



Capacity for Rail

***Towards an affordable, resilient, innovative
and high-capacity European Railway
System for 2030/2050***

Upgrading of infrastructure in order to
meet new operation and market
demands

Public deliverable D1.1.4

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EXECUTIVE SUMMARY

Very few recently built railway lines are constructed specifically for freight traffic. The trend today is that new lines are constructed for high-speed operations, whereas the remaining lines are transformed towards more freight. These existing lines were however built for the traffic demands at the time of construction. This means that lines need to be upgraded in order to meet the new demands from increased freight operations that also often have a different character than the freight traffic that existed before the transition (if any). Consequently upgrading of freight lines is, and will continue to be, an important activity to meet future demands from industry and society.

Freight operators often propose enhanced operations (e.g. increased longer trains, axle loads, more frequent operations). However, these are commonly not realised due to infrastructure limitations. The experience from countries that have allowed the enhanced operations by upgrading lines is that there are limitations to what can be done, but also possibilities to overcome and/or circumvent these limitations. Therefore, it is important that upgrading is carried out in a systematic manner and that state-of-the-art knowledge is employed.

Several Research & Development projects have increased the knowledge and understanding, and introduced tools to handle upgrading. This has enabled demands from freight operators to be met in an environmentally friendly and cost efficient manner. The picture below shows examples are given of some EU-projects carried out during the last 10 years that have provided input to enhance methods of upgrading freight lines.

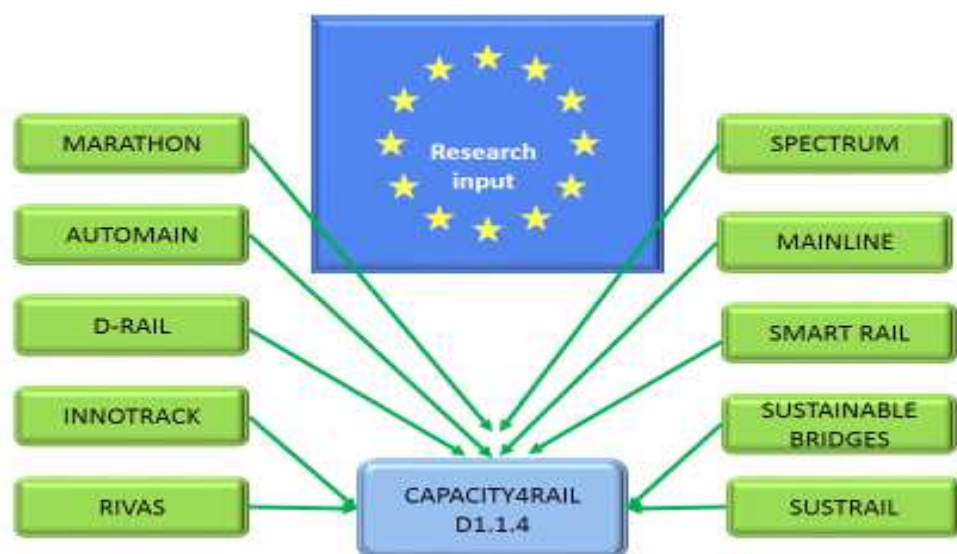


Figure E.1. Example of EU-projects that have providing knowledge to this guideline.

Despite the increasing need for freight line upgrading, and the increased knowledge on means and methods to carry out such an upgrading, there has until now not existed any good guideline or best praxis compilation founded on recent research and development. The present report – D1.1.4: Upgrading of infrastructure in order to meet new operation and market demands – aims to fill this need. The present report is a first step; when Capacity4Rail is close to finalisation it will be upgraded to a new version, D1.1.5, to allow incorporation of additional findings from Capacity4Rail (and additional new knowledge e.g. from the new European projects Shift2Rail and In2Rail) with bearing to freight track upgrading.

The target audience for this guideline is infrastructure managers, contractors and regulative bodies. In addition, the text is heavily referenced to allow for in-depth investigations of track experts and researchers.

The report sets out with establishing the background to the report (chapter 1) and by detailing the objectives (chapter 2).

In chapter 3 means of upgrading by modifying the traffic situation and the pertinent consequences are outlined.

Chapter 4 presents in detail upgrading possibilities, procedures and consequences for different types of infrastructure. Here section 4.1 focuses on substructures; 4.2 on structures like bridges, tunnels, culverts and retaining walls; section 4.3 focuses on track and switches and crossings (S&C). To provide the full picture, additional sections give input on topics such as energy supply, signalling and stations.

In sections 4.1 and 4.2 the upgrading is mostly focused on assessment and strengthening using new knowledge in a structured approach. In section 4.2 (structures) the impact of different alterations in the traffic situations is on different types of structures is described. Differences in safety concepts in codes for new and existing structures are outlined. Many of the uncertainties present when building a new structure that call for a high safety factor, can be resolved when you are dealing with an existing structure where actual loads, geometries and material properties (including deterioration rates) can be determined with a high accuracy.

An assessment of an existing structure is recommended to be carried out in three phases: Initial, Intermediate and Enhanced. The Initial phase includes a site visit, study of documentation and an overview calculation. The Intermediate phase may include material investigations, detailed calculations and/or analyses, and also additional inspections and monitoring. The Enhanced phase usually consists of refined calculations and analyses including statistical modelling and reliability based assessments, laboratory examinations and/or field-testing and economic decision analyses. That a thorough assessment as outlined above is often the most cost effective way to upgrade a structure is exemplified by cases where a high hidden capacity indicated by such an assessment has been confirmed by full-scale tests.

For tunnels, different cross sections and typical loading gauges are discussed, as well as possibilities for working in tunnels with rail service in operation. Examples are also given on variations in conditions in countries such as Germany, Austria, Switzerland, Sweden and the U.S.

For culverts, different types of damage and malfunction are described and examples are given on suitable measures. Examples are also given on how retaining walls can be constructed.

In section 4.3 a methodology is outlined for how consequences of upgraded operational conditions on track, switches and crossings can be investigated using three levels of successively more detailed investigations. In the first level, “low-resolution analysis”, an overview estimate is carried out of the consequences of upgraded operational conditions in terms of increased deterioration costs. In the second level, this analysis is refined to better identify major cost drivers. In the final “high-resolution analysis” the most important phenomena are investigated in detail. The section also contains an overview of different deterioration phenomena, and descriptions of potential consequences of different types of upgrading. It should be noted that the latter is not confined to increased deterioration – as an example the introduction of longer trains may also call for revisions of the geometry of the track infrastructure.

In chapter 5 maintenance routines are discussed. This includes proactive issues such as selection of suitable rail and wheel grade selections, monitoring of cracks, and establishing maintenance strategies. In addition, details on maintenance actions such as grinding and wheel re-profiling procedures, lubrication and welding are included. The chapter concludes with more “forward looking” topics such as automation of maintenance and implementation of new maintenance concepts.

The guideline ends with overall conclusions in chapter 6 and additional references in chapter 7.

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ABBREVIATIONS AND ACCRONYMS

Abbreviation / Acronyms	Description
BCM	Business Centred Maintenance
CARS	China Academy of Railway Sciences
CFRP	Carbon Fibre Reinforced Polymer
CMP	Common Mid-Point
CRP	Combined Resistivity and Reflection
ERTMS	European Rail Traffic Management System
FM	Friction Modifiers
GHG	Green House Gases
GPR	Ground Penetrating Radar
GPS	Global Positioning System
HSL	High Speed Line
HSG	High Speed Grinding
IM	Infrastructure Manager
IST	Instituto Superior Técnico (University of Lisbon)
KTH	Royal Institute of Technology in Stockholm
MBC	Mineral Based Composite
RCF	Rolling Contact Fatigue
RBCM	Risk Based Centred Maintenance
RFC	Rail Freight Corridor
RFC	Ride Force Counter
RSMV	Rolling Stiffness Measurement Vehicle
RU	Railway Undertaking
SAFT	Synthetic Aperture Focus Technology
SoS	System of System
TOP	Top of Rail
TPG	Two Pass Grinding
TTCI	Transportation Technology Center, Inc
UIC	International railway union
WARR	Wide Angel Refraction and Reflection
VDM	Value Driven Maintenance
VUC	Variable Usage Change

1 BACKGROUND

Development of railways has always raised new demands. Speeds and loads have increased quite significantly during the history of the railways. For newly constructed lines this has led to the introduction of larger rail profiles with better steel grades, improved rail support in the form of high-performing fastenings, sleepers and ballast layer, and in bridges designed for higher loads and speeds. At the same time the large majority of the railway infrastructure is old and built according to codes and demands of the time of their construction.

Very few new railway lines are designed and constructed specifically for freight traffic. The trend today is that the new lines are designed for high-speed operations and that the existing lines are transformed to carry more freight traffic. As mentioned, these existing lines are constructed for traffic demands from the time when they were built. This means that these lines have to be upgraded if they should fulfil the requirements of the new freight operations. Consequently, upgrading has and will be an important future measure to meet current and future demands from industry and society.

The big challenge is (and has for a long time been) how to upgrade existing infrastructure in a cost efficient and environmentally friendly manner. In particular this relates to some limitations are not easy to address, as will be outlined below. This deliverable aims to support Infrastructure Managers (IMs) in how to assess their infrastructure with respect to upgraded freight traffic in a methodical manner that ensures a (cost) efficient, safe and environmentally friendly upgrading.

Several recent R&D projects have increased knowledge and understanding of track upgrading. Figure E1 in the Executive Summary shows examples of EU-projects carried out during the last 10 years that have provided input to this guideline.

The guideline is structured such that after an overview of the objectives in chapter 2, chapter 3 investigates different traffic situations. The consequences of these traffic situations are assessed in focus on how they influence the infrastructure.

In chapter 4 the influence of upgrading on the infrastructure is investigated. Recommendations and examples of methods for assessment and analysis of consequences of upgrading are provided.

In chapter 5 different maintenance routines are evaluated with respect to consequences resulting from new traffic demands on infrastructure.

This guideline is intended to be upgraded to a new version – D1.1.5 – at the end of the Capacity4Rail project. In the upgraded version additional findings from Capacity4Rail, and also parts that were not possible to include due to the limited available budget will be added.

1.1 FREIGHT SYSTEM PARAMETERS

In addition to quality, the cost of freight transports (in a broad sense) is the most important parameter to consider. The cost will be affected by the following train and infrastructure parameters: The axle load, the load per unit length, the loading gauge, the train weight and the train length. The axle load and the wagonload per meter are depending on the infrastructure standard where the condition of the entire track including substructure and bridges etc. is important. The train weight is also limited by the tractive effort of locomotives, the couplings and the electrical power supply. For a customer also the quality – e.g. in terms of time for delivery, reliability, prevention of damage of goods etc. – is important.

The axle load often decides how much goods that can be carried in one wagon. The maximum axle load is therefore a good measure of the development of the railway systems, see figure 1.1. In the 19th century the highest allowed axle load was in the order of 10–15 tonnes, which in the 20th century increased to 20–25 tonnes. Today 22.5 tonnes is basically standard in Europe. With 22.5 tonnes axle load the gross weight of a 4-axle wagon can be $4 \times 22.5 = 100$ tonnes. If the wagon tare weight is 30 tonnes the payload will be 70 tonnes. In US it is common with 35.7 tonnes axle load which allows a payload of 120 tonnes on a 4-axle wagon.

Since the cost for a wagon and for train operations will only increase marginally with a higher axleload (and weight) the transportation cost will be lower the higher the allowed axle load. This line of reasoning is valid for heavy goods. For bulky goods, instead the loading gauge in terms of the wagons maximal height, width and shape can be the critical parameter. Regardless, a much cost efficient way of increasing capacity is to aim for “more freight in the wagon and more wagons in the train”.

However higher axleloads and higher load per unit length also means requirements for better infrastructure, which can be costly. However in situations where the track has been improved gradually and running characteristics of locomotives and wagons has improved, it has been possible to incrementally increase axle loads and speeds for freight trains – sometimes without any other measures than that the system has been continuously renewed and improved.

1.2 PASSENGER SYSTEM PARAMETERS

Although the focus of the guideline is on freight transport, it is important also to study the needs of passenger transports, not the least since most freight transport is carried out on lines where there are additional passenger operations.

The most important factors for passenger transports are the travel times, a competitive price and good quality (including reliability of services). To limit travel times, a high average speed is important. This is connected to a high top speed (in some cases including rapid braking and accelerations) and a proper timetable. To operate a train at high speeds, it first has to overcome running resistance. As speed increases, also the air resistance becomes important. At very higher speeds, this will be the most important factor. Therefore, faster trains need to be streamlined. In addition, passenger services are much more sensitive to track geometry deviation (both at large and small scales) than freight operations. This typically results in higher construction and maintenance costs for passenger lines as compared to freight lines.

Today the limit for daily operation of a high-speed train is some 300–350 km/h. This limit is mainly based on what is economically feasible. By technical development, this limit has

continuously been pushed higher. In the 1930s, the speed record for steam engines was some 200 km/h. The top speed record for an electric high-speed train today is 576 km/h, as set in 2007 by a TGV train. There is a clear connection between the top speed record and the highest allowed operating speed for commercial trains. The top speed record has been doubled every 60 years. If this trend will continue, there will be trains in operating at 500 km/h in 2050. The development is shown in figure 1.2.

In figure 1.3 the maximum train speed in Spain is shown. On conventional lines it increased from 160 to 200 km/h in the 1990s. On high-speed lines it has reached 300 km/h by the introduction of new lines and vehicles. The Spanish high-speed record was reached 2006 with 403.7 km/h. The axle load on the Spanish network was increased from 20 to 23 tonnes in 1980. On high-speed lines it is 17 tonnes/axle.

After investigating the diagrams in figures 1.1 and 1.2, we believe that requirements on upgrading infrastructure to meet new demands are a reality of great importance. At the same time it is known that some of today’s restrictions are easy to meet while some other are difficult, costly and sometimes impossible to overcome with realistic measures. Therefore, a systematic approach to evaluate the impact of track upgrading is of great value. This is the background to the current deliverable.

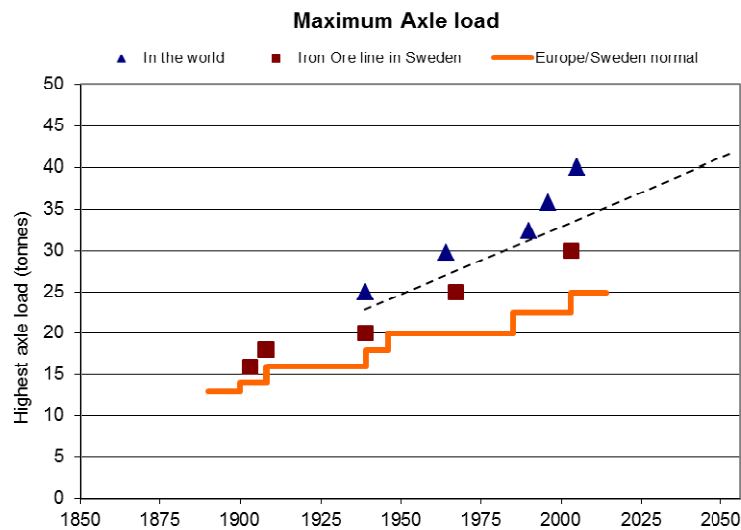


Figure 1.1: Development of maximum axle loads in the world, Europe and for the Iron Ore line in Sweden with indicated trend line to 2050. Source: KTH Railway Group (Fröidh-Nelldal).

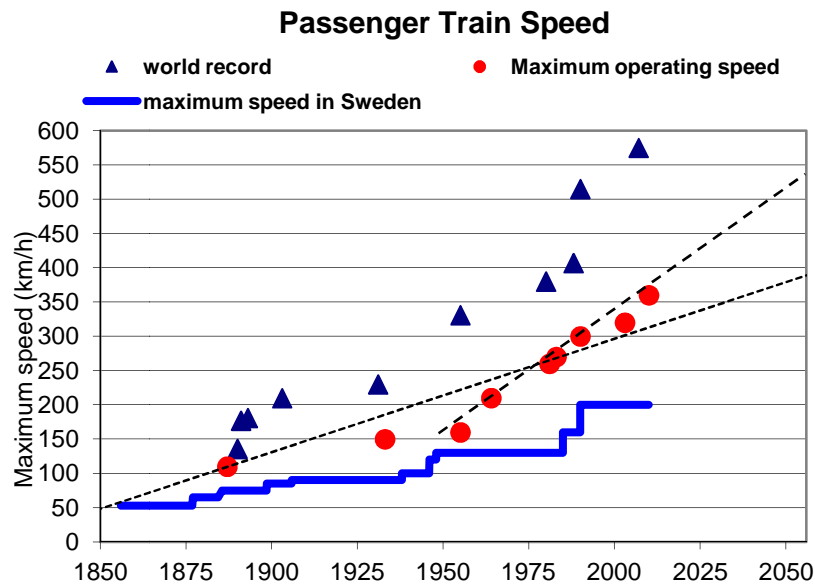


Figure 1.2: Development of world speed records for passenger trains, maximum operating speed in the world with indicated trend line towards 2050. Also maximum operating speed in Sweden is shown. Source: KTH Railway Group (Fröidh-Nelldal).

Passenger Train Speed in Spain

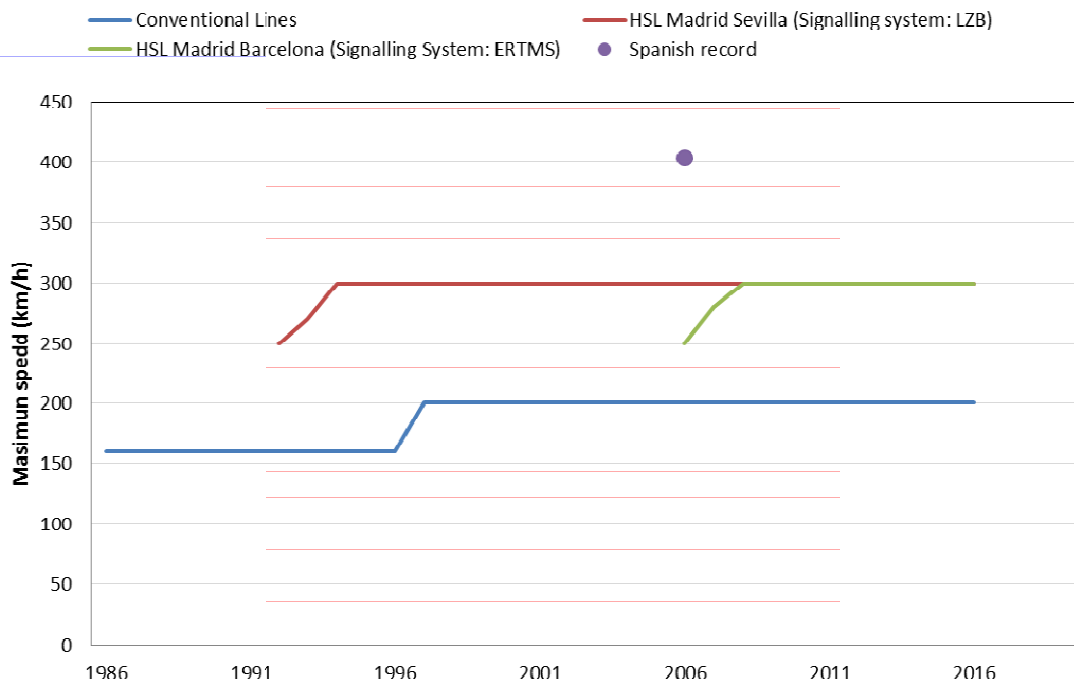


Figure 1.3: Passenger train speed in Spain 1986-2016.

