High-speed railway embankments — a comparison of different regulations

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Preface/Acknowledgement

This master thesis project was administered by the Department of Civil and Architectural Engineering at KTH. The Swedish transport administration initiated this report and provided me with literature, reports and information that I needed.

I would like to thank everybody that helped me finish my thesis. A special thank you to my supervisor Leif Jendeby and to all the people at COWI, especially Mikael Lindberg. And also a special thank you to my supervisor, professor Stefan Larsson. Thank you for your help and your encouragement.

Finally, I would like to thank the people closest to me, who has supported me during my education.

Thank you!

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Abstract

Swedish transport administration initiated this Master Thesis project and the aim was to compare regulations for the design of high-speed railways from three European countries: France, Germany and Spain. The reason why this is of interest for the Swedish transport administration is the design of the first Swedish high-speed railway, called Ostlänken. Therefore, a literature study of the regulations and other literature regarding high-speed railway has been carried out. A basic description of railway components, slab track and ballasted tracks is presented.

Ballasted embankments usually consist of a trackbed layer (ballast onto subballast), and the ultimate thickness of this layer is discussed, as there are a number of methods available to calculate the appropriate thickness, with a number of different design parameters. These design methods results in different trackbed thickness and choosing the "wrong" method might lead to an overestimation or underestimation of the trackbed layer. Constructing a ballastless railway line means that the ballast is replaced by another material, usually a slab made of reinforced concrete or asphalt, and the rail is cast onto this slab. Countries design their slab using different methods.

Germany has constructed high-speed railway lines with a slab track solution, generally slabs with low flexible stiffness. France has until recently constructed their high-speed line ballasted but is now developing a new slab track technique, called NBT (New Ballastless Track) and Spain uses various methods. It is difficult to compare the regulations, however, there are some factors that at least begin to explain the differences between the countries: the frost hazard, the inherent ground quality, purpose with the railway (mixed traffic, solely passenger traffic, etc.), design parameters (life, axle load, etc.). Furthermore, the settlement requirements, soil classification and bearing capacity are factors that varies from country to country, but the origin for this variation is harder to detect.

Sammanfattning

Detta examensarbete initierades av Trafikverket och syftet med arbetet är att jämföra de regelverk som gäller för höghastighetsbana från tre europeiska länder: Frankrike, Tyskland och Spanien. Anledningen till att olika länder reglering är av intresse för Trafikverket är att Sverige ska bygga sin första höghastighetsbana, Ostlänken.

Det här arbetet är huvudsakligen en litteraturstudie och för att genomföra arbetet så har de franska och tyska regelverken studerats noga. En grundläggande beskrivning av järnvägskomponenter, ballastfria banker och baner med ballast presenteras i rapporten. Ballast spår består vanligtvis av ett lager av ballast. Den optimala tjockleken på detta skikt är omdiskuterat, eftersom att det flera tillgängliga metoder som tar hänsyn till olika design parametrar, vilket resulterar i olika tjocklekar. När man bygger spåren, är det viktigt att man får tjockleken på ballast lagret rätt. Därför måste man välja vilken design metod för att bygga efter med omsorg, att välja en olämplig metod kan leda till en underskattning eller överskattning av tjockleken på ballast skiktet. När en järnväg konstrueras ballastfritt innebär det att ballasten ersättas med ett annat material (vanligen en platta av armerad betong eller asfalt) och rälsen gjuts sedan på plattan.

Tyskland konstruerar sina höghastighetsjärnvägslinjer med ballastfria spår, generellt med plattor med låg flexibel styvhet. Frankrike har tills nyligen konstruerade sin höghastighetslinje med ballast men utvecklar nu en ny ballastfri teknik, som kallas NBT (New Ballastless Track) och Spanien använder olika metoder.

Det är svårt att jämföra de olika regelverken, men det finns vissa grundläggande faktorer som åtminstone börjar förklara svårigheterna: frost, jordkvaliten, om järnvägen är tänkt för godstrafik, persontrafik eller både och, konstruktionsparametrar (livslängd, axellast , etc). Det finns även andra aspekter som skiljer sig, men det är svårt att identifiera orsaken till varför de skiljer sig år: sättning kraven varierar från land till land och jordens klassificeringssystem varierar också.

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Terminology

Geotechnical symbols, terms, definition and their units

Symbol	Term	Definition	Unit
d	Particle size	Particle size determined by sieve analysis or sedimentation	mm
d_{max}	Diameter of the largest particle	Particle size determined by sieve analysis or sedimentation	mm
D_{pr}	Compaction ratio		%
D_w	Wheel diameter		m
E	Young's Modulus	$\frac{\sigma}{\varepsilon}$	Pa
E_{v2}	Modulus of deformation obtained on 2nd loading in the plate bearing test	$E_{v2} = \frac{1.5 \cdot r \cdot \Delta\sigma}{\Delta s}$	r- plate radius Δσ-pressure under plate Δs-settlement of plate
E_{vd}	Deflection module	Dynamic plate load test	
I_P	Plasticity index	Difference between liquid and plastic limits	-
l_c	Consistency index	$\frac{w_L - w}{I_P}$	-
N	Number of repeated load applications		
p	Density	Total mass of the soil divided by its volume	$\frac{kg}{m^3}$

p_d	Dry density	Mass of solid particle divided by the total soil volume	$\frac{kg}{m^3}$
P_d	Dynamic wheel load		N
P_s	Static wheel load		N
R_t	Direct tensile strength		Pa
R_{tb}	Brazilian tensile strength		Pa
T	Depth of subgrade layer		m
U	Uniformity coefficient	d_{60}/d_{10}	-
v	Train speed		km/h
w	Moisture content	Water contained in material divided with weight of solid particles in material	%
w_L	Liquid limit	Water content at which the behaviour of a clayey soil changes from plastic to liquid	-
ε_p	Plastic strain	Total cumulative plastic strain at the subgrade surface for the deign period	%
ε_{pa}		Allowable plastic strain at the subgrade surface for the design period	%
θ'	Friction angle		°

ρ_{dfc}	Dry density in the bottom layer	: The average value on a segment 8 cm thick located in the lower part of the compacted layer	$\frac{kg}{m^3}$
ρ_{dm}	Average dry density	Average dry density throughout the thickness of the compacted layer.	$\frac{kg}{m^3}$
ϱdOPN	Optimum dry density		$\frac{kg}{m^3}$
σ	Stress		Pa
σ_d	Deviator stress		Pa
σ_s	Compression strength		Pa
τ_f	Shear strength		Pa
φ	Total cumulative plastic deformation of the subgrade layer for the design period		m
φ_a	Allowable plastic deformation of the subgrade layer for the Design period		m

Terms and definitions

Term	Definition	Comments
ADIF	Railway organisation in Spain	
CBR	Bearing load expressed as a percentage of a reference bearing load	
DB	Railway organisation in Germany: Deutche Bahn	
D_{pr}	Compaction ratio	
Fines	Soil with d<0.06 [mm]	Some railway organisations use other values of d, example 0.063, 0.08 [mm]
Hardness of stone	Resistance to impact and attrition.	May be determined by Los Angeles (LA), Deval and Microdeval test LA: [%] MDE: [%]
OPN	Standard optimum Proctor (compaction)	
Proctor density	Maximum dry density and water content.	Under standardized conditions of compaction
SNCF	Railway organisation in France	
Trafikverket	Railway organisation in Sweden.	
UIC	A worldwide railway organisation	

1 Introduction

1.1 Background

The Swedish transport administration has begun the planning and design of a proposed high-speed network system, which will link the Stockholm region with Jönköping, Gothenburg and Malmö (Fig. 1). The main reason to construct high-speed railways is to shorten the travel time but another important reason is the growing need for an enlarged passenger capacity. The system will be designed to operate trains with top speeds of 320 km/h and the line is planned to consist of 1500 km railway track, 200 bridges and 20 km tunnels. The construction of the high-speed railway will start with Ostlänken, which is a 150 km ling line from Järna near Stockholm to Linköping. Five stops are planned: Vagnhärad, Nyköping, Skavsta, Norrköping and Linköping. The construction is planned to begin in 2019 or 2020 and will be completed around 2035.

The type of ideal and cost effective construction is a debated subject in Sweden. Traditionally, railways have been constructed by ballasted track where the initial construction cost is relatively low, due to the great experience. However, since the train speed has increased so has the need to improve the railway embankments. Today, there are a number of alternative solutions available. A competitive solution is the ballastless embankment. The initial cost is higher than for ballasted, but the amount of maintenance is decreased and therefore is the availability of the track also increased. However, if maintenance is necessary, the cost and amount of work will be much higher than for traditional embankments. As a results of the maintenance difficulties there are strict restrictions on the total settlements, leading to high expands for ground improvement. These demands raises new technical question formulation regarding the embankments, heights, durability, degradation, demands on the material. Sweden has choosed to design the system with ballastless system.

High-speed railway has been built in number of countries, such as Germany, Spain and France. Each country has chosen different solution due to their different ground condition, experience and needs. There are different requirements in each country. However, the different condition makes it difficult to perform objective comparisons.

Figure 1. Ostlänken, the first planned high-speed network system in Sweden. Ostlänken is the line between Järna (50 km south of Stockholm) and Jönköping. (Svt, 2016)

1.2 Aim

The aim of this thesis is to compile and discuss a number of foreign regulations (Spain, France and Germany) regarding praxis of high-speed railways. The issue arose since the current review has given rise to a number of questions. Therefore, a closer study of different regulations from different countries is desirable. The comparison of the foreign regulations includes design praxis, current design philosophy, design criteria and which theoretical ground the design relays on. Furthermore, one should also consider the type of railway: which type of loads the railway is exposed to, the speed of the trains and type off embankment (ballastless or ballasted). The discussion focus on the differences and, if possible, the reasons regarding:

- Design parameters
- Deformation
- Classification systems
- Bearing capacity, frost protection and layer thickness

1.3 Definition of a high-speed train

First of all a definition of a train travelling at high speed is in place, since the definition is vague and there is no threshold value (Swedish Transport Administration, 2016: UIC, 2016). A reason for the vague definition could be that the technique is under constant development. As an example, when the first high-speed railway was constructed in Japan 1964 the maximum speed was 210 km/h, today, on the same railway, the maximum speed is 270 km/h.

Today, trains that operate in high speed are defined both as "high-speed trains" and "real high-speed trains". To separate them, one may start by separating a "high-speed train" from a "real high-speed train". The easiest way to do that is by separating a high-speed railway from a conventional railway.

A conventional railway is designed for trains with a top speed of 250 km/h. Furthermore,

- Conventional railways are commonly trafficked with mixed traffic (i.e freight and passenger). Therefore, due to the heavy freight trains the inclination should not be lager than 10‰.
- The curve radius is commonly 500-2000 m

A high-speed railway is designed, as indicated by the name, for trains with very high speed; over 250 km/h. There are more requirements on the railway system in order to enable trains to travel at these speeds:

- The curve radius must be sufficiently large, usually 4000-7000 m, so that the train can preserve the high speed through the curve.
- If there is no freight-traffic on the railway line, it may have an inclination of 35‰.

According to Swedish Transport Administration (Swedish Transport Administration, 2008) the term "high-speed train" is often used for fast trains operated at conventional railway and the term "real high-speed train" is commonly used for trains that can be operated at a high-speed railway. This is the definition that is used in this Master thesis.

1.4 Methodology

This master thesis is a literature study of standards from three countries regarding design of high-speed railway embankment.

France

- Technical reference ”Référentiel technique LGV dans le cadre de PPP ou de DSP tome 2 – ouvrages en terre” (English translation: Technical Reference LGV as part of PPP or DSP Volume 2 – earthworks)

Germany

- RIL 836 Erdbauwerke und geotechnische Bauwerke planen, bauen und instand halten (English translation: RIL 836 Earthworks and geotechnical structures design, build and maintainance)

- Nachweise und Bemessung Tragfähigkeit und Gebrauchstauglichkeit (English translation: Evidence and design resistance and serviceability)

Further literature studies

- UIC 719-R, UIC 714-R, part 1 and 2, the European draft “Railway application – Ballastless track systems”
- The Swedish “Teknisk systemstandrad för höghastighetsbanor”. (English translation: Technical system standard for high-speed tracks)
- The Spanish railway requirements have also been studied, however, the study is based on information that Swedish transport administration has gained from Caminos de Hierro, ADIF and RENFE.

Additional information comes from case studies and interviews made by Swedish transport administration, a general literature study of research regarding high-speed railway embankment and finally, interviews with the following people:

- Bernd Heer, DB International GmbH
- Coenraad Esveld, Professor of Railway Engineering at TU Delft and author of the book Modern Railway Track.

Different design methods developed to decide the optimized thickness of the trackbed layer is presented in chapter 4. These design methods and their design parameters are discussed and compared. In chapter 5 a summary of the regulations based on the survey of literature is presented. Design parameters, material parameters and the thickness and bearing capacity for the different layer in the substructures are compared and discussed based on the differences and the design methods.

2 Railway embankments

This chapter is a brief description of the main railway components and of ballasted and ballastless embankments. A railway embankment is traditionally constructed with ballast. However, designing the embankment ballastless has become a popular alternative construction technique. Countries such as Germany and Japan have used this technique since 1970.

2.1 Railway components

The components of a traditionally ballasted railway can be divided into two subcategories: superstructure and substructure (Bårström and Granbom, 2012). The superstructure consists of the rail, sleepers and the fastening system (Fig. 2). The substructure consists of ballast, subballast and the subgrade.

The rail rests on the sleepers and the fastening hold the sleeper and the rail together. The railway is under perfect conditions completely straight without irregularities or bumps. This is, however, impossible to obtain in reality. Therefore, one must reduce the forces that arise from an irregular railway. Since the wheels are stiff, one must have an elastic railway embankment to avoid wearing of the rail and also to limit the vibrations, i.e. ensure an acceptable train riding comfort.

Rail
The rail has several functions. First of all, the rails guide the wheels steadily and continuously (Selig and Waters, 1994).The rails also work as beams that transfer load from the train to the sleepers. Therefore, the rail must have adequate stiffness.

Fastening system
The fastening system maintains the rail to the sleepers and prevents movements caused by traffic and temperature changes of the rail. Different fastening system can vary a lot from each other.

Sleeper

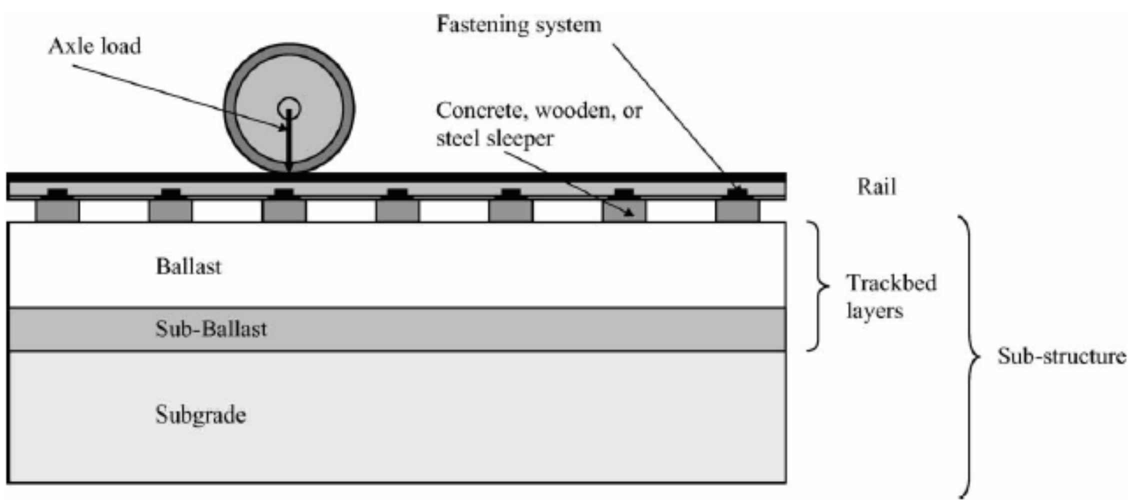


Figure 2. Simplified components of a typical ballasted track (Burrow, Bowness, and Ghataora, 2007).

Sleepers are generally made of reinforced/prestressed concrete or timber, but could also be made of steel. Sleepers do not only vary in material, but also in appearance. Three common sleeper formations are:

- Mono block: One concrete fragment that holds two rails (Fig. 3).
- Bi-block: Two lightly reinforced concrete fragments which is connected and held together with a steel tie rod (Fig 3).

Sleepers have a significant amount of important functions (Selig and Waters, 1994):

- Distribute the force from the rails into the substructure.
- Hold the fastening system in order to keep the correct track gage.
  The sleepers must be anchored to the substructure in order to restrain the rail movement

2.2 Forces

The track structures are exposed to mechanical forces: static, dynamic and thermal (Selig and Waters, 1994). The dynamic load caused by moving vehicles are often calculated as a factor >1 times the static load. The difference between the static load and the dynamic load is called the dynamic increment.

High frequency vibrations are a result from dynamic loading. Especially at high speeds, the vibration affects the performance of the superstructure and substructure.

The thermal forces caused by temperature changes expand or contracts the steel. If the rail is unable to expand or contract, it will cause stresses in the rail.

The forces that affect the track structure can be divided into three directions: vertical, lateral and longitudinal.

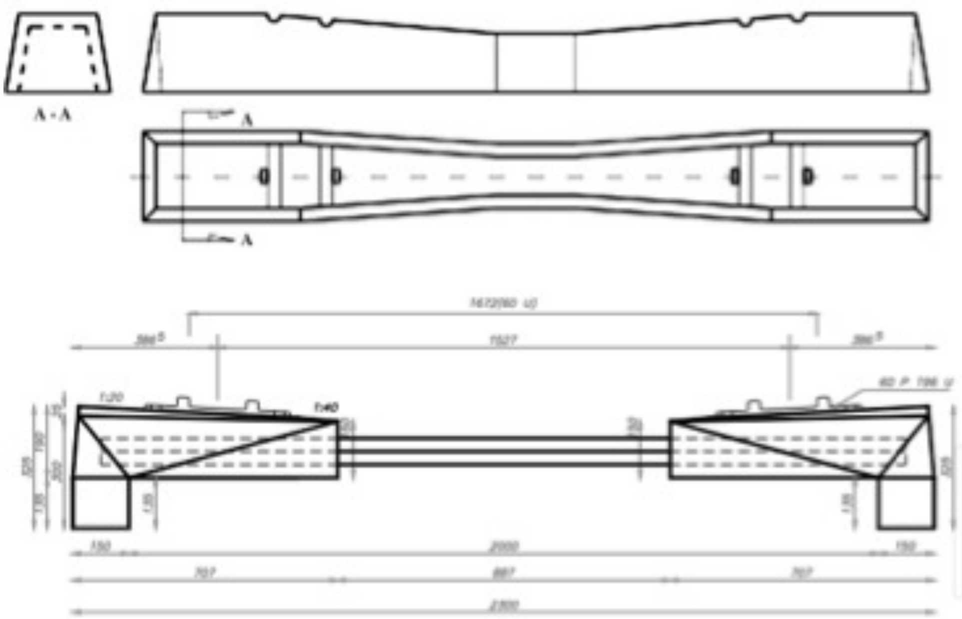


Figure 3. A mono block sleeper at the top of the figure and a bi-block sleeper at the bottom of the figure (Giannakos and Loizos, 2009: Guerrieri, Ticali, and Denaro, 2012).

Vertical forces

The vertical forces are perpendicular to the track plane and can be divided into wheel force and uplift force. The vertical wheel force causes, as a reaction, an uplift force, which lifts the rail upwards.

The vertical wheel force is often calculated as the load weight (this value have a wide range, depending on if it is a heavy freight train or a light passenger train) divided by the number of wheels plus a dynamic variation. The dynamic variations depends on (Selig and Waters, 1994):

- The geometry of the track (interaction between rail and train, e.g.: bounce)
- Wheel impact forces (e.g.: rail joints and wheel flats) these forces can be large and cause vibration in the track structure that for example contributes to track irregularities and powdering of ballast.

