unbound layers, these pavements are also quite sensitive to moisture content variations, and maintaining a good imperviousness of the surface course and good drainage during all their service life are essential for their performance.

3.4.2. Thick bituminous pavements

These structures consist of a bituminous wearing course, over one or two bituminous layers (road base and subbase). The base layers are typically between 15 and 40 cm thick.

**Thick bituminous pavements are suitable for any traffic levels, from medium traffic (100 HV per day), to very heavy motorway traffic (5000 HV per day or more).**

The mechanisms of deterioration of these pavements are mainly:

- Fatigue of the bituminous base layers, due to generation of high tensile strains at the bottom of the bound layers, due to bending of the pavement structure under vehicle loads.
- Rutting, generally affecting only the surface layer, and aggravated by high temperatures, low speeds and particularly heavy loads.

3.4.3. Semi-rigid pavements

These structures comprise a bituminous wearing course, and a base composed of materials treated with hydraulic binders, built in one or two layers (road base and subbase), and having a total thickness varying typically between 20 and 50 cm.

Hydraulic-bound materials have a more brittle behavior than bituminous materials, and so they are more subject to cracking, and also more sensitive to over-loading and to thickness variations of the treated layers. For this reason, hydraulic bound materials are mainly used for thick layers (generally more than 20 cm thick), and are therefore mainly used for pavements for **medium to heavy traffic (more than 150 HV Per day)**. Typical design lives are 20 to 30 years.

The main mechanisms of deterioration of these pavements are:

- Fatigue of the hydraulic-bound base layers, due to generation of high tensile strains at the bottom of these layers, due to bending of the pavement structure under vehicle loads.
- Transversal cracking, due to thermal shrinkage and setting. These cracks, which affect the layers treated with hydraulic binders rise up through the wearing course, appear on the pavement surface and then deteriorate due to traffic loading and water infiltration. Thermal cracking phenomena depend on the severity of temperature variations.
- Rutting of the bituminous surface layers. But the hydraulic bound layers are not subject to rutting.

3.4.4. Composite pavements

These structures consist of a bituminous wearing course, a bituminous road base (typically between 10 and 20 cm thick), and a subbase consisting of materials treated with hydraulic binders (typically between 20 and 40 cm thick). Generally, the ratio of the thickness of the bituminous layers to the total thickness is approximately ½.
Composite pavements are used for similar levels of traffic than semi-rigid pavements, namely **medium to heavy traffic (more than 150 HV Per day)**. Their main advantage is the higher thickness of bituminous materials covering the cement treated layers. For this reason, they are less sensitive to reflective cracking.

### 3.4.5. Cement concrete pavements

These structures consist of a layer of cement concrete (approximately between 15 and 40 cm thick), possibly covered with a thin bituminous wearing course. The concrete layer rests either on a subbase (material treated with hydraulic binder, or lean concrete), or directly on the pavement foundation (subgrade, or capping layer). For the concrete layer, different solutions can be used:

- A continuous slab, with a longitudinal reinforcement (continuous reinforced concrete),
- Discontinuous slabs with or without dowels at the joints.

The mechanisms of deterioration of concrete pavements are similar to those of semi-rigid pavements:

- Fatigue of the concrete, due to generation of high tensile strains at the bottom of these layers, due to bending of the pavement structure under vehicle loads. Fatigue cracking can be increased by the effect of temperature variations (temperature gradients), which tend to deform the slabs. These deformations modify the conditions of contact between the slabs and their support, and can increase the tensile stresses due to traffic loads.
- Transversal cracking, due to thermal shrinkage and setting. In concrete pavements, this cracking is generally controlled by transverse joints, or limited by continuous longitudinal reinforcement. In the case of transverse joints, load transfer is often improved by dowels.

Concrete pavements present a high structural resistance, but also a higher initial cost than other pavement types. For the reason, they are mainly used for **very heavy traffic pavements (motorways, more than 1500 HV Per day)**, and designed for long design lives (30-40 years). When the structural layers are deteriorated, their cost of rehabilitation is also generally very expensive.
The advantages and disadvantages of the various pavement types influence the choice of pavement structure when building a new road, or retrofitting an old one. Indeed, as explained above, selection
of the different types of pavement structures depends on several criteria, including: level of traffic, quality of the subgrade, service life, level of service, climate, availability and cost of different types of materials.

3.5. Pavement design principles

Most countries use so-called mechanistic-empirical pavement design methods, which are similar in their principle. They are based on two main steps:

- A calculation of the stress-strain response of the pavement to a reference load (generally defined as the “equivalent standard axle load”, or ESAL), using a multi-layer linear elastic pavement model.
- The application of several pavement design criteria, which allow to calculate the number of standard axle loads (ESALS) which can be supported by the pavement before failure (also called the pavement life), in function of the maximum level of stress or strain calculated in each pavement layer.

The design criteria used depend on the type of pavement, and on the nature of the pavement materials:

- For low traffic pavements, the design criterion is generally based on the maximum level of the vertical compressive strain at the top of the subgrade \( \varepsilon_z \). This criterion is defined as a “rutting criterion” of the subgrade, because the level of permanent deformations in the subgrade is strongly related with \( \varepsilon_z \).

- For thick bituminous pavements, there are generally two design criteria:
  - The first design criterion is based on the maximum tensile strain at the bottom of the bituminous layers, \( \varepsilon_t \). This criterion is defined as a “fatigue criterion” of the bituminous layers, because it relates the fatigue life of the bituminous material with the maximum tensile strain at the bottom of the bituminous layers, \( \varepsilon_t \).
  - The second criterion is the same rutting criterion of the subgrade, based on the maximum vertical strain \( \varepsilon_z \) at the top of the subgrade.

- For pavements with layers treated with hydraulic binders, there are two design criteria:
  - The first design criterion is based on the maximum tensile stress at the bottom of the bituminous treated layers, \( \sigma_t \). This criterion is defined as a “fatigue criterion” of the layers treated with hydraulic binders, because it relates the fatigue life of the hydraulic bound material with the maximum tensile stress \( \sigma_t \).
  - The second criterion is again the rutting criterion of the subgrade, based on the maximum vertical strain \( \varepsilon_z \) at the top of the subgrade.

All mechanistic pavement design methods are based on several main parameters:

- The traffic level and the service life, which can be converted into a number of Equivalent Standard Axle Loads (ESAL) that the pavement must support,
- The bearing capacity of the subgrade (elastic modulus), which is taken into account in the pavement model,
• The **mechanical properties** of the pavement materials (elastic modulus, Poisson ratio, fatigue properties...),

• The climate, and in particular the **temperature** (single value, or several climatic periods) considered for the bituminous materials, which have a strongly temperature-dependent behavior,

• A **factor of safety**, (called risk coefficient in the French method), which can be used to adjust the number of loads to failure. For example, in France, a low probability of failure is considered in the design for heavy traffic roads, on which a high level of service must be ensured.

### 3.6. Road structure catalogue. Methodology for assessment for SIAP

#### 3.6.1. Methodology for pavement assessment

According to the presentation of the different types of pavement structures, and of the pavement design principles, to establish a representative “catalogue”, or a representative library of pavement structures, for Falcon, the following main factors need to be considered:

• The **type of pavement structure**: Flexible, thick bituminous, semi-rigid, composite, concrete pavement,

• The **level of traffic**, expressed by the number of heavy vehicles (HV) per day and ESALs,

• The chosen **level of service**, which can be expressed by a factor of safety, or a risk of failure,

• The **bearing capacity of the subgrade**,

• The **mechanical characteristics of the pavement materials**.

#### 3.6.2. Example of pavement design catalogue

##### 3.6.2.1. French case

As an example, France has developed a pavement design catalogue ([Catalogue des structures de chaussées neuves, SETRA, LCPC, 1998](#)) still in use today. This design catalogue is made for the national road nework, this means mainly medium to heavy traffic roads.

Two categories of roads are distinguished:

- Main road corridors, designed for 30 years, and considering a high aggressiveness of the traffic
- Other national roads, designed for 20 years, and considering a lower aggressiveness of traffic.

For each category of roads, different catalogue sheets are proposed, for

- Each **type of pavement structure** (5 types): flexible, thick bituminous, semi-rigid, composite and concrete.

- **Different classes of base layer materials**, with different mechanical properties

Each catalogue sheet then proposes several structure thicknesses, depending on:

- The **level of traffic**,
- The **class of bearing capacity of the subgrade** (3 classes : 50, 120 and 200 MPa).
An example of catalogue sheet, for thick bituminous pavements, with a widely used base course asphalt material, called GB3, is presented in Figure 8: in this figure, the horizontal variable is the soil Bearing capacity (category) in MPa. The vertical variable is the cumulative traffic in millions of heavy vehicles, over the lifetime of the road structure to be designed.