2 OBJECTIVES

The objective of the current report is to facilitate upgrading in a methodical, (cost-) efficient and environmentally friendly way. To this end, the guideline will contain clear descriptions and recommendations. It can be considered as a combination of checklists, functional requirements and overviews of potential possibilities of analyses and actions. However, foremost it outlines a structured way of working with the upgrading process.

Sometimes clear recommendations on actions are not possible to give (e.g. because the recommendations are intended to cover a very broad range of upgrading scenarios). In such cases references to state-of-the-art knowledge in the area will be given.

The guideline is mainly intended to provide support to infrastructure managers in upgrading infrastructure in order to meet new operational and market demands. However, the infrastructure performance must be adopted to customer needs and allowed societal costs. This will also affect the wagon and train configurations and by that the rail network and the infrastructure standard, as described in next section. For this reason, the guideline should be valuable for all parties working with upgrading or operation under upgraded conditions, e.g. operators, industry (both on the train and on the trackside), maintenance contractors etc.

In upgrading substructures, the objective is a better understanding of existing methods for subgrade assessment, with special focus on high-performance methods such as stiffness measurement and/or geophysical inspections. The aim is to evaluate the subgrade quality along the whole railway line prior to infrastructure upgrading. In cases where the subgrade condition does not meet the required performance, strengthening will be necessary. To this end, the latest knowledge in improvement methods is reviewed, with a close look at the impact on train operations.

For bridges, the objective is to provide tools so that the assessment and upgrading can be done in a structured, (cost-) efficient and safe manner. Several EU-projects have shown that most bridges have a hidden capacity and that it is possible to quantify and utilize it. The objective is to present methods to do this for usual bridge types. If the added capacity found is not sufficient, adequate strengthening and upgrading methods are proposed and exemplified.

For tunnels increased loading gauges is the biggest challenge. The objective is to present the currently best practice and to provide examples of what can be achieved.

For culverts and retaining walls, the objective is to exemplify problems that may be encountered and to present methods to overcome these. In addition, the aim is to provide some good examples of how upgrading can be performed.

For tunnels increases in loading gauge is the main challenge. Here the objective is to provide good examples on how this can be handled.

To handle upgrading of tracks, switches & crossings is very much a question of having control over the resulting deterioration. Therefore the main objective is to understand, quantify and limit different modes of deterioration. To this aim, a scheme of successively higher resolution analyses is presented and suitable mitigating actions are outlined.

Some of the increased deterioration (related to different phenomena) can be controlled by enhanced maintenance routines. The objective of chapter 5 is to address how this can be done.

2.1 HOW MARKET REQUIREMENTS AFFECT TRAIN, VEHICLES AND INFRASTRUCTURE PERFORMANCE

There are different requirements for development in the markets where rail is used. These will affect the future performance needs, and thereby the requirements of the infrastructure, see figure 2.1. For heavy freight, such as steel and iron ore, there is often a request for higher axle load and load per unit length, which will affect the track, supporting structures, and the substructure. For semi-finished goods, there are sometimes requests for higher axle loads and wider load gauges. For voluminous goods, there is typically an ambition towards wider loading gauge and longer trains.

For intermodal transports, the typical request is for a higher loading gauge and longer trains. This will affect the requirements for infrastructure in different ways. A higher and wider gauge will affect the tunnel profile and bridge structure to accommodate the freight. Higher axle load will affect both the track and the substructure to allow heavier loads. A request for higher linear load will primarily affect bridges and culverts to be able to carry a higher load per unit length without premature deterioration.

Markets	Train parameters				Infrastructure parameters			
	Linear load	Axle load	Gauge	Train Length	Track	Sub-structure	Structure	Network
Raw material High density	High	High	Low	Short	Low speed	Very stable	Very stable	Normal sidings
Semi manufactured Medium density	Medium	High	Wide	Medium	Medium speed	Very stable	Very stable	Normal sidings
Manufactured Medium density	Medium	Medium	Wide	Medium-long	Medium speed	Stable	Widen gauge	Long sidings
Inter Modal Low density	Low	Low	High	Long	Higher speed	Stable	Higher gauge	Long sidings

Figure 2.1: Examples of markets, and pertinent affected train and infrastructure parameters (Nelldal, KTH 2015)

2.2 THE NEED FOR A SYSTEM APPROACH

All improvements of the rail system must be based on a system view of the railways, as illustrated by the freight system in figure 2.2. The customer's transportation needs put demands on the wagons – the wagons are coupled together into trains where available

tractive power is taken into account – the train utilises the infrastructure with a certain performance along a link and ultimately in a network from origin to destination. The intention is to analyse the railway system from its actual performance today to what is planned for the future and what is optimal from the point of view of markets if the entire system is considered.

For the traffic situations in chapter 3 the objective is to point out what is possible to do with the trains and operation of the freight system. It will include effects on cost and capacity and also relation to infrastructure performance as further described in the subsequent chapters.

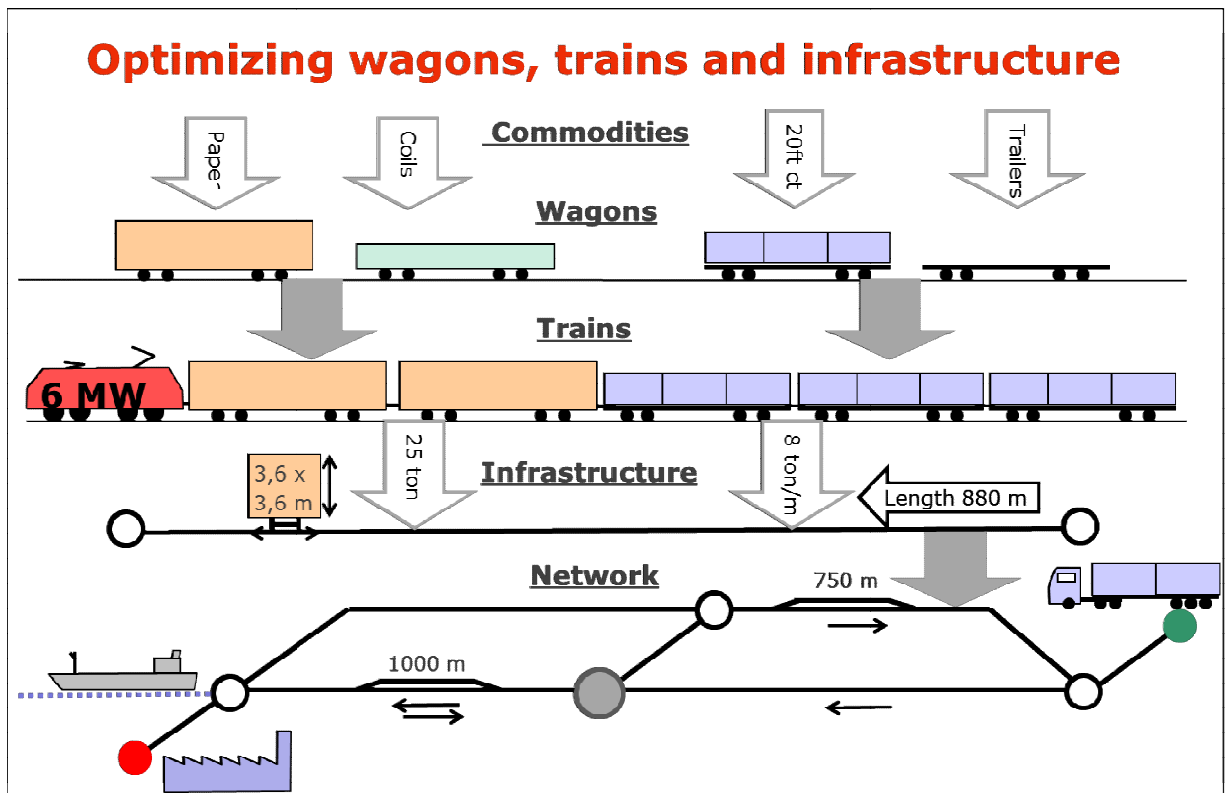


Figure 2.2: Principles for optimising wagons, trains and infrastructure. Source: KTH Railway Group "Roadmap for development of rail and intermodal transportation" (Nelldal, KTH 2013).

3 TRAFFIC SITUATIONS

3.1 INTRODUCTION

What do we mean with upgrading in this deliverable? By upgrading, we mean that the traffic situation is changed so that more goods can be transported on a railway line and/or can be transported in smaller timeslots.

The total demand for freight in Europe has increased rapidly in recent decades, but trucks have handled most of the increase, even if the rail market share has increased in many countries. Rail deregulation has not been implemented in practice in all countries while at the same time truck deregulation has resulted in a low-cost truck market. Moreover, the cost of external effects has not yet been implemented.

Most forecasts show an increase of 60% in total freight demand by 2050 and an approximately constant rail market share. To fulfil the targets in the EU white paper, it is necessary to double the market share of railway traffic from 18% today to at least 36% in 2050. This means that the demand for rail will increase by 3-4 times (Nelldal-Andersson 2012).

To reach this white paper target, it is necessary to both increase quality and capacity and to lower the cost of rail freight. Rail must be better to meet the customer requirements and the cost must be competitive with road freight. To solve this, a system approach is needed since the performance of rail operations is not performance better than the weakest link of the railway transportation chain.

To increase the capacity of the rail system, the following measures can be taken:

More efficient timetable planning: On double track: Bundling of trains with the same average speed in timetable channels. During the day faster freight trains are an option.

Use of trains and vehicles with higher capacity: For freight: Longer trains, higher and wider gauge, higher axle load and metre load. For passenger trains: Double-decker and wide-body trains.

Differentiation of track access charges to avoid overloaded links.

Better signalling system, such as ERTMS with shorter block lengths is a potential measure on double track.

Adaptation of freight corridors for long and heavy freight trains.

Investment in HSR to increase capacity for freight trains and regional trains on the conventional network and dedicated freight railways.¹

There is a target in the white paper to triple the length of High Speed Rail (HSR) lines by 2030, which means that there would be approximately 18,000 km HSR covering about 8% of the rail network. According to current plans (UIC 2013) this seems to be realistic. The planned Rail Freight Corridors (RFC) are of approximately the same length. Longer and heavier trains will

¹ C4R SP2 WP 2.1 Progress beyond State of the Art on Rail Freight Systems, Bo-Lennart Nelldal, KTH. 2014.

make it possible to roughly double the capacity for freight trains without building new railways. With ERTMS even more traffic can be handled, see figure 3.1 below from in Capacity4Rail WP2.1 report.

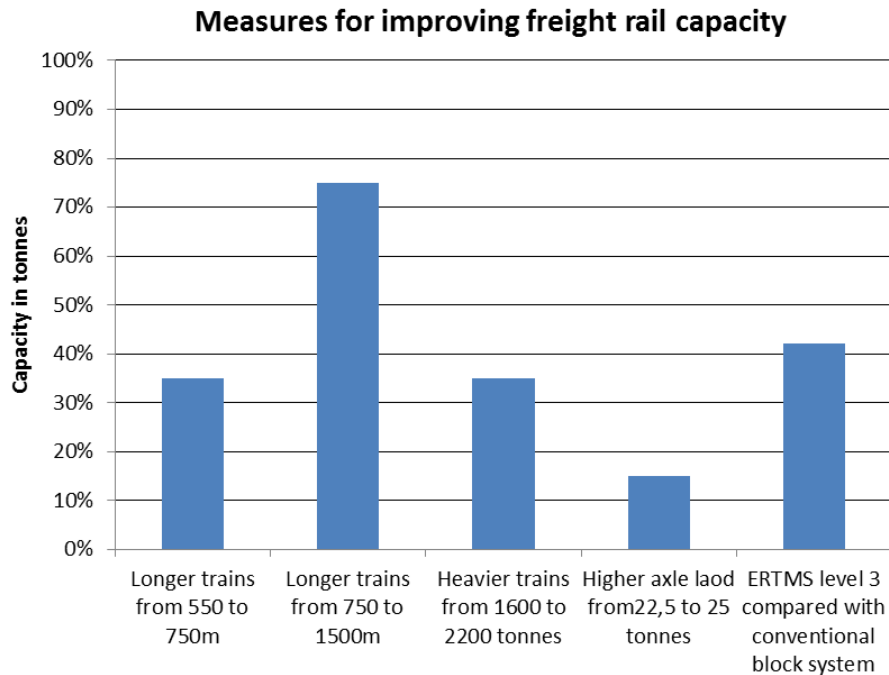


Figure 3.1: Capacity gains for different freight train measures. Source: TRANSFORUM freight road map (Nelldal 2014).

An estimation of the effects of a mode shift to rail transport applying the world’s “best practice” shows that such a mode shift to rail can reduce Green House Gas (GHG) emissions due to EU transports over land by about 20%–30%, compared with a “business as usual” scenario. The rail system can thus substantially contribute to the EU target of reducing GHG emissions in the transport sector by 60% compared to 1990 levels. To enable such a mode shift there is a need for strong development of the rail system.²

3.2 STAKE HOLDERS PREFERENCES

As a pre-view to the following subchapters, it is also important to keep in mind the demands from the rail freight market stakeholders. In order to visualize the priorities of the rail freight market stakeholders, the graph below gives an over-view.

The graphs in figure 3.2 indicate the importance of the most important factors to increase capacity for rail freight, as stated by stakeholders of the rail freight transport market. The graphs in figure 3.3 visualize the specific viewpoints regarding the four factors of length, weight, axle load and speed.

² C4R SP2 WP 2.1 Progress beyond State of the Art on Rail Freight Systems, Bo-Lennart Nelldal, KTH. 2014.

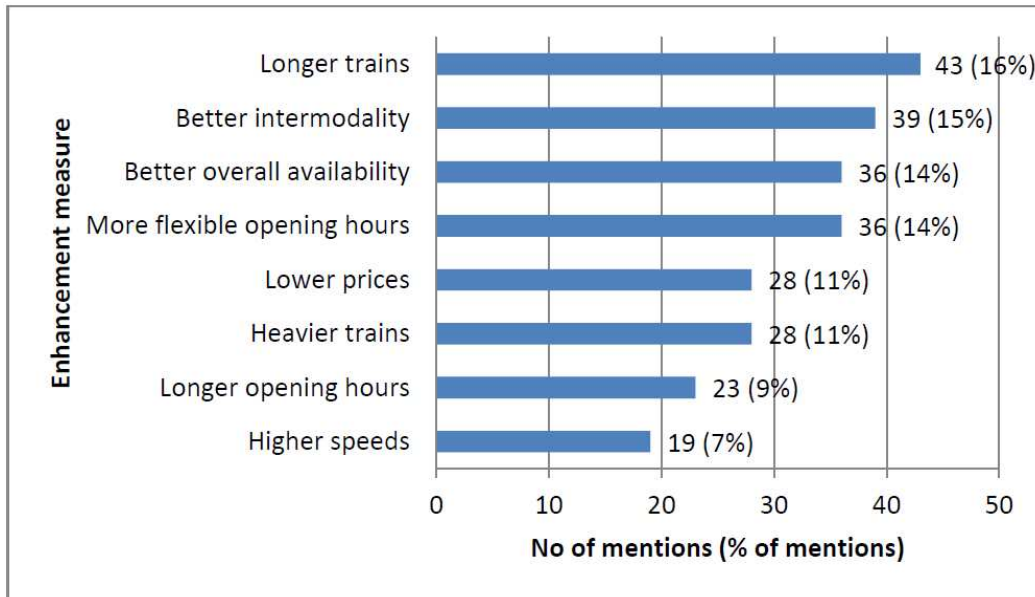


Figure 3.2. Improved measures suggested by stakeholders. Source: Transport Market Study for the Scandinavian-Mediterranean Rail Freight Corridor, page 212. ETC Transport Consultants, 2014.

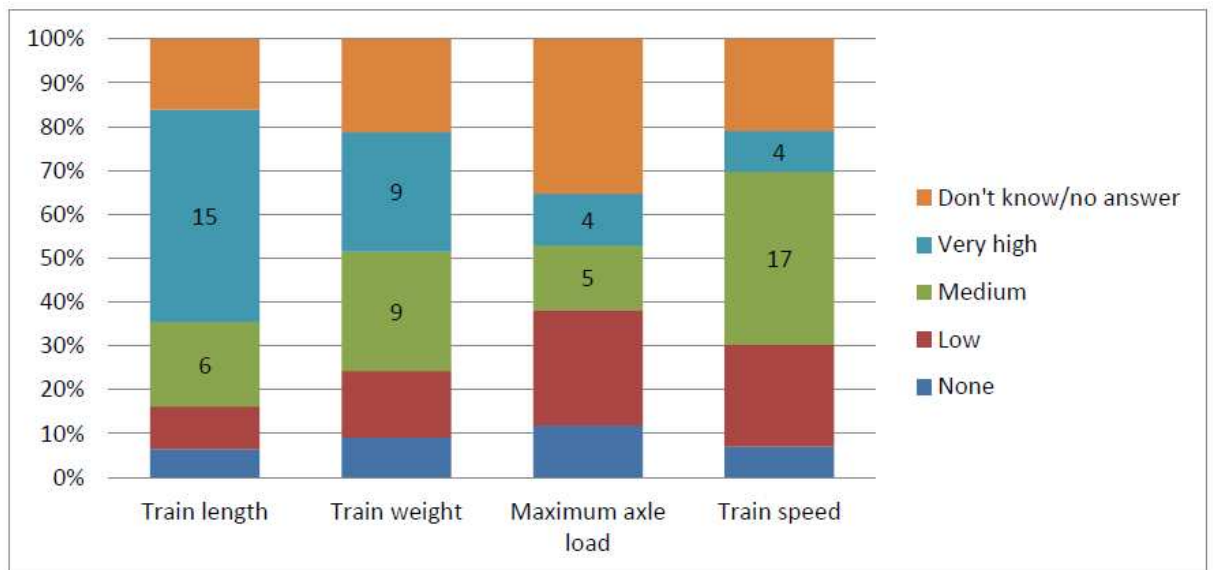


Figure 3.3: Rating of Importance of Technical Parameters. Source: Transport Market Study for the Scandinavian-Mediterranean Rail Freight Corridor, page 187. ETC transport Consultants, 2014.