Lateral forces

These forces are parallel to the long axle of the sleepers. They could be divided into lateral wheel force and buckling reaction force.

Longitudinal force

These forces are parallel to the rails and come from the train traction force, braking force from the train, thermal expansion of rails and rail wave action.

2.3 Trackbed layers

The trackbed layers usually consist of ballast and subballast (Fig. 2) (Esveld, 2001). Different material may serve as an adequate ballast layer: crushed stone, gravel and crushed gravel. However, the most suitable ballast material is igneous rock (basalt, diabase or granite). Less suitable material is softer sedimentary rock (limestone, dolomite or sedimentary rocks) (Lichtberger, 2005). The prime factors when selecting the ballast material are the availability, under the assumption that the material available is of adequate quality, and economy. But there are also other important aspects: stone size (a common choice for main track is 25-60 mm and for secondary lines are 15-30 mm), size distribution, shape of each stone, the rate of undersize respectively oversize (usually 5% is tolerated)

The ballast layer have several functions, the most important functions are listed below (Lichtberger, 2005):

- The ballast layer should distribute contact forces from the axial load in order to preserve the track in the required positions and keep the stresses in the subgrade at a tolerable level.
- The ballast must work as drainage layer.
- The ballast must be sufficient elastic to avoid wearing out the rail and also limit the vibrations.
- The ballast can be periodically maintained to adjust the track line and level and will thus ease the maintenance work.
- The ballast provides voids for storage of fouling material in the ballast.

The subballast is the layer between the subgrade and ballast. The material of choice is broadly-graded crushed natural aggregates or slags. The main function of the sub-ballast is:

- Separate the ballast layer from the subgrade layer.
- Prevent the subgrade to be worn out by the ballast.
- To work as a drainage system; partly to shed water inflow from the surface and direct water away. From the subgrade and partly to drain water leaking up from the subgrade.
- Frost protection.
- Reduce traffic induced stresses.

Ballast material is more expensive than subballast material; therefore, an appropriate thickness of subballast reduces the expenses.

2.4 Drainage system

An important function of the trackbed layers is that they work as a drainage system (Lichtberger, 2005). Water primary enter the track substructure in three different ways: Rain and snow from above, water from nearby slopes enters the ballast and the underlying material, and finally, water can seek upwards to the substructure. To prevent frost heave, the trackbed layers must have sufficient thickness to give the required insulation, and the drainage system must keep the water out of the trackbed.

Each of these sources of water ingress to the embankment demands different drainage solutions. Excess of water may lead to several difficult problems, which leads to high maintenance costs. The problems that might occur is for example (Lichtberger, 2005):

- Irregularities in the track, due to heave by frost or less decreased bearing capacity due to periods of thaw.
- Decomposition of the ballast
- The water and subsoil creates a slurry which can cause attrition on the sleepers
- Volume increase
- Increased pore pressure

The trackbed layers also work as insulation of the subgrade, which will reduce the risk of frost heave (Peppin, Majumdar, Style, and Sander, 2011). This phenomenon occurs during freezing and thawing and can cause large vertical deformations of the track.

2.5 Settlements

A central problem in geotechnical engineering is to predict settlements and their development with time (Larsson, Bengtsson, and Eriksson, 1997). Many structures that are constructed today are sensitive to settlements but they can be sensitive to different types of settlements. For example. Large settlements in road and railway embankments may often be accepted, under the criteria that the settlements are even and that the time dependent settlements are possible to predict sufficiently accurate. These embankments are sensitive to *differential settlements* in both longitudinal and lateral directions.

Many countries have restricted the settlements in the earthwork to almost no settlement at all. Restrictions from Germany, Spain and France will be presented and discussed section 5.2

Subgrade deformations

The intention with this section is to give an understanding of the important role the subgrade plays when constructing the railway embankment; the subgrade is the foundation to the entire railway structure (Selig and Waters, 1994: Li and Selig, 1995). If the quality of the soil is not adequate, the soil must either be replaced with a stiffer material or be reinforced. Besides having adequate knowledge of the subsoil one must also have understanding of which deformation modes that may occur, in order to prevent them. If the subgrade deforms, it can no longer serve as a stable platform and, furthermore, it is difficult and expensive to maintain the subsoil.

The subgrade is the foundation on which the entire railway structure rests and influences the track performance and maintenance. The stresses from traffic can extend down to the subgrade, which means that the subgrade is an important part of the substructure and influences the track performance and the maintenance work of the track. The substructure influences the superstructures support resiliency and contributes to the elastic deflection of the rail under wheel loading. The differential settlements of the rail are also influenced by the subgrade.

Progressive shear deformation and excessive plastic deformation is two major problems that the subgrade is exposed to. The common denominator between these problems is that they are mainly caused by repeated traffic loading. It is important to consider both the magnitude of the traffic load and the number of load repetitions.

Progressive shear deformation

Progressive shear deformation arises due to cyclic stresses from the traffic is of that magnitude that it causes the soil to shear and remould (Selig and Waters, 1994: Li and Selig, 1995). Naturally, the soil is sheared upwards to the side of the track since this direction presents least resistance (Fig 4). The heave causes depression underneath the track, which leads to irregularities in the track. This problem is usually adjusted by adding ballast at the sag beneath the track, which will decrease the stresses in the subsoil. However, there is no such thing as free lunches, and the depression at the subgrade surface must also be corrected; otherwise, since ballast is it will trap water and reduce the affect of the increased layer of ballast.

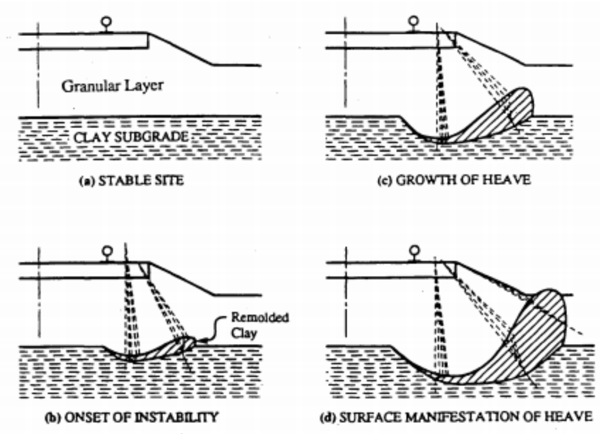


Figure 2. Progressive shear deformation (Selig and Waters, 1994: Li and Selig, 1995)

Excessive plastic deformation

Excessive plastic deformation is partly associated with progressive shear deformation and partly with progressive consolidation and compaction of the subsoil caused by the repeated traffic load (Selig and Waters, 1994: Li and Selig, 1995). Due to the latter case, this process is more rapid with newly constructed embankments. The plastic deformation is due to repeated traffic induced stresses and may cause tracks with unacceptable irregularities. Another problem that may occur if the subgrade consists of fine-graded material is that the depressions in the soil are filled with water. This will cause the soil to shear and create larger depressions. If ballast continuality is added underneath the sag of the track, a difficult problem named ballast pockets may occur (Fig 5).

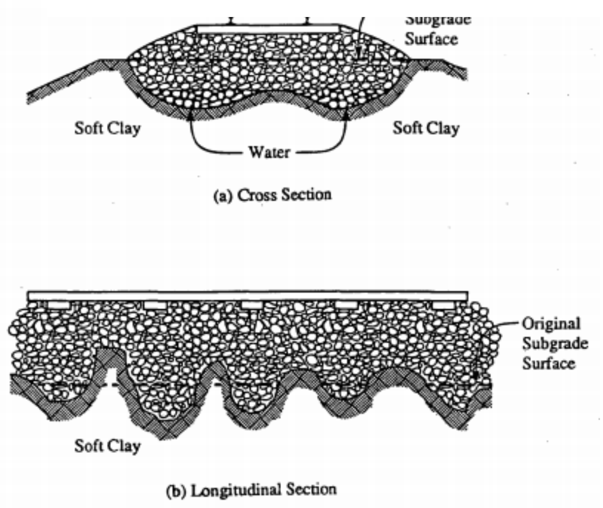


Figure 5. Excessive plastic deformation (Selig and Waters, 1994: Li and Selig, 1995)

2.6 Ballastless substructure and superstructure

Substructure: The substructure has significant influence of the performance of the slab track. The installation can be onto different earthwork (e.g.: embankment, tunnel, bridges, on piles etc) that vary in stiffness (Fig.6) and hence demands different construction methods (Esveld, 2001: SSF Ingenieure, 2010). Different countries focus on different structures, some countries primarily construct slab tracks on stiff ground, e.g.: tunnels or bridges and some countries design high-speed lines on softer earthwork, e.g.: embankments, soft soil). The earthwork has, naturally, a significant influence of the both the construction cost and also the track performance.

Superstructure: According to Esveld (2001), there are two generally designs principles. The German design, based on highway knowledge, the reinforcement is placed in the neutral line of the slab and is cast onto a stiff bearing layer (Esveld, 2003). The second method that (Esveld, 2001) describes in his article "Developments in high-speed track design" is a method that suggests the reinforcement to be placed on top and at the bottom of the slab to increase the bending resistance, and will from now on be referred to as "the other method".

The German slab track solution and design concept origins from experience received from highway design (Esveld, 2001, 2003, 2008: Liu, Zhao, and Dai, 2011). The slab track is supported by a layer of high bearing stiffness, a modulus of elasticity, $E_{v2,}$ of at least 120 MPa. The layer underneath shall also have a suiffiecient bearing stiffness, $E_{v2}\geq$60 MPa. Standard proceedure in the "German design" is to place the reinforcement in the neutral axis in order to control the crack width. That works well at rigid support such as bridges and tunnels, however, it does not provide any extra bending resistance to the slab. Today, most of the existing slab track is based on this German slab track solution and thus, if the slabs are installed onto week subgrades, massive soil reinforcement is necessary.

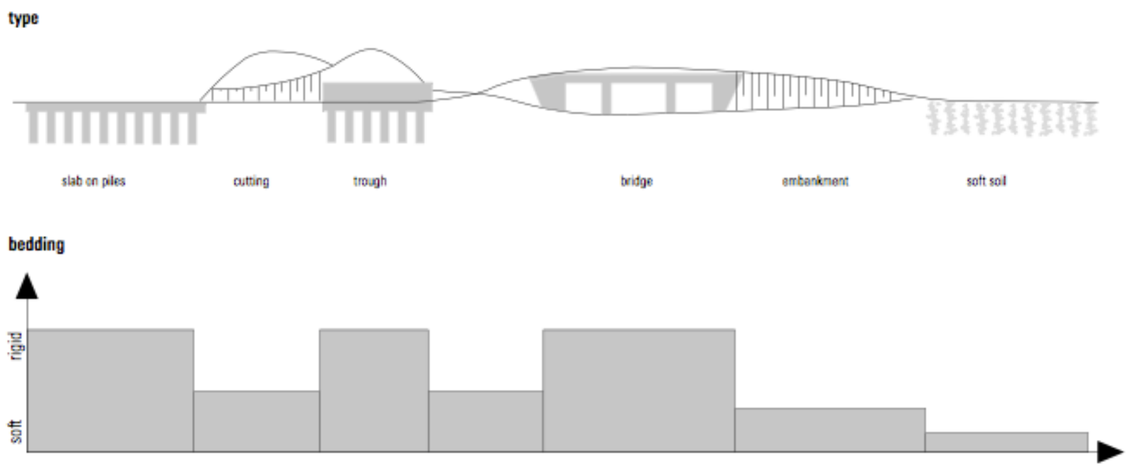


Figure 6. Different earthwork and the variation in stiffness (SSF Ingenieure, 2010)

In an e-mail interview with Esveld (Esveld, 2016), he write, that with this high E_{v2} value ($E_{v2}\geq$120 MPa), large settlements are unlikely. In Esveld´s opinion, this leads to a non-optimal design. This is partly based on the lack of bending reinforcement in the German slab tracks. If the slabs were created with higher bending stiffness, the cost could be reduced and also allow lower E_{v2} values in the slab. The ground in Sweden is of poor quality, therefore would this type of construction be an option.

The second design method is a good choice if the subsoil is of poor condition (Esveld, 2003). Hence, when the slab is installed onto a weak subgrade one can expect a greater bending moment in the slab, and settlements in the subsoil. To avoid this, the slab may be reinforced at the bottom and at the top, which increases the bending stiffness and, furthermore, helps the slab to take axial forces. This will reduce the need to use soil reinforcement significant. However, on the other hand, research at TU Delft (Esveld, 2001) has shown that this design technique requires a high amount of reinforcement (1.5% for B35 concrete).

Three typical ballastless constructions from Germany and one from France can be seen in Appendix A.

2.7 Ballastless track versus ballasted track

The most common railway construction among the current railways is still the ballasted track (Esveld, 2001: Lichtberger, 2005). However, ballastless tracks (also called slab track) are becoming a competitive track system solution. It is especially competitive for high-speed railways. A comparison between the different techniques may be seen in Table 1 below.

Table 1. Comparison between ballastless and ballasted systems (Esveld, 2001: Lichtberger, 2005).

	Ballastless	**Ballasted**	**Comments**
Life time	50-60 years	30-40 years	**Ballasted systems:** Previous experience on ballasted high-speed railway shows that maintenance work must be done after the track has deformed about 20 mm (usually after the track has been exposed to a traffic load of 30-60 Mt) and the track must be cleaned after 15 years. Furthermore, the ballast bed must be renewed after about 30 years.
Availability	Higher availability than on ballasted tracks.	Lower availability than on ballastless tracks.	
Maintenance	Lowered maintenance requirements (e.g: no ballast cleaning or tamping is needed), which lead to lower maintenance costs and higher availability	Higher maintenance requirements compared to ballastless systems.	**Ballastless systems:** If maintenance is needed is more difficult to adjust the track position for (e.g.: the height change in each rail fastening point must be calculated) therefore it is more expensive and difficult to make adjustment after the construction is done which means that it is difficult to upgrade existing slab track to future innovative techniques. **Ballasted systems**: Even thus ballasted tracks requires maintenance, the work is relatively simple; the track component can easily be replaced if needed and lesser adjustments of track lay-out (curves) are achievable.
Material type	Reinforced concrete or asphalt.	**Advantage:** Due to the ballast in the embankment the elasticity, drainage and damping of noise is good. **Disadvantage:** The ballast may be damage and contamination in the embankment	**Ballasted systems:** The Deutsche Bahn has gathered experience from high-speed lines operated on ballasted tracks, the investigations shows that the ballast is damaged in certain zones associated with rail deflection, welded joints, insulation rail joints, bridge crossing.

		which leads to lowered drainage capacity.	
Weight and height of the structure	Reduced weight and height compared to ballasted systems	Relatively heavy and high structures.	**Ballasted systems:** Requires strong construction for bridges and viaducts.
Construction costs	Higher construction costs compared to construct ballasted embankments.	Lower construction costs compared to constructing ballastless embankments.	**Ballastless systems:** The construction work is more expansive. **Ballasted systems:** Established technology that has relatively low production costs.
Airborne noise	Higher than for ballasted tracks.	Due to the ballast, the damping of noise is good	
Settlement requirements	More sensitive to settlements and therefore requires either a more stable ground or excessive ground improvement.	Irregular settlements on high-speed lines.	

3 Design methods

Chapter 4 describes four different methods to calculate the required thickness of the trackbed layer. There are a numbers of different methods for the design of the trackbed (Burrow et al., 2007). However, the different methods use different design parameters, and they also give different trackbed thickness. It is important to compare and understand the different methods, understand why they differ and how the design parameters influence the methods. To that end, these methods are presented in section 4.1 - 4.4 and a comparison between them may be seen in section 4.5.

3.1 Li et al. method

Li et al's method targets to prevent both excessive plastic deformation and progressive shear deformation (Li, 1994: Li, Sussmann Jr, and Selig, 1996: Li and Selig, 1998). To that end, a serial of design charts (Fig. 9, 10 and 11) and design methods was developed using GEOTRACK[1] and extensive test results of diverse subgrade soils exposed to repeated loadings.

Since this method contains two different design criterions, excessive plastic deformation and progressive shear deformation, must both of these criterions be evaluated in order to determine which criteria that require the largest trackbed layer. The required thickness is calculated with respect to plastic deformations according to Method 1 and with respect to excessive plastic deformation according to Method 2. The first set of design charts was developed to use with Method 1, example of such design chart may be seen in Figure 9 and 10. The second set of design charts were developed together with Method 2, and example these design charts may be seen in Figure 11.

3.1.1 Design procedure

Excessive plastic deformation and progressive shear deformation can be related to cumulative plastic strain and cumulative deformation. Therefore, several triaxial tests were made on different fine-grained subgrade soil. These tests aimed to determine the allowable plastic strain and deformation under repeated stress application. Based on the results from these tests, two Equations (1 and 2) were derived.

In order to determine cumulative plastic strain ε_p, Equation 1 may be used:

$$\varepsilon_p = a(\frac{\sigma_d}{\sigma_s})^m \cdot N^b \qquad (1)$$

Where,

a,b and *m*- Parameters that relates to the soil type, e.g.: moisture content, dry density and soil structure. (Tab. 2).
σ_d- Soil deviator stress
σ_s- Soil compression strength
N- Number of repeated stress applications

Figure 7 shows an example the variation in ε_p depending on the material parameter and the $\frac{\sigma_d}{\sigma_s}$. The number of repeated stress application is set to N= 10^7.

[1] GEOTRACK is a three-dimensional program that was developed in order to calculate the train-induced stresses in the trackbed layer and in the subgrade.

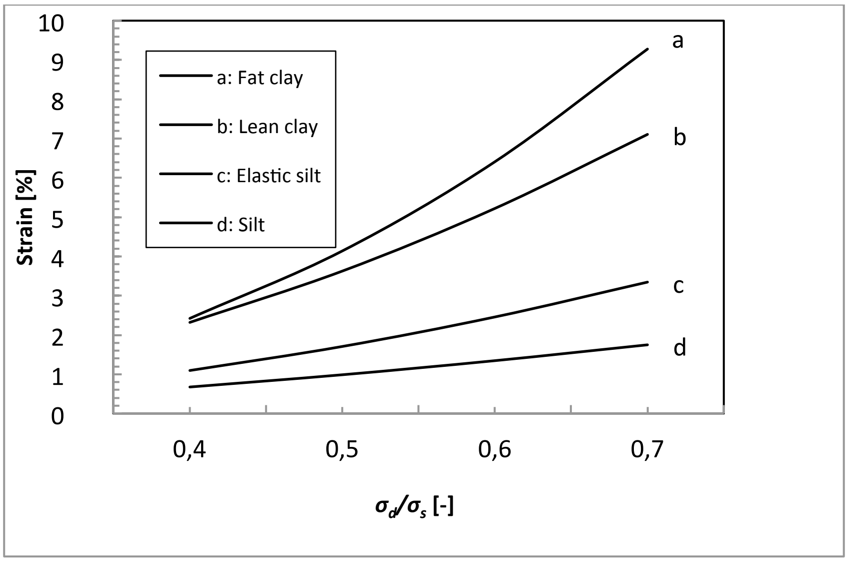


Figure 7. Cumulative plastic strain variation, depending on ratio of deviator stress and compression strength

In order to determine cumulative deformation, p, Equation (2) may be used:

$$p = \int_0^T \varepsilon_p \cdot dt \qquad (2)$$

Where,

T- Depth of subgrade layer

Two design criterion is set (Tab. 3) in order to limit the total ε_p and the total p for the design period. The design procedure for each method is presented below, both of these criterions must be evaluated and the trackbed is dimensioned after the method that requires the largest trackbed.