Figure 8: Example of French pavement catalogue sheet – thick bituminous pavements structures with base course material GB3, proposed for different levels of traffic and subgrade categories (CS means “couche de surface) which means top layer).
3.6.2.2. Belgian case

In Flanders, there are a number of standard road structures, see [14] about the standard structures with bituminous upper layers, and [15] and [16] describing standard road structures of “all” types. For example, for in [14], the structure of the pavement structure for various traffic classes is given, see Figure 9.

![Figure 9: Road pavement structure with classic undercoat, cement stabilized stone foundation, thicknesses in cm, for various traffic classes.](image)

In Wallonia, the software design tool “Qualidim” is used for the design of new road structures. However, many roads have old lower layers that are built before this software existed (before approximately 2006), so they are designed according to an internal document of the NRA.

The Qualidim software for design of road structures (as in use in Wallonia, Belgium) is described here: [http://qc.spw.wallonie.be/fr/qualiroutes/qualidim.html](http://qc.spw.wallonie.be/fr/qualiroutes/qualidim.html) [17] or [18].

Globally, all these road structures are quite similar to the French, although they may not be entirely identical.

This is also the case for Swedish pavements.

3.6.3. Pavement structure catalogue proposed in FALCON

As explained in the previous sections, European road networks consist of different types of pavement structures, built with different types of materials (bituminous, unbound, treated with hydraulic binders), presenting different deterioration mechanisms. These different pavement types are also designed for different traffic levels. Finally, the effect of heavy vehicles on pavements depends on the type of pavement structure, because load spreading is different on a thick pavement, built with stiff materials, and on a thin low traffic pavement.

To cover most possible situations, it is proposed to consider the following pavement structures:

1. **One flexible, low traffic pavement** (granular base, bituminous wearing course) designed for a level of traffic T5, according to the French pavement design guide. This level corresponds to 25 heavy vehicles / day during 20 years,
2. **two thick bituminous pavements**, designed for 2 different levels of traffic (medium and high): T0, which corresponds to 1200 heavy vehicles/day during 30 years, and T1, which corresponds to 500 heavy vehicles/day, during 20 years,
3. **Two semi rigid pavements** (cement treated base, bituminous wearing course), also designed for the levels of traffic T0 and T1.
4. **Two concrete pavements**, also designed for the levels of traffic T0 and T1.

In addition, it has been considered that for the highest level of traffic (T0) a subgrade with a high bearing capacity (120 MPa) will be used, to guarantee a high level of service, and for the medium traffic (T1), a subgrade with a lower bearing capacity (50 MPa).

The different types of pavements are illustrated on Figure 10. The precise characteristics of the selected pavement structures are summarised in Table 2.

---

**Figure 10.** Types of pavement structures considered in FALCON project.
## Table 2. Pavement structures proposed for evaluation of impact of heavy vehicles on pavements.

<table>
<thead>
<tr>
<th>Traffic</th>
<th>Pavement structures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thick Bituminous</td>
</tr>
<tr>
<td>T0</td>
<td>1200 HV/day during 30 years</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>T1</td>
<td>500 HV/day during 20 years</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>T5</td>
<td>25 HV/day during 20 years</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The different pavement material characteristics correspond to standard material classes widely used in France. Their characteristics are summarized in Table 3, for the bituminous materials, and Table 4 for the materials treated with hydraulic binders.

### Table 3. Main characteristics of the selected bituminous materials

<table>
<thead>
<tr>
<th>Material</th>
<th>E (MPa)</th>
<th>ν</th>
<th>εₜ (μstrains)</th>
<th>-1/b</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At 15 °C, 10 Hz</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BBSG</td>
<td>5500</td>
<td>0,35</td>
<td>100</td>
<td>5</td>
</tr>
<tr>
<td>GB3</td>
<td>9000</td>
<td>0,35</td>
<td>90</td>
<td>5</td>
</tr>
</tbody>
</table>

*E*: elastic modulus  
*ν*: Poisson ratio  
*εₜ*: limit tensile strain leading to failure for $10^6$ cycles  
*b*: exponent of the fatigue law
Table 4. Main characteristics of the selected materials treated with hydraulic binders

<table>
<thead>
<tr>
<th>Material</th>
<th>E (MPa)</th>
<th>v</th>
<th>σ₆ (Mpa)</th>
<th>-1/b</th>
</tr>
</thead>
<tbody>
<tr>
<td>GC3</td>
<td>23000</td>
<td>0,25</td>
<td>0,75</td>
<td>15</td>
</tr>
<tr>
<td>BC5</td>
<td>35000</td>
<td>0,25</td>
<td>2,15</td>
<td>16</td>
</tr>
<tr>
<td>BC2</td>
<td>20000</td>
<td>0,25</td>
<td>1,37</td>
<td>14</td>
</tr>
</tbody>
</table>

E: elastic modulus    - v: Poisson ratio
σ₆: limit tensile stress leading to failure for 10⁶ cycles    - b: exponent of the fatigue law

To summarize, the flexible design is generally for a low traffic road and the other designs are for heavier traffic.

But the choice between bituminous or cement concrete roads is not only traffic related but also made in function of life-time expectancy (in years, already mentioned above), initial construction costs, direct maintenance costs (how often maintenance is needed and how much the maintenance costs) and indirect costs (road closure for maintenance, mobility related issues).

4. Bridges

4.1. Catalogue of existing bridge structures

Bridges are of various structural natures and various materials, see Figure 11.
Figure 11: Various types of bridge structures, from left to right and from top to bottom: masonry arch bridge on river l’Aurence in middle France, prestressed concrete viaduct near Orléans, prestressed viaduc de Meaux near Paris and viaduc de Millau which is a steel orthotropic deck.

Bridges can be classified in different categories: (1) by type of material (masonry, prestressed concrete, reinforced concrete, ...) which is often done in the governmental reports, (2) by number of spans, or (3) by mode using the bridge (railway, road or canal), ...

The span length varies from approximately 5 meters (integral concrete frames) to several hundreds of meters (mostly steel bridges).

To design a bridge, one chooses the number and position of supports (and thus the span length). This already leads - more or less - to a choice in material (steel, concrete) and construction method. The characteristics of the materials and the final sections of the elements are determined by calculated the stresses induced by the load models of Eurocode 1 [20] in the structure. The stresses have to stay below of the design stresses of the materials.

The best way to obtain a panel of these structures is to study the influence lines, which are a physical function corresponding to the response function (shear, bending moment, ...) of the structure as the downward unit load moves across the structure.

4.2. Damage phenomena: extreme effects and fatigue. Methodology for bridge assessment

A bridge should be designed or assessed in a static behaviour and mostly also in a dynamic behaviour. The dynamic assessment corresponds to a complete VBI (Vehicle-Bridge Interaction) model and makes it possible to compare the eigenfrequencies of the bridge with those of the actions applied on the structure. This is quite important but not always done in a first approach of bridge design.

Therefore, we will focus on the static assessment of bridges. Two damage phenomena have to be considered: extreme effects and fatigue. To do that, the static effects of vehicles on a bridge have to be calculated.

4.2.1. Static effect of traffic on a bridge

The effect on moving loads is given by the convolution of these moving loads with the influence line of the effect. In particular, a moving vehicle with N axles would be considered as N moving loads, with given axle loads and given distance between the axles.

If \( I_i(x) \) is the value of the influence line of effect \( i \) at coordinate \( x \) and we consider truck \( j \), the global effect at coordinate \( x \) \( E_i(x) \) is given by the sum of the effects of all axles:

\[
E_{i,j}(x) = \sum_{n=1}^{N} P_{j,n} l_i(x - d_{j,n}),
\]

(10)

where:

- \( P_{j,n} \) is the axle load on axle \( n \) of truck \( j \),
- \( d_{j,n} \) is the distance between axle 1 and axle \( n \) for truck \( j \) (so by definition, \( d_{j,1} = 0 \)).

The values of \( P_{j,n} \) and \( d_{j,n} \) are given by the vehicle configurations and the function \( l_i(x) \) is representative of the chosen infrastructure.
The effect $E_{i,j}(x)$ is generally a stress (units: MPa or $kN/m^2$) or a strain (units: $\mu m/m$) in the structure.