3.3 TECHNICAL AND OPERATIONAL DEVELOPMENT

Much of today's rail freight system is based on an old standard 3–4 MW locomotive. This means trains of some 1,500 gross tonnes and a maximum length of 650–750 metres. Nevertheless, modern locomotives have a tractive power of 5–6 MW capable of hauling 2,000–2,500 tonne trains of up to 1,000m in length. In Europe, train lengths up to 850m already exist and experiments have been made with 2x750m=1,500m long trains. With higher axle load on the loco's, the train weight can be even more increased due to the improved frictional capacity. For example, today's Traxx-locomotives have 21 ton axle load. An increase to 22.5 ton – as in most cases are allowed for the wagons – will increase the potential train weight with approx. 7%.

Concerning the wagons, the question is whether development will be incremental or if it is possible to make a system change. An incremental change means successively higher axle loads, wider gauge, higher payload, better and more silent brakes and some added electronic equipment. A system change will include electro-pneumatic brakes, disc brakes, full electronic control of the wagons loads and automatic central couplers. The automatic couplers are the most critical component – important because not only it will make shunting and marshalling safer and cheaper, but also because it will make it easier to operate longer trains.

Duo-locomotives equipped with both normal electric traction and diesel traction has been introduced. These can shunt the wagons at a yard or an un-electrified siding. The operators thus need only one loco instead of two. It will also facilitate introduction of linertrains that can stop along the line to change wagons.

For intermodal rail transport, the terminal costs are critical. If the terminals are located on an electrified side track where the train can drive straight in and out, liner trains can be used without switching to a diesel engine (or a duo loco). This requires a horizontal transfer technology that can function under the overhead contact wires so that the train can change unit-loads during a stop of 20 minutes. The terminals can be made compact since there is no need to park the wagons. Most trailers today are not designed to be hauled onto a railway wagon. Solutions where trailers can be rolled on and off can thus widen the market considerably.

In the following sections of this chapter, different measures of train and wagon performance will be analysed. Note that modifications of these will also affect the requirements of the infrastructure as discussed in the subsequent chapters.

- Increased train weight
- Longer trains
- Higher axle load and meter load
- Higher speed
- Increased loading gauge
- Combinations of these measures

3.4 INCREASED TRAIN WEIGHT

With a model developed at KTH the differences in cost and capacity of a typical freight train with increased performance have been calculated. The model has been used to evaluate different measures. The cost has been calculated for a Swedish freight train on a 600 km line with an average speed of 75 km/h. It is the operating cost for the train including capital cost for the vehicles and administrative costs. Track access charges are included but not the actual costs for the infrastructure. Feeder transports and marshalling is not included. This means that it is only line haul costs that are calculated. The capacity is calculated for the train itself and not for the line.

The result is visualized in diagrams. They show the relative difference between trains with different parameters i.e. train weight, train length, axle load. The cost is shown as the difference in percent of cost per payload in tonne-kilometre. Capacity is indicated by the gross weight in tonnes for the train.

Cost and capacity differences for heavier trains is shown in figure 3.4. The basic data is for a train with 1650 gross-tonnes and one locomotive. If you can haul 2000 tonnes, the capacity of the train will increase by 22%. At the same time, the cost per tonnes-km will decrease by 9%. You can also see that if you decrease the train weight to 1400 gross-tonnes, the capacity will decrease by 14% at the same time, as costs will increase by 9%.

If we continue with investigating potential improvements, it is seen that if 2600 tonnes can be handled with one locomotive, the capacity per train will increase by 58% and the cost per tonnes-km will decrease by 19%. Nevertheless, if we increase to 4000 gross-tonnes the capacity will increase by 144% as compared to the basic alternative of 1650 tonnes. However with today’s traction you will then need two four-axle locomotives instead of one, so the cost will only decrease by 18% (as compared to 19% with one engine and 2600 tonnes). At 5200 tonnes, you will need a train with two modern locomotives. The increase in capacity will then be 219% and the pertinent decrease in cost by 25%.

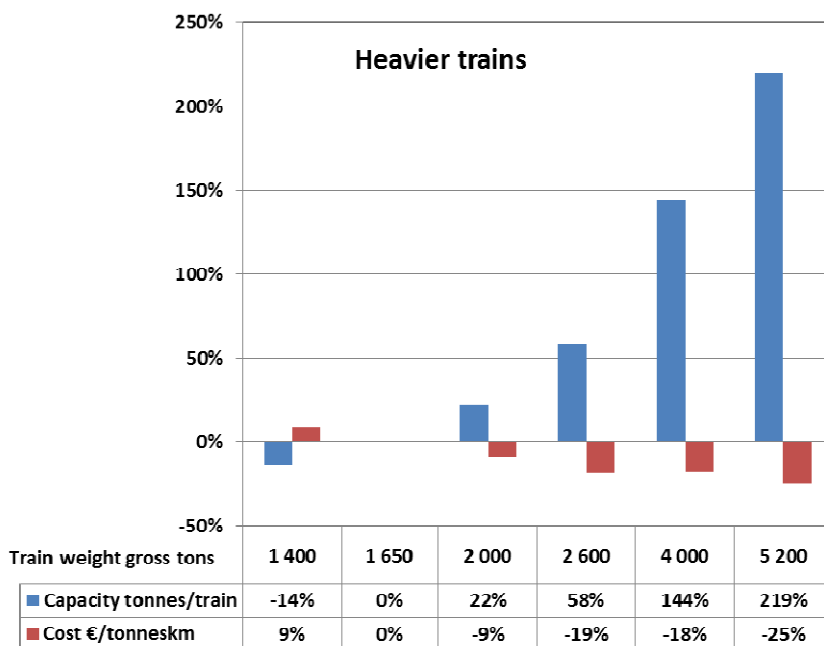


Figure 3.4: Effects of heavier trains in capacity per train and cost per tonnes-km. Source: Roadmap for development of rail and intermodal freight transportation, KTH Railway Group. 2013

3.5 LONGER TRAINS

As could be seen in figure 3.1, the issue of increasing the train length is the single most important operational requirement of stakeholders on the rail freight market. Increasing the length of the trains will increase train path capacity and freight volume capacity, respectively.

Furthermore, railway undertakings (RU) revenues could substantially increase while costs would rise only marginally. The graphs in figure 3.5 below clearly indicate that there is a demand for increased length as well as for increased weight of trains. Between 2010 and 2013 the train length was increased from 650 meter to 835 meter, and the train weight was increased from 1600 metric tons up to 2300 metric tons on the line between Germany and Denmark in the relation Maschen - Fredericia.

The increased utilization of about 19 % led to a decrease in costs per unit by about 14%. The strengthened competitiveness led to an increase in market share by more than 25 % between 2010 and 2013. At the end, the RU operated not only better-utilized trains but also more trains (+ 7%).

It is also interesting to notice that both graphs still indicate the same pattern even after the implementation of increasing the train length and train weight. The conclusion must be that cargo volumes available for rail freight still are higher than the supply of freight capacity.

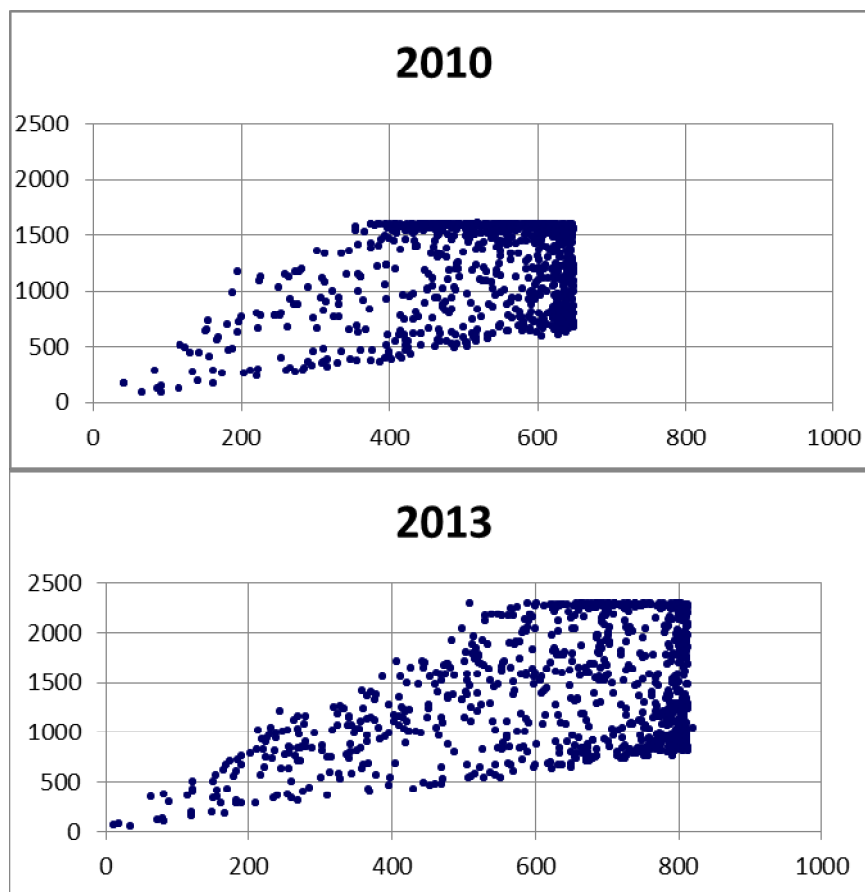


Figure 3.5: Trains Utilisation Effects 2010-2013 between Maschen and Fredericia when train length was extended from 650 to 835m and train weight was increased from 1600 to 2300 tonnes. Source: Longer Trains Utilisation Effects DB Schenker, 2014

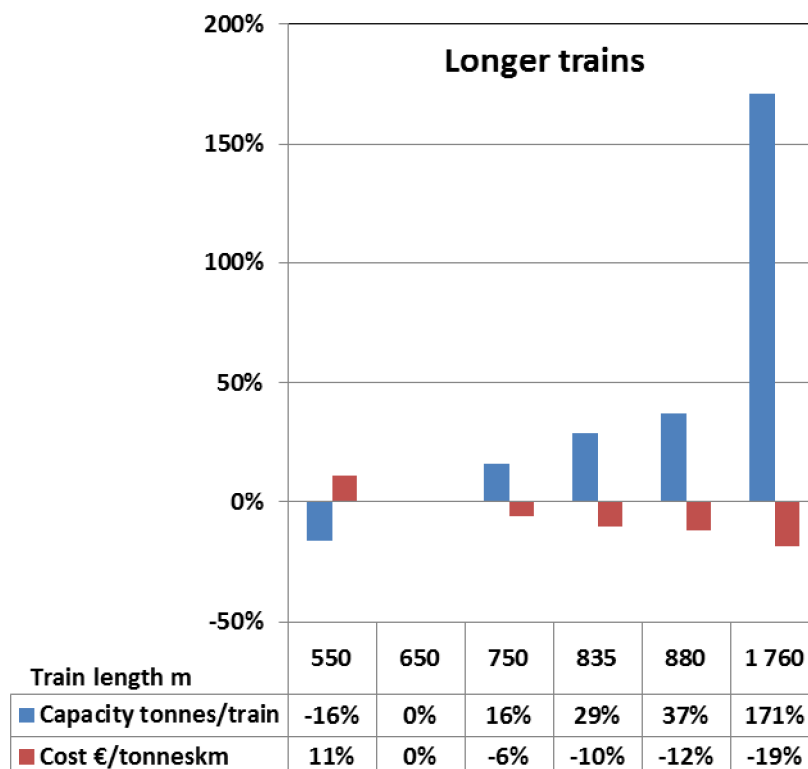


Figure 3.6: Effects on train capacity and operation costs of longer trains. . Source: Roadmap for development of rail and intermodal freight transportation, page 21. KTH Railway Group. 2013.

The effects of longer trains on train capacity and operational costs are shown in figure 3.6. If the train is extended from 650 to 750 m with one loco, capacity will increase by 16% and costs will increase by 6%. If the length is 835 m with one loco (e.g. as today between Hamburg and Copenhagen, capacity will increase by 29% and the cost will decrease by 10%. These are effective measures if you calculate the train costs and capacity. However, note that infrastructure costs are not taken into account.

Longer trains are easier to handle on double track since there are normally no meetings on these. However, passenger trains must overtake sometimes freight trains and then there must passing stations of sufficient length. However, during night, when freight trains dominate, it will be easier to find proper paths. In addition, the yards must be adopted to longer trains. On single track, it is necessary to adopt most crossing stations to longer trains. This can be expensive, however in general less so than to build a double track. This is common in the US, where it is rather common with very long trains (2000–3000m) on single-track lines.

3.6 INCREASED AXLELOAD AND METERLOAD

A high axle load is favorable for freight traffic, as more weight can be loaded on each wagon, or there can be fewer axles per ton payload. It is also an advantage if the locomotive has a higher axle load, as the adhesion, weight can be increased, which reduces the risk of slipping and allows for higher train weights. The maximum permitted axle load applied on most of the

main lines in Europe is 22.5 tons. This weight has been gradually raised; previously, it was 20 tons.³

In some countries, an upgrade of the axleload to 25 tons is in progress on selected sections of network with heavy transports, and most new lines are designed for 25 tons axleload. On the Iron Ore Line in Sweden, 30 tons axleload applies. Today 32.5 tons axleload is tested.

Figure 3.7 shows the correlation between axle load/train weight and axle load/train length and how the cost €/tonkm decreases at higher axle loads.

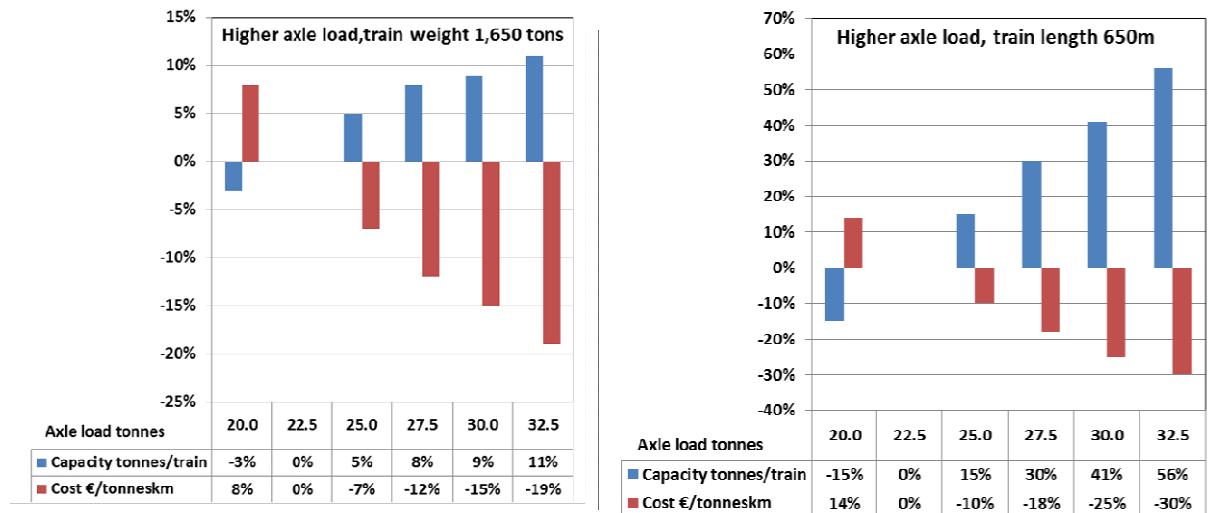


Figure 3.7: Evaluation of measures taking capacity and cost into account. Source: Roadmap for development of rail and intermodal freight transportation, page 21. KTH Railway Group. 2013.

With a model developed at KTH the cost and capacity of a typical freight train have been calculated. The model is described earlier.

Higher axle load: An increase from 22.5 to 25.0 tonnes axle load will increase the capacity of a train by 1,650 tonnes, gross weight by 5% and decrease the cost by 7%, not considering the infrastructure cost. This is due to fewer wagons required for the same payload. If the train length is constant at 650 m, the train can be extended with more wagons, capacity will increase by 15%, and the cost will decrease by 10% with 25.0 instead of 22.5 tonnes axle load.

A *high permitted load per unit length* is important for freight with high density, and allows for high loading factors on shorter wagons. This in turn means that train length can be limited, and that heavier trains can be operated on lines where passing sidings are short. It also allows more wagons to fit into yards and sidings. A high load per unit length is thus important for efficiency, especially for ore, steel and paper products. To give an example of current potential, a large number of wagons for steel industry transports are designed for 8.3

³ Cross-border freight Transports by Rail, page 66, Bo-Lennart Nelldal, KTH, 2014.

tons/meter and 25 tons axle load. Further, new bridges in Sweden are designed for 11 tons/metre.⁴

Higher axle loads and meter load are of interest to the stakeholders on the rail freight market, however this topic cannot be considered to be of prime interest, as visualized in figure 1.⁵

STORA ENSO introduced a new transport concept in 1999. The new transport concept was based on increased axleload and increased loading gauge. The new transport concept gave savings on the costs for paper products with some 20 million €/year.

In 1995–1997 LKAB evaluated the consequences to increase axleloads from 25 tonnes to 30 tonnes. More cost-efficient transports were a necessity for LKAB since the mines in Kiruna and Malmberget in Sweden are the only subterranean iron ore mines among the major competitors. The new transport concept including higher axle loads, increased speed etc. resulted in a reduction of rail transport costs with more than 50%.

Experience from Sweden where more than 40% of the railway network today is upgraded to 25 tonnes or more is very positive. The most profitable examples are when the whole transport concept is evaluated and where higher axleloads and/or speeds is one of several measures.

3.7 HIGHER SPEEDS ON FREIGHT TRAINS

As could be seen in figure 3.2, the specific issue of higher maximum speed is of low concern. Interviews with several RU's and forwarders indicate that this issue is of considerably lesser interest than activities to increase the average speed of the trains (the time it takes from A to B) and that the last kilometer is very often the determining factor concerning reliable overall travel times⁶. The market stakeholders would prefer to have a rail freight solution with higher reliability and punctuality than to have faster – but more unreliable – services.