Table 2. Suggested parameters for different soils (Li et al., 1996)

Soil type	a	b	m
Fat clay	1.2	0.18	2.4
Lean clay	1.1	0.16	2.0
Elastic silt	0.84	0.13	2.0
Silt	0.64	0.10	1.7

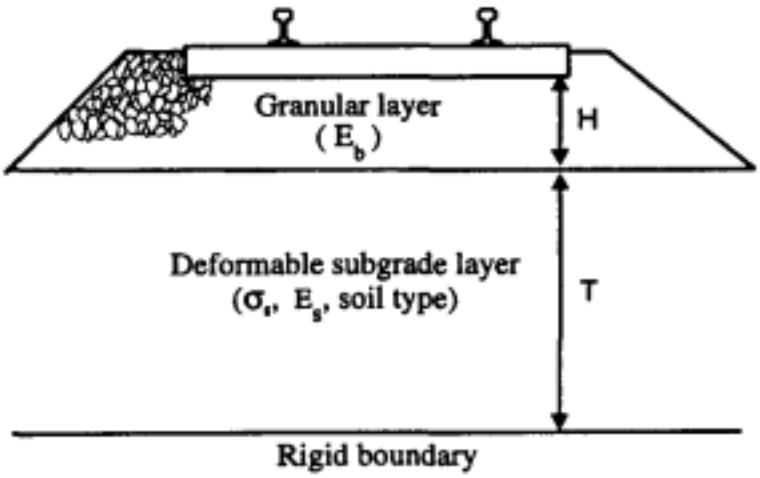
Table 3. Design criteria (Li 1998)

Design criteria 1 Design criteria that aims to protect from progressive shear deformation is:	Design criteria 2 Design criteria that aims to protect from excessive plastic deformation is:
$\varepsilon_p \leq \varepsilon_{pa}$ Where, ε_p- Total cumulative plastic strain at the subgrade surface for the design period ε_{pa}- Allowable plastic strain at the subgrade surface for the design period	$p \leq p_a$ Where, p- Total cumulative plastic deformation of the subgrade layer for the design period p_a- Allowable plastic deformation of the subgrade layer for the design period

Presentation of Method 1 and Method 2

The material parameters that are included in this method (Fig. 8) must be known in order to determine the thickness of each layer.

Method 1



- Trackbed layer:
  -Height, *H*
  -Resilient modulus, E_b
- Subgrade layer
  -Height, *T*
  -Soil type (Tab. 2)
  -Resilient modulus, E_s
  -Compressive strength, σ_s

Figure 8. Definition of layer height (Li and Selig, 1998)

Method 1 aims to calculate the required *H* to prevent cumulative plastic strain at the surface of the subgrade. Step 1 and 2 aims to determine the σ_{da}. If the σ_{da} can be determined directly, step 1 and 2 is not necessary.

Step 1

First of all, the material information needed for the design must be prepared. This includes the characteristics of the subgrade and trackbed layer (Fig. 8). Furthermore, the magnitude of the design criteria, ε_{pa}, must be determined for the design period.

The traffic condition must also be determined. Li et al method uses the computer program GEOTRACK to do so. GEOTRACK is a three-dimensional program that was developed in order to calculate the train-induced stresses in the subgrade. To account for that, three design traffic parameters: static wheel load, train speed and traffic (represented by MGT) is transformed into two design parameters. These parameters is: the design dynamic wheel load, P_d, and the total number of repeated load applications, *N*.

The trackbed is modelled as a uniform layer supported by a layer of homogenous subsoil overlaying a rigid layer (Fig. 8). Li et al (Li, 1994) suggest that P_d shall be used in order to account for track and vehicle irregularities. If P_d is not known, it is possible to calculate it from Equation 3:

$$P_{di}=(1+\frac{0.0052\cdot v}{D_w})\cdot P_s \tag{3}$$

Where,

P_{di} -A dynamic wheel load
P_s -A static wheel load
v- Train speed
D_w- Wheel diameter

The number of load applications N_i is determined by:

$$N_i=\frac{T_i}{8P_{si}} \tag{4}$$

Where,

T_i-Total traffic tonnage for wheel load P_s

To convert N_i cycles of a wheel load P_{di} to a design wheel load P_d under N_i^0 cycles of wheel load, equation 5 is used:

$$N_i^0=N_i\left[\frac{P_{di}}{P_d}\right]^{m/b} \tag{5}$$

Total number of load application, N, for P_d is calculated by:

$$N=N_1^0+..+N_i^0+... \tag{6}$$

Step 2

Step 2 is to determine the allowable deviator stress, σ_{da}. First of all, the soil type is chosen from Table 2 and the corresponding chart is chosen. For example, if soil type "Fat clay" the chart from Figure 9 is chosen and the point β $(\frac{\sigma_{da}}{\sigma_s})$, that corresponds to N, and ε_{pa}. Equation 7 is used to calculate σ_{da}.

$$\sigma_{da} = \sigma_s \cdot \beta \tag{7}$$

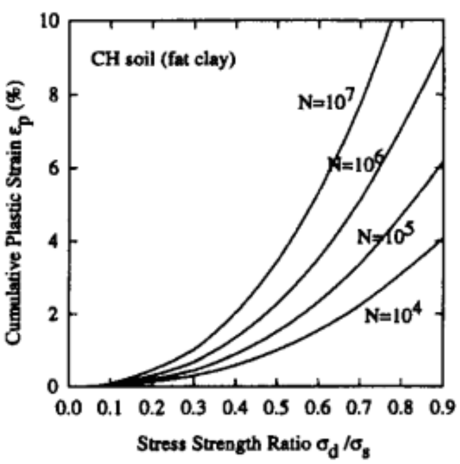


Figure 9. Example of design chart, the soil type is selected from Table 2. (Li and Selig, 1998)

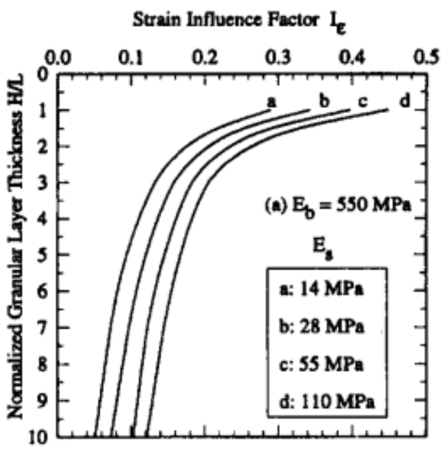


Figure 10. Example of design chart (Li and Selig, 1998)

Step 3
This step determines the required thickness of the trackbed layer. The strain influence factor, I_ε, is determined by:

$$I_\varepsilon = \frac{\sigma_{da} \cdot A}{P_d} \quad (8)$$

Where,

σ_{da} – Allowable deviator stress at subgrade surface (the deviator stress is the difference between the major and the minor principal stresses)
A- Area factor used to make the strain influence factor dimensionless, when SI units is used the factor 0.645 m^2 is used.

Thereafter the chart that has the E_b-value that agrees mostly to the real E_b is selected. For example, if E_b is close to 550 MPa, Figure 10 is selected. The point from the chart that corresponds to I_ε and E_s is selected and the value for H/L is obtained and H is determined by Equation 9 as follows:

$$H = \frac{H}{L} \cdot L \quad (9)$$

Where,

L- Length factor[2], equal to 0.152 m

Method 2
Method 2 aims to calculate the required *H* to prevent from total plastic deformation of the subgrade layer.

Step 1
The material parameter that is required in Method 1 is needed for Method 2 as well. In addition the thickness of the subgrade layer, *T,* and the design criteria ε_{pa} is switched to φ_a.

The traffic parameters P_d and N are calculated as in Method 1.

Step 2
The strain influence factor, I_φ, is determined by:

$$I_\varphi = \frac{\frac{\varphi_a}{L}}{a \cdot \left[\frac{P_d}{\sigma_S \cdot A}\right]^m \cdot N^b} \cdot 100 \quad (10)$$

Where,

φ_a- Allowable total subgrade plastic deformation for the design period
N- Total equivalent number of load repetitions for the design period
Pd- Design dynamic wheel load
σ_s- Soil compressive strength
a,m,b- Material parameters (Tab.2)
A,L – length factors as defined previously

[2] The length factor is used to make the design chart dimensionless for both SI and English units and has no physical meaning.

Step 3

Thereafter the chart that corresponds best to E_s, E_b and the soil type is selected, for example Figure 11. The point in the chart that corresponds to the T/L curve and I_p is selected and H/L is calculated as before.

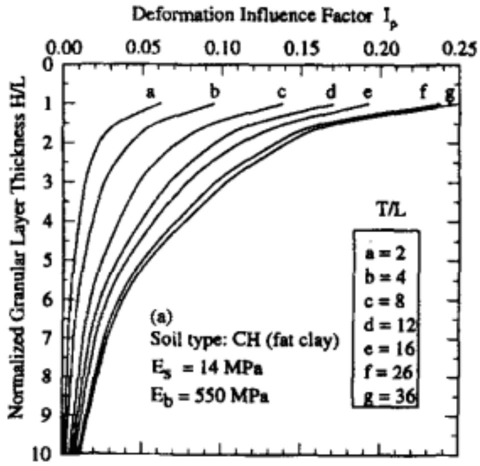


Figure 11. Example of design chart (Li and Selig, 1998)

3.2 The international union of railways method

This is an empirically founded set of recommendations for the design and maintenance of the substructure of the railway (Burrow et al., 2007: UIC, 2008: Peter et al., 2011). The recommendation suggests that the substructure should consist of, some or all, of the following layers: ballast and granular subballst (however no individual thickness of the trackbed layer is presented), a geotextile and a prepared subgrade. The method is valid for track gauges between 1435-1668 mm and with a c/c sleeper spacing of 0.6 m. The maximum axle load is 25 t.

3.2.1 Design procedure

The soil quality is classified according to Table 4. The subsoil is classified primarily based on the percentage of fines in the soil, i.e. frost sensitivity. There are four classes that the soil may be divided into:

- QS0: the soil is of poor condition and need improvement to be suitable to serve as a foundation.
- QS1: the soil is suitable, if provided with adequate drainage system and maintenance. However, reinforcement should be considered.
- QS2: the soil is of average quality.
- QS3: the soil that is considered to be of good quality

Table 4. Soil classification according to UIC 719-R

	Soil type (geotechnical classifications)	Soil quality class
0.1	High-organic soils	QS0
0.2	Soft soils containing more than 15% of fines[a], with a high moisture content, unsuitable for compaction	
0.3	Thixotropic soils[b] (e.g. quick clay)	
0.4	Soils containing soluble material (e.g. rock salt or gypsum)	
0.5	Contaminated ground (e.g. industrial waste)	
0.6	Medium-organic soils[b]	
0.7	High plasticity soils with more than 15% of fines, collapsible soils[c] or expansive soils[d]	
1.1	Soils containing more than 40% of fines[a] (except for soils classified under 0.2 or 0.7)	QS1
1.2	Rocks which are very susceptible to weathering E.g.: -Chalks with p_d<1.7 $\frac{t}{m^3}$ and high friability -Marl -Weathered shale	
1.3	Soils containing more than 15 to 40% of fines[a] (except for soils classified under 0.2 or 0.7)	QS1[e]
1.4	Rocks which are moderately susceptible to weathering E.g.: -Chalks with p_d<1.7 $\frac{t}{m^3}$ and low friability -Unweathered shale	
1.5	Soft rock E.g.: Microdeval wet (MDE) > 40 and Los Angeles (LA) > 40	
2.1	Soils containing from 5-15% of fines[a] except collapsible soils[c]	QS2[f]
2.2	Uniform soils containing less than 5% of fines[a] (C_u≤6) except collapsible soils[d]	
2.3	Moderately hard rock E.g.: If 25 < MDE ≤40 and 30 < LA ≤40	
3.1	Well graded soils containing less than 5% of fines[a]	QS3
3.2	Hard rock E.g.: If MDE ≤ 25 and LA ≤30	

a. These percentages are calculated from particle size distribution analysis undertaken on material passing a 0.063 mm sieve. The percentages indicated here have been rounded down (practices vary slightly from one railway to another); they may be increased by up to 5% if a suiffciently representive number of samples is taken.
b. Certain railway someteimes include these soils in quality class QS1.
c. Collapse settlement higher than 1% for undistributed samples or for remoulded samples with the standard Proctor density and a normal pressure of 0.2 MPa
d. Free swelling higher than 3% for undistruibuted samples or for remoulded samples with the standard Proctor density.
e. These soils can come under quality class QS2 if the hydrogeological and hydrogical conditions are good.
f. These soils can come under quality class QS3 if the hydrogeological and hydrogical conditions are good

After the soil is classified, the bearing capacity of the soil, *P*, and the minimum thickness of the subgrade, e_f, is determined by using Table 5. For high-speed lines, bearing class P3 shall be considered.

Table 5. Required bearing capacity, depending on the soil quality of the embankment or excavation surface

Embankment or excavation surface		Bearing class for the subgrade	Requirements of prepared subgrade		
Soil class	CBR[a]		Soil class	CBR[b]	Minimum thickness of subgrade, e_f (Fig. 12)
QS1	2[c]-3	P1	QS1	2[c]-3	-
		P2	QS2	5	0.50 [m]
		P2	QS3	10-17[c]	0.35 [m]
		P3	QS3	10-17[c]	0.50 [m]
QS2	5	P2	QS2	5	-
		P3	QS3	10-17[c]	0.35 [m]
QS3	10-17[c]	P3	QS3	10-17[c]	-

a. CBR corresponding to the saturated in situ conditions of the material
b. CBR corresponding to a remoulded saturated sample compacted to the design conditions of the material
c. Proposed values according ERRI Report D117/RP 28 (1983)

The total thickness (Fig. 12) is decided based on the bearing capacity of the subsoil (P1, P2 or P3), type of sleepers and their spacing and also the traffic characteristics.

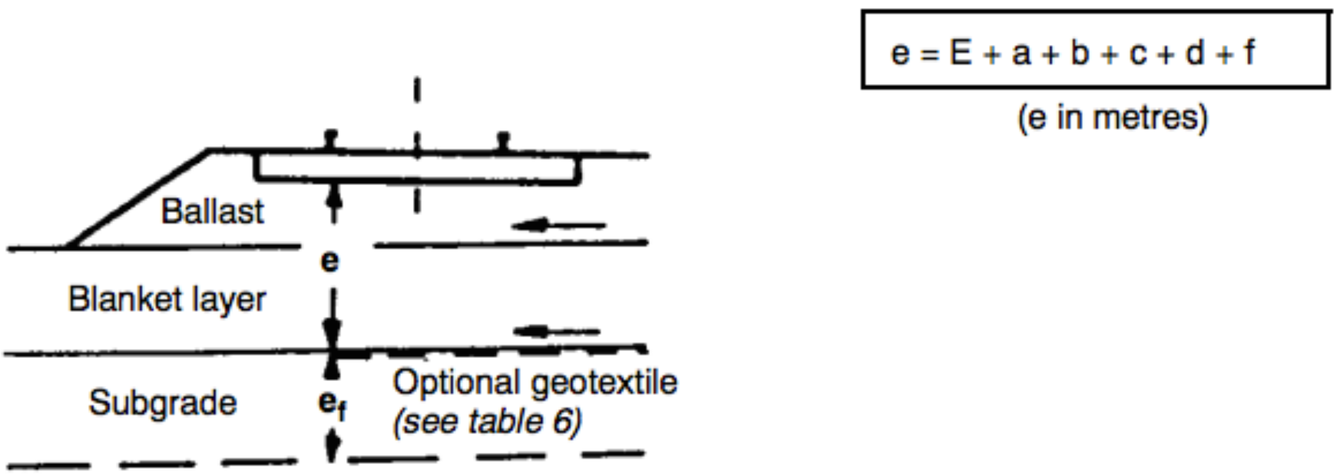


Figure 12. Minimum thickness of trackbed layer, e[m] (UIC, 2008).

Table 6. Parameters to calculate the minimum thickness according to Figure 11 (UIC, 2008).

Parameter	Thickness	Definition
E	+0.7 [m]	Bearing capacity: P1
E	+0.55 [m]	Bearing capacity: P2
E	+0.45[m]	Bearing capacity: P3
a	0	UIC groups 1-4[a]
a	-0.10 [m]	UIC gruops 5 and 6[a]
b	0	Wooden sleepers, length 2.60 m
b	$+\frac{(2.5\text{-L})}{2}$	Concrete sleepers of length *L* (if *L* >2.50 <m b may be negative)
c	0	For usual dimensions
c	-0.10 [m]	For difficult working conditions on existing lines
d	0	Axle load < 20 t
d	+0.05 [m]	Axle load < 22.5 t
d	+0.12 [m]	Axle load < 25 t
f[b]	+	Geotextile should be included if the prepared subgrade is formed from class QS1 or QS2
f[b]	0	If the prepared subgrade is formed from soils of quality class QS3 no geotextile required,

a. The UIC groups are defined under the rubric traffic

b. Meaning that a geotextile is needed if the soil is is formed from a soil class of less than geotextile.

Traffic

The UIC 719 R uses a method, which converts the daily traffic to a theoretical traffic load. The UIC 719-R refers the traffic characteristics to the UIC 714 (UIC, 2009). The UIC 714 classifies railway lines into 6 different groups, based on a theoretical traffic load, *Tf* (Eq. 11).

$$Tf = Sv \cdot (Tv + Kt \cdot Ttv) + Sm \cdot (Km \cdot Tm + Kt \cdot Ttm) \qquad (11)$$

Where,

Tv Mean daily passenger tonnages in gross tonnes hauled
Tm Daily freight tonnage in gross tonnes hauled
Ttv Mean daily tonnage of tractive unit used in passenger traffic, [t]
Ttm Mean daily tonnage of tractive units used in freight traffic, [t]
Km Coefficient allowing both for the influence of the load and wear effect of freight bogies. Normally corresponds to:
Km=1.15 For traffic that consists more than 50% of trains with 20 t axle load or traffic that consists of at least 25% of trains with an axle load of 22.5 t.
Km=1,45 For traffic that consists more than 50 % of trains with an axle load of 22.5 or traffic that consists of at least 75% of trains with an axle load of at least 20 t.
Kt Coefficient that allows for the traction-motor axle wear factor, equal to 1.40
Sv Coefficient that allow for the fastest passenger trains speeds
Sm Coefficient that allow for the speed of ordinary freight trains

Based on the UIC 714 (Eq. 11), the thickness of the trackbed increases due the lines with heavier and faster traffic.

3.3 British Rail method

The British Railway developed a method from research and aims to protect against excessive plastic deformation by a threshold stress value (Heath, Shenton, Sparrow, and Waters, 1972: Li et al., 1996: Burrow et al., 2007). If the stresses in the subsoil are above the threshold value it causes large plastic deformations in the subsoil and if the stresses is below this value the cumulative deformations are small.

To this end, two models were developed .The first model is a linear elastic model of the track system that consider the induced stresses from traffic. The other model determined the threshold stress in the subsoil by a cyclic triaxial compression test, were the cumulative soil strain in the sample is seen as a function of the number of loading cycles applied.

3.3.1 Design procedure

Track model

The elastic stress distribution in the substructure was theoretically calculated by a computer program designed for this purpose. The subsoil and the trackbed layers are in the model simplified to homogenous layers and the stress distribution is calculated with various sleepers spacing and contact pressures.

The theoretically calculated results were compared to field test, at East Coast Main Line at Wood Green. The results showed that the mean vertical stress distribution is possible to estimate by the simple elastic theory.

Subsoil parameters

The soil parameters must be determined in an accepted method in order to use the result in a design method. This time the choice of technique fell at a triaxial compression test. Hence, principal stresses were applied from two direction; major vertical direction and minor horizontal direction. The minor principal stresses were held constant and the major principal stress was pulsed. The maximum cycling frequency was 30 c/min. The choice of soil fell at London clay collected at Wood Green at an average depth of 900 mm below ground level. The clay sample was provided with an adequate drainage system.

The cumulative soil strain in the sample is seen as a function of the number of loading cycles applied. The test showed that a threshold value existed; stress above the threshold value causes large plastic deformations and under the threshold value cumulative deformations are small.