4.2.2. Extreme effects

The maximum of this effect $E_{i,j}(x)$ is then assessed for various vehicle configurations. Two vehicle configurations can then be compared in terms of extreme effects by calculating the ratio of the effect $E_{i,j}(x)$.

More precisely, to compare the extreme effects induced by two different vehicles, the ratio of each vehicle is given by:

$$R_{i,j} = \frac{E_{i,j}}{E_{i,ref}},$$

where:

- $R_{i,j}$ depends on the type of effect $i$ and the vehicle $j$,
- $E_{i,j}$ is the maximum effect $i$ of vehicle $j$,
- $E_{i,ref}$ is the maximum effect $i$ of the reference vehicle (for example, the 40-t conventional trailer).

By definition, for all effects, $\forall i, R_{i,ref} = 1$.

4.2.3. Fatigue

Fatigue is a slow damaging process affecting mainly steel bridges, or steel elements of composite bridges. It consists of a progressive and localized structural damage identified by crack propagation in a material is subjected to cyclic loading. Fatigue is governed by GVW and/or axle loads, depending on the span length and the active load effect.

The lifetime in fatigue of a structure is either the total number of stress cycles to failure, or the duration to get them. The number of cycles to failure $N$ for repeated loads of constant amplitude depends on the stress amplitude $\Delta \sigma$ as given by the S-N (Woehler) curves (Figure 12):

$$\begin{cases} 
N \times \Delta \sigma^3 = 5.10^6 \Delta \sigma_D^3 & \text{if } \Delta \sigma \geq \Delta \sigma_D , \\
N \times \Delta \sigma^5 = 5.10^6 \Delta \sigma_L^5 & \text{if } \Delta \sigma_D > \Delta \sigma \geq \Delta \sigma_L , \\
N = \infty & \text{if } \Delta \sigma < \Delta \sigma_L ,
\end{cases}$$

where:

- $\Delta \sigma$ is the stress cycle amplitude,
- $\Delta \sigma_L$ is the fatigue limit (depends on the material and the structural element shape, and is given in the Eurocode 3 or other standards),
- $\Delta \sigma_D$ is the endurance limit (depends on the material of the structural element).

The elementary damage induced by one cycle of amplitude $\Delta \sigma$ is $1/N$, and for a series of $n$ cycles of this amplitude, the cumulated damage is $n/N$. For a series of cycles with variable amplitudes, counted with the “rain-flow” method, represented by an histogram ($n_i, \Delta \sigma_i$), the total damage induced is given by the Miner’s law: $d=\Sigma (n_i/N_i)$. Finally, the lifetime is: $T=t/d$, where $t$ is the duration to get the series of cycles considered.
The aggressiveness of a given vehicle is proportional to the elementary damage caused to a detail (a structural element) by this vehicle crossing a bridge, and is generally given by an equivalent number of standard vehicles (to be defined) causing the same damage.

![Figure 12: S-N Woehler curves (extracted from Eurocode 3, EN1993).](image)

This damage phenomena makes it possible to compare the effect of different vehicles. But to design bridges, one would use the load models of Eurocode 1 [20].

### 4.3. Design models for bridge design

Indeed, bridges are designed according to the European standards called Eurocodes, whose application is mandatory since 2010; for example, prestressed and reinforced concrete bridges are designed according to Eurocode 1992; steel bridges are designed according to Eurocode 1993. This means that these standards define how the bridge has to be drawn, regarding the stress state in the structure. Eurocode 1991-2 provides the traffic load models, both for extreme loads and fatigue loads.

The stresses are calculated by the structural resistance theory and applying the relevant load models. There are also load models for climatic actions (temperature, snow loads, wind loads, ...).

For the project FALCON, we focus on the Eurocode 1991-2 on traffic on road bridges [20]. The extreme effects in a structure result from the load models 1 or 2 (LM1 or LM2). For calculation of fatigue lifetime, the load model 3 (LM3) is applied. These models consist generally in a uniform load spread over the traffic lanes of the bridge, plus some additional local loads (representing axles).

These load models integrate some safety margins, above all dynamic impact factors, and thus may be too conservative in some cases. Therefore the possibility exists to use recorded traffic data (WIM data), i.e. load model 4 (LM4).
These LM1, LM2 and LM3 are only valid for span length below 200 metres. Above this limit, the LM4 must be used.


As stated before, European bridges are designed according to load models defined in the Eurocode 1991-2. These load models integrate safety margins and are supposed to represent the whole traffic being at a same time on a bridge.

This means that bridges are designed and assessed according to standards that are applied Europe-wide. The only differences from one country to another are due to some National $\alpha$-factors applied to the load intensity of the load models to adjust them to the National traffic conditions. The design criteria will be listed in another deliverable (D3.4).

This catalogue of bridge structures contains an exhaustive list of theoretical structures to which these loads models are applied. These theoretical structures are represented by influence lines for the various load effects to be considered as shown in Table 5.

For these bridge structures, the effects of the various vehicle configurations determined in Task 3.1 of FALCON will be calculated. The outer envelope of these effects will be searched and provided as the upper limit for design criteria of a PBS.

Table 5: Catalogue of infrastructure to be assessed.

<table>
<thead>
<tr>
<th>Bridge structure</th>
<th>Effect</th>
<th>Span length</th>
<th>Damage model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Simply supported, single span</td>
<td>Bending moment at midspan 1, shear at support 0</td>
<td>10m, 20m, 35m, 50m, 100m</td>
<td>Extreme effects &amp; Fatigue</td>
</tr>
<tr>
<td>2 Two-span, continuous bridge</td>
<td>Bending at midspan 1 and support 1, shear at support 0</td>
<td>10m, 20m, 35m, 50m, 100m</td>
<td>Extreme effects &amp; Fatigue</td>
</tr>
<tr>
<td>3 Three-span, continuous bridge</td>
<td>Bending at midspans 1 and 2, at support 1, shear at support 0</td>
<td>10m, 20m, 35m, 50m, 100m</td>
<td>Extreme effects &amp; Fatigue</td>
</tr>
</tbody>
</table>

These calculations are numerous (90 calculations), there the way to determine the outer envelope will have to be discussed.

5. Tunnels

Tunnels belong to another infrastructure to be designed according to the expected traffic actions.
The main parameters to be known are:

- The inner tunnel dimensions which limit the outer vehicle dimensions: height and width,
- The axle loads for design of the road wear in the tunnel,
- The calorific power of the truck (payload and truck itself, incl. the fuel),
- The type of payload (e.g. dangerous good).

The design rules for tunnels are the same all over Europe.

Tunnels are classified according to [32]:

- length of tunnel;
- and traffic flow;
- and possibly also: one way or two way;
- urban or inter-urban;
- oversea or subsea; and
- level of service (type or class of road).

It classifies safety equipment for tunnels into: (1) emergency and evacuation aid equipment; (2) traffic regulations; and (3) tunnel operation systems.

The vehicle dimensions can be monitored at the entrance of entrance tunnel in order to filter the trucks which do not comply with the limits of the tunnel.

The emergency garages in the tunnels are designed nowadays to accommodate existing trucks (up to 18.25 metres in Europe).

The most important parameter linked with the weights and dimensions of the trucks is the calorific power of the truck. Several tunnel management strategies may be implemented: to limit the number of dangerous good trucks being in the tunnel at the same time, to escort these trucks, to only allow them crossing the tunnel without other vehicles, or even to forbidden them.

The heat release for fires classified as causing some damage to the tunnel is estimated to be below 20 MW. Only for the very serious fires a heat release of more than 20 MW is considered. So, major fires are very rare events even in relation to the whole number of lorry fires in tunnels.

Different fire characteristics are needed depending on whether the purpose is to design the tunnel structure or the ventilation facilities (smoke control).

- The design of structures for fire resistance is based on the temperature of the hot air (°C) versus time.
- The design of ventilation is based on the heat release rate (thermal power in MW) or the smoke release rate (flow at the temperature of the hot smoke in m3/s) versus time.

The dependence upon time is very important to evaluate the conditions at the beginning of the fire, taking into account the evacuation phase (time for fire brigade to arrive and get organised).

For design purposes, it is necessary to choose typical fire characteristics corresponding to the traffic which uses a particular tunnel. Conditions like the permits of hazardous transports have to be taken into account.
According to the EU Directive concerning minimum safety requirements for tunnels (Directive 54/2004/EC), a risk analysis should be performed regarding the transportation of dangerous goods, see Figure 14.