To reach a higher average speed it is most important is to avoid stops for overtaking by passenger trains and stops at borders and marshalling yards. By a higher top speed you can avoid overtaking especially during daytime, see figure 3.8. It is also necessary to bundle trains with different average speed to utilize capacity better and – in certain corridors and times windows – to give priority to freight trains.

Improved acceleration and breaking performance of freight trains is also a measure to make them more flexible in operation with mixed traffic. This is a rather complex technical question where investments and maintenance of the wagons must be taken into account.

Additionally, regarding maximum speed, speed restrictions are in place on certain sections and though 120–140km/h would be technically possible e.g. in Germany. A constant

⁴ Cross-border freight Transports by Rail, page 66, Bo-Lennart Nelldal, KTH, 2014.

⁵ Transport Market Study for the Scandinavian-Mediterranean Rail Freight Corridor. ETC transport Consultants, 2014

⁶ Transport Market Study for the Scandinavian-Mediterranean Rail Freight Corridor, page 188. ETC Transport Consultants GmbH, Berlin 2014.

maximum velocity of 100–120km/h would be sufficient and less costly⁷. Thus, better train path utilization (allowing for faster and more reliable travel speeds) is usually more important than maximum speed.

Many wagons and most freight locomotives are prepared for 120 km/h top speed, so this may be the next step in increasing speed for some freight trains, preferably on daytime in mixed traffic lines. The step to 140–160 km/h is more demanding mainly due to today’s breaking systems.

A additional important factor is that higher maximum speed and average speed also improves the competitiveness of rail freight transports by lowering the cost of productivity since it may be possible to get one more turn of a train set (or a locomotive) per day.

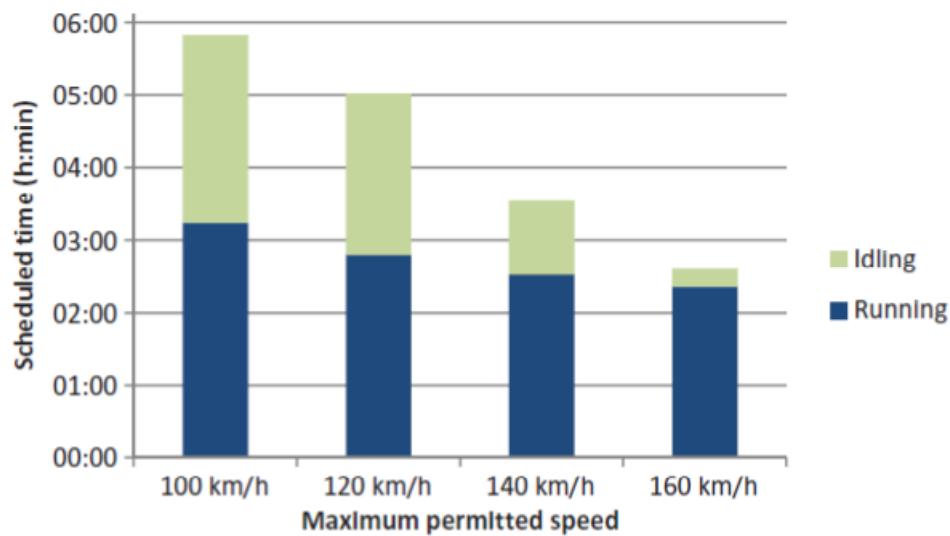


Figure 3.8: Scheduled time for a freight train between Mjölby and Hässleholm in Sweden (273 km), daytime, when passenger services dominate. Idling time is primarily stops for overtaking’s. Source: Capacity for Express trains on mixed traffic lines, Fröidh, Sipilä, Warg 2013.

⁷ Transport Market Study for the Scandinavian-Mediterranean Rail Freight Corridor, page 188. ETC Transport Consultants GmbH, Berlin 2014.

3.8 INCREASED LOADING GAUGES

The issue of increased loading gauge is one very interesting opportunity to increase freight volumes on rail in the future.

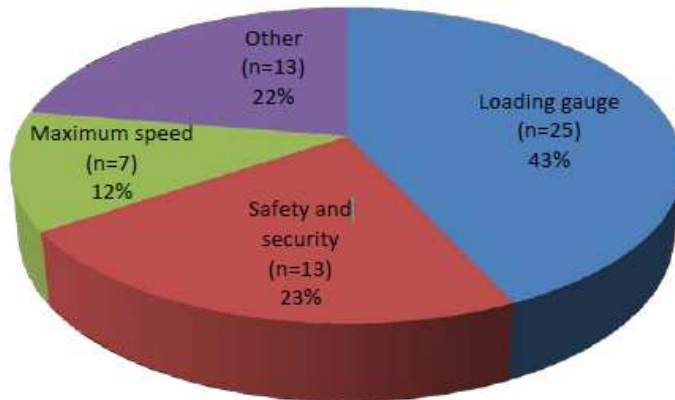


Figure 3.9: Demand to Improve Transport Equipment and Facilities. Source: Transport Market Study for the Scandinavian-Mediterranean Rail Freight Corridor, page 189. ETC transport Consultants, 2014.

43% of stakeholders identified the need to improve the loading gauge, rendering this the most commonly mentioned component-requiring improvement, see figure 3.9. This was followed by safety and security. Maximum speed, received the smallest proportion of mentions. The category "other" included:

- Harmonization of brake laws/usage
- Use of same locomotive throughout
- Train length and weight
- Track quality
- Standardization of rolling stock.

These reasons must be regarded posing a very heterogeneous list since it is a very important issue for improvement.⁸

The development of intermodal transports on rail is increasing over time but with *increased and standardized unit profiles* in Europe as a whole, intermodal freight traffic would without any doubt increase at a higher rate. One important aspect for this conclusion is the fact that the semitrailer transport is one of the most demanded techniques of the combined transport with the highest growth during the last years, see figure 3.10.

⁸ Transport Market Study for the Scandinavian-Mediterranean Rail Freight Corridor, page 189. ETC Transport Consultants GmbH, Berlin 2014.

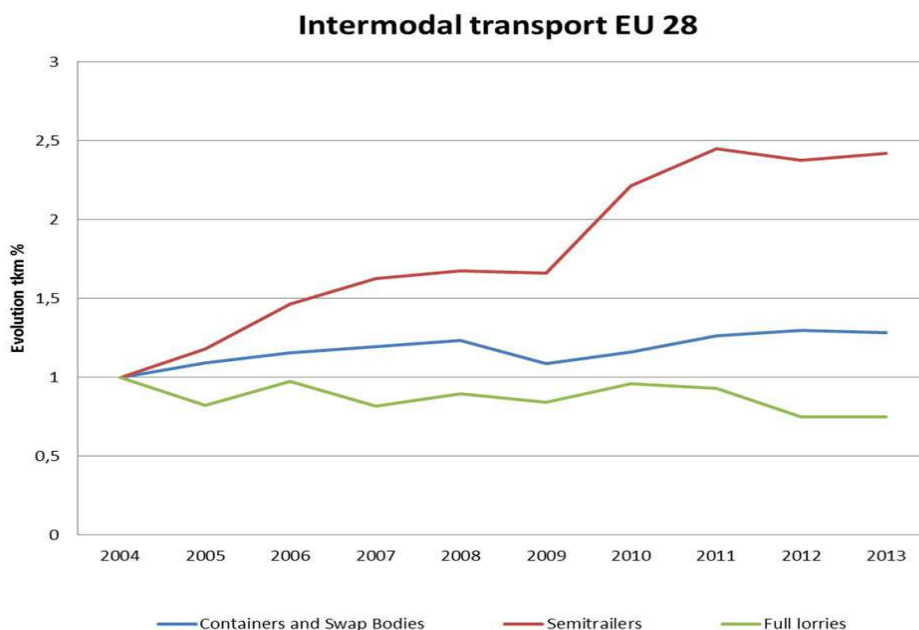


Figure 3.10: Evolution of Container Transport vs. Semitrailer Transport in EU27 (in tkm).
Source: 2014 Eurostat.

The current trends indicate a strong growth and demand for semitrailers. In second place comes the growth of long 45' domestic containers or bodies, in third place the 30' containers (mostly bulk containers, silos and tanks) for domestic transport, finally 26' containers (swap bodies and smaller tanks) are proportionally decreasing their share.⁹

The necessary intermodal gauge to transport standard semitrailers on standard pocket wagons is P/C400. This gauge is large enough to allow 4 m high semitrailers transportation. With smaller intermodal gauges, either the semitrailers have to be lower or the pocket level has to be lower¹⁰. It is most likely that a further growth for semitrailers in combined traffic to and from southern Europe (France, Spain and Italy), would be a reality if the loading gauge could include also the P/C400 code, minimum GB2. (A small loading gauge)

In northern Europe the prerequisites for transports of units with profile P/C 450 are almost fulfilled. In figure 3.11 it is shown how a 4.5m high trailer can be loaded in a pocket wagon on a railway with P/C470 gauge i.e. 4.83m allowed height and at least 2.60 m width.

As an example, a relatively large and growing part of the Swedish railway network is fully available to loading gauge C (3.60 m × 4.83 m), i.e. both height- and width-wise. An even greater part of the Swedish railway network permits loading gauge C in height, intermodal gauge P/C 450 (2.60 m × 4.83 m). See figure 4.2.26. According to a survey conducted by the Trafikverket and KTH there are only about 20 obstacles to gauge P/C 450 on roughly 6,200 km of Sweden's most important freight routes with 5 of these being eliminated by projects

⁹ Future Prospects on Railway Freight Transportation, page 89, Armando Carrillo Zanuy, Technische Universität Berlin. 2013.

¹⁰ Future Prospects on Railway Freight Transportation, page 89, Armando Carrillo Zanuy, Technische Universität Berlin. 2013.

already planned. A cohesive network for P/C 450 already exists from Haparanda to Gothenburg, Öresund and Trelleborg.¹¹

The Öresund Bridge is prepared for the Swedish loading gauge C. This “Öresund standard” is in many respects the highest in Europe. The new fixed link across the Fehmarn Belt is also being planned for this standard. The French rail administration authority has preliminarily assessed P/C 450 (2.60 m × 4.83 m) and 3.15×4.83 to be practicable applicable in northern France and wishes to collaborate on this. If the Fehmarn Belt link is also adapted, this will enable a future corridor for the transportation of both trailers 4.5 meters tall and wagonloads to be created between Scandinavia and France–Great Britain see figure 3.13.¹²

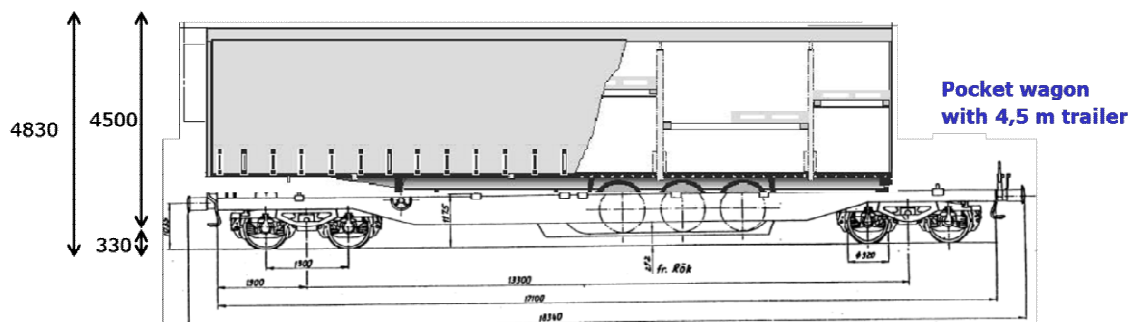


Figure 3.11: Pocket wagon with profile P/C 450 means that it is possible to load a trailer with 450 cm height with a floor height of 33cm. The total height will be 483 cm. Source KTH Railway Group.

As for today semitrailer transport are more common in countries where the loading gauges are more allowing, see figure 3.12. Increased gauge would indicate that modal split would increase in favour to the semitrailer, thus also increasing the total freight volumes transported in the future.

Sweden	Denmark	Switzerland	Norway	Finland	Italy	Austria	Germany	France	Hungary
31%	30%	16%	16%	16%	16%	14%	12%	7%	6%

Figure 3.12: Percentage of semitrailer transport over total intermodal transports. Source: Eurostat, 2014

Double stack container trains are common in US. It is possible because most railways are not electrified in US so the loading gauge is rather high. In Europe it is not easy to adopt the network to double stack trains not only because many lines are electrified but also because the gauge are more restricted in many countries because of bridges and tunnels. The Betuwe

¹¹ Roadmap for development of rail and intermodal freight transportation, page 30. KTH Railway Group. 2013.

¹² Roadmap for development of rail and intermodal freight transportation, page 30. KTH Railway Group. 2013.

line, which is a new freight line from Rotterdam, have been adopted to double stack but this has not been used yet because it is an isolated link. In Europe, it is more important to gain intermodality by making it possible to load full height trailers on a pocket wagon and to have a good length utilization of wagons and trains for intermodal transports.

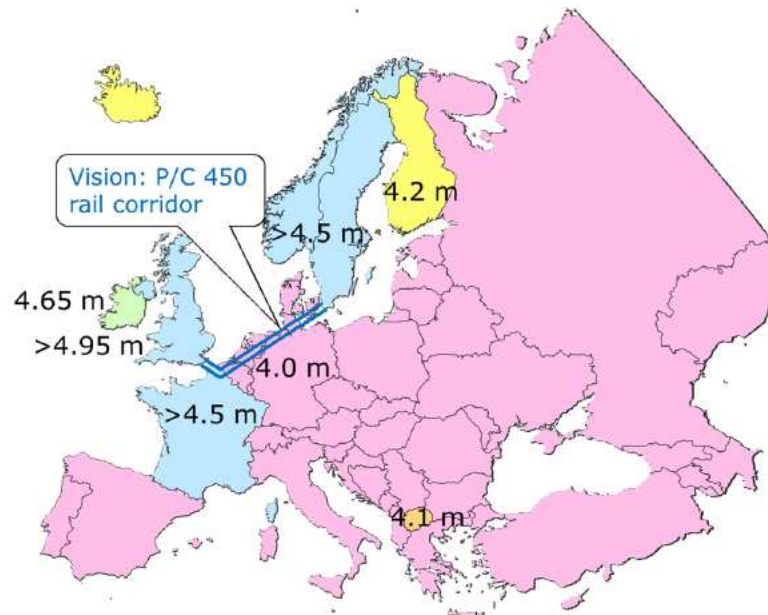


Figure 3.13: Permitted vehicle height on the road network and future railway corridor between Scandinavia and France–Great Britain (“Fran-Scan”). Source: Roadmap for development of rail and intermodal freight transportation. KTH 2013.

3.9 COMBINATIONS OF HIGHER AXLE AND METER LOAD, WIDER GAUGE AND LONGER TRAINS

According to the EU’s 2011 white paper on transports a larger percentage of long-distance freight transports should go by rail and maritime shipping. For the railways’ future competitiveness, it is therefore important that new construction and reconstruction of railways strive for as high a standard as is practically possible, as its marginal cost in the current situation is deemed to be low.

As an example, the Öresund Bridge has a standard with an α -factor 1.10 which means a permitted axle load of 25 tons, 8.3 tons weight per metre, high loading gauges and 1000-metre trains; see Table 3.14. The “Öresund standard” is, in several regards, the highest in Europe. The tunnel across Fehmarn belt is also planned for this standard, as is the Betuwe Line between Rotterdam and the Ruhr area.

The width can also be utilised by passenger trains in order to obtain more seats and/or higher comfort. Example on this is the Green Train approximately 3.54 m wide with lateral bump stops corresponding to the Regina (3.45 m). High trains – double-decker trains – higher than G1 for increased comfort can possibly be arranged with P/C 450.

	Öresund Link Network statement 2014	Fehmarn Belt Planned
Speed	200 km/h (passenger)	√
Train length	1000 m	√
Wagon weight	4000 tons	√
Loading gauge	SE-C (3.60 m x 4.83 m) planned	√
Intermodal gauge	P/C 450 (2.60 m x 4.83 m)	√
Weight per metre	8.3 tons/metre	√
Axle load	25 tons	√
Distant signals	2200 m	1800 m
Gradient	WB ≤12,4‰ (bridge), ≤15,4‰ (tunnel) EB ≤15,6‰ (bridge), ≤15,4‰ (tunnel)	≤12,5‰

Figure 3.14: The Öresund standard for infrastructure applied to the Öresund Bridge and planned for application to the fixed service over Fehmarn Bält.

A common standard that can be applied during investments in infrastructure in the future should be developed. For freight traffic, a high standard is desirable regarding loading gauges, train lengths, train weights, gradients, linear loads and axle loads. We propose that the following standard be considered in new construction and larger reconstruction:

- Maximum gradient 10.0‰–12.5‰. (In Sweden, 10‰ is applied and on Fehmarnbelt up to 12.5‰ is planned, while the Öresund Link has 15.4–15.6‰.)
- Sidings adapted as much as possible to 1050-metre trains over the longer term.
- Loading gauge 3.60 m x 4.83 m (corresponding to Swedish loading gauge C), or –if this is not possible– 3.15 m x 4.83 m with full width for the whole height (i.e. rectangular)
- Axle load 25 tonnes and linear load 8.3 tons/metre
- Train weight of approximately 5,000 tons on 10‰ gradient and 4,000 tons on 12.5‰ gradient
- The ETCS signalling system level 3 over the long term
- Maximum speed 200–250 km/h for passenger trains on lines with mixed traffic
- Freight train paths for 120–140 km/h during the day and 100 km/h at night

Most of this standard already exists on the Öresund Link, and is also planned for the fixed service under Fehmarnbelt. This is a good example for planning future infrastructure, which would make railway traffic considerably more competitive and meet EU goals for a long-term sustainable transport system.

The combination of higher axle load, meter load, wider loading gauge and longer trains sometimes means that the capacity in tonnes or volume can be remarkable extended with the right measures. Figure 3.15 gives some examples of the consequences of different standards. The lowest standard is 630m long trains (common in Sweden), 22.5 tonnes axle load (standard in Europe) and loading gauge G1 (also common in Europe). This has index 100

in terms of payload capacity in tonnes (blue columns) and volume in m³ (green columns). If the train length will be extended to 740m and the gauge to G2 the weight capacity will increase by 15% and the volume capacity by 29%.

Next step is 835m long trains, 25 tonnes axle load and the Swedish profile SE-A (width 3,40m) which will increase the weight by 50% and the volume by 66%. With a combination of today's best standards in Europe: train length 1050m, as for new built lines in in Denmark, axle load 25 tonnes as for new built lines in Denmark and Sweden, and loading gauge C as for many lines in Sweden, the weight will increase by 88% and the volume by 331%. In this case it is possible to almost double the payload in tonnes and more than triple the payload in m³.