The threshold value is usually significantly smaller than the static strength of cohesive soils. The threshold strength is in practice often assumed to be 50% of the static strength of the soil.

Design chart

Based on the track model and the subsoil parameters a series of design charts were made for different soil types and axle loads. The design charts (Example of design chart may be seen in Fig. 13) related the required thickness (the distance from the bottom of the sleeper to the top of the subgrade) of the trackbed to the threshold stress in the soil obtained from the heaviest axle load that is operating on the line.

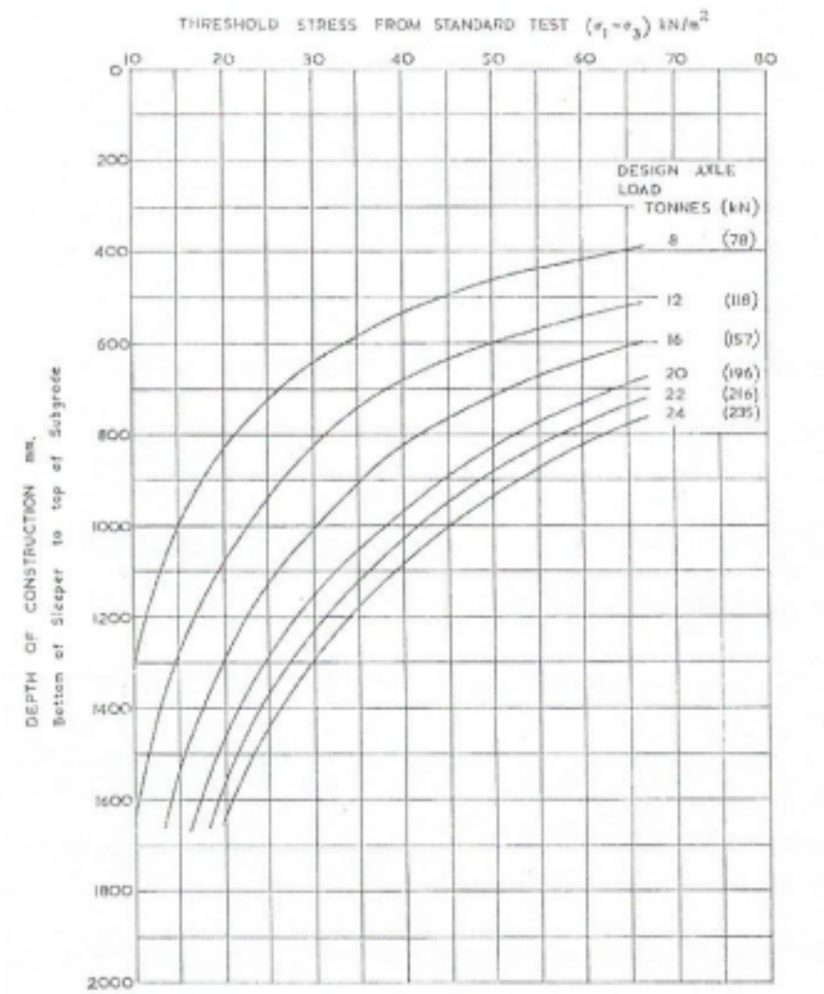


Figure 13. Design chart used to determine the stress threshold value, from (Heath et al., 1972) .

3.4 Network Rail code of practise

The Network Ril code of practise, NR/SB/TRK/ 9039 "Formation treatments", gives recommendations of the required thickness of the trackbed layer (Burrow et al., 2007). At sites where excessive maintenance and irregular track geometry have been observed, the required thickness can be determined with charts. Furthermore, the code suggests that the subgrade has sufficient strength and stiffness at sites with acceptable track irregularities and with a "normal" maintenance period.

3.4.1 Design procedure

Trackbed layer

The chart relates the required trackbed thickness to Young modulus of the subgrade for different values of the dynamic sleeper support stiffness: 30, 60 and 100 kN/mm sleeper end. According to Burrow et al (Burrow et al., 2007), the code does not give any technical information on how the charts were developed only that the charts were "derived using a combination of empirical data and multi-layer elastic theory".

3.5 Comparison between the methods

A series of test were made in order to see how the trackbed thickness varies between the different methods (Burrow et al., 2007: Peter et al., 2011). Parameters that influence the trackbed thickness: subgrade, axle load, speed and cumulative tonnage were varied. Different methods take different design parameters in consideration, which results in varied thickness of the trackbed.

Table 7. Factors accounted for in the design procedures reviewed. Table modified from (Burrow et al., 2007))

	Li et al's method	UIC 719 R	British Rail Method	NR Code 039
Static axle load	From GEOTRACK model used to formulate their design charts	Yes	From an elastic model (charts only go up to an axle load of 24 tonnes)	No – but 25.4 tonne axle load limit on UK network
Sleeper type, length and spacing	GEOTRACK	Yes	No difference in stresses found for sleeper spacing of 630–790 mm	No
Rail section	GEOTRACK	No	No	No
Speed	By using a dynamic axle load or by the AREA equation.	Yes	No – field results showed response was quasi-static up to 100 km/h, but could be incorporated by using a dynamic axle load	Via minimum requirements for the dynamic sleeper support stiffness. Also 125 mile/h is fastest speed on UK network
Annual tonnage	Yes	Yes	No	No
Cumulative tonnage	From annual tonnage multiplied by the design life	No	No	No
Subgrade condition	Charts are provided for different subgrade types in terms of the resilient modulus and soil strength	Yes (using soil quality determined primarily from the number of fines in the soil)	Using a threshold stress for the material in question	Undrained subgrade modulus or undrained shear strength

Subgrade

Depending on which method one is using, the trackbed thickness varies significantly (Fig. 14). For example, resilient modulus is 15 MPa the trackbed thickness varies from 1-1.9 m and for a resilient modulus of 100 MPa the trackbed thickness varies between 0.2-1 m.

Stiffer subgrade may resist higher loads and a higher stress level and the other way around, soils of less resistance may resist lower stress levels. Therefore, should the engineering properties of the subsoil be a design parameter. This is also the case for all methods except for the UIC 719-R. This method, as described earlier takes the percentages of fines in consideration, the strength and stiffness in the soil is solely included indirectly, which means that soil of different strength and stiffness is classified in the same category, as long as the percentage of fines is the same. This may lead to an overestimated or underestimated thickness of the trackbed layer.

Subgrade stress analysis

According to Li et al (Li, 1994, 1998), the primarily limitation compared with other design methods is that they assume homogenous half-space for the trackbed layers, which results in an overestimation of the train induced stresses in the subsoil (Li and Selig, 1998). The thickness of the trackbed layer and the resilient modulus of both the trackbed layer and the subsoil have a significant influence on the deviator stress level in the subsoil. However, the resilient modulus of the trackbed layer is, according to GEOTRACK analyses (Fig. 15 and Fig. 16) made by Li Dingqing (Li and Selig, 1998), the material parameter that has the greatest influence on the stress level in the subsoil.

Figure 15 shows the deviator stress at the subgrade surface below the embankment tie at varying trackbed thicknesses. The stresses in the subgrade are reduced with an increased trackbed layer and, furthermore, the stresses are distributed more uniformly. Moreover, Figure 15a and 15b also shows that a higher stiffness in the trackbed layer results in lower σ_{da} in the subgrade, which indicates the limitation with homogeneous half-spaced theory.

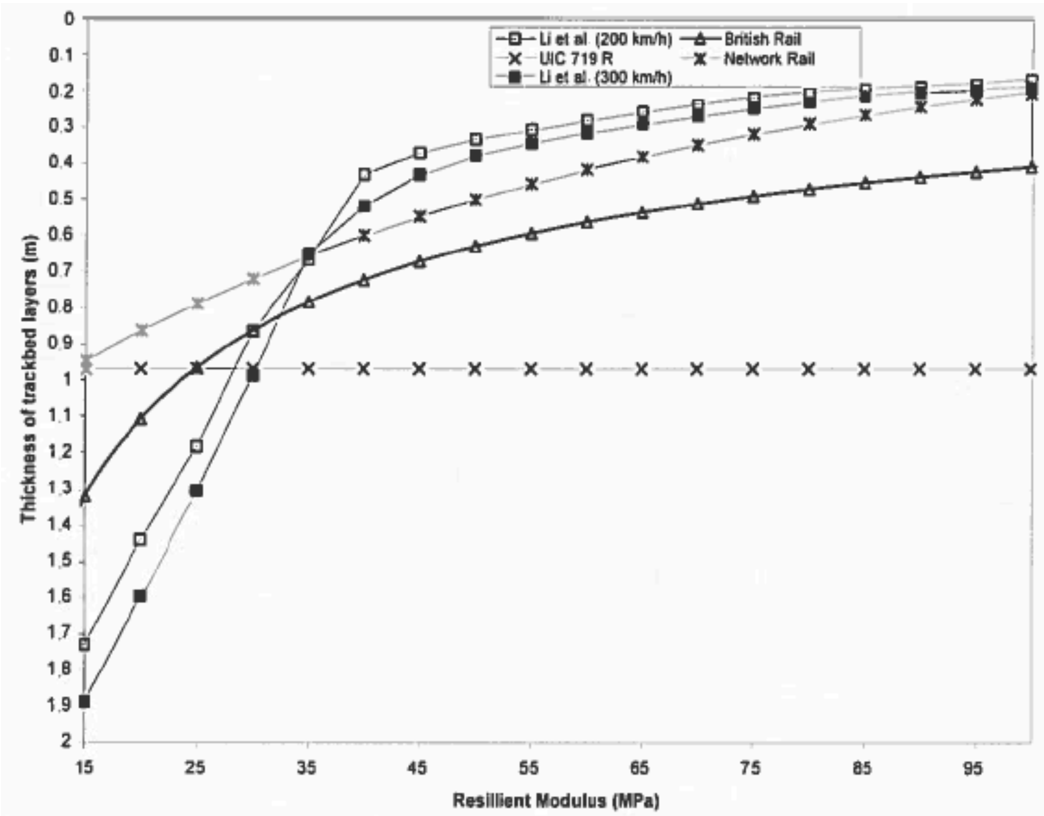


Figure 14. Variation in designed trackbed thickness, depending on design method and the resilient modulus in the subgrade (Peter, Burrow, Ghataora, and Evdorides, 2011).

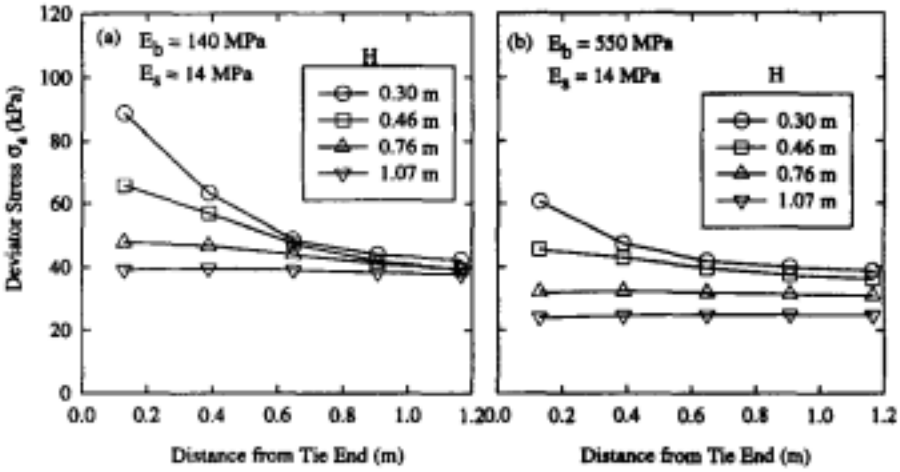


Figure 15. Deviator stress at subgrade surface for different thickness and resilient modulus of the trackbed layer. Figure a) Trackbed resilient modulus = 140 MPa, b) trackbed resilient modulus 550 MPa (Li and Selig, 1998)

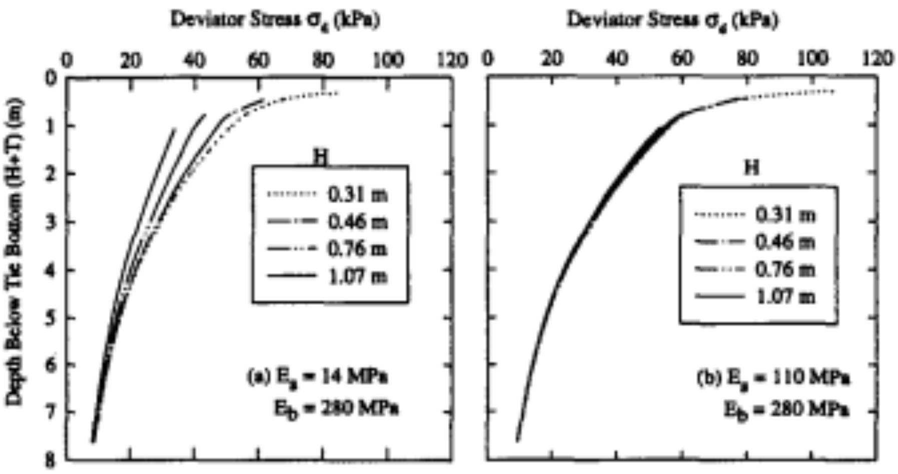


Figure 16. Deviator stress at subgrade surface for a) soft subgrade, 14MPa and b) stiff subgrade, 110 MPa for varying thickness of the trackbed layer (Li and Selig, 1998)

The resilient modulus of the subsoil also has an influence on the stress level in the subsoil, the stiffer subsoil the higher the stress level (Fig. 15a and b). However, a high stress level corresponds to a high strength of the subsoil and the strength increase is usually greater than the stress increase (Li and Selig, 1998).

Axle load

The load from the train causes stresses in the substructure, which are transmitted to the subgrade producing strain and deflection. Hence, the axle load is suggested to be a design parameter when designing the thickness of the trackbed layer. The Network Rail code of practice is the only method that not considers the axle load as a design parameter. However, even though the other three methods consider the axle load as a design parameter the suggested thickness varies significant. The design thickness varies at 140 kN between 0.2-0.6 m and at 340 kN between 1.2-0.8 m (Fig. 17).

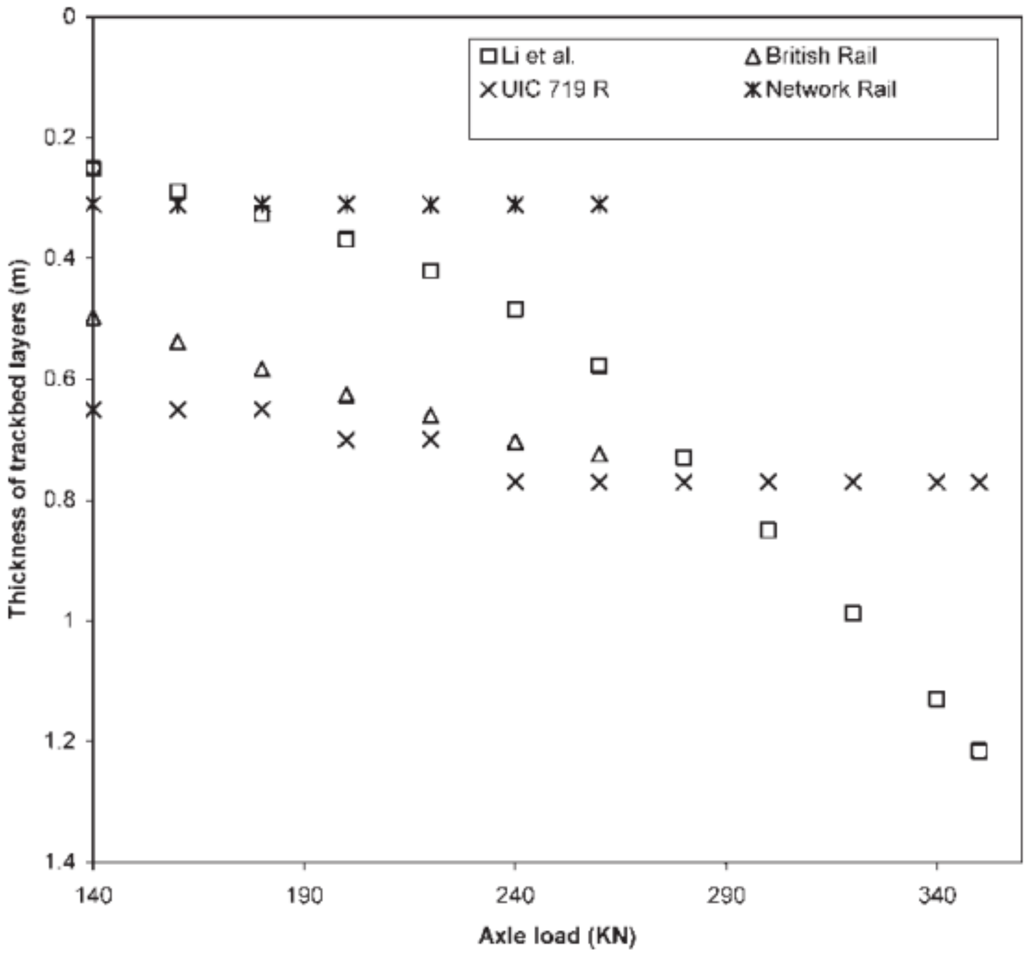


Figure 17. Figure modified from (Burrow et al., 2007)

The analytical methods used in the different design methods simulate the stress distribution in the subsoil differently. As mention earlier, the British Rail code uses a single layer to simulate the substructure, which means that the higher stiffness in the trackbed layer is neglected. Also, the changes in resilient modulus due to loading are neglected. Li et al's model separates the trackbed layer from the subsoil and considers the changes in the resilient modulus due to the loading. The Network Rail code of practise uses an elastic multilayer theory, but there is no further technical information given. Therefore it is difficult to track the reason why the thickness varies from the other methods; if it is due to the model or other parameters. The UIC 719 R method is believed to be based on empiricism (Burrow et al., 2007).

Speed

The dynamic effect is considered in the Li et al's method using the AREA formula (Eq. 1). However, this formula is based on empiricism and is believed to overestimate the influence of the dynamic increment at high speed, which leads to an overestimated thickness of the trackbed layer (Burrow et al., 2007). The variation of the trackbed thickness varies between 0.25-0.7 m for a speed of 80 km/h and 0.57-0.88 m at 350 km/h for the different design approaches (Fig. 18).