**Figure 13:** Examples from calorific power.

**Figure 14:** Risk analysis flowchart, from [32].
6. Road Geometry

The geometry of the road is an important point when talking about dimensions of trucks. Indeed, trucks have to be compliant, in a static and a dynamic way, with the existing infrastructure, meaning the roundabout, the right turn, left turn, the ramps...

Some provisions of the Directive 96/53/CE modified by the Directive 2015/719/CE [9], are very similar to PBS, e.g. the minimum radiuses the trucks are supposed to be able to drive. But most of the provisions are prescriptive, such as the vehicle length, width and height.

These rules will be reviewed in the task on the vehicle policy review, linked to report D3.3. Therefore, we will give here the example values or definitions for France for the sake of understanding, but we will not review the European situation. Indeed, road geometry is one of the major issues when dealing with longer and/or heavier trucks, see Appendix 1: List of infrastructure components to take into consideration when selecting longer and/or heavier vehicle (LHV) routes.

6.1. Definitions of main road geometrical characteristics

Design of roads is based on definition of different road categories, which define the main geometrical characteristics of the road, such as (see Figure 15 and Figure 16):

- Road alignment,
- Longitudinal profile,
- Cross section,
- Other characteristics like types of intersections.

Figure 15: Longitudinal profile (slope, from Wikipedia: https://en.wikipedia.org/wiki/Grade_(slope)) and cross-section of road (cross-slope can also be said “crossfall”, from Wikipedia https://en.wikipedia.org/wiki/Cross_slope)

Figure 16: Examples of road alignment (top, left), curves and transition lane (bottom, right).
These design guidelines take into account elements like vehicle speed, vehicle type, road grade (slope), distance of visibility, and stopping distance.

Road geometry is strongly related to vehicle dynamics. The stability of vehicles depends on the observance of rules linking vehicle speed, radius of curves and cross slope. Road alignment also influences sight distance, which is an important factor for safety.

6.1.1. Road alignment
Horizontal alignment in road design consists of straight sections of road, known as tangents, connected by circular horizontal curves. Circular curves are defined by their radius and deflection angle (extent). The design of a horizontal curve entails the determination of a minimum radius (based on speed limit), curve length, and objects obstructing the view of the driver.

6.1.2. Longitudinal profile
The profile of a road consists of road slopes, called grades, connected by parabolic vertical curves. Vertical curves are used to provide a gradual change from one road slope to another, so that vehicles may smoothly navigate grade changes as they travel. Sag vertical curves are those that have a tangent slope at the end of the curve that is higher than that of the beginning of the curve. Crest vertical curves are those that have a tangent slope at the end of the curve that is lower than that of the beginning of the curve.

The longitudinal profile also affects road drainage.

6.1.3. Cross section
The cross section of a roadway defines the number of lanes, their widths and cross slopes, as well as the presence or absence of various features like shoulders, curbs, sidewalks, drains and ditches.

6.1.4. Transition curve
A transition curve is a horizontal curve of varying radius which is used to connect straight line to circular curve. It is defined according to its length L, which depends on the type of road.

6.2. Safety effects of road geometry
The geometry of a road influences its safety performance. While studies of contributing factors to road accidents show that human factors predominate, roadway factors are the second most common category.

The safety of a horizontal curve is affected by the length of the curve, the curve radius, whether spiral transition curves are used, and the superelevation of the roadway. Cross slope and lane width also affect the safety performance of a road. Finally, Road geometry affects the sight distance available to the driver. Sight distance is defined as "the length of roadway ahead visible to the driver." Sight distance is how far a road user (usually a vehicle driver) can see before the line of sight is blocked by a hill crest, or an obstacle on the inside of a horizontal curve or intersection. Insufficient sight distance can adversely affect the safety or operations of a roadway or intersection. Sight distance is linked with the maximum speed allowed on the road.

6.3. Road categories for geometry design
International road classifications distinguish 3 main types of roads:
6.3.1. Motorways
Motorways are roads reserved for motor vehicles, designed for long distance traffic, with two distinct carriageways, separated by a central reservation. They have no at-grade intersection with other types of infrastructures (other roads, railway tracks, pedestrian paths...). They are specifically designated as motorways.

6.3.2. Express roads
Express roads are also reserved to motor vehicles, and on which it is forbidden to park or stop, except in case of emergency. Their junctions and intersections are also generally grade-separated. They can have one or two carriageways.

6.3.3. Ordinary roads
Ordinary roads are open to all categories of road users and vehicles. It can have one or two carriageways, and can vary in quality, from newly built local roads with two carriageways to narrow local roads.

6.3.4. Example of extended road categories
In France for example, this classification is expanded into 4 categories (see Figure 17):

- Motorways, (type L),
- Express roads (type T),
- Interurban arterials (Type R, with 2 carriageways). They are designed for heavy traffic, but cannot be classified as express roads, because they have occasional at-grade intersections, possibly with traffic lights that stop traffic,
- Ordinary roads (type R, with one carriageway).

![Motorway](image1)
![Express road](image2)
![Interurban arterial](image3)
![Ordinary road](image4)
Each of these categories has a reference speed, and a series of main characteristics, summarized in Table 6. The reference speed can be 60, 80, 100 or 120 km/h, and for example a road defined by the category R80 is an ordinary road, with a reference speed of 80 km/h.

**Table 6: Main characteristics of the different road categories.**

<table>
<thead>
<tr>
<th>Road category</th>
<th>Ordinary road (R)</th>
<th>Interurban arterial (R)</th>
<th>Express road (T)</th>
<th>Motorway (L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of carriageways</td>
<td>1</td>
<td>2</td>
<td>1 or 2</td>
<td>2</td>
</tr>
<tr>
<td>Intersections</td>
<td>At grade or roundabout</td>
<td>At grade or roundabout – no crossing of central reservation</td>
<td>Not at grade</td>
<td>Not at grade</td>
</tr>
<tr>
<td>Access</td>
<td>Possible to all road users, and road neighbors</td>
<td>Possible to all road users, and road neighbors</td>
<td>Restricted access</td>
<td>Restricted access</td>
</tr>
<tr>
<td>Speed limit (outside urban areas)</td>
<td>90 km/h</td>
<td>90 or 110 km/h</td>
<td>90 km/h</td>
<td>110 or 130 km/h</td>
</tr>
<tr>
<td>Reference speed categories</td>
<td>R60 or R80</td>
<td>R100 or R80</td>
<td>T80 or T100</td>
<td>L100 or L120</td>
</tr>
<tr>
<td>Crosses urban areas</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>no</td>
</tr>
<tr>
<td>Main destination</td>
<td>Short to medium</td>
<td>Short to medium</td>
<td>Medium to long</td>
<td>Medium to long</td>
</tr>
</tbody>
</table>
The link between those road categories with pavement design is not completely direct: indeed, the design of the road structure must provide sufficient bearing capacity for the expected heavy traffic and therefore the flexible design is generally for a low traffic road (road of type “R”) and the other designs are for heavier traffic.

But one should not forget that the choice of materials is also related to other criteria than traffic only.

6.3.5. Choice of reference speed

The reference speed is the highest continuous speed at which vehicle can travel with safety when weather condition is conducive. It is generally taken as the 85th percentile speed. It is the single most important factor that affects most of the geometric design of roads.

For each type of road, different levels of driving comfort (dynamic vehicle behavior) can be defined, depending on the reference speed.

On R category roads:
- Speed category R60 is used when the topography is hilly. It represents a good compromise between road construction cost and comfort.
- Speed category R80 is used when the topography is relatively flat.

On T category roads:
- Speed category T80 is used in hilly areas.
- Speed category T100 is used in relatively flat areas.

On L category roads:
- The choice is between L100 and L120.

Mountain roads, in particularly steep areas, may be outside these categories. Their characteristics result from a compromise between the expected performance and the topographical constraints.

6.4. Geometrical characteristics of different road categories (In France)

6.4.1. Road alignment

6.4.1.1. Minimum radius

For questions of safety and dynamic comfort, the minimum radius of curves is limited, for each road category.

The minimum radius values for France are given in Table 7.