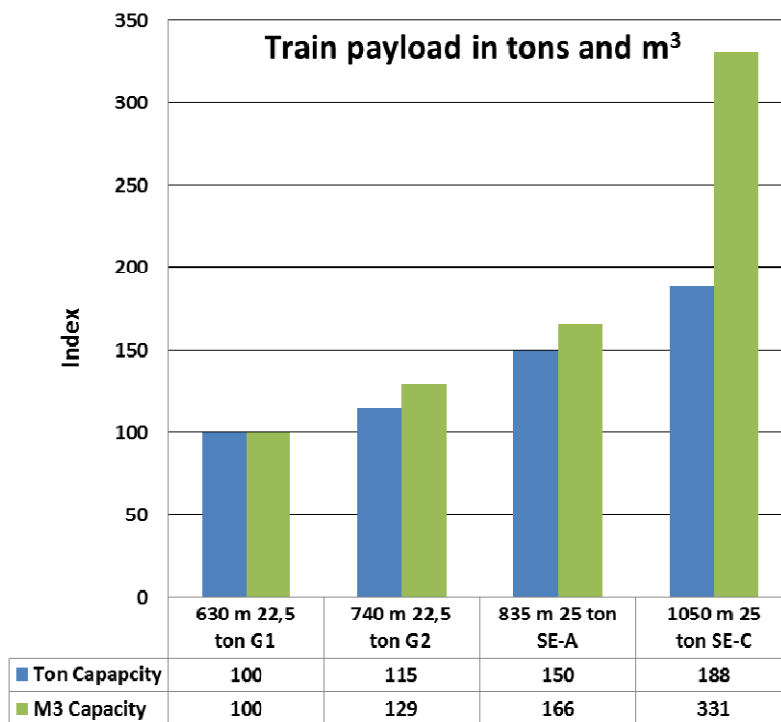


Figure 3.15: Combination of different standards for train lengths, axle load and gauge and the consequences for the capacity of a wagon-load train.

Source: KTH calculations 2015.

4 INFRASTRUCTURE

This chapter has been divided into sections. The first two: Substructure and Structures static and quasi-static loads are the dominant parameters that need to be dealt with in upgrading. In the subsequent section Rails and S&C, dynamic loads are the dominating cause of deterioration and the main parameter that need to be considered in upgrading.

4.1 SUBSTRUCTURE

In the course of time, permanent ways have been adapted to increased requirements. Ballast track today allows speeds of more than 250km/h and carry axle loads in excess of 25 tons. The substructure and earth structure, however, has not progressed in the same way. Due to future demands for faster and heavier transports (see section 3), the railway subgrade can thus experience problems, such as reduced stability, increase of settlements, and possibility of extensive vibrations. These issues have an adverse effect on the safety, reliability and economy of the railway operations. Therefore, it is necessary also to upgrade the subgrade in such a way, as it will withstand the new loading conditions.

Improvement of the subgrade could be also necessary in cases where failures occurs. In both the case of upgrading and repair, it will be necessary to first assess the substructure condition, then to evaluate existing and required subgrade quality, and finally to decide on and design strengthening measures. The following flow chart shows both scenarios and the processes involved.

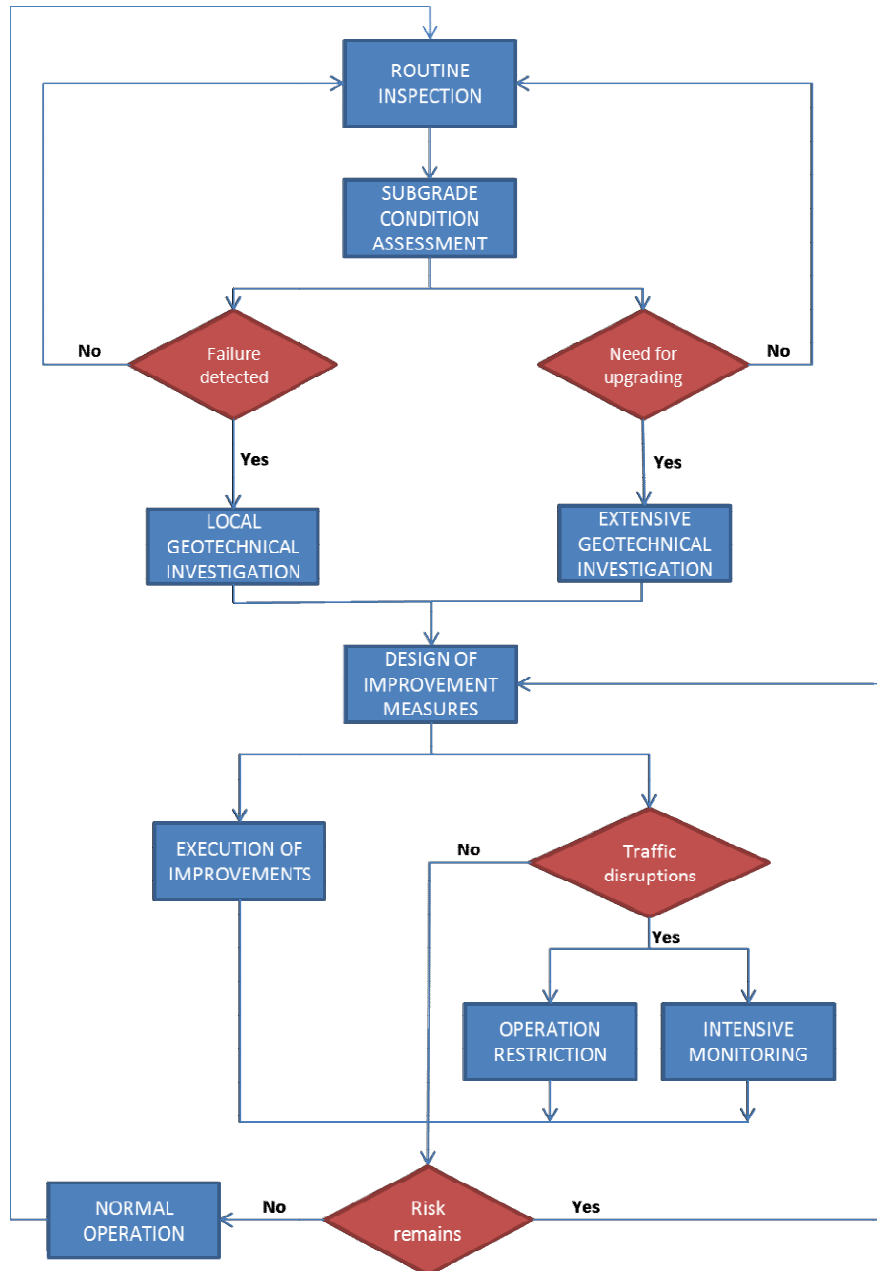


Figure 4.1.1 Flow chart for assessment, improvement and follow-up of subgrade conditions.

4.1.1 SUBGRADE ASSESSMENT

A key point in improving subgrade conditions is an understanding of what the current conditions are and what effect these will have on the operating traffic. To this end, this guideline will support IMs in subgrade assessments. One key issue is here to interpret and compare results obtained by different measurement techniques and recommend suitable methods. In for example INNOTRACK a number of techniques including various geophysical methods, rolling stiffness measurement methods and methods able to obtain point-wise depth dependent profiles of the subgrade resistance were investigated.

Knowledge about the condition of the different track components along the railway lines is vital in performing optimal track maintenance. This knowledge is gathered with the help of inspections as well as manual and automatic measurements. Regarding railway substructure, most of the imperfections are visible as track geometry irregularities. Therefore, the most important inspection is an accurate measurement of spatial and temporal track geometry profiles. Furthermore, to be able to determine the root cause of a problem or to conclude how changed traffic conditions will affect the track, more measurements or visual inspections might be needed.

The following sections deal with the assessment of the railway substructure. First, the factors and mechanical parameters, which can better inform about the substructure condition are discussed; and then, different methods and techniques appropriate to assess the substructure quality are described.

4.1.1.1 Assessing substructure conditions

Railway tracks inevitably cross-regions of less appropriate subgrade materials, as well as high stiffness structures (e.g. bridges, tunnels etc.). Hence, the occurrence of variations in vertical stiffness is frequent. Track stiffness is a mechanical parameter deserving special focus, both from a theoretical and practical point of view. In contrast to infrastructures like bridges, which in some sense are known structures in terms of materials and can be subjected to visual inspection, the substructure of the track is often unknown and only limited visual inspection is possible. The possibility to measure the vertical track stiffness could here be a help for determining which sites along the track that need some kind of substructure reinforcement or further investigation.

4.1.1.1.1 Subgrade conditions and track geometry deterioration

Permanent deformations (settlements) along the track and in transition zones are related to the geotechnical properties and conditions of substructure materials. Drainage plays a relevant role regarding the stability of the subgrade: the layers of sub-ballast and subgrade generally contain moisture. These layers perform best under cyclic load when sustaining an intermediate moisture state. The change of the moisture level over time affects the stress state in the granular material leading to variations in pore pressures and – as a result – in overall material behaviour. In regions subjected to frost drainage this effect can be dramatic.

Under load, track settlements often tend to be progressive and may increase in magnitude over time. This effect is due to dynamic effects and induced by the trains as well as by the temperature effects. Two parameters related to the substructure especially influence the track geometry degradation:

- Differential settlement between adjacent sections of the track;
- Stiffness characteristics of the subgrade.

Differential settlements occur whenever two segments of the railway track settle at different rates. The running surface deviation that develops in this situation can contribute to dynamic loads as high as some three times the static wheel load. Settlements also at plain track can be highly variable due to geotechnical issues affecting the subgrade performance. These can be low strength soils, deficient soil compaction, poor drainage, and erosion. Moreover, environmental factors such as wet/dry and freeze/thaw cycles can also affect the subgrade behaviour. In summary, settlements in the granular materials of the substructure as

well as the deformations in the superstructure components are influenced by the track operational conditions, the geotechnical conditions of the involved materials in terms of compaction and saturation.

The stiffness characteristics of the subgrade can also have a relevant impact on track degradation, as discussed in greater detail in the following section.

4.1.1.1.2 Track stiffness and its impact on the mechanical behaviour of the track

It is generally assumed that increasing track vertical stiffness contributes to better support the applied traffic loads (static and dynamic) thus reducing track deterioration geometry rates. A minimum track bearing capacity and stiffness is required in order to ensure that the rolling stock would never submit the structure to higher loads than the elements bearing strength, if that happened, faster deterioration rates would exist derived from excessive track settlement and high bending stress in the rails might occur. However, excessive vertical stiffness can also be a problem due to the increase of track dynamic loads, as shown since the results published by Prud'homme (1970). Based on this reasoning, in the last decades, several authors and research projects have been discussing in the literature the importance of track vertical stiffness and which should be its appropriate or optimum value for a given railway track. Example on this is, e.g. Sussman et al., 2001; Dahlberg, 2003; Teixeira, 2003; Hunt, 2005; López Pita et al. 2004; Iwnicki, 2006; Li and Berggren, 2010; Puzavac et al., 2012; Andersson et al., 2013. These are mentioned from the last decade and among many others. The global conclusion is that a high track stiffness of the substructure is recommended, although modulated with a good elasticity of the superstructure to reduce dynamic loads and its impact (e.g. acting soft railpads, sleeper spacing, sleeper pads...).

Track stiffness of the substructure is highly influenced by the natural geological profile characteristics. Notice that the substructure may include several layers of soil material with different characteristics, hence its track stiffness will be the combination of stiffness's of the different soils. The subgrade is in fact the most variable element of the track structure: it consists on different materials, which may be vulnerable to weather conditions and suffer seasonal thickness variations due to hydrologic and climate conditions and therefore induce variations on its elasticity modulus.

Since the subgrade condition is subject to weather, extremes of temperature and moisture, the track stiffness may also vary with seasonal changes.

4.1.1.1.3 Influence of track stiffness and its variations

Track stiffness varies randomly along the track. Variations between bridges, tunnels and natural platforms are the most common examples of important stiffness variations. However, even in plain track on a natural platform important stiffness variations may often occur. Several causes can contribute to these stiffness variations: transition zones between landfill or excavations sites or other types of construction, geology of the site, embankment material type, thickness of track bed layers, deficient compactness, presence of culverts or small underpass structures, moisture content on the subgrade, harsh seasonal climate conditions where freeze-thaw cycles may induce variations on track stiffness if the track is not correctly designed for dealing with such conditions, etc. Surprisingly, research has shown that even in high-quality railways, where the design and construction of the track structure are carefully

monitored, the same variability in support conditions appear (Teixeira, 2003). Several authors, such as Li and Berggren (2010), have demonstrated that varying track stiffness causes variations in dynamic loads. This usually causes differential settlements and consequently differential track geometry deterioration that continually grows if not submitted to maintenance. Experimental measurements have shown that larger geometry defects tend to coincide with sections of sudden stiffness variations (Berggren, 2009; Burrow et al., 2009).

A recent study performed by IST (Ribeiro, 2014) in collaboration with EBER Dynamics AB further confirmed the possible influence of vertical track stiffness on track geometry degradation. The study assessed data from stiffness measurements recorded in 2012 with the IMV100 (see below) against the maintenance history and longitudinal level deterioration rates of the Iron Ore line. The analysis suggests a correlation between higher rates of degradation of local defects and sudden stiffness variations, as well as evidence that higher stiffness variations tend to have a correlation to higher frequencies of maintenance operations.

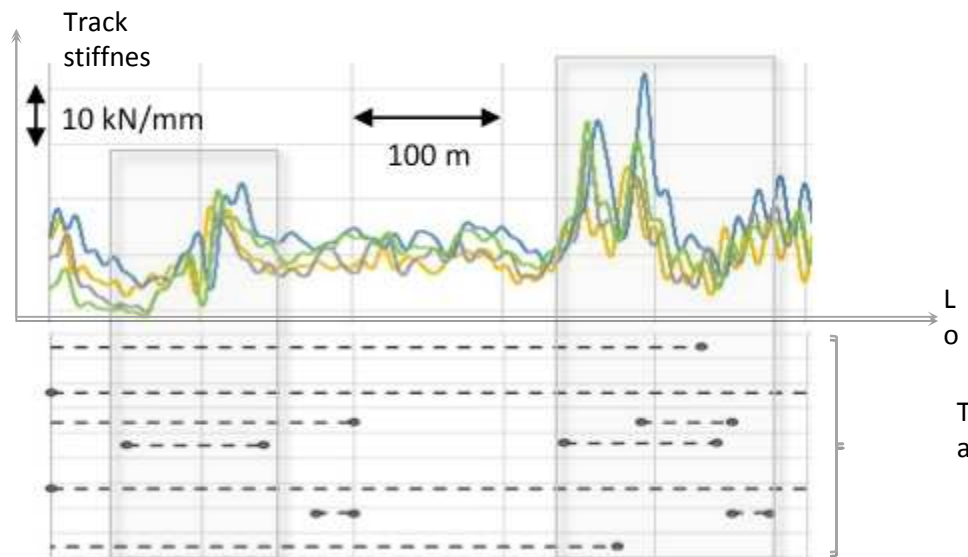


Figure 4.1.2 – Registers of tamping operations associated to sudden stiffness variations.

Source: Ribeiro (2014)

Given the important role played by subgrade stiffness, the measurement of track stiffness has a great potential for track maintenance management as an indicator of the root causes at problem sites as well as revealing needs for track upgrading especially when the accommodation of higher speeds and/or axle loads are considered.

4.1.1.1.4 The role of track stiffness measurements in track maintenance and upgrading

Measurement of track irregularities is the most used automated condition assessment technique in railway maintenance. Most problems with the track (at least the ones concerning ballast and substructure) will be visible as track irregularities. However the root cause of the problem cannot be detected with the help of track geometry measurements. In these cases, track stiffness measurements is a valuable complement. However, it is also important to be aware that track stiffness measurements cannot indicate the root cause of a problem in all cases.

The main causes of subsoil problems are described by Li and Selig (1995) as:

- Load factor: there are two types of load factors, material dead-weights and repeated dynamic loading;
- Soil factor: a problematic subsoil will not generally consist of coarse-grained soils (gravel and sand) but most likely will be fine-grained soils (silt and clay) due to the lower strength and permeability of the latter materials;
- Soil moisture: almost every subsoil problem can be attributed to high moisture content in the fine-grained soil. The presence of water in the subsoil can reduce the strength and stiffness of soils dramatically;
- Soil temperature: soil temperature is of concern when it causes cycles of freezing and thawing.

Most often, a subsoil problem area will involve several of the abovementioned factors.

In general, the common track problems may be related to track stiffness and maintenance actions as summarized in Table 4.1.1.

Table 4.1.1 – Relation between stiffness and track problem/maintenance.

Parameter	Problem	Maintenance/Rehabilitation
Low track stiffness	Poor or weak subgrade soil	Substructure design Stabilization of subgrade soils Reinforcement of soil strength
Variable track stiffness	Variable track support (stiffness or modulus)	Matching rail seat pads Substructure redesign

Whenever low values of track stiffness are recorded, it most likely indicates a weak subgrade condition at the location since subgrade properties most influence the value of track stiffness. Maintenance for these conditions can involve design and reconstruction of the track substructure layers, possibly in conjunction with some geotechnical measure regarding soil reinforcement (Sussmann et al., 2001).

Variable track stiffness indicates conditions in which a rapid variation in track support at the design wheel load occurs. The variation in track stiffness results in distortion of an initially smooth running surface due to differential elastic track deflection. Methods used to improve this track condition include design of rail seat pad stiffness, design of the substructure, and use of ballast mats. Suiker and Esveld (1997) and Lundqvist (2005) have presented research

within the area of stiffness variations. López-Pita and Teixeira (2001; 2002) and Grainger et al. (2001) also argued for the importance of adopting a maximum variation criterion, i.e. the need to assure the homogeneity of track stiffness.

4.1.1.1.5 The role of track stiffness measurements in mitigating noise and vibration

Possible noise problems originate most often from the superstructure and especially the wheel–rail contact. The main contributors are rail and wheel roughness, track receptance, wheel receptance and the decay rate of the rail. For noise, only frequencies in the audible band are of interest (20 – 20000 Hz). The upper limit as seen from the track is 2000 – 5000 Hz (Hildebrand, 2001; Aknin and Chollet, 1999). There is of course noise at higher frequencies, for example wheel and braking noise, but the track (rail) is not dominant. Jones and Thompson state (2001) stated that the noise radiated by the track is strongly related to the stiffness of the rail fastening, in particular the rail pad between the rail and the sleeper. Soft pads cause the rail to become uncoupled from the sleeper, which minimizes noise from the sleeper but allows the rail to vibrate more freely. Waves can therefore travel over a greater distance and the noise from the rail is increased. Conversely, with stiff pads the contribution from the rail is reduced but that from the sleeper is increased.