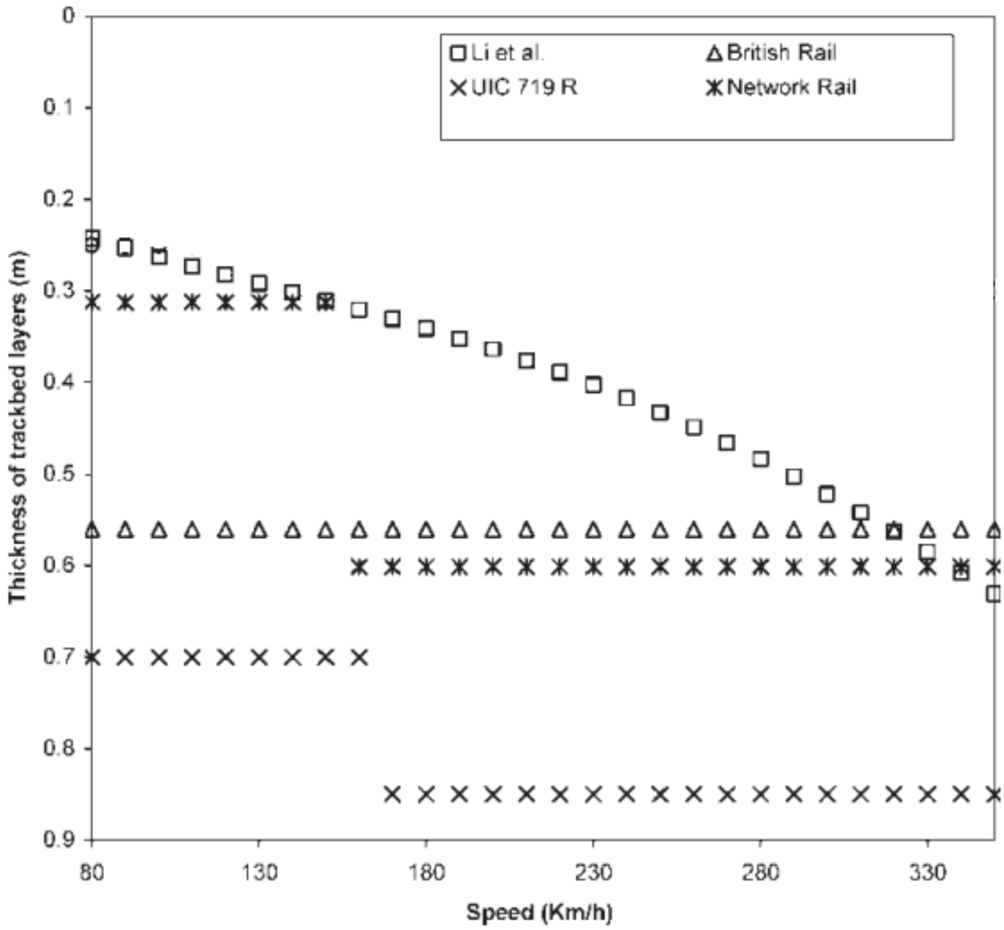


Figure 18. Figure modified from (Burrow et al., 2007)

In the British Rail method, the speed is not a design parameter. Network Rail code standard states that high-speed line requires stiffer sleeper support and therefore also thicker trackbed layers. UIC 719 R takes the speed in consideration with the UIC 714.

Cumulative tonnage

Different lines may have different cumulative tonnage even though the maximum single axle load is the same. If only the single axle load is taken under account, a line with 100 MGT may be designed the same as a line with 10 MGT if the maximum axle load is similar. The only methods that consider the cumulative tonnage is the Li et al's method and the UIC 719 R. Li et al's method convert the assumed variation of loads during the design life into one design load. The variation of the trackbed thickness varies between 0.3-0.85 m for cumulative tonnage of 30 MGT and 0.25-0.85 m for 900 MGT for the different design approaches (Fig. 19).

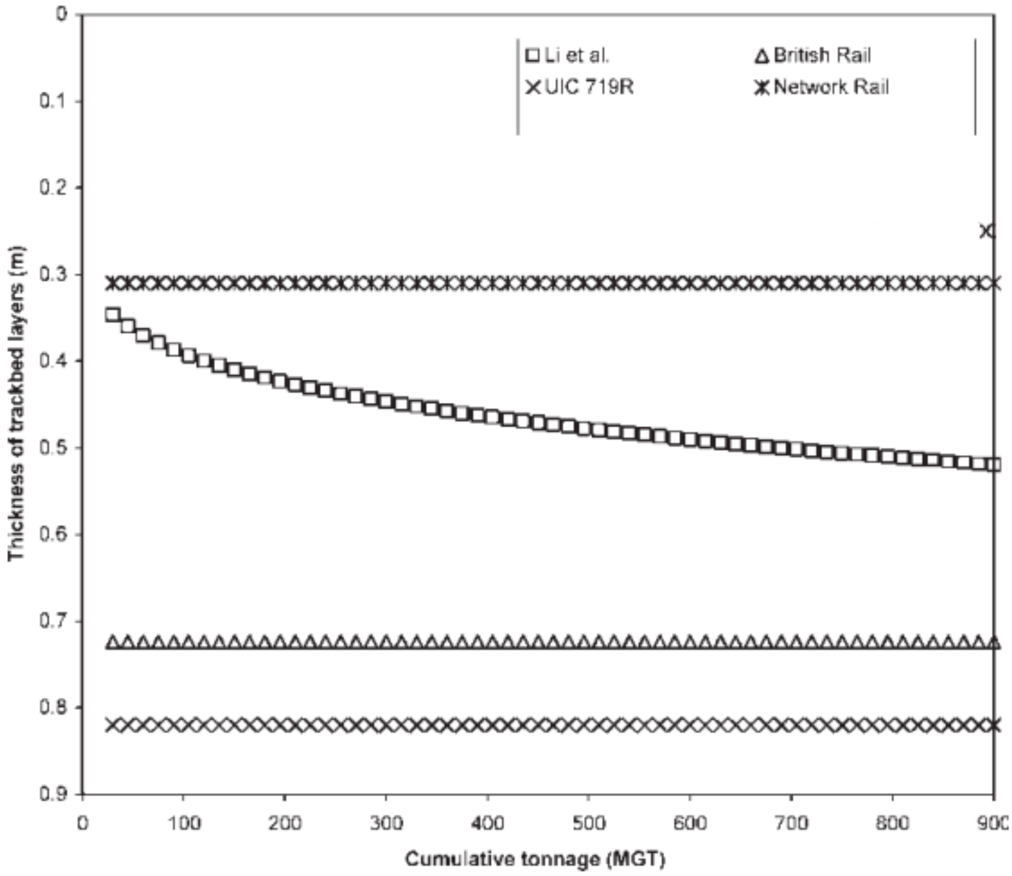


Figure 19. Figure modified from (Burrow et al., 2007)

4 Results/ Discussion

Chapter 5 is divided into 5 subchapters, each chapter presents results from Sweden, Germany, France and Spain. At the end of each subchapter the results are discussed. To be able to compare with the foreign regulations, the demands relevant for this thesis are briefly presented in section 5.1-55.

Sweden
The Swedish transportation administration has developed technical requirements regarding design, construction, establishment and maintenance of the planned high-speed railway (Ostlänken) in Sweden (Swedish Transport Administration, 2016). The requirements states that it is important that:

- The freight traffic shall not effect the punctuality and durability of the passenger train.
- Few stations along the line and also few connections to the conventional railway line, in order to keep a high average speed
- The track shall be designed ballastless for speed > 200 km/h and for speed < 200k m/h the track may be designed ballasted.

Foreign regulation
Germany, Spain and France have developed high-speed railways for a long time (Swedish Transport Administration, 2011: Smekal, 2012: Swedish Transport Administration, 2012). Each country has different priorities and has focused on different aspects during the constructing and as a result developed knowledge in different areas. Germany has constructed high-speed railways since 1972 and hence has learned to master the entire process. Furthermore, Deutsche Bahn is responsible for both the construction and also, the train traffic operating the railway. Germany has an integrated system with a lot of stations (the distance between two stations is usually less than 100 km). The philosophy behind the railway system is that it shall be easy travelling between point A and B; a “door-to-door” system. Moreover, Germany has been in the forefront developing ballastless high-speed railways and has constructed two ballastless lines: Stuttgart-Ulm and Frankfurt Main-Köln. For speed over 250 km/h, Germany has selected ballastless track solution, because they believe that it is the most economical solution. Spain and France has separated freight traffic from conventional traffic and their high-speed railway is constructed ballasted. Both France and Spain has decided to construct a high-speed network separated from the freight train network and Spain have Europe’s longest high-speed network. The main focus in Spain is to have a high average speed with few stations, and as a result short travel time between the end-stations. These different priorities between the countries result in different design parameters. For example a railway track dimensioned to carry freight train traffic must have a lower tilt and be dimensioned for heavier trains. Shorter distance between stations means that the top speed is already reached, and thus makes it pointless and uneconomical to design the railway for higher speeds.

4.1 Design parameters

The design lifespan, load and speed for each country is presented in Table 8-19 below.

4.1.1 Sweden

Table 8. Design lifespan in Sweden (Swedish Transport Administration, 2016)

Construction	Design lifespan
Slab track [years]	60
Geotechnical structures [years]	80
Bridges [years]	120
Supporting structures, i.e piles etc. [years]	120

Table 9. Design load in Sweden (Swedish Transport Administration, 2016)

Type of load	Rail	Geotechnical structures
Axle load [t]	17	25
Distributed load [t/m]	8	8

Table 10. Design speed in Sweden (Swedish Transport Administration, 2016)

	Design speed
Dimensioned speed [km/h]	320

4.1.2 Germany

Table 11. Design lifespan in Germany ((Swedish Transport Administration, 2012)

Construction	Design lifespan
Ballasted track [years]	30-40
Slab track [years]	60
Geotechnical constructions [years]	100
Bridges [years]	80
Tunnels [years]	100

Table 12. Design load in Germany ((Swedish Transport Administration, 2012)

Type of load	Design load
Axle load [t]	22.5
Distributed load [t/m]	8

Table 13. Design speed in Germany from ((Swedish Transport Administration, 2012)

	Design speed
Dimensioned speed [km/h]	300

4.1.3 France

Table 14. Design lifespan in France ((Swedish Transport Administration, 2011)

Construction	Design lifespan
Ballasted track [years]	-
Slab track [years]	-[a]
Geotechnical constructions [years]	100
Bridges [years]	100
Tunnels [years]	100

[a] According to Robertsson, Masson etc. (Robertson et al., 2015), The ambition with France ballastless high-speed track, NBT, is a design life of at least 100 years.

Table 15. Design load in France ((Swedish Transport Administration, 2011)

Type of load	Design load
Axle load [t]	17[a]
Distributed load [t/m]	-

[a]The ambition with the France NBT, is a design load up to 25 t.

Table 16. Design speed in France ((Swedish Transport Administration, 2011)

	Design speed
Dimensioned speed [km/h]	-[a]

[a]The goal speed at the NBS constructed in France is 360km/h.

4.1.4 Spain

Table 17. Design lifespan in Spain (Smekal, 2012)

Construction	Design lifespan
Ballasted track [years]	40
Slab track [years]	-
Geotechnical constructions [years]	100
Bridges [years]	100
Tunnels [years]	100

Table 18. Design load in Spain (Smekal, 2012)

Type of load	Design load
Axle load [t]	30
Distributed load [t/m]	8

Table 19. Design speed in Spain (Smekal, 2012)

	Design speed
Dimensioned speed [km/h]	350

4.1.5 Discussion

Spain has the highest design speed, however, the highest operated speed today is 300 km/h. The reason why they choose to design for 350 km/h is so that they have the possibility to increase the speed of the trains in the future. Germany has a design speed of 300 km/h and has no interest to increase the speed in the future, because the distance between the stations is too small (100 km approximate). The design life does not vary significant between the countries.

The design load in Spain is significantly higher than the design loads in France and Germany. However, the maximum axle load in Spain is 17 t. The reason for the high design load is due good subgrade and as a result will the total cost not be so much higher than for a lower design load. Furthermore, Spain constructs their railway so that it will be possible for freight trains to operate, even though that is not the case today. Germany construct their railway for mixed traffic, freight train shall be able to operate the railway at night. France has the lowest design load, the main reason to that is that no freight traffic is operated at the railway.

4.2 Deformations

The origin of the settlement requirement in each country is unclear. It is interesting to notice that Germany is the only country, of the ones studied in this Thesis that uses formulas to calculate the settlements.

4.2.1 Sweden

The settlement requirements for the substructure and subgrade must be further evaluated according to the current system standard (Swedish Transport Administration, 2016)

4.2.2 Germany

The following regulation may be used regarding settlement deformation. (DB Netze AG, 2008, 2014). The settlement restrictions only concern the geotechnical parameters and the driving dynamics. Not to the technical limitations of rail height adjustability.

Settlement requirements for ballastless embankments
Relative limitations: Differential settlements must be limited to that extent that the track at no point exceeds 1/500 from the reference length.

Total limitations: The settlements are generally allowed to be 15 mm for a designed speed of 300 km/h. If the subgrade only consists of soil longer than 100 m, under the condition that the relative limitations for settlement still apply, 30 mm is allowed.

The allowable settlements after construction can in exceptional cases be extended to 60 mm. However, the driving dynamic parameter, R_a, for the track must still be fulfilled. The extended settlement limitation is only valid for embankments designed according to standard, with a height over 10 m and long transitional zones onto stable grounds. The limitations of the driving dynamic parameter can be defined as:

$$R_a \geq 0.4 \cdot (v_d)^2 \qquad (12)$$

Where,

R_a - Vertical curve radius
v_d - Design speed

The coefficient can be reduced to 0.25, under the condition that relevant authorities approves.

The previously described limitations are only valid if the constructed earthwork has been carried out to given specifications. Furthermore, the combined settlements, caused by traffic and the embankments own weight, shall not exceed 10 mm and the earthwork construction and foundation must be controlled due to vibrations caused by the train.

Settlement limitation at transition zones
The differential settlements relative to a fixed track structure between the back wall of the abutment and a given point 30 m from the back wall shall not exceed 20 mm. The gradient of the top of the frost protection layer due to settlement shall not exceed 1:1000.

Settlement requirements for ballasted embankments
Relative limitations: The acceptable differential settlements for the foundation of newly constructed or essential improved embankments within a certain maintenance cycle (usually 6-10 years) are shown in Figure 20.[3]

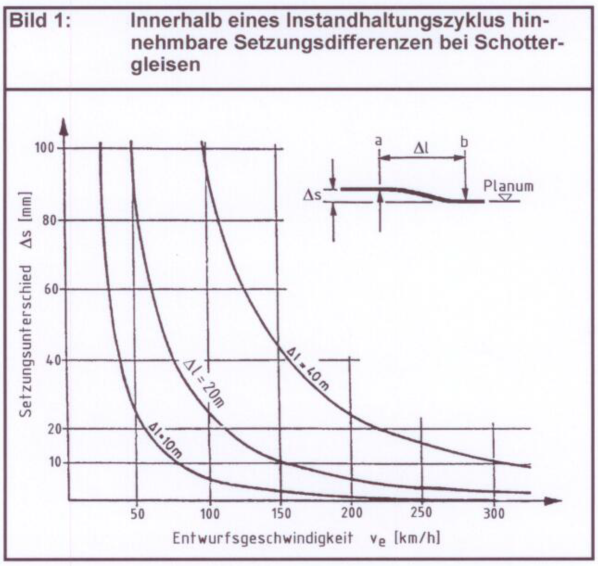


Figure 20. Requirements for the differential settlements. On the x-axis shows the design speed [km/h] and on the y-axis shows acceptable differential settlements [mm].

Total limitations: 30 mm settlements are accepted. Furthermore, the total settlements shall not exceed 3 times the value of the differential settlements after commissioning within a reference length of 40 m.

[3] This diagram is only adequate for ballasted embankments since slab track can not be tamped up or down to achieve an acceptable vertical alignment again (Heer, 2016).

4.2.3 France

Settlement

Total limitations: 100 mm settlements from the top of the rails down to the solid rock, during the track design life (100 years) (Réseau ferré de France, 2010). Settlements to this extent are possible to correct by adding more ballast to correct the thickness of the layer back to its origin position. Furthermore, the settlement must be limited to a maximum of 10 mm/year and is only allowed the first 25 years, thereafter no additionally settlements are allowed.

Relative limitations: The longitudinal differential settlements must be limited to 3 mm between two sections that are located 15 m apart (1:5000). If the sections are 30 m apart the differential settlements shall not be more than 8 mm (1:1000). Furthermore, the lateral differential settlements (i.e between the rails) must not differ more than 3 mm at a 3 m long section (1:3750).

Swelling

Swelling soils on high-speed lines must be avoided and can solely be tolerated in special cases and must then be limited to 10 mm/year.
Some soils are likely to increase in volume, which leads to irregularities in the track. The three main types are:

- Swelling clay due to water absorption.
- Materials that consists of more or less rocky clay or very over-consolidated material that reorganise after unloading
- Soils containing particles (e.g. sulphates, nitrates, ...) and swells under due to chemical treatment (e.g. lime or hydraulic binders) and thus create new mineral species.

4.2.4 Spain

Almost no settlements due to consolidation are accepted and, therefore, there is no stated time depended limitations (Smekal, 2012). However, settlements that does not exceed 50 mm 100 days after the construction of the superstructure is completed may be accepted. Under the condition that the subgrade consists of clayey soils and appropriate drainage is installed. The settlements must than be controlled and evaluated (Tab. 20). For example, if the topography controlled settlements is > 4 mm/month, the risk is considered to be high and therefore, the controls per year must be 6.

Table 20. Control of settlements (Smekal, 2012)

	Topography controlled settlements [mm/month]	Risk	N° of controls per year [n°/year]
Embankments and technical blocks	> 4	High	6
	Between 1-4	Medium	4
	< 1	Low	2
	Approximately 0	None	General

4.2.5 Discussion

The origin of the settlement requirements is rather unclear, it is hard to determine on what they are based. If possible, it would be interesting to clarify why they differ. Looking at the ballasted settlement requirements, the French are based on how much ballast it is possible to add to the ballast layer after the construction is finished. Spain has no stated requirements; the project manager herself may in principle set a limit. Germany has settlement requirements regarding both total limitation as well as relative limitation. For the ballastless requirements, Germany has strict and regulated settlement demands. The parameters that influence the total limitation are: dimensioned train speed, embankment height and vertical curve radius.

4.3 Classification systems

The reason why it is interesting to compare the different classification systems is that different countries classify the trackbed material differently. Therefore, the following chapter presents and discuss the different classification systems.

4.3.1 Sweden

The material is divided into different material type as shown in Table 21 and is classified according to sieve analysis. The soil susceptible classes is divided from 1 to 4, were 1 is non susceptible and 4 is very susceptible. The backfill shall be made with material from class 1 or 2 (Swedish Building Services, 2014).

Table 21. Swedish soil classification system (Swedish Transport Administration, 2014)

Material type	Rock type	Ball mill value	The content of [weight-%] x/y[a] Fine soil 0.063/63 mm	The content of [weight-%] x/y[a] Clay 0.002/0.063 mm	The content of [weight-%] x/y[a] Organic soil[c] %/63 mm	Example of soils[b]	Soil susceptible classes
1	1	≤ 18	< 10		≤ 2		1
	2	19-30					
2			≤ 15		≤ 2	Bo, Co, Gr, Sa, saGr, grSa, GrTi, SaTi	1
3A	3	>30	≤ 30		≤ 2		2
3B			16-30		≤ 2	siSa, siGr, Ti	2
4A			31-40		≤ 2	clTi, siTi	3
4B			> 40	> 40	≤ 2	Cl, ClTi,	3
5A			> 40	≤ 40	≤ 2	Si, clSi, siCl, SiTi	4
5B					3-6	gyCl, gySi	4
6A					7-20	clGy, siDy	1-4
6B					> 20	Pt, Gy	1
7			Other materials Special investigations			Alternative materials	

[a]The concentrations (x/y) applies to the amount of material passed through the sieve x mm relative to the total amount of material that passed through the sieve y mm.
[b] The soils nominations are in accordance with the European standard.
[c]Organic content shall be determined according to SS 271 07 (SIS)

The materials, design and inspection are decided based on the "AMA Anläggning" (English translation: AMA Construction) and "TRVAMA Anläggning" (English translation: TRVAMA Construction).

4.3.2 Germany

German soils are classified according to DIN 18 196 (Tab. 22). Each layers in the embankment has specific requirements regarding the bearing capacity, compaction and soil type (Fig. 21 and 22 and Tab. 23 and 24) (SSF Ingenieure, 2010). Measures such as subgrade treatment, drainage and preloading must be taken if the requirements can not be fulfilled by compaction or if soil of the requested type is not available. Complete soil replacement may also be an alternative, if the bearing capacity of the existing soil is insufficient.