<table>
<thead>
<tr>
<th>Road category</th>
<th>R60</th>
<th>T80 or R80</th>
<th>T100</th>
<th>L100 or L120</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum radius Rmin (m)</td>
<td>120</td>
<td>240</td>
<td>425</td>
<td>400m and 600</td>
</tr>
</tbody>
</table>
6.4.1.2. Transition curves

Minimum lengths of spiral transition curves, for each road category, are given in Table 8:

**Table 8: Minimum Lengths of transition curves, depending on road transverse profile.**

<table>
<thead>
<tr>
<th>Transverse profile</th>
<th>Minimum length of transition curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 lanes</td>
<td>L = inf (6R^{0.4}, 67)</td>
</tr>
<tr>
<td>3 lanes</td>
<td>L = inf (9R^{0.4}, 100)</td>
</tr>
<tr>
<td>2x2 lanes</td>
<td>L = inf (12R^{0.4}, 133)</td>
</tr>
</tbody>
</table>

6.4.2. Longitudinal profile

For questions of dynamic comfort, visibility and safety, the geometry of the longitudinal profile should satisfy the limits given in Table 9:

**Table 9: Limit values of slope and vertical curves, depending on road category and reference speed.**

<table>
<thead>
<tr>
<th>Road category</th>
<th>R60</th>
<th>T80 or R80</th>
<th>T100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum slope</td>
<td>7 %</td>
<td>6 %</td>
<td>5 %</td>
</tr>
<tr>
<td>Minimum radius of crest vertical curve</td>
<td>1500</td>
<td>3000</td>
<td>6000</td>
</tr>
<tr>
<td>Minimum radius of sag vertical curve</td>
<td>1500</td>
<td>2200</td>
<td>3000</td>
</tr>
</tbody>
</table>

6.4.3. Cross section

A road cross section consist mainly of (see Figure 18):

- The traffic lanes,
• The road shoulder, which can be divided in a sealed shoulder (which can support the vehicles), and an unsealed shoulder,
• The traffic lanes and shoulders are characterized by minimum and maximum cross slopes.

Figure 18. Main characteristics of a road cross section (also see Figure 16).

6.4.3.1. Width of traffic lanes
The normal traffic lane width is 3,50 m on all road categories

On ordinary roads (type R), the lane width can be reduced to 3m in case of site constraints, and if the traffic is low. On mountain roads, the lane width can be even lower.

In case of curves of small radius ( R <200 m), lane width is increased in curves, by a factor equal to 50/R (in meters).

The width can be higher in curves with small radius (radius inferior to 200 meters). In Belgian for example, this additional width E is given by:

\[ E = \frac{50}{R} \text{ until 1998,} \]

\[ E = \frac{100}{R + \sqrt{R^2 - 100}} \text{ after 1998.} \]

Where R is the radius of the curve.

6.4.3.2. Shoulder width
Minimum shoulder widths, depending on road category, are given in Table 10.
Table 10. Shoulder widths for different road categories

<table>
<thead>
<tr>
<th>Road category</th>
<th>Shoulder width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary roads R - 2 or 3 lanes</td>
<td>2 m (minimum 1,75 m), paved or stabilized shoulder</td>
</tr>
<tr>
<td>Interurban arterial R - 2x2 lanes</td>
<td>2 m paved or stabilized shoulder</td>
</tr>
<tr>
<td></td>
<td>a different solution is possible in urban areas (1m + sidewalk for example)</td>
</tr>
<tr>
<td>Express road T 2 or 3 lanes</td>
<td>2,5 m (minimum 2m) paved shoulder</td>
</tr>
</tbody>
</table>

6.4.3.3. Cross slopes

Cross slopes are different on straight sections and in curves. They are defined in Table 11.

Table 11. Cross slopes for traffic lanes and road shoulders

<table>
<thead>
<tr>
<th></th>
<th>Cross slope - traffic lane</th>
<th>Cross slope - shoulder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight section, or curve</td>
<td>2,5 % - bi-directional slope</td>
<td>4 % for stabilized shoulder</td>
</tr>
<tr>
<td>radius R &gt; Rnd</td>
<td></td>
<td>2,5 to 4 % for paved shoulder</td>
</tr>
<tr>
<td>Curve with radius</td>
<td>2,5 %-- uni-directional slope</td>
<td>4 % for stabilized shoulder</td>
</tr>
<tr>
<td>Rdm &lt; R &lt; Rnd</td>
<td></td>
<td>2,5 to 4 % for paved shoulder</td>
</tr>
<tr>
<td>Curve with radius</td>
<td>Slope P varies linearly with</td>
<td>Slope P on inside of curve</td>
</tr>
<tr>
<td>Rmin &lt; R &lt; Rdm</td>
<td>1/R, between 2,5 % and 7 %</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(maximum cross slope)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>uni-directional slope</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Rnd = minimum radius of curve with no cross slope
Rdm = minimum radius of curve with 2,5 % cross slope
Rmin = minimum curve radius
The values of Rnd, Rdm and Rmin are defined in Table 7, as a function of road category

6.5. Geometrical characteristics – special case of mountain roads (the most “difficult” case)

Mountain roads are roads built in mountainous areas, with such topographical constraints, that they cannot respect the geometrical characteristics of other road categories.

Such roads are generally two lane roads, of category R (for other categories of roads, T and L, due to the heavy traffic, the geometrical characteristics are generally respected).
6.5.1. Road alignment – curves

Due to constraints in mountainous areas, the radius of curves can be very small (internal radius 10 m or even less in very sharp curves). For very small curve radii, the lane width must be increased.

In France, the minimum lane width \( W \) in curves, to allow traffic of semi-trailers, can be defined by the relationship: \( W = 3.5 + 25 / R \), where \( R \) is the internal radius of the curve. This gives approximate minimum lane widths (for one direction), depending on inside curve radius, which are given in Table 12.

\[ \text{Table 12. Minimum approximate lane widths in curves, for passage of semi-trailers} \]

<table>
<thead>
<tr>
<th>Minimum inside curve radius</th>
<th>Minimum lane width</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 m</td>
<td>8 m</td>
</tr>
<tr>
<td>11 m</td>
<td>6 m</td>
</tr>
<tr>
<td>17 m</td>
<td>5 m</td>
</tr>
</tbody>
</table>

These values may be compared with the Belgian ones given in Section 6.4.3.1.

6.5.2. Longitudinal profile

The following maximum slopes can be recommended:

- roads open to traffic all year round, and exposed to snow or icing in the winter: maximum slope 8%
- roads not exposed to snow or icing, or open only in the summer period: maximum slope 10 %

6.5.3. Cross section

Depending on the level of traffic, different minimum road widths can be recommended, as given in Table 13.

\[ \text{Table 13. Minimum recommended widths for mountain roads, depending on level of traffic} \]

<table>
<thead>
<tr>
<th>Traffic level</th>
<th>Width of traffic lanes (m)</th>
<th>Width of stabilized shoulders (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy traffic</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 500 HV / day</td>
<td>7 m</td>
<td>1,5 m</td>
</tr>
<tr>
<td>or &gt; 6000 vehicles / day</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Significant traffic</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 50 HV / day</td>
<td>6 m</td>
<td>1,5 m</td>
</tr>
<tr>
<td>or &gt; 2000 vehicles / day</td>
<td></td>
<td>1 m</td>
</tr>
<tr>
<td>Other cases</td>
<td>5,5 m</td>
<td>0,75 m</td>
</tr>
</tbody>
</table>
6.6. Skid resistance characteristics

6.6.1. Skid resistance levels

The skid resistance of a road surface depends on several parameters, such as the stone skeleton, the mastic, the composition of the material and the compaction. For cement concrete road surfaces, also the cement and sand skeleton may influence the skid resistance. For some materials (e.g. fine cement concrete and stone mastic asphalt) the stones and their texture may have a higher contribution to the skid resistance than for other but this is very difficult to quantify.

The force needed to brake with an LHV may well be bigger than the force needed to brake an ordinary lorry but the LHV also disposes of more tyres and thus the friction force due to tyre-surface contact is also higher than for an ordinary lorry. If quantification would have to be made, the Longitudinal Friction Coefficient is the better candidate for the characterization of the skid resistance of a road surface. If the skid resistance to sideway slipping is to be modelled, the Sideway Friction Coefficient is the more appropriate candidate for the road surface characteristic.

As mentioned in [33] the objective usually is to adapt the vehicle to the existing road infrastructure rather than the other way around. This also holds for skid resistance. Some accidents with LHV in The Netherlands indeed included skidding of the vehicle. Nevertheless, a UK study [34] showed that the LHV is less likely to encounter safety hazards related to skidding than ordinary lorries when making use of an adequate assisted braking system (ABS).