Concerning ground borne vibration problems, we can divide the vibration path into source (train/track), way of propagation and receiver (buildings etc.). With track stiffness measurements, it is obvious that only the track can be measured and not the way of propagation between the track and the receiver. Ground borne vibrations are propagated through the soil, meaning that a soil with low stiffness and/or a clear resonance can propagate vibrations easily. However, the measured track stiffness is the total stiffness and does not only consist of the soil stiffness. A vibrating clay soil with a high embankment can have the same average stiffness as a normally vibrating gravel soil with a low embankment. Smekal et al. (2003) showed an example of how the low frequency soil properties can be filtered out with the help of stiffness measurements at different frequencies, thus leading to identification of potentially problem sites regarding vibrations. Berggren (2005) demonstrated how soft soil properties could be estimated from dynamic stiffness measurements. See also section 4.1.3.

4.1.1.2 Methods for subgrade quality assessment

Several methods might be used to assess the quality of the substructure. The main techniques regarding measurement of substructure stiffness and geophysical inspection of the subgrade condition are discussed in the following sections.

4.1.1.2.1 Track substructure stiffness measurement

Different ways of measuring vertical track stiffness have been developed by organisations throughout the world. Until recently, the most used methods consisted in standstill measurements by simulation of vehicle loadings or measurement of parameters associated to subgrade material properties, which were often used for research purposes (Berggren, 2009; Hunt, 2005). In contrast, rolling measurement techniques have the potential of being used on a regular basis for maintenance purposes. They have only been applied successfully in the last years and research for improvements is still on going.

The research and improvement of measurement techniques, especially rolling measurement techniques, will allow getting an improved data interpretation. These results will be a key for planning optimal upgrading and maintenance activities and consequently optimal use of the maintenance budget. The appropriate equipment must be selected based on the frequency of measuring track stiffness (Berggren, 2009). As the same author states, “static and low frequency dynamics of the track are mostly related to geotechnical and geodynamical issues” such as bearing capacity of the subgrade, ground borne vibrations and soft-soil related problems while “high frequencies relate to problems associated with noise and train-track interaction forces”, being useful when considering the upper layers (Andersson et al., 2013).

Standstill measurements

Standstill measurements are made at discrete intervals. According to Berggren (2009), there are four main standstill devices used for measuring vertical stiffness:

i. Instrumentation of track components

The instrumentation of track components – e.g. with the attachment of accelerometers, displacement sensors or strain gauges – to measure the effect of the rolling stock on the structure in a set of sleepers, and/or rails, is the simplest method for measuring the vertical track stiffness. Stiffness is then related to the obtained load-deflection diagrams.

ii. Impact hammer

The controlled impact of a handheld hammer equipped with a force transducer on the head, to measure impulse load, on an instrumented rail or sleeper is another way of standstill vertical stiffness measurement. This technique consists on measuring the impulse force of the hammer and the acceleration registered on the transducer. In this manner the transfer function on the impact surface is obtained. Depending on the material used to cover the hammerhead, different frequencies can be recorded (from 50 Hz up to around 1500 Hz). For noise and vibration work this is a reasonable method but not for track deterioration investigation (Hunt, 2005).

iii. Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer was adapted by the railway industry after being originally developed for evaluation of the static stiffness of the trackbed layers, including the subgrade, on highways (Bold, 2011; Shein et al., 2007). The testing method consists in dropping a mass from a known height onto a footplate placed upon a sleeper disconnected from the rails, to simulate traffic loading. Geophones placed at various distances from the point of impact automatically record the deflection through the measurement of the surface velocity and its integration (Berggren, 2009). The requirement of disconnecting the sleepers for applying the test makes this procedure very time-consuming and expensive (Bold, 2011).

iv. Track Loading Vehicle (TLV)

The aim of this technique is to use hydraulic jacks for loading the track with the equipment’s own weight and then measure the applied deflection. The latter can be made through a laser system (McVey et al., 2005). The weight is usually applied on the rails but similarly to the Falling Weight Deflectometer technique, sleepers can also be disconnected and loaded. Some available equipment is able to excite dynamically the track up to 200 Hz (Berggren, 2009).

The Swedish TLV (Figure4.1.) has a weight of 49 tons and can load each rail statically up to 150 kN and excite dynamically up to 200 Hz. In addition, lateral stability/stiffness can be measured.

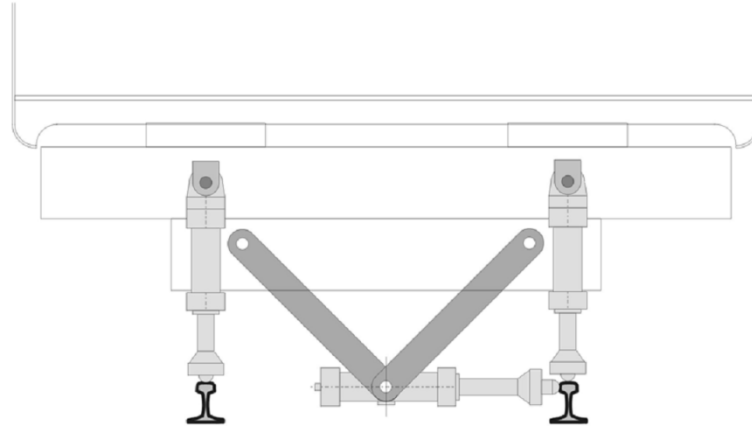


Figure4.1. 3. – The vertical and lateral hydraulic actuators of the Swedish TLV.

Rolling measurements

If track vertical stiffness is to be investigated over large distances or even the whole track, standstill measurement are virtually impossible to carry out, as they are very time-consuming and expensive. Therefore, models of rolling measure vehicles have been developed by several organisations (Berggren, 2009). There are two main development approaches: one is based on dynamic measurements on a single axle, and the other is based on laser-measurement of the vertical deflections under applied loads.

In all approaches, it is important to discriminate different conditions or defects that may influence the continuous measurement of track vertical stiffness. That discrimination is made based on the wavelength (λ) interval of some of these defects (Berggren et al., 2002), Table 4.1.2. This is an important aspect for choosing excitation frequencies for measuring (Berggren et al., 2005).

Table 4.1.2 – Wavelengths of imperfections that influence stiffness measurement (Berggren et al., 2002).

Imperfection	Wavelengths
Track geometry irregularities: Longitudinal level Cross level	$1 < \lambda < 150 \text{ m}$
Rail shape faults: Rail corrugation/roughness Welds/joints	$0 < \lambda < 3 \text{ m}$
Discrete foundation of the rail by sleepers	$0.5 < \lambda < 0.7 \text{ m}$

Below some examples of measurement techniques for rolling vertical track stiffness employed in different countries are summarized:

Sweden

The RSMV (Rolling Stiffness Measurement Vehicle) operates at maximum speeds of 50-60 km/h and can measure dynamic stiffness up to 50 Hz (Berggren, 2006, 2009; Berggren et al., 2002). It was developed in Sweden, by Banverket (now Trafikverket), initially as a prototype trolley that could be used together with the Swedish TLV (Berggren et al., 2002). After successful tests, it was upgraded and built on a modified two-axle freight wagon. The measurement technique is based on a hydraulic system that dynamically excites the track at one or several selected discrete frequencies, with two oscillating masses connected to force transducers mounted on top of the axle box of one of the wheel axles (Figure 4.1.4). The applied forces are a minimum of 180 kN regarding the static axle load and a maximum amplitude of 60 kN (30 kN per oscillating mass) for the dynamic axle (Berggren, 2009; Berggren et al., 2002, 2005). Resulting forces and accelerations are measured by force transducers and accelerometers placed on both axle boxes of the wheel-set enabling the estimation of track stiffness at different excitation frequencies (Berggren, 2009; Berggren et al., 2005).

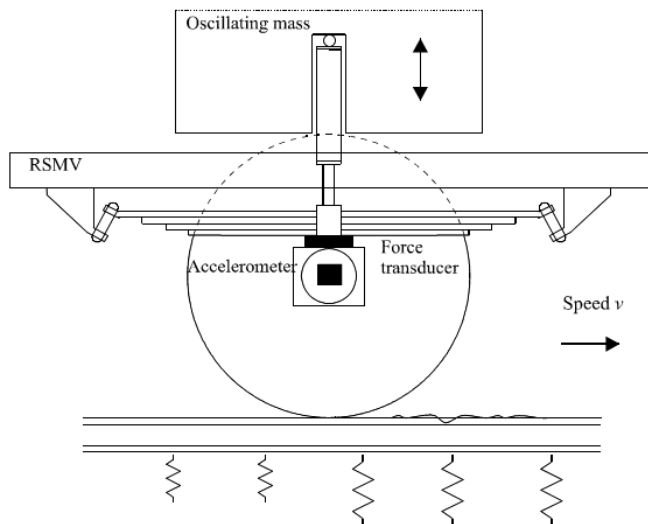


Figure 4.1.4 – Measurement principle (one side only) of RSMV (Berggren et al., 2005).

USA

The Transportation Technology Center, Inc. (TTCI) developed a track loading vehicle (TLV) for measuring track stiffness at standstill and at speeds up to 16 km/h, when coupled with an empty tank car. For standstill measurements, the TLV works as previously described. The lighter vehicle has a pneumatic system for loading the track and a measurement system for measuring deflections. For obtaining the track stiffness, the difference between the measurements with heavy (178 kN) and light (44 kN) loads is calculated (Berggren, 2009; Burrow et al., 2009). Some results from the TTCI measurements are shown in Figure 4.1.5.

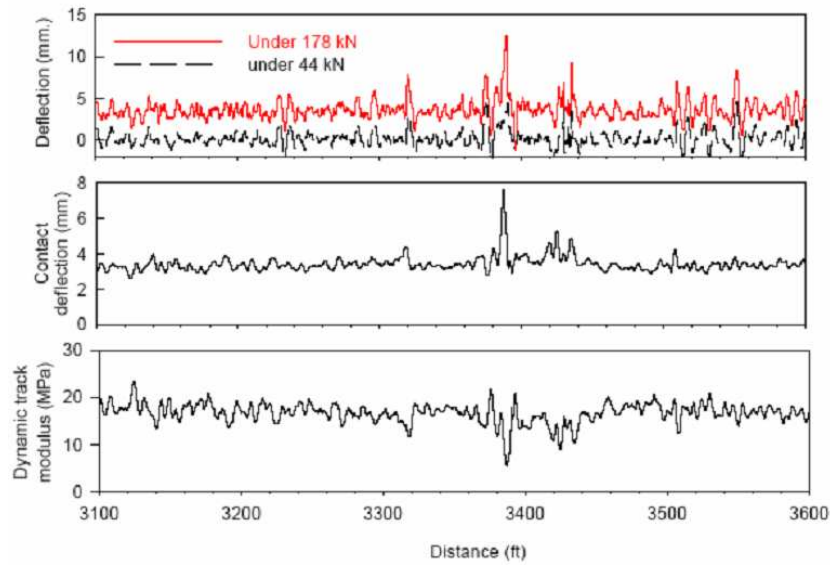


Figure 4.1.5 – Examples of results from TPCI measurements (Li et al., 2002).

China

The China Academy of Railway Sciences (CARS) developed an inspection equipment with a system that uses two cars of different weight for loading the track. The displacement is measured with two track geometry chord measurement systems. The heavier vehicle measures the track stiffness and its weight can be varied for enabling the measurement of non-linear characteristics of the track structure. The lighter vehicle is used to reduce the effect of track geometry irregularities for the recordings made by the heavier vehicle. It can measure at travel speeds up to 60 km/h (Berggren, 2009; Burrow et al., 2009).

France

The CETE-Normandie Centre created the Railway Portancemetre by adapting a technology used for monitoring stiffness on roads to railways (Figure). In an approach similar to that of the RSMV (although in a lighter version), a dynamic load is applied to the track through a vibrating wheel suspended by a spring and a damper. For road testing, 10 kN of static weight, 0.5 mm of theoretical amplitude, at 35 Hz is used. For rail testing these characteristics will be changed with increased loads (both static and dynamic) and possibilities to alter the frequency.



Figure 4.1.6 – The “Portancemetre” apparatus (INNOTRACK, 2010).

Spain

In the INNOTRACK project, procedures to measure the track stiffness based on the use of external sensors mounted on rails were developed. These are currently still in use in Spain. They have been used to detect the wheel loads and rail movements induced in track by trains operating the railway lines. The wheel loads were assessed by a method based on the determination of the maximum shear stress induced in the rail cross section by the pass of the trains, while both direct and indirect methods have been adopted for measuring rail deflections.

The measurements have been achieved with the following pieces of equipment:

- Extensometer shear bands
- Laser beam sensors
- 2 Hz geophones

The procedure was explained in detail in the document *Track stiffness evaluation procedures set up by Cedex for ADIF* within the project INNOTRACK.



Figure 4.1.7 – Laser system used for measuring rail deflections directly



Figure 4.1.8 – Geophone of 2 Hz provided with the adequate support to be clamped to the base of a standard rail

Outcome of different measurement techniques

Vertical track stiffness measurements are unlikely to match exactly if two or more methods are used to test the same track. Some of the circumstances that might be different are summarized below (Berggren, 2009; Hosseingholian et al., 2009):

- The *static preloads* applied by each measurement equipment differ and those differences are likely to cause different recorded stiffness values;
- Some of these techniques are based on running wheel-sets that load and excite the track with the vehicle's speed. Depending on the running speed, the track will be excited by a specific frequency. As the dynamic track stiffness is frequency dependent, the *excitation frequency*, hence *speed*, will make the determined stiffness vary;
- The *spatial resolution* from each measurement technique may be different;
- To calculate the rail deflection based on deflections not measured directly under the wheel-set, approximations are invoked in mathematical models. These may introduce uncertainty in obtained results. Therefore, *model dependency* is another factor that may lead to variations in measured vertical track stiffness;
- *Track geometry irregularities* are likely to influence stiffness measurements since most of the available equipment usually measures track stiffness through sensors that obtain results that are a combination of deflection due to track flexibility and track geometry irregularities (mainly longitudinal level).

4.1.1.2.2 Geophysical inspection of substructure defects

Geophysical investigations are based on observation of changes in physical fields. Research and retrieval of information from archival reports enables the set-up of an optimal combination of geophysical methods for an efficient diagnose. Based on the problem nature, the methods listed below are used in full combinations, incomplete combinations or separately.

- GPR (Ground Penetrating Radar);
- Resistivity profiling;
- Multielectrode (resistivity) measurements;
- Seismic methods;
- Gravimetry.

GPR (Ground Penetrating Radar)

Ground penetrating radar (GPR) can provide a rapid, non-destructive measurement technique for evaluating the substructure condition of a railway track. Methods of applying GPR to railways have been developed to provide a continuous evaluation of the track substructure conditions relative to subsurface layering, material type, moisture content and density. Typically, for railroad applications the GPR equipment is mounted on a rail-vehicle with clearance between the antennas and the track surface, which permits continuous measurements to be made from above the top-of-rail at normal vehicle speeds (Figure4.1.9). Usual penetration depth in performing measurement beyond the track is 8 m (maximum) and 2.5 m within the track skeleton.



Figure4.1.9 – GPR rail-setup with three pairs of 1-GHz horn antennas (Selig et al., 2003).

The GPR method requires transmitting pulses of radio energy into the subsurface and receiving the returning pulses that have reflected off interfaces between materials with different electromagnetic properties. This is illustrated in Figure 4.1.10. It is a pulse echo technique using radio energy as shown in Figure 4.1.10b. Antennas are moved across an area emitting a continuous series of radar pulses, and thus receiving a profile of the subsurface.

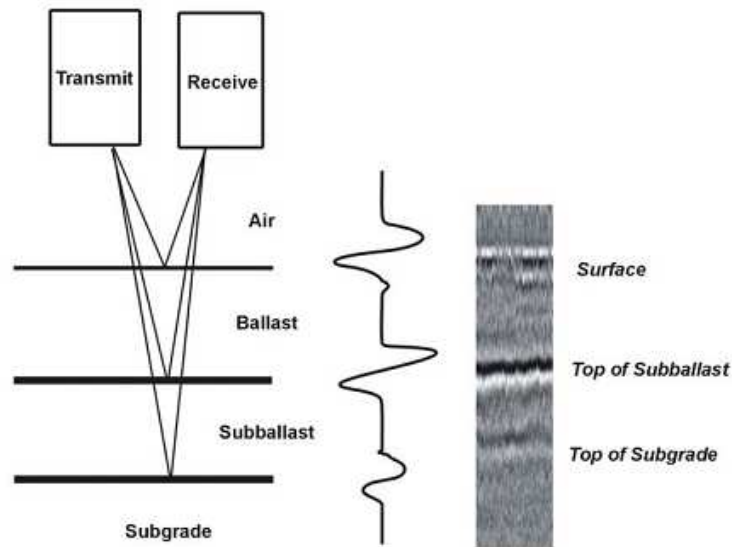


Figure 4.1.10 – The generation of a GPR profile: a) Transmission principles; b) single GPR scan (Selig et al., 2003).

The key material properties influencing the measurement are the dielectric permittivity, the electrical conductivity, and the magnetic permeability. These control the velocity with which the radio pulses travel through the material and the decay in amplitude of the pulse with distance. The pulse travels faster through a low dielectric permittivity material than a material possessing a higher dielectric permittivity. In contrast to permittivity, the conductivity of the material dictates how quickly the pulse of radio energy decays in amplitude (attenuates) with distance. It thus controls how deep the pulse will penetrate. These two properties are the mostly influential.

Reflections of the GPR pulse occur at boundaries in the subsurface where there is a change in the material properties. Only a portion of the pulsed signal is reflected and the remaining part of the pulse travels across the interface to be reflected back to the receiver from another interface boundary. The time it takes the pulse to travel through the layer and back is controlled by the thickness and properties of the material. The travel time between upper and lower boundaries of a layer can be used to calculate the layer thickness by employing a known velocity.

Material and moisture variation within the track substructure causes distinct material differences that are easily recognized by GPR. Water has the highest dielectric permittivity of the materials found in substructures. It produces a strong effect on the GPR profiles. In an area where water has been trapped in the substructure, the dielectric permittivity is increased, resulting in a stronger reflection than if the material was more dry. The GPR profiles are manipulated to extract layer thickness and permittivity (related to the material density and water content).