Table 22. Soil classification system, according to DIN 18 196 (ICP mbH, 2009)

Main group	**Grain size table [weight-%]**		**Group (general)**	**Group (Specific)**	**Symbol**
	***d*≤0.06 mm**	***d*>2.00 mm**			
Coarse-grained soil	≤5	>40	Gravel	Poorly graded gravel	GE
				Well-graded gravel-sand mixtures	GW
				Gap graded gravel-sand	GI
		≤40	Sand	Poorly graded Sand	SE
				Well-graded Sand-gravel mixtures	SW
				Gap graded Sand-gravel	SI
Mixed grained soil	5-40	> 40	Gravely silt	5 to 15 weight-% ≤ 0,060 mm	GU
				15 to 40 weight-% ≤ 0,060 mm	GU*
			Gravely clay	5 to 15 weight-% ≤ 0,060 mm	GT
				15 to 40 weight-% ≤ 0,060 mm	GT*
		≤40	Sandy-silt	5 to 15 weight-% ≤ 0,060 mm	SU
				15 to 40 weight-% ≤ 0,060 mm	SU*
			Sandy-clay	5 to 15 weight % ≤ 0,060 mm	ST
				15 to 40 weight % ≤ 0,060 mm	ST*
Fine grained soil	> 40		Silt	Light plastic silts ($W_L \leq 35$)	UL
			Clay	Light plastic clays ($W_L \leq 35$)	TL

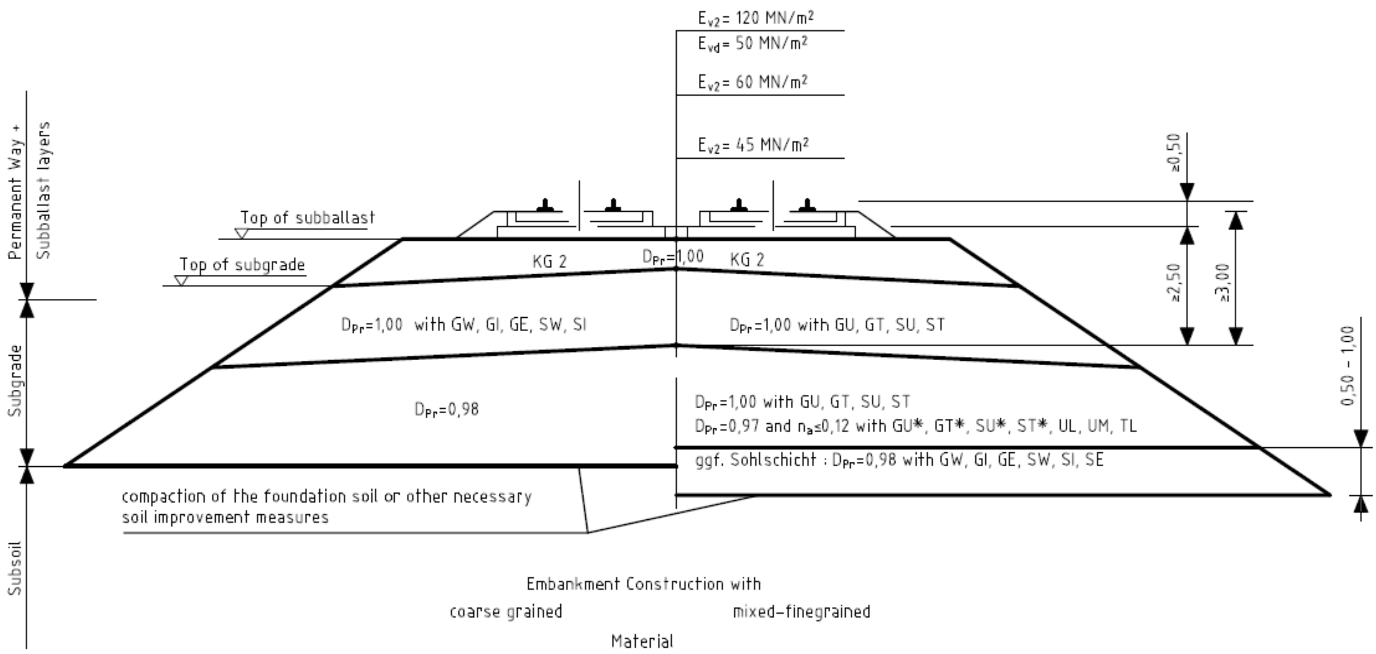


Figure 21. General German design principle for a ballastless high-speed embankment (Swedish Transport Administration, 2012)

Table 23. Material, bearing resistance and compaction ratio for ballastless embankments (Swedish Transport Administration, 2012)

Layer	KG	E_{v2} [MPa]	E_{vd} [MPa]	D_{pr} Course grained soil	D_{pr} Mixed-fine grained soil
Hydraulic bonded layer: HBL	-	-	-		-
Frost protection lager: FPL	2	120	50	1.00	1.00
Subgrade material > 3,00 m	-	60	-	1.00 GW, GI, GE, SW, Si	1.00 GU, GT, SU, ST
Subgrade material	-	45	-	0.98	1.00 GU, GT, SU, ST 0.97 and na <0.12 GU*, GT*, SU*, ST*,UL, UM, TL
Subgrade material if necessary improvement of surface of subsoil	-	-	-		0.98 GW, GI, GE, SW, SI, SE

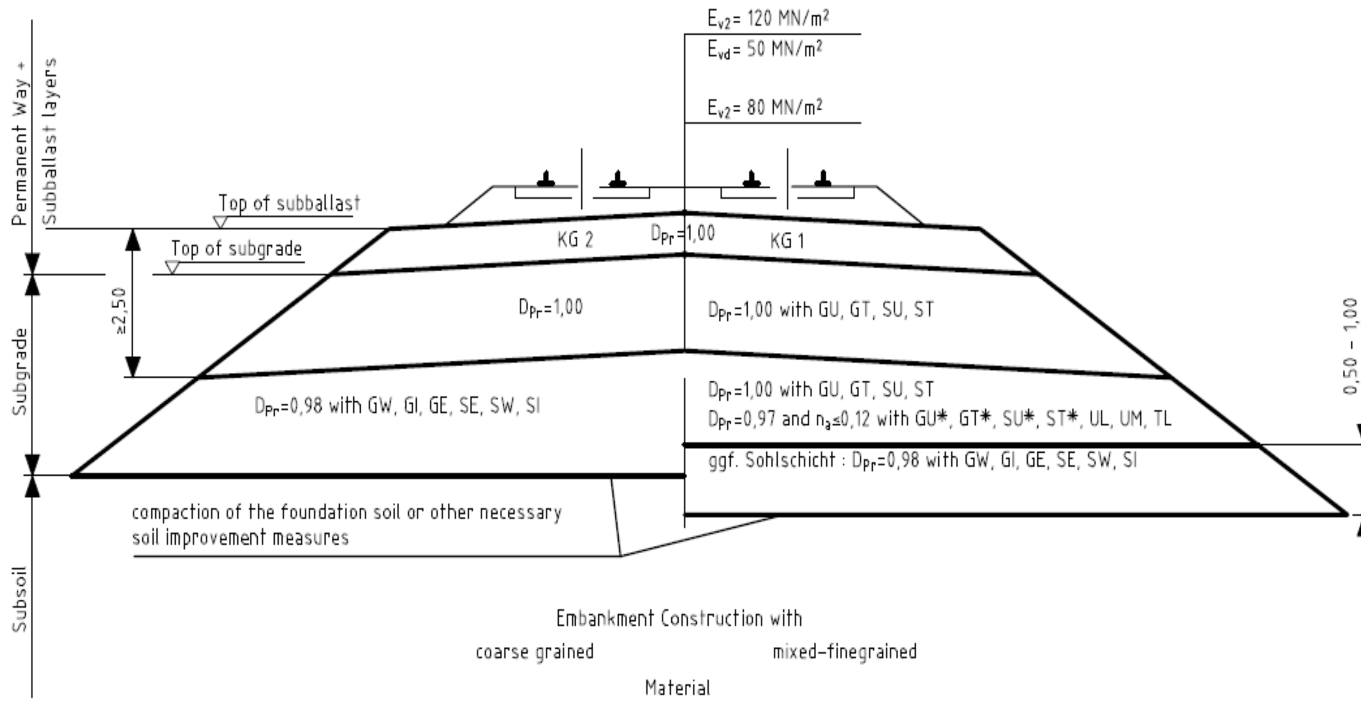


Figure 22. General German design principle for a ballasted high-speed embankment (Swedish Transport Administration, 2012)

Table 24. Material, bearing resistance and compaction ratio for ballasted embankments (Swedish Transport Administration, 2014)

Layer	KG		E_{v2} [MPa]	E_{vd} [MPa]	D_{pr} Course grained soil	D_{pr} Mixed-fine grained soil
	Course grained soil	Mixed-fine grained soil				
Ballast			-	-		-
Subballast[a]	2	1	120	50	1.00	1.00
Subgrade material > 3,00 m			80	-	1.00	1.00 GU, GT, SU, ST
Subgrade material				-	0.98	1.00 GU, GT, SU, ST 0.97 and na <0.12 GU*, GT*, SU*, ST*, UL, UM, TL
Subgrade material if necessary improvement of surface of subsoil			-	-		0.98 GW, GI, GE, SW, SI, SE

[a] Percenteges of fines shall be < 5.7%

4.3.3 France

Soils with organic content exceeding 3% are graded separately Class F with industrial by-products.

The material that is used in the embankment and in the formation layer (Fig 26) is classified according to GTR (Corte et al., 2000: Réseau ferré de France, 2010). And they are classified into three different main classes:

- Soils (class A, B, C and D),
- Rocky material (class R) and
- Organic soil and industrial products (class F)

The soils are classified with laboratory tests based on three parameters: Nature (classified according to their particle size and clay content), mechanical behaviour (classified according to resistance to fragmentation, wear resistance and friability) and the water status (classified into five states: very dry (ts), dry (s), average state (m), wet (h) and very wet (th)).

The soils are divided into four subclasses, based on the mentioned laboratory tests:

- Class A: Fine soils (e.g.: silt, marl, clay, clay loam, sandy clay, silty clay)
- Class B: Sandy and gravel material (e.g.: sand loam, silty serious, clayey sands, clay gravel)
- Class C: Soils with fine and large elements. (e.g.: scree, alluvium)
- Class D: Soils water insensitive (e.g.: scree, alluvium)

An example of a soil classification table of soil type A may be seen in Appendix D.

Compaction

The compaction level in France are classified into four different classes: q1, q2, q3 and q4. These classes are based on the mean density in the compacted layer, ρ_{dm} and the density at the bottom layer, ρ_{dfc}, (the average value on a segment 8 cm thick located in the bottom of the compacted layer). The standard requirement for the compaction level in the formation layer and the embankment is symbolized q3, respectively q4 and are defined as Figure 23.

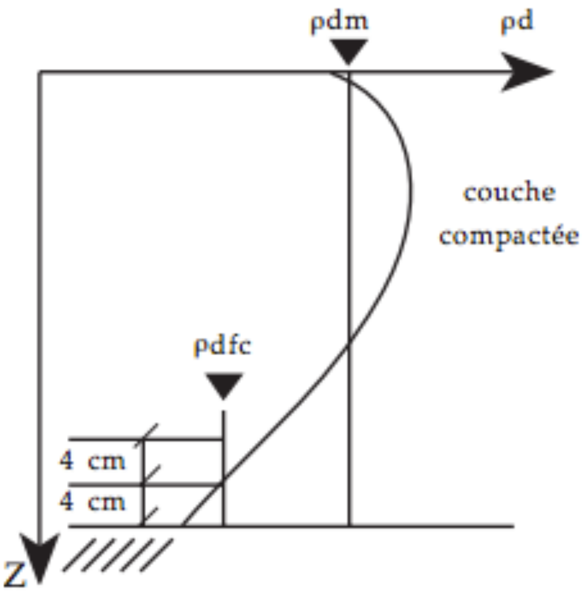


Figure 23. Variation in dry density on the height of the compacted layer.

q3:
$\rho_{dm} \geq 98.5\%$ ρdOPN and,
$\rho_{dfc} \geq 96\%$ ρdOPN

q4:
$\rho_{dm} \geq 95\%$ ρdOPN and,
$\rho_{dfc} \geq 92\%$ ρdOPN

Where,
ρdOPN are the optimum dry density determined by a Standard Proctor test.

4.3.4 Spain

The Spanish regulation was not possible to study for this thesis, therefore there is no information regarding the Spanish soil classification.

4.3.5 Discussion

There are several different ways to classify the soil, and the methods vary from one country to another. Germany classifies the soil according to DIN 18 196, Spain classifies it according to ADIF or UIC 719-R, and the French classification system is called GTR. Table 32 in appendix D links the GTR system to UIC 719- R. The Swedish and German soil classification system is primarily based on grain size distribution. In the GTR system, the soil is classified primarily based on the water content, particle size distribution and mechanical parameters.

Different European countries classify the ballast material differently as well. Sweden and Germany classify the ballast according to their size using sieve analysis, while the classification in France not only uses sieve analysis, but also mechanical behaviour and water status. Even though Germany and Sweden both use sieve analysis, their classifications still disagrees as can be seen in table 21 and 22. For instance, they use different percentage of weights of fine materials and different group names.

This is very important to remember, because if we in Sweden want to use empirical data from Germany or France, we must work under the same conditions. If Germany says they have used gravel, that might not be what we call gravel in Sweden; we must avoid making preventable mistakes due to unfortunate terminology.

Another thing worth consideration is the French soil qualification system. As briefly mentioned in the result, it is more complex than the Swedish and German qualification systems. Indeed, it takes a lot of parameters into account. Naively, one would think that such a thorough qualification of the soil should lead to them using a more suitable soil in the sublayer, what could else be the reason for the complexity if not to increase the suitability of the soil? If indeed a more suitable soil is used, it ought to be more efficient and hence less of the soil should be needed. This would then in turn reduce the cost. One must remember that such a classification of the soil in itself is time consuming and less transparent, so one has to weigh the possible advantages against the disadvantages.

4.4 Bearing capacity, frost protection and layer thickness

Each of the subchapters in this chapter presents a typical cross-section of the railway embankment in each country. The bearing capacity, layer thickness and frost criterion are also presented, if it was possible for me to get access to them.

4.4.1 Sweden

Figure 24 presents a typical cross-section of a ballastless embankment. The column on the right side of the figure presents which part of the structure that the railway component "belongs to".

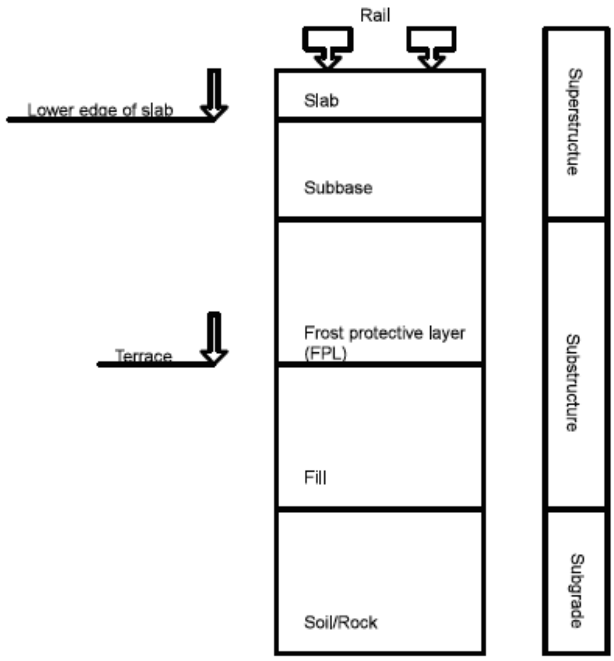


Figure 24. Principal definition of ballastless embankment, with superstructure and substructure.

Bearing capacity

The required bearing capacity (Tab. 25) is stated in the Swedish regulations as follows: (Swedish Transport Administration, 2016)

Table 25. Bearing capacity (Swedish Transport Administration, 2016)

Layer	Bearing capacity (E_{v2}) [MPa]
On top of frost insulation layer	120
Terrace surface	60
Subgrade	45

Frost

The geotechnical constructions shall be frost resistance for at least 100 years and are project depending.

4.4.2 Germany

Figure 25 presents a principal definition of the ballasted embankment and Figure 26 presents a principal definition of a ballastless embankment. The column on the right side of the figure presents which part of the structure that the railway component “belongs to”.

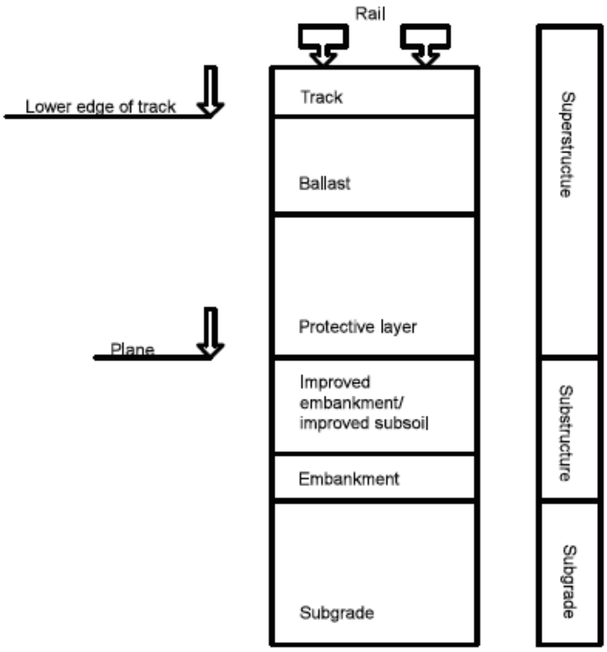


Figure 25. General ballasted track solution

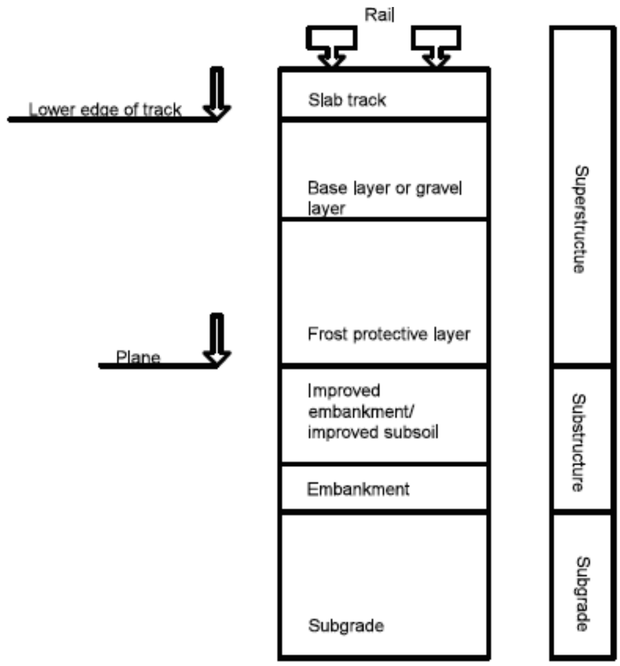


Figure 26. General ballastless track solution

Bearing capacity and layer thickness
An overview of the design criterions for the subgrade and subballast layers are shown in Table 26. The freeze influences zone I,II and III that is referred to in the Table are presented in Appendix D

Table 26. Overview over the design criteria for new constructions the subgrade and subballast layers. (Swedish Transport Administration, 2012).

Type of track		Subballast		Protection layer					Subgrade	
Track class	Permanent way	E_{v2}	E_{vd}	Material	D_{pr}	Normative thickness [cm] Freeze influence zone I	II	III	E_{v2}	E_{ve}
P300[a]	Ballast	120	50	KG 1 / 2	1	70	70	70	80	40/35
	Slab track	120	50	KG 2	1	40	40	40	60	35/30

[a]P300=High-speed transport routes (design speed 300km/h)

Frost
The depth of frost protection layer is based on the exceptional cold winter, 1962-1963 (DB Netze AG. 2014). Frost susceptibility cohesive loose rock of the earthwork must be protected against frost penetration:

- The achievable frost range below the ballast is a frost protective layer.
- If the construction is built under ground water level, measures must be taken to avoid water sipping upwards into the frost protection layer.
- The material in the frost protection layer must follow the Casagrande frost criterion.

The soils are graded in a scale from soils not susceptible to frost (F1) to soils that are very susceptible to frost (F3) (Tab. 27). The soils are graded based on the fine content and the coefficient of uniformity, U (Fig. 27).