In [35], the results reported in [34] are summarized as follows. [34] concludes that the increased length and weight of the vehicles could cause an additional accident risk during braking operations, but a requirement that all components of the vehicle should comply with current braking regulations – namely, ABS and requirements for the distribution of braking amongst the axles – would be expected to minimize these additional risks. Tests by Daimler AG carried out on test tracks in Sweden and Germany – reported in [1] – have proven the high braking performance of LHV. Compared with a conventional truck trailer combination, an LHV could decrease the braking distance on a dry surface by up to 5%, and on a slippery surface by up to 17%. This is the result of LHV having a reduced axle load due to more axles and a bigger footprint, allowing higher brake forces to be transmitted.

The New Zealand Transport Agency published a Note on the specification for state highway skid resistance management [36]. As explained in [37], this is a document that supports road controlling authorities in their approach to asset management. This specification addresses “road segmentation”, where each segment gets a minimum skid resistance level. This level depends on several factors, such as:

- approaches to railway level crossings, traffic signals, pedestrian crossings, stop and give way controlled intersections, roundabouts;
- one lane bridges (approaches and bridge deck);
- urban and rural curves (other levels in function of radius);
- down gradients higher that 10%;
- on ramps with ramp metering;
- state highway approach to a local road junction;
- down gradients between 5 and 10%;
- motorway junction area including on/off ramps;
- roundabouts, circular section;
- undivided carriageways;
- divided carriageways.
These levels are expressed in the “Equilibrium SCRIM co-efficient (ESC)”, which is the sideways force coefficient measured with a SCRIM device and normalized over seasonal differences. Existing road segments are then inspected and the actual ESC is compared to the expected level. Then this information is used in the asset management decision process in order to plan the interventions with the highest benefit/cost ratio. However, these levels do not seem to be chosen in function of the use of the road by LHVs.

So skid resistance characteristics are related with the macrotexture and microtexture of the road surface. The macrotexture influences mainly the water evacuation properties, and is measured on roads by profilometer measurements, giving a mean texture depth value (MTD). **Significant levels of mean texture depth** values in France for interurban roads are:

- On roads with slopes ≤ 5%: Mean specified MTD: ≥ 0.6 mm, minimum value 0.4 mm
- On roads with slopes > 5%: Mean specified MTD: ≥ 0.8 mm, minimum value 0.6 mm

The microtexture is not measured directly, but is evaluated by friction coefficient measurements, using different methods (longitudinal friction or transversal friction). A widely used measurement is the measurement of the side friction coefficient (SFC) (using in particular the SCRIM apparatus). In France, the following levels of SFC (measured at 60 km/h) can be considered significant: SFC = 0.4: minimum acceptable friction level, SFC = 0.6: good mean friction level, 0.8: very high friction level.

### 6.6.2. Skid resistance and safety

For stability and safety of heavy vehicles, it is the combination of geometrical parameters (slope, curve radius, crossfall) and friction properties of the road surface which is important. In France, a study [22], based on simulation of behaviour of semi-trailer vehicles, has been performed to evaluate the maximum speed a heavy vehicle can drive without skidding or rolling over, depending on the characteristics of the infrastructure (slope, crossfall, radius). This study has led to identify configurations of infrastructure (i.e. combinations of geometric and surface characteristics values) that may lead to a risk of accident for heavy vehicles. The main “dangerous” configurations, or “warnings” identified in this study, are summarized in Table 14 and Table 15.

**Table 14. Definition of safety warnings in curves for dual carriageway roads (after [22])**

<table>
<thead>
<tr>
<th>Warning</th>
<th>Radius of curvature (m)</th>
<th>Skid resistance</th>
<th>Slope (%)</th>
<th>Crossfall (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>R &lt; 120 m</td>
<td>SFC &lt; 0.40</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>R &lt; 120 m</td>
<td>-</td>
<td>-</td>
<td>Gap with theoretical value &gt; 2%</td>
</tr>
<tr>
<td>3</td>
<td>R &lt; 120 m</td>
<td>-</td>
<td>&lt; - 3% (descent)</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>R &lt; 120 m with geometrical problems</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>Risky successive curves</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>R &lt; 120 m</td>
<td>-</td>
<td>&gt; 4% (ramp)</td>
<td>-</td>
</tr>
</tbody>
</table>

**Table 15. Definition of safety warnings in curves for single carriageway roads (extracted from [22])**
The different combined situations of geometry and skid resistance of tables 14 and 15 can be used as basis for the comparison of the behavior of different vehicle configurations.

6.7. Catalogue for road geometry

The normal lane width is 3.50 meters. In case of curves of small radius (R < 200 m), lane width is increased in curves, by a quantity equal to 50/R (in meters). In hilly regions, the minimum lane width W in curves is given by: \( W = 3.5 + 25/R \), where R is the internal radius of the curve. The geometries given here are extracted from [29], [30] and [31].

6.7.1. Curves

<table>
<thead>
<tr>
<th>Speed</th>
<th>60 km/h</th>
<th>80 km/h</th>
<th>100 km/h</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum radius Rmin (m).</td>
<td>120</td>
<td>240</td>
<td>425</td>
</tr>
<tr>
<td>Minimum radius with a standard cross slope of 2,5 % Rdm</td>
<td>450</td>
<td>650</td>
<td>900</td>
</tr>
<tr>
<td>Minimum radius without cross slope Rnd (m)</td>
<td>600</td>
<td>900</td>
<td>1300</td>
</tr>
<tr>
<td>Maximum slope</td>
<td>7%</td>
<td>6%</td>
<td>5%</td>
</tr>
<tr>
<td>Minimum radius of crest vertical curve</td>
<td>1500</td>
<td>3000</td>
<td>6000</td>
</tr>
<tr>
<td>Minimum radius of sag vertical curve</td>
<td>1500</td>
<td>2200</td>
<td>3000</td>
</tr>
</tbody>
</table>

6.7.2. Spiral transition curves

<table>
<thead>
<tr>
<th>Transverse profile</th>
<th>Minimum length of transition curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 lanes</td>
<td>( L = \inf (6R^{0.4}, 67) )</td>
</tr>
<tr>
<td>3 lanes</td>
<td>( L = \inf (9R^{0.4}, 100) )</td>
</tr>
<tr>
<td>2x2 lanes</td>
<td>( L = \inf (12R^{0.4}, 133) )</td>
</tr>
</tbody>
</table>

6.7.3. Cross-slope

<table>
<thead>
<tr>
<th>Cross slope - traffic lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight section, or curve</td>
</tr>
</tbody>
</table>
radius $R > R_{nd}$

<table>
<thead>
<tr>
<th>Curve with radius $R_{dm} &lt; R &lt; R_{nd}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5 % uni-directional slope</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Curve with radius $R_{min} &lt; R &lt; R_{dm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope varies linearly with $1/R$, between 2.5 % and 7 % (maximum cross slope) uni-directional slope</td>
</tr>
</tbody>
</table>

These three parameters are combined for example in the case of highway exit lanes (Figure 19, Figure 20):

**Figure 19**: Highway exit lane for straight road, where the values for the length of the straight section ($L_m$), the length of the transition curve ($L_{cl}$) and the radius of the curve ($R$) are within the ranges given before.

**Figure 20**: Highway exit lane for road curved to the right, where the values for the length of the straight section ($L_m$), the length of the transition curve ($L_{cl} + L_{AD}$) and the radius of the curve ($R$) are within the ranges given before.

### 6.7.4. Skid resistance

- On roads with slopes $\leq 5$ %: Mean specified MTD $\geq 0.6$ mm, minimum value $0.4$ mm
- On roads with slopes $> 5$ %: Mean specified MTD $\geq 0.8$ mm, minimum value $0.6$ mm
6.7.5. Left-turn maneuver, right-turn maneuver and other intersections in cities

\[ D = 160 \text{m} (120 \text{m minimum}) \]

Figure 21: Intersection number 1.

Figure 22: Intersection number 2.

Figure 23: Intersection number 3.
6.7.6. Roundabouts

The radius of the roundabout should be between 15 and 25 meters.

7. Road equipment and services

7.1. Safety barriers

7.1.1. Crash Tests according to EN 1317

For the assessment of the performance of safety barriers resp. vehicle restraint systems (VRS) different crash tests have to be conducted. Dynamic deflection and working width of a safety barrier as well as vehicle intrusion are usually determined in a crash test with a heavy vehicle (containment test, maximum level) whereas the impact severity is measured with a light vehicle representing the worst case for the injury of occupants (impact severity test). Depending on the test type the vehicle weight and test parameters vary according to the following Table 16.

The highest containment level given in the European standard EN 1317 is H4b, this means an impact test with an articulated truck of 38 tonnes that hits the safety barrier with an impact speed of 65 km/h and an impact angle of 20°.
As stated in [2], in several countries, the safety barriers and their anchorages are designed for older trucks that tend to be lighter than those currently in use. The up-grading of these devices would incur additional expenses, but should be considered in order to maintain the safety level provided.