The implementation of predictive maintenance policies requires firstly a detailed knowledge of the state of the railway platform, and secondly a comparison over time to determine performance thresholds. To achieve these goals a non-invasive diagnostic tool is needed. This tool must be able to investigate continuously large sections of track with sufficient resolution and high sampling rates. In this regard GPR has been proven in the past to be the most suitable choice.

GPR is a shallow geophysical technique capable of generating 2D- and 3D-pseudoimages of the subsurface structure. The advantage over other conventional techniques such as boreholes or trenches is that it is a non-invasive method of investigation, fast and versatile, with acquisition speeds up to 80 km/h.

The propagation wave speed depends on several properties of the medium that produce absorption and scattering of the signal. These variations on the pulse shape can be directly related to the various conditions of the platform. The most important parameters whose properties can be investigated with GPR in railway applications are:

- Thickness of the different layers (ballast, sub-ballast, subgrade).
- Degree of contamination of the ballast
- Moisture content of the ballast
- Possible settlement areas

To determine the thickness of each layer it is necessary to identify first the reflector for each interface (e.g. ballast–sub-ballast interface) and then the pulse velocity in that layer. Usually, the velocity is obtained by correlating the GPR information with data obtained from direct sampling of the platform (trenches or boreholes). The traditional approach is based on the interpolation of sparse data throughout long distances with the associated uncertainties. With the aid on GPR data this approach is no longer necessary and continuous information can be derived. It is also possible to directly measure the velocity of the GPR pulse in each of the layers using variable offset measurements like WARR (Wide Angle Refraction and Reflection) or CMP (Common Mid-Point) (Jol, 2009). Such tests were done manually in the past making data collection painfully slow. The use of modern antenna arrays significantly reduces data acquisition times, which makes these kinds of techniques very valuable.

GPR pulses in ballast are affected by contamination from the fine-grained material from the subgrade. The most obvious effect is an attenuation in higher frequencies (Leng & Al-Qadi, 2009). Other visible effects are the appearance of reflectors inside the ballast, the loss of intensity of the reflector at the base of the ballast and a decrease in propagation velocity.

The pulse velocity depends on the composition of the materials and the degree of moisture and contamination (Figure 4.1.11). In the case of clean ballast, the velocity ranges between 0.12 m/ns and 0.21 m/ns. For ballast contaminated with fine-grained material, it ranges between 0.08 m/ns and 0.12 m/ns (Göbelet al., 1994). This property makes pulse velocity determination of outmost importance.

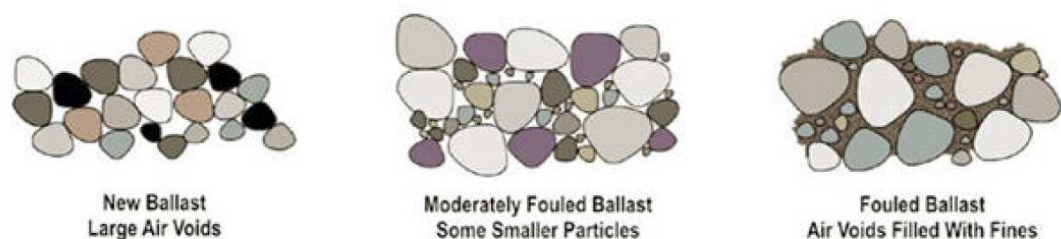


Figure 4.1.11 – Different degrees of contamination in ballast

Water content is one of the parameters that mostly affects the GPR pulse (Huisman et al., 2003) producing a large reduction of the pulse velocity and the appearance of reflectors of high amplitude and low frequency. Moreover, the thickening of the ballast layer indicates

subsidence areas. The investigation of these zones by three-dimensional models contributes to detailed information of the morphology and extension, which allows considering the intensity of the deformation.

To determine the most suitable GPR system from those available in the market, a series of tests were carried out in Spain. Equipment from three different manufacturers were tested. The first test was carried out in June 2009 with a Ground Coupled system (250 and 500 MHz) from MALA Geoscience. Data was acquired every 2cm referencing the data with a submetric GPS. The second system was the Norwegian built 3D-Radar composed by an array of 21 antennae spaced 7.5cm apart (across measuring direction) with step-frequency technology allowing acquisition of data between 100 and 3000 MHz. The GPR array was installed in the front of an ADIF maintenance train (Figure 4.1.12). Tests were performed at low speed (4 km/h for maximum resolution) and high speed (80km/h for low resolution). Positioning was achieved using a submetric GPS. The last system employed in this comparison was from the Italian manufacturer IDS and composed of three aircoupled 400 MHz antennae. The system was also installed in front of an ADIF maintenance train. Measurements were done every 6 and 12cm at acquisition speeds between 60 and 80 km/h.



Figure 4.1.12. 3D-Radar GPR system tested mounted on the front of an ADIF maintenance train.

GPR ability as an efficient tool for ballast and subgrade condition assessment was tested. CMP measurements allow determining the pulse velocity inside the ballast layer. Using this parameter, the total thickness of the ballast layer and its degree of contamination can be determined.

Knowledge about ballast thickness variations with time will allow quickly identifying areas under subsidence and adopting correction measures before further major defects are developed.

The final goal is to efficiently plan maintenance works and to increase the safety of day-to-day operations. Pulse velocities are very sensitive to humidity and contamination of the ballast layer. This property makes pulse velocity determination a tool to pin down areas prone to suffer defects in the near future.

Finally, a better understanding of current ballast and subgrade conditions is crucial to understand future evolution and to plan maintenance works ahead of time (even before defects appear). This will reduce economical costs and increasing operational reliability levels.

4.1.1.2.3 Long term settlements

In 2007 an investigation “Long term settlements in the track bed and subsoil” was carried out. It was a follow up from the EU-project Eurobalt II. Banverket continued to perform long-term measurement at Swedish test sites. See more in chapter 4.3.

Different kinds of track stiffness measures were evaluated within Eurobalt II and within INNOTRACK. These included stationary (standing still) measurements and measurements with the Track Loading Vehicle (TLV); and continuous measurements performed with the Rolling Stiffness Measurement Vehicle (RSMV) (the latest).

The measured data included the evolution in time of track stiffness as well as the difference registered in these measures for adjacent sleepers and between the left and right side of the track. Track stiffness measurements from the RSMV were registered based on noise excitations performed with a speed of 5 km/h and a frequency of 3–20 Hz that allowed measuring the stiffness magnitude at different frequencies.

The obtained results enabled the identification of problematic situations such as spots where water was trapped within the substructure layers and track sections with excessive track stiffness variations due to subsurface drainage components or trenches. The evolution of the track stiffness for a given track section and an investigation of the evolution of settlements over time also permitted the recognition of weaker sections of the track. Moreover, parallel monitoring of weather conditions (soil and air temperature, precipitation and relative humidity) allowed establishing some causality between track stiffness variations and the atmospheric conditions.

Resistivity profiling

This technique relies on the measurement of electric resistivity of ballast. The basic measurement array is characterized by the use of two inner – potential – electrodes and two outer – current – electrodes (symmetric resistivity profiling – SRP). To detect thin conductors, one additional distant current electrode is used combined resistivity profiling (CRP). Resistivity magnitude increase with a decline of water content and a decline of mineralization. Resistivity magnitude further rises in relation to the soil grain size curve (resistivity magnitude rises with increasing size of soil grains). The measurement proceeds in profiles/survey lines. The lines, if allowed by field conditions, are laid out in parallel with one another. The measured data is largely presented in the form of graphs. In the event that the measurement covers a more extensive area, it can be processed also in the form of resistivity contour lines.

The method of resistivity profiling is used, for example, to differentiate clays from gravels or to follow the profile of bedrock. The method may also be applied to observe intensively water-bearing areas and to detect tectonic lines (thin conductors). In a railway context, it is recommended to use this method to observe the geological conditions close to the railway body and/or changes in embankment composition. Usual measuring intervals at a profile is 5 m to 10 m. Nevertheless, the method is quite undemanding regarding the instrumentation and shows high working productivity. A disadvantage of the method is potential measurement interference caused by stray currents and by the presence of conductors in the ground.

Multielectrode method (resistivity tomography)

As is the case for the resistivity profiling, this method is based on the measurement of electric resistivity in stones. However, the measurement proceeds using a computer, which allows controlling the interconnected input from tens to hundreds of electrodes. The measured data are recorded, and stored in extensive and detailed databases of apparent resistivities related to the points in the studied medium. The database of apparent resistivities is further interpreted, which allows for a reliable interpretation model of resistivity conditions with much higher confidence levels than for resistivity profiling. An example is presented Figure 4.1.13. Certain special apparatuses allow interpretations in three dimensions and isoohmic cross sections.

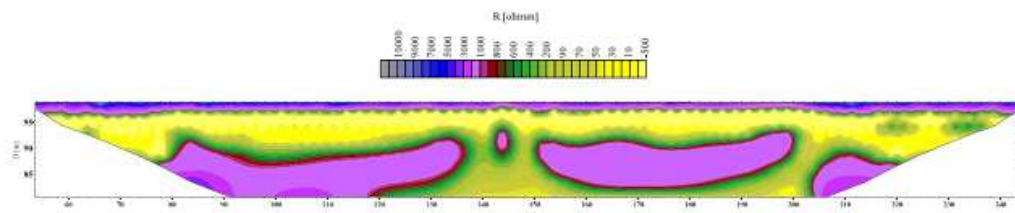


Figure 4.1.13 – Example of resistivity cross section produced by multielectrode method on track (INNOTRACK, 2010).

In a railway context, the method can be recommended for detailed observation of resistivity conditions. In performing the measurement on railway tracks, most often the employed distance between the electrodes is 0.4 to 4 m. The acquired volume of information is a manifold higher than if GPR, SRP or CRP methods are employed. Disadvantages of the method is the relatively large labour needs, and the high price of the geophysical apparatus and interpretation software.

Seismic methods

Seismic methods are based on observation and measurement of propagation of elastic waves. Seismic hammer blows on pad (blow seismics) most often excite seismic impulses. For versions with penetration depth of more than 30 m, explosives are often used as a source of seismic impulses (dynamite seismics). In addition, vibrators can be used source of seismic impulses. These are able to excite frequencies in a wide range (vibrator seismics).

In the measurement, seismic sensors (geophons) interconnected with the apparatus are used. The measurement outputs have a direct link to the geotechnical properties of the

medium: The velocity of seismic wave propagation is dependent on bulk density and modulus of elasticity. Rising values of the modulus of elasticity are associated to a medium showing higher solidity (the highest values of modulus of elasticity are shown by solid rocks and low values of modulus of elasticity correspond to soils). It is recommended to include seismics, especially in detailed measurement phases where shorter track segments are examined. A disadvantage of the seismic measurements is that they are labour intensive, have high processing complexity and costly.

Gravimetry

This method is based on highly accurate measurement of changes in gravity. To introduce topographical corrections, it is necessary to simultaneously perform accurate levelling. The works proceed at profiles (survey lines) and also in a grid. Solid objects, such as elevations of solid rocks, show positive anomalies. Deficits of mass (such as cavities, increased porosity) show a resulting negative anomaly. A characteristic output of gravity measurements are graphs with the course of the measured and corrected gravity values, or interpreted gravity models. The models express the measured distribution of mass in a way best fitting the measured values.

The measurement interval at the profiles on railway tracks largely ranges between 0.5 and 20 metres. The measurement results have a direct relation to bulk density of the studied structures. The advantage of the work with a gravimeter is a relatively small effect of the presence of stray currents on the measurement. The final data processing requires the availability of not only the gravity data, but also precise hypsography of all measured gravimetric points and a topographic plan of the broader surroundings of the track.

4.1.1.2.4 Geomechanical investigation of substructure problems

The depth of the substructure that influences the track stiffness is at least several meters below the top of the subgrade. Given that subgrade resilient modulus vary by a large amount from point to point along the track, the subgrade plays a central role in track global stiffness and its overall performance. Within the process of substructure assessment regarding upgrading needs, it is fundamental to be able to perform an adequate analysis and prediction of the track performance as well as to identify the main causes of subgrade related performance. Hence, in order to correct substructure problems, a previous knowledge concerning of root causes is required.

The necessary extent of the knowledge related to the substructure condition depends on the specific problem and might include relevant information such as the depths of the distinct soil or granular layers; the identity and physical properties of the materials; and the groundwater conditions. Nevertheless, the above information is usually not known. Hence, a field investigation, often supplemented by laboratory tests, is required. The following sections identify means for obtaining information related with the causes of the substructure problems.

Characterization of major substructure problems

The root causes of an unstable substructure might be associated to any of the ballast and subballast layers or to the subgrade. Very often, unstable substructures are also associated with poor drainage. Due to poor drainage, the trackbed layers may become weak and deform

excessively. This leads to poor track geometry, which in turn causes higher wheel/rail forces (INNTRACK, 2010).

In the ballast layer, fouled ballast is often the major problem given that it has very high stiffness and little damping, and therefore is not capable of accommodating high wheel/rail forces associated with a poor track geometry. Moreover, a saturated subballast layer due to fouling materials and poor drainage can lose its strength under repeated dynamic wheel loads, thus representing a source of instability for the substructure.

A subgrade built with soft or marginal soil can become unstable either in a progressive manner or suddenly. Sudden failures rarely occurs, unless there is a dramatic change of environment (such as high rainfall and flooding) or load conditions (such as a large increase in wheel loads). However, progressive deformation (shear) is relatively common whenever the subgrade is built with soft or marginal soil types. This progressive deformation can become rapid, leading to rapid track geometry degradation when speed or axle load increases since the bearing capacity of the subgrade soils may not be sufficient to withstand the stresses caused by the increased traffic loads. In addition, poor drainage or entrance of water into the subgrade may reduce soil strength, leading to excessive deformation or unstable substructure.

Table 4.1.3 presents the causes and features associated to major subgrade problems. Relating a problem to its root cause is of great importance when assessing the need for rehabilitation or upgrading of the substructure.

Table 4.1.3 – Major subgrade problems, causes and features (after Selig, 2004).

Type	Causes	Features
Progressive shear failure	<ul style="list-style-type: none"> - Repeated loading over-stressing subgrade - Fine-grained soils - High water content 	<ul style="list-style-type: none"> - Squeezing of subgrade into ballast shoulder - Heaves in crib and/or shoulder - Depression under sleepers trapping water
Excessive plastic deformation	<ul style="list-style-type: none"> - Repeated loading of subgrade - Soft or loose soils 	<ul style="list-style-type: none"> - Different subgrade settlement - Ballast pockets
Subgrade attrition with mud pumping	<ul style="list-style-type: none"> - Repeated loading of subgrade stiff hard soil - Contact between ballast and subgrade - Clay-rich rocks or soils - Presence of water 	<ul style="list-style-type: none"> - Muddy ballast - Inadequate sub-ballast
Softening subgrade surface un sub-ballast	<ul style="list-style-type: none"> - Dispersive clay - Water accumulation at soil surface - Repeated train loading 	<ul style="list-style-type: none"> - Reduces sliding resistance of subgrade soil surface
Liquefaction	<ul style="list-style-type: none"> - Repeated dynamic loading - Saturated silt and fine sand - Repeated train loading 	<ul style="list-style-type: none"> - Reduces sliding resistance of subgrade soil surface

Type	Causes	Features
Massive shear failure (slope stability)	<ul style="list-style-type: none"> - Weight of train, track and subgrade - Inadequate soil strength 	<ul style="list-style-type: none"> - Steep embankment and cut slope - Often triggered by increase in water content
Consolidation settlement	<ul style="list-style-type: none"> - Embankment weight - Saturated fine-grained soils 	<ul style="list-style-type: none"> - Increased static soil stresses from weight of newly constructed embankment - Fill settles over time
Frost action (heave and softening)	<ul style="list-style-type: none"> - Periodic freezing - Free water - Frost-susceptible soils 	<ul style="list-style-type: none"> - Occurs in winter/spring period - Heave from ice lens formation - Weakens from excess water content on thawing - Rough track surface
Swelling/shrinkage	<ul style="list-style-type: none"> - Highly plastic or expansive soils - Changing moisture content 	<ul style="list-style-type: none"> - Rough track surface - Soil expands as water content increases - Soil shrinks as water content decreases
Slope erosion	<ul style="list-style-type: none"> - Surface and subsurface water movement - Wind 	<ul style="list-style-type: none"> - Soil washed or blown away - Flow onto track fouls ballast - Flow away from track can undermine track
Slope collapse	<ul style="list-style-type: none"> - Water inundation of very loose soil deposits 	<ul style="list-style-type: none"> - Ground settlement
Sliding of side hill fills	<ul style="list-style-type: none"> - Fills placed across hillsides - Inadequate sliding resistance - Water seeping out of hill or down slope is a major factor 	<ul style="list-style-type: none"> - Transverse movement of track

Substructure investigation

Being one of the most important steps in geotechnical engineering, a proper identification, classification and characterization of subgrade soils must be performed whenever an investigation of the substructure is carried out.

A limited substructure examination can be made by hand, or by machine through excavating inspection holes at the side of the track. Generally, these procedures are associated with relatively small practical limits for digging distance below the subgrade surface. Because conditions at the top part of the subgrade are often different beneath the track than at the side of the track, a better inspection can be made by removing the ballast shoulder. This allows inspection of the ballast and sub-ballast layers as well, which is usually needed whenever subgrade information is required.

The next level of investigation may involve excavating a cross trench from one side of the track to the other. This is important in fully evaluating the ballast, subballast and upper subgrade conditions because conditions often vary with position across the track and with

depth. In cases where subgrade depressions and ballast pockets occur, they might represent significant deviations from a plane boundary between layers that will not be known without a full (or at least half) width cross trench. Depths of excavation to at least 1200 mm below the top of the sleeper are often quite possible (for example through the use of coring machines).

Identification of subgrade layers and materials to depths below those possible with coring machines or cross trenches require the use of standard drilling techniques. Common in situ geotechnical procedures include: standard penetration tests (SPT); cone penetration tests (CPT); piezocone (CPTu); flat dilatometer (DMT); pressuremeter (PMT); and vane shear (VST). Each test applies different loading schemes to measure the corresponding soil response in an attempt to evaluate its characteristics. Figure 4.1.14 depicts these various devices and simplified procedures in graphical form.

Boreholes are required for conducting SPT and normal versions of PMT and VST. A rotary drilling rig and crew are essential for these tests. In the case of the CPT, CPTu and DMT, no boreholes are needed. These are thus termed “direct-push” technologies. Specialised versions of the PMT (i.e. full displacement type) and VST can be conducted without boreholes. As such, these may be conducted using either standard drill rigs or mobile hydraulic systems in order to directly push the probes to the required test depths. A disadvantage of direct-push methods is that stiff trackbed layers and bedrock will prevent further penetration. In such cases, borehole methods prevail as they may advance by coring or non-coring techniques.