Table 27. Definition of frost classes: F1, F2 and F3 (Netze 2014).

Frost susceptible classes			Fine content <0.063 mm	Safety of frost, fine content for
F1	Soils not susceptible to frost	U>15	< 5	5
F2	Soils susceptible to frost	U < 5	5 - 40	15
F3	Soils that are very susceptible to frost	U < 5	> 40	-

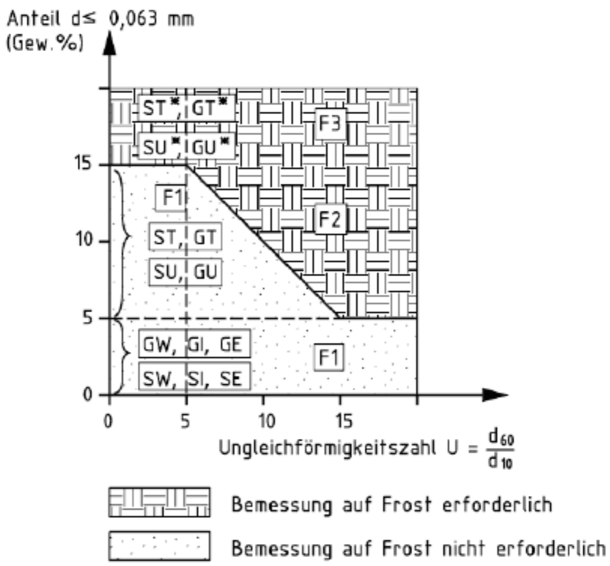


Figure 27. Classification of frost susceptibility as a function of percentage fine-grain in the material, d, and coefficient of uniformity, U. The doted area shows the materials were frost dimensioning is not required and the hatched area shows the material were frost dimensioning is required. (DB Netze AG, 2014)

4.4.3 France

Figure 28 presents a principal definition of the ballasted embankment.

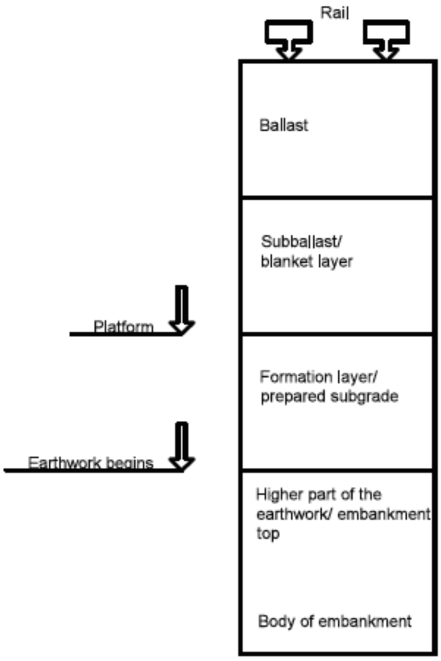


Figure 28. Definition according to the French regulations

The following requirements solely apply for ballasted track system (Réseau ferré de France, 2010). The requirements regarding the construction and the underground considering high-speed railway embankments were developed by several analyses. The durability criterion is considered to be fulfilled, if the railway structure is constructed accordant to the requirements and is regularly maintained.

Bearing capacity and layer thickness

The soils are classified according to their bearing capacity, assessed by plate loading tests (Tab. 28). The layer thickness and material is determined based on the soil class (Tab 29).

Table 28. Soils classified according to their bearing capacity

Class	Denomination	E_{v2} modulus [MPa]
S0	Must be improved to serve as a stable subgrade	Not applicable
S1	Soil quality "fair"	30
S2	Soil quality "good"	50
S3	Soil quality "very good"	80

Table 29. Structure type

Class	Geotextile	Formation layer thickness [cm]	Material formation layer	Subballast thickness [cm]	Ballast thickness
S1		50	Granular material	20	The dimensions of this table agrees with concrete sleepers and ballast under sleepers 0.35 m
S1		35	Treated granular material, class 4*	20	
S1		40	Treated granular material, class 5*	20	
S2	x	-	Integrated 30 cm	35	
S2		35	Granular material	20	
S3		-	Integrated 30 cm	20	

* The formation layer can be executed in granular materials (class B and D, from GTR) hydraulically bound optionally combined with lime. The treated material must be frost-resistant and take Rt = 0,8.Rtb

France constructs their railway structures ballasted. In the Table below (Tab. 30), bearing capacity, material size and layer thickness are presented. If the subsoil is poor and needs to be improved or reinforced, the blanket layers bearing capacity must be higher than if the soil quality is good.

Table 30. Required bearing capacity of natural soil

Layer	Bearing capacity (E_{v2}) [MPa]	Compaction ratio	Material size [mm]	Nature	Layer thickness [cm]	Comments
Ballast	120					
Subballast	80	100% Compared to modified Proctor	0/31.5	Usually untreated granular material or as an alternative a bituminous layer	20-35	Percentage of fines < 4-8 % < 40 (LA + M_{DE}) for v > 160 km/h < 50 (LA + M_{DE}) for v < 160 km/h
Formation layer	45/120[c]	q3 (as described before)	0/125	Treated[a] or untreated[b] material	35-50	Visual inspections supplemented with appropriate control of bearing capacity are always made after freeze-thaw period.
Upper part of embankment	30	q4 (as described before)		If the soil is classified into soil class S0 it must always be improved to at least soil class S1 or replaced by a suitable material	The improvements shall be made of suitable thickness, minimum of 35 cm	The bearing capacity improvement shall be sustainable (according to GTR or GTS) The improved bearing capacity shall be controlled with recognized methods. (e.g. Plate loading test)

[a] Untreated material may include a geotextile.

[b] The treated material is evaluated with approved geotechnical investigations (sample is taken every 250 m, and 95% of the samples must be of approved mechanical class). The treated subgrade is protected with a coating adapted to the execution day.

[c] If the soil quality is good the bearing capacity from a plate loading test shall be 45 MPa and if the soil must be reinforced the bearing capacity must be 120 MPa

Frost
There is strict regulations regarding frost protection; the entire rail structure must be protected against frost during the entire design life. A component of the railway may either be protected against frost based on the current winter or an exceptional winter, depending on the environment and how sensitive that part of the structure is.

The subballast must consist of frost-resistance materials. If the railway is constructed in areas were exceptional winter is possible the formation layer must consist of frost resistance material. The formation layer and the embankment, if it is not frost-free, shall not be affected by the depth corresponding to an exceptional winter.

4.4.4 Spain
Figure 29 presents a typical cross-section of a ballasted embankment.

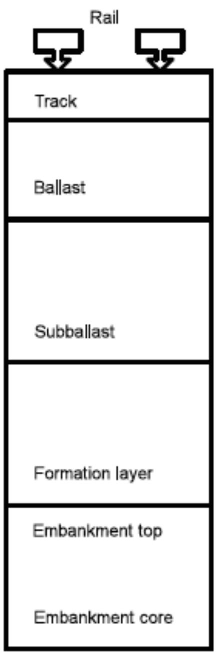


Figure 29. Definition according to UIC 719-R

Bearing capacity and layer thickness
The design praxis during the construction of railway is partly based on ADIF and partly based on UIC 719. The ADIF demands are stricter, except for the E-modulus criteria (Tab. 31) (Smekal, 2012).

Table 31. Spanish requirements regarding thickness of each layer, modulus and also compaction rate (Smekal, 2012).

Layer	UIC 719-R			ADIF		
	e [m]	CBR	GC and E	e [m]	CBR	GC and E
Trackbed layer	0.45	-	GC> 103% PN; E> 120 MPa	0.65		GC> 100% PM; E> 120 MPa
Formation layer	0.35	10-17	GC> 100% PN; E> 80 MPa	0.40	10	GC> 95% PM; E> 80 MPa
Embankment top	-	-	-	-	5	GC> 95% PM; E> 60 MPa
Embankment core	-	5	GC> 95% PN; E> 45 MPa	-	5	GC> 95% PM; E> 30 MPa

Frost

The high-speed lines are placed at areas where no frost dimension is necessary.

4.4.5 Discussion

One reason why it is difficult to compare the codes from different nations is that they partly origin in empiricism. Section 3.3 Trackbed layer shows different approaches when determining the trackbed thickness. These design methods were developed for ballasted embankments. However, the significant variation of trackbed thicknesses indicates that the best way, or rubust, of determining the thickness trackbed in still unknown. Ballasted embankments have been constructed for a long time and if there is reason to question the design methods, there are reasons to believe that is the case during the ballastless constructing as well.

The design praxis in Spain is partly based on ADIF and partly on UIC-719-R. Table 31 shows that, when comparing ADIF to UIC 719-R, the ADIF suggests thicker layers and higher compaction ratio. The UIC 710-R suggest that the trackbed layer should have a thickness 20 cm greater than ADIF and the formation layer shall be 5 cm thicker. Looking back at Figure 14 and according to analysis by Li et al (Li, 1994,1998), the trackbed thickness has a significant influence of the stresses that are spread to the subgrade. Figure 15a shows a trackbed onto a soft subgrade, the graph that represents a trackbed layer of 30 cm shows that the deviator stress at the top of the subgrade is ca 90 MPa and with a trackbed thickness of 46 cm, the deviators stresses in about 70 MPa. This shows that it is important to construct railway with an appropriate thickness, if the trackbed is under dimensioned it might lead to too high stress levels which will cause deformation and irregularities in the railway, and as a result, it will lead to high maintenance cost.

Building a railway embankment it is important that one gets the dimensions right. Therefore, one must choose which method to build after with care. Looking back at Figures 17-19 we see that different design parameters, e.g: train speed or axle load, changes the required thickness needed of the trackbed layer. The thickness is important. If the trackbed is too thick it will result in a higher cost than necessary. However, if the thickness of the trackbed layer is not adequate, the deviator stress in the subgrade will be too high. This in turn leads to unnecessary maintenance of the tracks. The question is which, if any, of them is best suited for high-speed trains in Sweden? The plot in Figure 17 is made by varying axle load with the other values, as for instance the speed of the train, fixed. The different methods differ with the speed of the train differently so extrapolating the fixed low speed to a high speed will affect the four methods differently. So the risk of under- or over dimensioning the ballast layer will be high.

Ballasted versus ballastless

According to current Swedish system standard shall the high-speed line be constructed with a slab track. Germany has the longest experience in Europe and therefore, it is important to gain knowledge from their structure solutions and consider what the best alternative for the Sweden would be.

Looking back at chapter 2.6 at the two different general ballastless design principles: German design and "the other design". Due to the soft subgrade in Sweden it would be interesting to study the method with bending reinforcement in the top and at the bottom of the slab that shall reduce the need of soil reinforcement. Of course, this must be evaluated due to the higher reinforcement cost.

5 Concluding remarks

First of all, there are some basic factors that explain the difficulties to compare the regulations. For example: the frost hazard, the inherent ground quality, purpose with the railway (mixed traffic, solely passenger traffic, etc.), design parameters (life, axle load, etc.). But there is also other aspects that differ, that is hard to identify. The reason for this, is in my opinion, that it is hard to identify were the regulations origins. For example, it is not stated why the stiffness of the trackbed layer must be of a certain magnitude.

It is clearly important to gather information from different countries based on their qualifications and also, based on the requirements stated in the Swedish system standard ("Teknisk system standard för höghastighetsbanor"). This standard states that the Swedish high-speed railway network will be constructed ballastless, which states that it is important gain knowledge from Germany, which has most experience in the area. Furthermore, the effect from e.g. freight traffic must affect the punctuality and durability at the high-speed line as little as possible. Spain has the most punctual train schedule and, compared to Germany, both France and Spain has separated passenger traffic from freight traffic and significant fewer stops along the line.

Some linguistic complications that I did come across when I did the research for this master thesis is, that some terms and words might be lost in translation, as some native words are translated into the same English word. Another complication might be that railway organisations refer to the same thing but they have different definitions, for example the name of the substructure layers. Both the France and the German regulation states that if the railway is not designed according to the guideline praxis, it must be extensive tests and proves that the other design method work. However, there is no clear definition of what that means. Therefore, it is possible that the definition is not coherent.

6 Further research

This thesis has focused on the high-speed train rail systems of Germany, France and Spain. It has as well studied four different design methods for ballasted tracks, with special focus on Li et al's method and The International Union of Railways' method. In the discussion section we have seen that the question of which country's technique, or what specific design method, we should implement in Sweden is a question that requires great consideration; if not done right it could become unnecessarily expensive. Therefore, further research to find an optimal method could greatly reduce the overall costs.

Further research should focus on trying invoking typical Swedish conditions. What thickness would the different design methods suggest for Swedish soils? How much impact does changing the axle load to an axle load of a typical Swedish train have on the thickness? Swedish soil, ballast material, typical axle load and speeds, traffic frequencies; these are all things that should be taken into account.

Another thing that could be very rewarding is creating a 3D model using Plaxis. Then one could model and examine how each variable affect the strain and shear strength specifically. What parameter has the biggest impact on the strain? This would tell you a lot about how to design the rail.

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Appendix A

Germany has developed a series of different slab track solution, ever since the Rheda design 1970, the German engineers has been in the forefront when it comes to develop new technical slab track solutions (Esveld, 2003). The three most commonly used slab track systems are: the RHEDA 2000, Züblin and the Bögl.

Rheda system

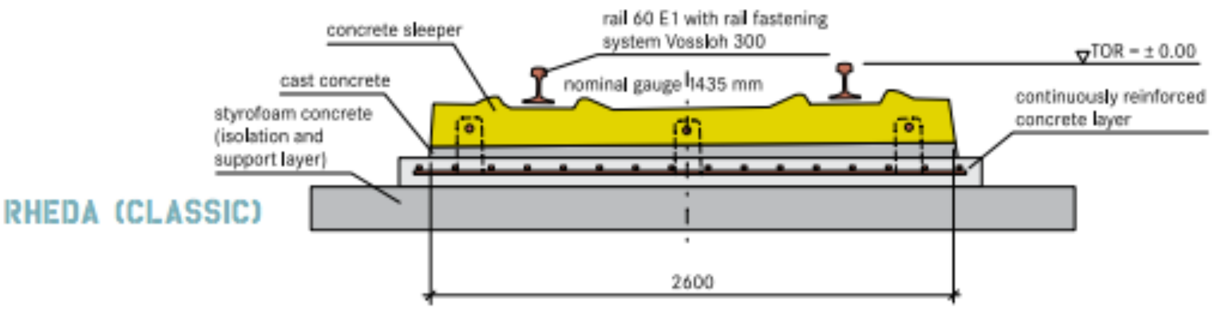


Figure 1. Classic RHEDA design (Rail.one GmbH Pfleiderer track systems, 2011)

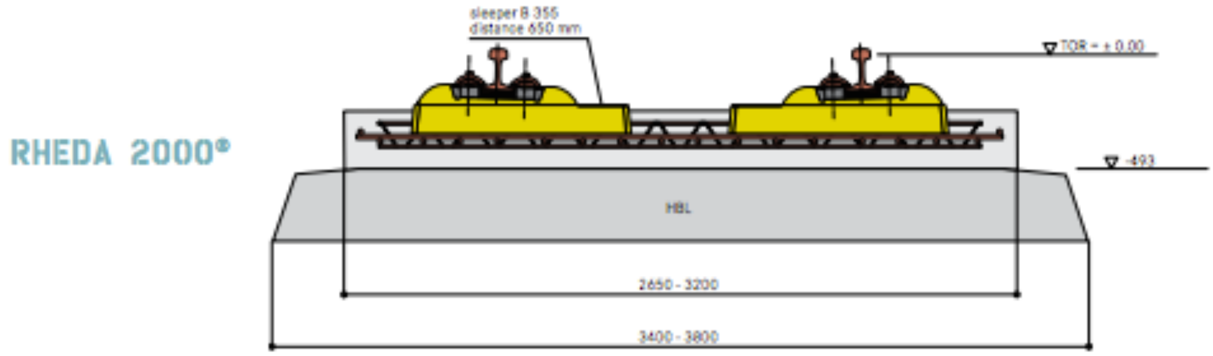


Figure 2. Rheda 2000 (Rail.one GmbH Pfleiderer track systems, 2011)

The Rheda system is named after the station Rheda, located on the line Bielefeld to Hamm that was constructed 1972. The classic Rheda design (Fig. 1) was originally constructed with mono-block sleepers. However, it has been developed through the years and the new design is called Rheda 2000, and is constructed with bi-blocks sleepers (Fig. 2). The Rheda 2000 design was first applied at a section in the Dutch high-speed rail line between Leiozig and Halle.

The Rheda 2000 system is a discrete rail support with modified bi-block sleepers. The sleepers is embedded in a monolithic concrete slab, the minimum strength of the concrete slab must be 30/37 MPa (cube/cylinder). The substructure consists of: hydraulically bonded layer (HBL), frost protection layer (FPL) and subsoil.

The reinforcement bars are placed in the neutral line of the concrete slab, in both lateral and longitudinal direction. The placement of reinforcement helps to control transmits lateral forces and the crack width of the slab rather than provide a stiff slab. Therefore, the superstructure depends exclusively on the soil bearing capacity and stiffness: the superstructure requires a foundation that is free from settlements. If the subgrade is of poor condition one can make the slab stiffer by adding an adequate amount of reinforcement steel in the bottom of the slab (inside the cemented trough).

Züblin system

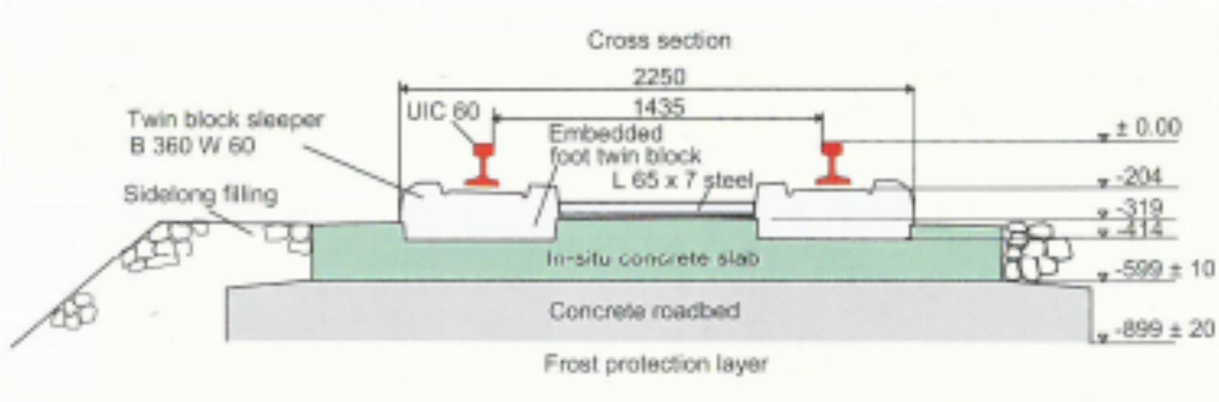


Figure 3. Züblin system

The Züblin design is a discrete rail support with sleepers (Fig. 3). The Züblin design consists of concrete twinblock or monoblock sleepers, which are embedded in a monolithic concrete slab. The concrete slab rests on a concrete roadbed (Hydraulic bonded layer) and under the roadbed there is a frost protection layer.

The longitudinal reinforcement in the Züblin slab is mainly to control the cracks; the goal is to achieve a regular crack pattern and tolerable crack-widths. The reinforcement percentage is normally 0.8 -0.9. This leads to free cracking, the width between the cracks is less than 2m and the width of the cracks is less than 0.3m. The reinforcement steel rods in the Züblin slab is not placed in the neutral line, it is placed nearer the bottom of the slab. As a result, the slab obtains certain flexural stiffness. If the height and the amount of reinforcement are increased, the Züblin slab can be constructed as a rigid slab if the bearing capacity of the soil is poor.

The main different from the Rheda 2000 design is the construction work. The Züblin design aims to separate the individual constructions stages in order to optimize each stage.