When considering increasing the maximum permitted weight of trucks, the issue of containment should be considered. However, it is apparent that barriers that just comply with the minimum standards applicable in Europe are not designed to cater for the worst case that might occur (i.e. 40 or 44-tonne truck travelling at 90 km/h).


This work can also be done by numerical simulation, in the case of the tentative assessment of the behaviour of the restraint system regarding a given truck. The description of the restraint quality of a given barrier for a given vehicle is then rather qualitative [12].

7.2. Parking lots

Parking lots, in terms of design (dimensions) and number have to be considered in the list of infrastructures sensible to traffic characteristics.
It should be noted that even today, the service in terms of parking lots is not sufficient for the existing traffic (volume, dimensions).

Here the interlink with the driving schedules of the drivers should be mentioned. Indeed, parking lots could be an example for the possible coming-together of two smart-technologies, from side of the road infrastructure (parking tracking sensors which detect if a parking lot is free) and predictive parking needs from side of the driver/truck (technology indicating the hours and the distance remaining until the rest).

These technologies will even more useful with a new legislation coming, forbidding to drivers to sleep in their trucks during week-ends.

Finally, it should be mentioned that some parking lots now offer the possibility to leave (some) load there, in the sake of intermodality.

7.3. Freight terminals

Finally, especially in the frame of multimodality, the locations, the numbers and the dimensions of the freight terminals have to be carefully assessed and updated to cater for new types of trucks and truck loads.

This is a difficult question as there are often issues of non-civil engineering nature.

7.3.1. Classification of the terminals

The classification is based on the fact that the various classes have different needs in terms of localization, design, equipment and exploitation [13]. The main categories of intermodal terminals are:

- Freight terminal in ports,
- Rail terminals.

It is also possible to classify the terminals by, see Figure 26: Terminal characteristics for main categories, extracted from [13]:

- Spatial or economic importance,
- Access modes,
- Type of loading units that are handled (ISO containers, bulk, ...),
- Transhipment technology,
- ...
7.3.2. Issues for the infrastructure and the equipment

Existing terminals are considered as old when they are older than 20 years. In this case, the issues, which are expected to increase, are linked to (Figure 27):

- Disposition and design of the terminals,
- Rail access,
- Road access,
- Transhipment zone.

**Figure 26: Terminal characteristics for main categories, extracted from [13].**
Most of the problems that arise with terminals is that its design (layout, dimensions, ...) does not fit the needs anymore. Especially, the possibility to increase the terminal should be also foreseen, in order to cater for increasing demands. For example, the loading tracks should be long enough (> 700m).

Other problems are linked to the road access. Everything should be done to avoid traffic jams, in particular parking lots should be present in a sufficient number.

The storage capacity should be sufficient, and should be designed in order to cater for the demands in the following 20 years (the increase should be foreseen).

8. Conclusions

This report has listed the most typical infrastructure elements, namely pavements, bridges, tunnels, road geometry, safety barriers. For each of them, the links and interactions with traffic have been highlighted. Then for pavements and bridges, the description has been made by details explaining their structural behaviour (e.g. influence lines for bridges).

This catalogue is extensive, but not exhaustive. Moreover, the infrastructure elements presented here are those that could be designed according to the present design codes.
For pavements and bridges, the report lists typical structures that will have to be studied in the following tasks of this project, in the frame of development of Performance Based Standards.

9. References


[3] Walloon region DGO1, List of criteria for the selection of itineraries suitable to longer trucks, Walloon Region (Belgium), 2016


[23] Walloon region DGO1, List of criteria for the selection of itineraries suitable to longer trucks, Walloon Region (Belgium), 2016.


[29] ICTAAL, Instruction sur les conditions techniques d’aménagement des autoroutes de liaison, Cerema, 2015.


10. Appendix 1: List of infrastructure components to take into consideration when selecting longer and/or heavier vehicle (LHV) routes

10.1. Rejection components for the selection of the route

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Unit</th>
<th>Threshold</th>
<th>Experimental assessment</th>
<th>Elements/aspects that could be affected</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Road environment: residential area</strong></td>
<td></td>
<td></td>
<td>Pilot studies in the NL</td>
<td>Road safety, conflicts with VRU, truck maneuverability</td>
<td>[26] (p29) ; [23] ; [28]</td>
</tr>
<tr>
<td>The presence of LHV should be prohibited in residential areas to avoid conflicts with vulnerable road users (e.g. children, pedestrians, cyclists). In addition, the infrastructure in residential area might not be suitable to LHV (e.g. narrow streets).</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Road environment: inner city</strong></td>
<td></td>
<td></td>
<td>Pilot studies in the NL</td>
<td>Road safety, congestion, truck maneuverability</td>
<td>[26] (p29) ; [23] ; [28]</td>
</tr>
<tr>
<td>The presence of LHV in the inner-city should be prohibited to avoid conflicts with vulnerable road users and to limit road congestion. In addition, the maneuverability of LHV in narrow streets might be difficult.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Bridges/road structures with insufficient load-bearing capacity</strong></td>
<td>Maximal vehicle charge</td>
<td>Kg</td>
<td>Variable</td>
<td>Not specifically</td>
<td>Truck safety, bridge deterioration</td>
</tr>
<tr>
<td>The LHV itineraries must exclude any</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
bridge and road section with an insufficient load-bearing capacity. If not, the loading of the LHV might cause serious damages and safety issues.

### 10.2. Components that must be carefully analyzed to select a route

<table>
<thead>
<tr>
<th>Geometrical constraints</th>
<th>Dimensions</th>
<th>Unit</th>
<th>Threshold</th>
<th>Experimental assessment</th>
<th>Elements/aspects that could be affected</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length of the itinerary</strong></td>
<td>Maximum length</td>
<td>Km</td>
<td>50</td>
<td>No</td>
<td></td>
<td>[26] (p28)</td>
</tr>
<tr>
<td>The Netherlands recommends a maximum length of 50km for the itinerary.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Lane width | Minimum width | m | ? | | | [26] (p40) |
| The Netherlands recommends a "sufficient" lane width without specifying any value. The value of the minimum width depends on the operational speed on the road, the volume and composition of the traffic, the presence of obstacles along the road, etc. |

<table>
<thead>
<tr>
<th>Roundabout radius (180° turn)</th>
<th>Minimum</th>
<th>m</th>
<th>18¹</th>
<th>Software simulations</th>
<th>Truck</th>
<th>[24] (pp54-55) ; [23] ; [26] (p30) ;</th>
</tr>
</thead>
</table>

¹ Taking a roundabout with a radius of 18m is technically possible (180° turn) but not very comfortable, so that the literature suggests 20m.
In order to guarantee the manoeuvrability of the LHV when performing a 180° turn in a roundabout, a minimum radius of 18 meters is recommended. However, this manoeuvre is more comfortable with a radius of 20 meters (BRRC2007, p55) ([26], p30).

**Roundabout radius (270° turn)**

In order to guarantee the manoeuvrability of the LHV when performing a 270° turn in a roundabout, a minimum radius of 20 meters is recommended. However, this manoeuvre is more comfortable with a radius of 22 meters. In roundabouts with a radius of 18 meters, the manoeuvre is not possible with LHV ([24], p56).

<table>
<thead>
<tr>
<th>Minimum radius (m)</th>
<th>Software simulations (TrAC)</th>
<th>Pilot studies in the NL</th>
<th>Truck manoeuvrability</th>
</tr>
</thead>
<tbody>
<tr>
<td>20^2</td>
<td></td>
<td></td>
<td>[24] (pp55-56)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[23] ; [25] (p234)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[26] (p30) ; [27] (p53-55)</td>
</tr>
</tbody>
</table>

**Right-turn radius (intersections)**

In order to guarantee the manoeuvrability of the LHV when performing a right turn (90°), a minimum radius of 15 meters is recommended. Below this value, serious manoeuvrability issues occur. From 20m and above, the manoeuvre is performed without any problem ([24], p54).

<table>
<thead>
<tr>
<th>Minimum radius (m)</th>
<th>Software simulations (TrAC)</th>
<th>Pilot studies in the NL</th>
<th>Truck manoeuvrability, road safety, road congestion</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>[24] (p54) ; [26] (p30) ; [27] (p43-45)</td>
</tr>
</tbody>
</table>

^2 Taking a roundabout with a radius of 20m is technically possible (270° turn) but some manoeuvrability issues could occur, so that the literature suggests 22m.
### Height under bridges/tunnels

This aspect was not considered in the pilot studies in the Netherlands. However, the Flanders region recommends to consider the maximal height of the trucks (generally 4.10m) and to add 50cm if the speed of the LHV is higher than 50km/h.

<table>
<thead>
<tr>
<th>Minimum height</th>
<th>m</th>
<th>Pilot studies in the NL</th>
<th>Road safety, road congestion</th>
<th>[26] (p37)</th>
</tr>
</thead>
</table>

### Merging lanes (entrance/exit off highways and motorways)

In order to avoid safety issues due to lack of visibility of LHV drivers, the minimum length of merging lanes should be at least 250m. This value is recommended by the Flemish region ([26], p38) and the Netherlands on the basis of 2 pilot studies ([26], p28).

<table>
<thead>
<tr>
<th>Minimum length</th>
<th>m</th>
<th>Pilot studies in the NL</th>
<th>Road safety, driver visibility</th>
<th>[23] ; [26] (p28, p38)</th>
</tr>
</thead>
</table>

### Infrastructure equipment

#### Run-out lane (on downhill lane)

On downhill lane, the presence of run-out lanes might avoid road safety issues in case of loss of control/brake problem of the LHV. The length of the run-out lane should be defined by considering the length of the LHV.

<table>
<thead>
<tr>
<th>Minimum length</th>
<th>m</th>
<th>Road safety</th>
<th>[23]</th>
</tr>
</thead>
</table>

#### Arrester beds (on downhill lane)

<table>
<thead>
<tr>
<th>Minimum length</th>
<th>m</th>
<th>Road safety</th>
<th>[23]</th>
</tr>
</thead>
</table>
Similarly to the run-out lanes, the presence of arrester beds on downhill lanes might avoid road safety issues in case of loss of control/brake problem of the LHV. The length of the arrester beds should be defined by considering the length of the LHV.

<table>
<thead>
<tr>
<th>Non-separated cycle lanes</th>
<th>OK/Not OK</th>
<th>Not OK</th>
<th>Pilot studies in the NL</th>
<th>Road safety (especially cyclists)</th>
</tr>
</thead>
<tbody>
<tr>
<td>In general, it is preferable to avoid the selection of road sections with non-separated cycle lanes in order to limit the exposure of cyclists to LHV ([26], p29). However, if selected, such sections must be carefully analyzed.</td>
<td>OK/Not OK</td>
<td>Not OK</td>
<td>Pilot studies in the NL</td>
<td>Road safety (especially cyclists)</td>
</tr>
<tr>
<td>Road restraint systems (compatibility)</td>
<td>Minimum restraint</td>
<td>H4b barriers</td>
<td>Impact tests at BASt</td>
<td>Road safety</td>
</tr>
<tr>
<td>&quot;Some impact tests were undertaken at BASt to assess different safety barriers of concrete and steel intended for installation on German bridges. It resulted from these tests that H4b-barriers should be installed on new bridges over highways and in the central reservation of German highways ([25], p231). The Flemish administration also suggests to install H4b-barriers on LHV itineraries ([26], p39).&quot;</td>
<td>Minimum restraint</td>
<td>H4b barriers</td>
<td>Impact tests at BASt</td>
<td>Road safety</td>
</tr>
</tbody>
</table>

[23]; [26] (p29)
<table>
<thead>
<tr>
<th><strong>Rest area (and parking)</strong></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Lack of adequate parkings for LHV might lead to dangerous behavior (e.g. trucks parked on access driveways or emergency lanes). Then, it is important to consider the development of such facilities when selecting LHV itineraries.</td>
<td>Not specifically</td>
<td>Road safety</td>
<td>[24] (p63); [25] (pp237-238); [26] (p39)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Level crossings</strong></th>
<th>OK/Not OK</th>
<th>Not OK</th>
<th>Road/train safety</th>
<th>[26] (p30); [23]</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;In order to avoid severe train/truck accidents, the Walloon region and the Netherlands recommend to avoid level crossings on LHV itineraries. In a pilot study, the Netherlands allowed level crossings if the speed of trains is lower than 40 km/h.&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Infrastructure environment</strong></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th><strong>Limited traffic zone (30 km/h)</strong></th>
<th>OK/Not OK</th>
<th>Not OK</th>
<th>Pilot studies in the NL</th>
<th>Road safety, conflicts with VRU, truck manoeuvrability</th>
<th>[26] (p29, p42); [23]</th>
</tr>
</thead>
</table>
verified (e.g. right turns).

The Netherlands recommends to limit to 5 kilometers the total length of road sections that are not highways or motorways with physical separation with VRU."

<table>
<thead>
<tr>
<th>Accident-prone zones</th>
<th>OK/Not OK</th>
<th>Not OK</th>
<th>Road safety (risk exposure)</th>
<th>[23]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whenever possible, the Walloon region recommends to avoid the selection of LHV itineraries with road sections that are considered as accident-prone zones. The presence of LHV in these zones might increase the risk exposure (e.g. due to speed differential, lack of visibility for other vehicles, etc.)</td>
<td>OK/Not OK</td>
<td>Not OK</td>
<td>Road safety (risk exposure)</td>
<td>[23]</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Roadworks (driving through)</th>
<th>Layout (min. radius)</th>
<th>m</th>
<th>Maneuverability, road safety, road workers safety</th>
<th>[26] (p37)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;In some cases, the configuration of roadworks temporarily transforms the layout of a road section (e.g. lane closure, lane shift, reduction of the lane width, work zone delineation, etc.). Consequently, it might become temporarily difficult for the LHV to drive through the road section (manoeuvrability) in safe conditions (road users and workers). Then, when selecting an itinerary for LHV, it is important to analyse the configuration of</td>
<td>Layout (min. radius)</td>
<td>m</td>
<td>Maneuverability, road safety, road workers safety</td>
<td>[26] (p37)</td>
</tr>
</tbody>
</table>
eventual roadworks in order to guarantee the manoeuvrability of the LHV, the safety of road users and the safety of road workers ([26])."

<table>
<thead>
<tr>
<th>Road design</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type of intersections (traffic lights)</strong></td>
</tr>
</tbody>
</table>

"Intersections are important conflict points between LHV and cyclists. To that extent, the Flanders region recommends to avoid intersections without traffic lights (and specific pedestrians/cyclists crossings). A separate green phase for cyclists and pedestrians at intersections with traffic lights is also recommended to avoid conflicts.

In the Netherlands, LHV are not allowed on intersections with ""priority to the right"" rule ([26])."

<table>
<thead>
<tr>
<th>Manoeuvre time at crossroads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum time s</td>
</tr>
</tbody>
</table>

Road safety [23] ; [26]

<table>
<thead>
<tr>
<th>Winding/sinuous route (if 2x1 lanes)</th>
</tr>
</thead>
</table>

Visibility, road safety (overtaking) [23]
Visibility issues might occur on sinuous routes with LHV. Due to their size, the drivers that follow LHV might not be able to see the oncoming traffic (and then perform dangerous overtaking manoeuvres).

<table>
<thead>
<tr>
<th><strong>Long and steep slope (without slow lane)</strong></th>
<th>Maximum percentage</th>
<th>Road safety, LHV driving</th>
<th>[26] (p37) ; [23]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long and steep slopes might be strong obstacles for LHV on road sections without slow lanes ([26] p37). In particular, LHV might drive more slowly than other vehicles and then cause potential safety issues (due to speed differential).</td>
<td>%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Others

<table>
<thead>
<tr>
<th><strong>Overtaking limitations (for trucks)</strong></th>
<th>Yes/No</th>
<th>FR, DE, GE, UK, DK</th>
<th>Road safety (positive impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;An overtaking ban for trucks over distances of several kilometers can improve the coexistence of heavy and light vehicles on highly trafficked roads. The Netherlands has tested a dynamic overtaking ban on two 2x2-lane motorway sections that provides greater benefit than static bans ([25], p236). Permanent bans are preferred on sections with truck crash problems, while intermittent or dynamic bans are favored on sections with congestion and traffic problems.&quot;</td>
<td></td>
<td></td>
<td>[25] (pp235-237) (including thresholds from BASt)</td>
</tr>
<tr>
<td>Traffic volume</td>
<td>Maximum AADT</td>
<td>Veh/h</td>
<td>Road congestion, road safety</td>
</tr>
<tr>
<td>----------------</td>
<td>-------------</td>
<td>------</td>
<td>-----------------------------</td>
</tr>
</tbody>
</table>