In the Züblin design the sleepers are pressed in to the fresh concrete by vibration. This means that the concrete must not allow the sleepers to sink but also, the sleepers must be permitted to be pushed and vibrated into the fresh concrete.

Bögl slab

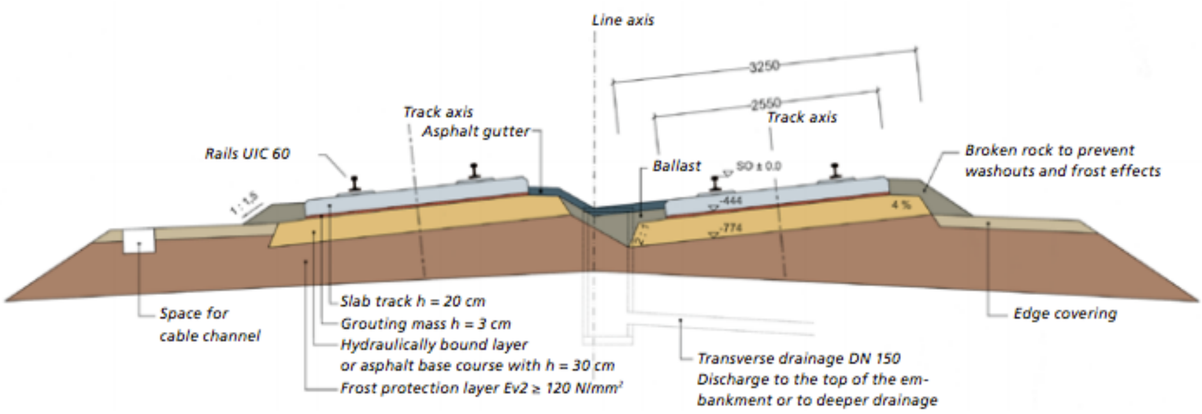


Figure 4. Tyypical cross-section for a Bögl slab track on an earth structure (Max Bögl).

The main difference from the Rheda and the Züblin design is that the Bögl slab track does not have sleepers. The bögl slab track is a discrete rail support without sleepers (Fig. 4). The Bögl slab track was first used at 1977 at Karslfeld, Germany. The Bögl slabs consist of B55 steel fibre reinforced concrete. The reinforcement bars are prestressed in the lateral direction, in the longitudinal direction conventional reinforcement is used. It is easy to adjust the slab due to the spindles, which are integrated in the slab. The Bögl slab is prefabricated and can be used at lines were no bending resistance is needed. Prefabricated slab track can contribute to:

- Higher quality of the track
- High level of mechanisation
- Less work is needed at site hence saves labour at site
- One does not need to wait for the concrete to dry and therefore it can be adjusted and fixed directly
- It is easier to repair

France slab track system

There is a on going development of a slab track system in France, named "New Ballastless Track" (NBT) (Fig. 5) (Robertson et al., 2015). The design principle is primarily based on knowledge from roads and airport pavements. The goal is a high design life, 100 years. To be able to achieve that, experience from structural engineering also influenced the design. The research that Robertson etc. performed showed that the thermal loading has a significant influence of the slab performance.

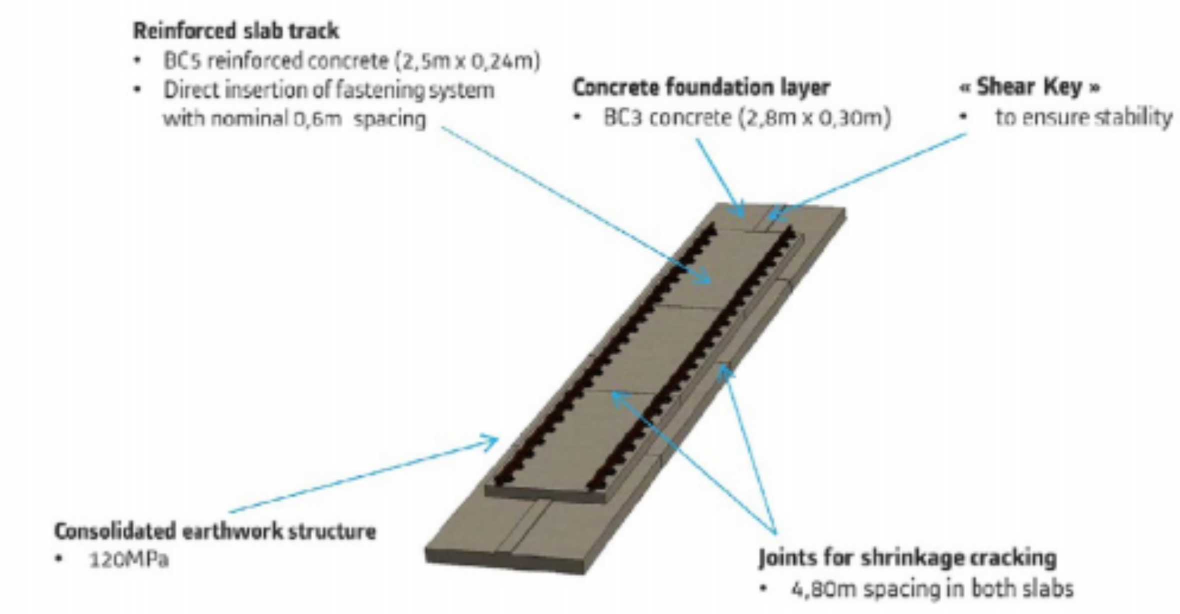


Figure 5. NBT (Robertson et al., 2015)

The system is designed for very high-speed trains, 360km/h, and axle load up to 25 t. It is possible to adjust the slab track +/- in lateral direction and 30 mm in the vertical direction. A problematic area is the transition zone between ballasted and ballastless foundation, due to the significant stiffness difference. To minimise these settlements, the NBT has designed the slab so that the dynamic stiffness is approximately 55 MN/m.

Appendix B

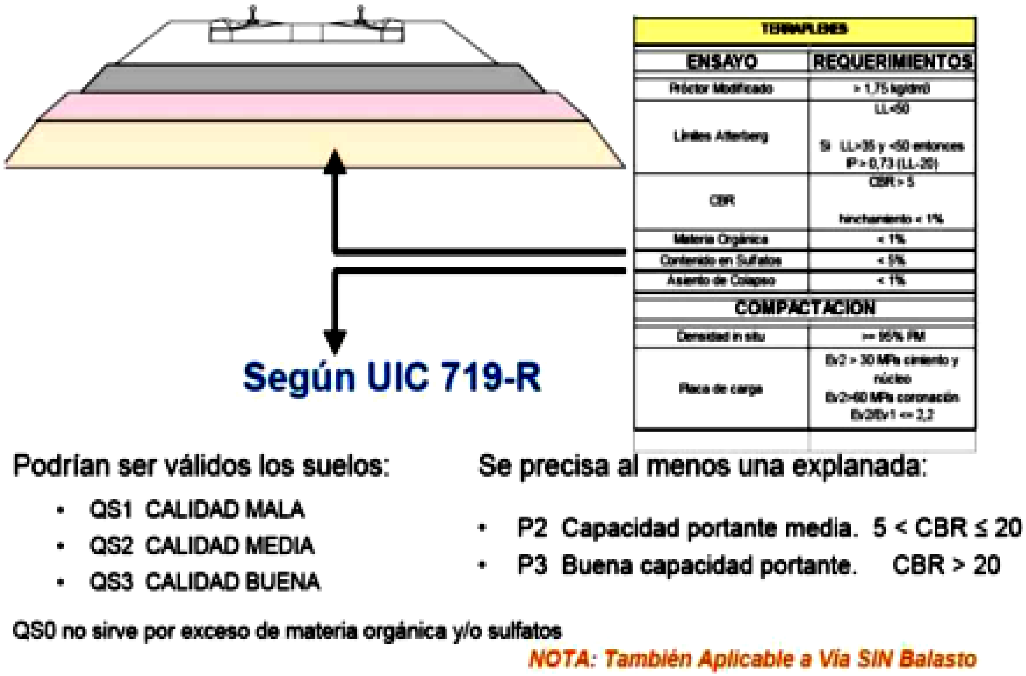


Figure 6. Embankment layer according to UIC 719 –R (Smekal, 2012)

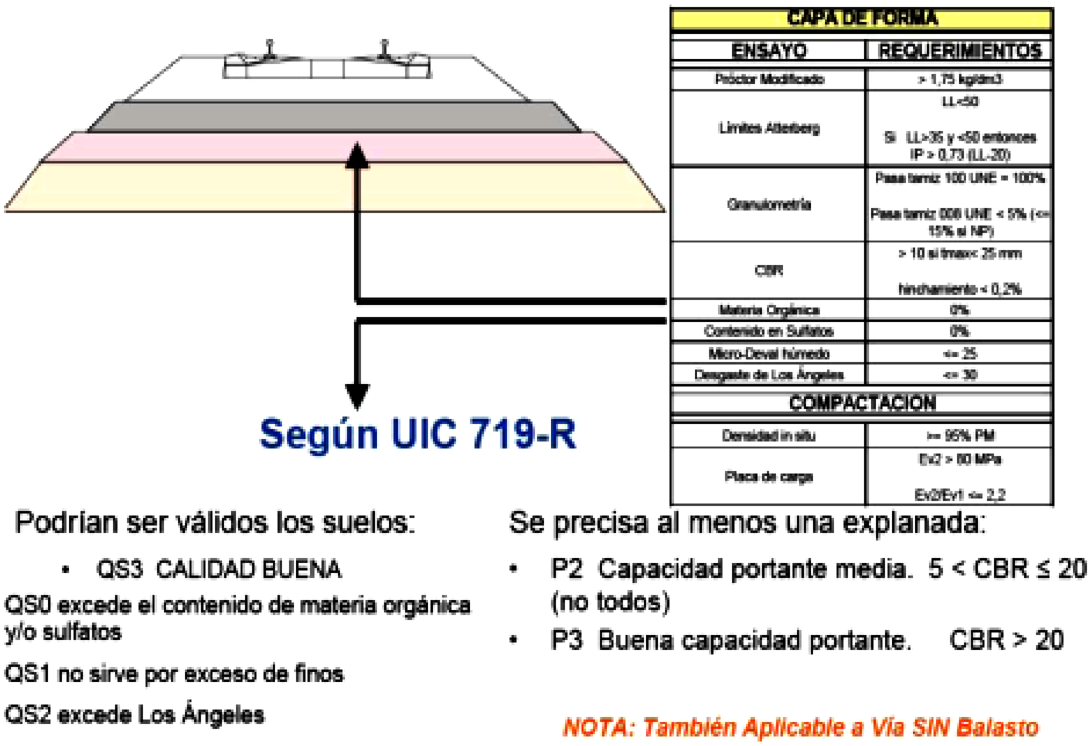
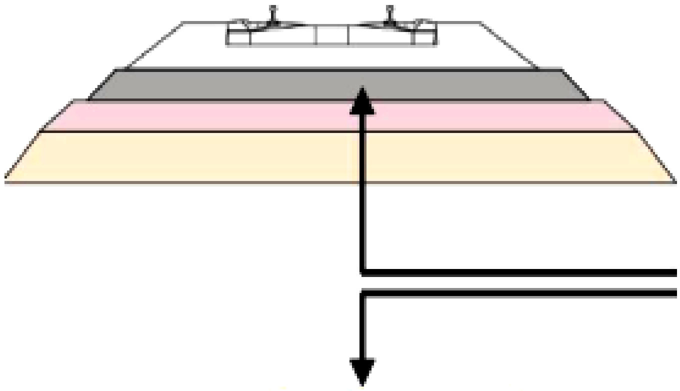


Figure 7. Formation layer according to UIC 719 –R (Smekal, 2012)



Según UIC 719-R

SUBBALASTO	
MATERIAL	
Próctor Modificado	> 1,75 kg/dm3
Granulometría	% Pasa tamiz 40 mm: 100 % Pasa tamiz 20 mm: 72 - 100 % Pasa tamiz 10 mm: 50 - 82 % Pasa tamiz 5 mm: 35 - 64 % Pasa tamiz 2 mm: 20 - 42 % Pasa tamiz 0,4 mm: 7 - 20 % Pasa tamiz 0,08 mm: 3 - 5 (<= 15% si NP)
Coeficiente de uniformidad	< 14
Materia Orgánica	0%
Micro-Deval húmedo	< 16
Desgaste de Los Ángeles	< 24
Contenido en sulfatos	0%
Permeabilidad	<= 10^{6} m/s
Índice de lajas	< 28
COMPACTACION	
Densidad in situ	>= 98% PM
Placa de carga	Ev2 > 120 MPa Ev2/Ev1 <= 2,2

Podrían ser válidos los suelos:

- QS3 CALIDAD BUENA

(No todos, puesto que algunos no cumplen que el coeficiente de LA<24)

Se precisa al menos una explanada:

•P3 Buena capacidad portante. CBR>20

(No todas las P3 cumplen si relacionamos el CBR > 20 con el Ev2 necesario del subbalasto>120 MPa)

NOTA: También Aplicable a Vía SIN Balasto

Figure 8. Subballast layer according to UIC 719 –R (Smekal, 2012)

Appendix C

Table 1. This table links the quality classes to the French earthworks classification. (Modified from (UIC, 2008))

Classification of standard NF P 11 300			Quality group of soils (See Tab.4)			
Class	**Name**	**Sub-class**	**QS 0**	**QS 1**	**QS 2**	**QS3**
A	Fine soils	A1	A1h	A1m & A1s	A1s B.C Hydro. dry season	
		A2	A2h	A2m & A2s	A2s B.C Hydro. dry season	
		A3	A3h	A3m & A3s		
		A4	Ah4	A4m & A4s[a]		
B	Sand and gravel soils with fine particles	B1				
		B2	B2h	B2m	B2s B.C Hydro.	
		B3				Clean and well graded Very hard
		B4	B4h	B4m	B4s	Clean and well graded Very hard
		B5	B5h	B5m	B5s	
		B6	B6h	B6m	B6s B.C Hydro.	
C	Soils containing fine and coarse particles	C1	The quality class corresponds to a ratio of 0/50			
		C2	The ratio of class Qs 0 w≥ 1.25 wopn	0/50 ratio of class QS 0 w < 1.25 wopn	0/50 ratio of class QS 1	Clean and well graded Very hard
D	Soils insensitive to water	D1			Ensure suitability for traffic test bed	
		D2		Low hardness	Medium hardness	Clean and well graded Very hard
		D3		Low hardness	Medium hardness	Clean and well graded Very hard

h: material classified as having high (h) or very high (th) water content in standard NF P 11 300
m: material classified as having average water content in standard NF P 11 300
s: material classified as being dry (s) or very dry (ts) in standard NF P 11 300
w: water content
wopn: optimum water content as per Standard Proctor
B.C. Hydro: good hydrological and hydrogeological conditions
Test bed: obligatory test bed

[a]Does not consider expansive soils, which must be exanimated specially

Table 2. Example of the France soil classification system. Soil type A. (Réseau ferré de France, 2010)

1.1 - Classe A : sols fins
(exemple : limons, marnes, argiles, limons argileux, argiles sableuses, argiles limoneuses,…)

Classification selon Norme NF P 11 300		Etat Hydrique	Classe de qualité du sol en place				Sensibilité au gel
			S0	S1	S2	S3	
A1	VBS ≤ 2,5 (ou Ip ≤ 12)	th	S0 →				SGt
		h	S0 →				
		m		S1 →			
		s		S1 →			
		ts	S0 →				
A2	12 < Ip ≤ 25 (ou 2,5 < VBS ≤ 6)	th	S0 →				SGt
		h	S0 →				
		m		S1 →			
		s		S1 →			
		ts	S0 →				
A3	25 < Ip ≤ 40 (ou 6 < VBS ≤ 8)	th	S0				SGt
		h	S0 →				
		m		S1 →			
		s		S1 →			
		ts	S0				
A4	Ip > 40 (ou VBS > 8)	th	S0				
		h	S0 →				
		m	S0 →				
		s	S0				
		ts	S0				

Appendix D

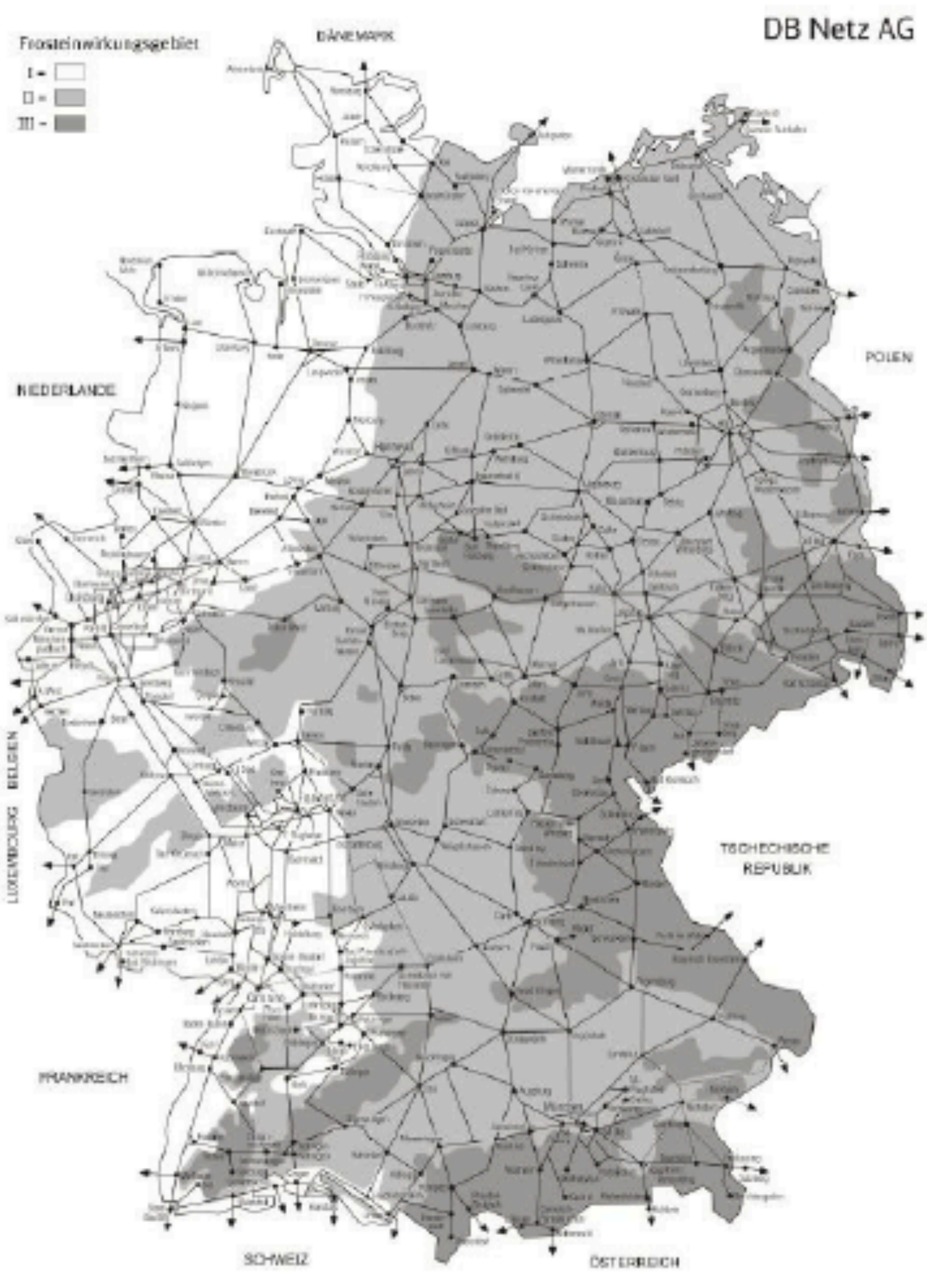


Figure 9. Frost areas (DB Netze AG, 2014)