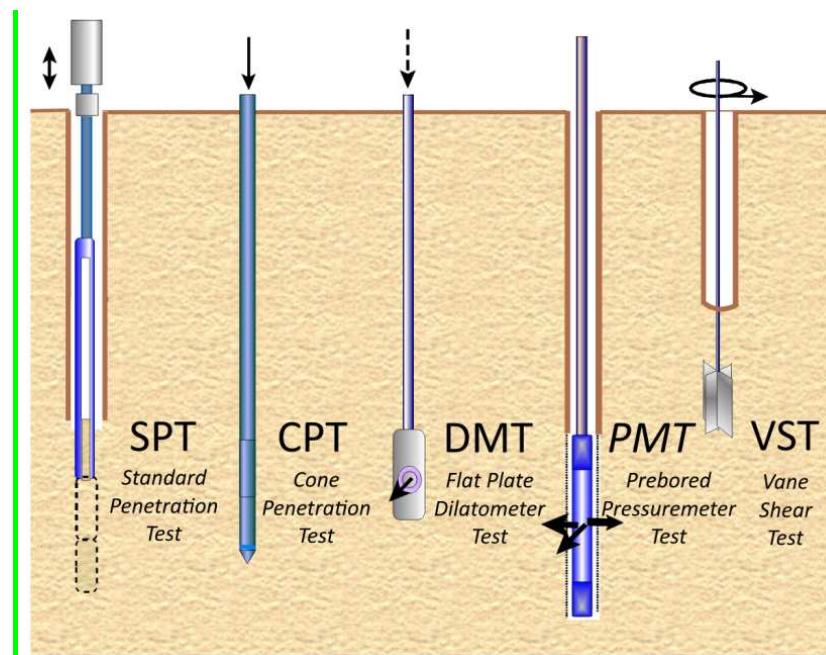


Figure 4.1.14 – Common in-situ tests for geotechnical investigation.

Physical properties of the layers can be measured in the field using in situ techniques, nevertheless direct measurement of properties such as the resilient modulus, plastic strain or strength characteristics require the elaboration of specific laboratory tests. Furthermore, for subgrade materials, undisturbed field samples will usually be required.

Identification of drainage rehabilitation needs

Along with structural measures with the aim of strengthening the substructure, it is of great importance to assess the condition of the drainage system, its adequate design and the well-functioning of all its components. In this regard locations where poor drainage might be the major cause for substructure weakening must be identified.

Excluding erosion by surface runoff, water generally causes the most problems after it infiltrates railway embankments and track substructures. Water in the soil and in cracks that have opened in the ground has the potential to destabilise embankments or decrease soil strength. This water is often a major contributor to track failures. Draining the track section and embankment is thus fundamental to improving track and embankment performance.

Water tends to accumulate in track locations where the drainage is impaired such as railway crossing approaches, bridge approaches, low points in vertical curves, and at other locations where track settlement has occurred and the track has been raised on additional ballast. Typical places within an embankment where water may be expected to accumulate or become trapped, include (IHHA, 2009):

- Ballast pockets;
- Cracks in embankments;
- The interface between the subgrade and a relatively stiff layer above (below ballast);
- At the contact of more permeable rock or soil with less permeable materials.

Interception and efficient drainage of non-track groundwater is another important aspect when assessing the performance of the existent drainage system and its need for rehabilitation. Groundwater in soils below embankment, flowing toward embankment from upslope areas, or flowing toward track constructed in cuts can lead also to unstable track conditions. Interception and removal of this water and lowering of the groundwater table to a satisfactory depth below the embankment or track substructure may improve the performance of embankments and track sections (Figure 4.1.15). Collection and removal of this groundwater may also improve stability of the slope through which the water is flowing, whether located above or below the track.

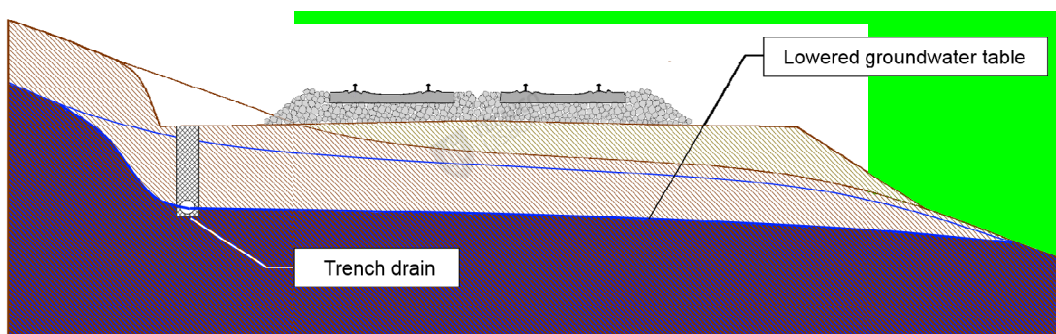


Figure 4.1.15 – Interception of water inflow trough trench drain.




The construction of trench drains (Figure 4.1.15) is a common solution to an effective interception and drainage of near-surface groundwater flowing toward the track. These drainage components consist of an excavated trench backfilled with clean, well-graded gravel products that provide a high permeability path for water to escape from the embankment slope or subgrade. These drains may be constructed parallel to and relatively near the track

or at some distance upslope from the track. Shallow drains with pipes are sometimes installed parallel and near the track to improve subgrade performance.

Depending on the groundwater depth, topography and geology, other drainage systems may be required. Horizontal drains, small diameter perforated pipes installed in holes drilled into the ground are common alternatives. High groundwater conditions and springs also contribute to landslides. In a landslide susceptible terrain, trench drains, horizontal drains or other subsurface methods may be components of a landslide stabilization program.

A proper geotechnical investigation to the railway substructure and its surroundings might allow the identification of different indicators regarding groundwater conditions and inadequate functioning of drainage system components (Table 4.1.4).

Table 4.1.4 – Indicators of high groundwater conditions and poor drainage.

Indicators of inadequate drainage	
Wet ground near or on the embankment	
Ponded water adjacent to track	
Vegetation normally associated with wetlands growing on nearby slopes	

Indicators of inadequate drainage

Water ponded in ditch and wetlands plants



The presence of sinkholes on track or nearby slopes



- Collecting and diverting surface water away from the track structure may be the most cost-effective portion of any subgrade maintenance or rehabilitation program. The more water that is intercepted and diverted away from the track, the less water available to infiltrate and potentially weaken the track structure. Recommended practices which must be part of a regular maintenance scheme and that should be carefully considered when assessing drainage rehabilitation needs in the scope of substructure upgrading include (IHHA, 2009):
 - Improve surface drainage to make sure runoff runs off and does not pond;
 - Divert surface water prior to it reaching problem area;
 - Keep surface water at surface. Do not let it pond or infiltrate the embankment or ground upslope of the embankment;
 - Deepen ditches below the bottom of the subballast (do not deepen ditches excessively such that support is lost for the track section);
 - Grade ditch bottoms to improve runoff;
 - Clean ditches and culverts
 - Check culverts and ensure they are in good condition, have not rusted through, and not pulled apart;
 - Replace culverts that are too small with culverts of adequate capacity;
 - Maintain culvert inlets and install systems to reduce potential for the inlet to plug or for debris to wash into the culvert and plug it;
 - Remove from embankment shoulders any debris produced by maintenance activities such as undercutting, shoulder ballast cleaning and ditch cleaning;
 - Grade shoulders to drain water over the side of the embankment without restricting the flow. Erosion protection measures may be required on the slope.

An effective drainage of shoulders and ditches contributes to a better drainage of the ballast and subballast layers. Ballast drainage has to be maintained to keep the proper resiliency under the sleeper. This area is affected when there is not sufficient ballast under the sleepers. Train loading leads ballast to break down. The residue, along with dirt blown into the ballast section allows the ballast to become fouled. Fouled ballast and shoulder fouling lead to water retention in the track structure. As the crushing and grinding of ballast particles progresses, drainage from track structure is impaired. The water retained in the ballast because of this impaired drainage accelerates the ballast breakdown process. Ballast failure and the pumping associated with it may cause loss of sleeper support.

In summary, the presence of water weakens the substructure. Considerable experience is required to differentiate between causes of different types of failure and to select an appropriate remedial measure to correct the problem. No single remedy has been identified to correct all of these problems; but since most of these failure modes involve water, drainage improvement is usually part of the remedy. In some situations, remedial measures may lead to an acceptable solution. However some remedial measures only provide a short term solution by only treating the symptoms of the problems rather than analysing its root cause for longer term eradication. In these cases, the upgrading of the substructure must take into account that a full redesign of the drainage system might be necessary. An upgrading analysis should thus analyse available observations, measurements and the requirements for both the railway and its surroundings.

4.1.2 SUBGRADE IMPROVEMENTS

To mitigate poor subgrade conditions is often very costly. This includes not only the cost of material and work efforts, but also the often-extensive traffic disruptions that are caused by the mitigating actions. There is thus a significant cost saving potential in developing new, LCC efficient improvement methods. However, these methods need also be thoroughly verified if they are to be employed in large-scale use. It is here important that not only the benefits, but also the drawbacks of each method are highlighted since the methods usually are suited mainly for certain conditions

In the INNOTRACK project a number of improvement methods were investigated, tested and verified. These were inclined piling using lime-cement columns, the use of geogrids in transition zones and bad drainage areas, and the use of short vertical soil-cement columns. These and other methods will be described in this section together with benefits and outlined drawbacks.

4.1.2.1 Introduction

Traditional open-cut improvement methods require the interruption of railway traffic for a long time. The different elements of the track used to be removed. Rails, sleepers, ballast, sub-ballast and often the embankment fill have to be withdrawn to perform the strengthening works. Apart from being time consuming, the traditional remedial works are costly in terms of material, resources and (often extensive) traffic disruptions.

Fig. 4.1.16 shows how the distribution of cost that can look like for an actual project of subgrade improvement with Deep Mixing Method.

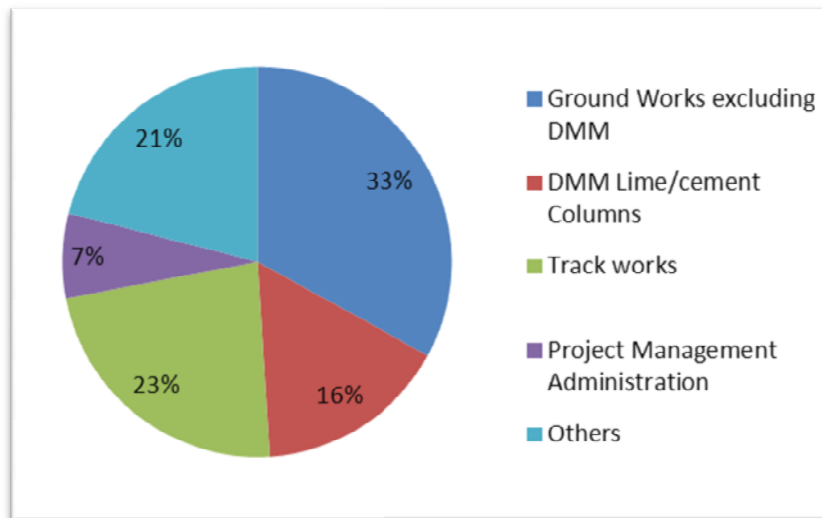


Fig. 4.1.16 - Distribution in a typical strengthening work with lime/cement columns (SUSTAINABLE BRIDGES, 2006)

Figure 4.1.16 shows costs for strengthening works, not the costs due to traffic disruption. One can see that the cost for installation of lime/cement columns is only the 16% of the total cost. In contrast, 56% of the total cost is for ground and track works. There is a great economical and operational interest to use soil improvements methods that minimize track works and disturbance.

There is thus a significant cost saving potential in developing new, LCC efficient improvement methods. However, these methods need also to be thoroughly verified if they are to be employed in large-scale use. It is here important that not only the benefits, but also the drawbacks of each method are highlighted since the methods usually are suited mainly for certain conditions.

Following there is a description of current practises in subgrade improvement, followed by some special use cases in the Northern and Southern countries of Europe, where the geotechnical conditions are significantly difficult compared with the middle region.

4.1.2.2 Overview of strengthening methods

As mentioned before, there are many different methods available for subgrade improvement depending on the geotechnical conditions, costs, construction time, performance, presence of buildings in the surroundings, tradition and experience of the contractor and the infrastructure manager, national regulations, etc.

With the aim of selecting the correct method, it is first necessary to identify the problem to be solved. The most common problems in embankments are settlements, instability and excessive vibrations. In all these cases increasing subsoil stiffness usually helps in solving the problem. There are two different approaches for this improvement works: from the track or from side of the railway embankment. Both methods have advantages and disadvantages.

4.1.2.2.1 Subgrade improvement from the track

Usually this type of work is carried out by equipment and tools mounted on railway vehicles. In most cases the works are done directly under the track. If the traffic should not be restrained, installation has to be carried out between sleepers. The space between a two sleepers, 40 cm in the best case, leads to the need for special tools. There are other restrictions as the time that the workers have available to conclude the installation. Traffic requirements usually reduce this time to the maintenance window.

Other considerations have to be taken into account. Catenaries, wires and poles represent especially dangerous elements in contact with the machines and tools needed for the installation. In addition, the methods that use bindings can pollute ballast during their installation. In this case, special tools have to be employed to keep the ballast clean.

4.1.2.2.2 Subgrade improvement from sides of railway embankment

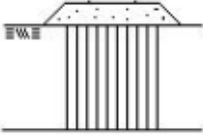
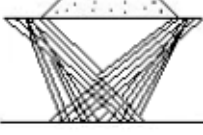

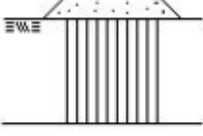
Execution of subgrade improvement or installation of strengthening elements from the sides of the railway track is an easier way to deal with these kinds of works, especially on electrified lines. However, there is not in every case enough space next to the railway embankment, especially in urban areas.

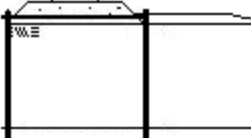
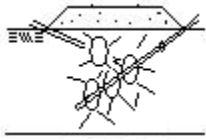
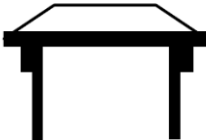
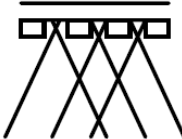
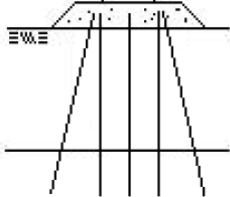
All methods for soil improvement have impact on the track geometry during the installation. It is important to assess the expected movements and the influence on the track during the works, which typically have to allow train circulation at normal service speeds. When the line remains open to traffic during the execution, it is therefore recommended to use real-time automated methods to detect possible movements.

The impacts of geotechnical issues are also very important and have to be taken account in the design phase. Characteristics of soil before, during and after strengthening must be assessed with consideration of time related changes. The design of the soil improvement has to assess the safety factor for the improved structure and for all stages of strengthening work. The risk of slide failure, subsidence and lateral movements have to be evaluated in advance and controlled during the execution of works.

Table 4.1.5 shows strengthening methods that have been used in the upgrading of railway subgrade. Of course, there are many other improvement techniques but these are less suitable for existing embankments. In addition, the table shows the influence on rail traffic and the kind of soils for which the method is applicable. It is noteworthy that, continuously, contractors develop new methods or adapt existing ones to their machines, local geotechnical conditions and railway related needs.

Table 4.1.5. Overview of existing strengthening methods (SUSTAINABLE BRIDGES, 2006) (INNOTRACK)

Scheme	Method	Principle	Can be performed without affecting traffic	Applicable soils	Increase of stability	Reduce settlements
	Deep mixing installed through the track and embankment	Mixes in-situ soils with cementitious materials to form a vertical stiff inclusion in the ground	No	Wet method: most soft soil Dry method: soft fine-grained soils	Yes	Yes
	Deep mixing installed inclined under embankment	Mixes in-situ with cementitious materials to form an inclined stiff inclusion in the ground	Yes	Wet method: most soft soil Dry method: soft fine-grained soils	Yes	Yes
	Deep mixing beside railway embankment	Mixes in-situ soils with cementitious materials to form a vertical stiff inclusion in the ground	Yes	Wet method: most soft soil types Dry method: soft fine-grained soils	Yes	No
	Jet grouting	Erodes soil in situ and mixes with cementitious materials to form stiff inclusion in the ground	Yes (unless installed beneath embankment)	Most soil types	Yes	Yes

Scheme	Method	Principle	Can be performed without affecting traffic	Applicable soils	Increase of stability	Reduce settlements
	Stabilizing berms, alone or in combination with anchored walls	Compacted material constructed adjacent to embankment. Driven walls.	Yes	Clay	Yes	No
	Compaction grouting	Low slump grout is pumped into the ground, which displace and density the soil or fill voids in rock	Yes	Granular soil cavities	Yes	Yes
	Concrete slab on piles	Installation of piles on both sides of railway, precast slabs fixed on piles in short stages with traffic interruption	No	All soil types	Yes	Yes
	Soil nailing	Soil nails installation between sleepers	No	All soil types	Yes	No
	Vibro compaction Vibro replacement Vibro concrete columns	Compaction of soil, transfer of loads to more competent strata through friction or end-bearing	No	Granular soils	Yes	Yes

4.1.2.3 Subgrade improvement methods

4.1.2.3.1 Deep Mixing Method

The Deep Mixing Method (DMM) was initially developed in 1970 in Japan (wet method) and Sweden (dry method) to stabilize subsoil under embankments. The deep mixing procedures increases the strength, lower ductility, and lower compressibility as compared to the original ground. The degree of improvement depends on the amount and type of stabilizer, installation process, characteristics of the original ground, and the curing time. This method can be applied from the sides of the embankment, as shown in Figure 4.1.17.



Figure 4.1.17 Example of installation of a lime column. Left: Inclined; Right: Vertical (INNOTRACK, 2008)

The principle of this method is quite simple. In-situ soils are mixed with cementitious materials to form a stiff inclusion in the ground. A mixing tool is rotated down to the design depth, and once it is reached, the direction of rotation of the mixing tool is reversed and the tool is withdrawn at a constant rate. During advancement and/or withdrawal of the mixing tool, agents such as quicklime, slaked lime, cement, and fly ash are forced into the ground. The deep mixing method produces columns in the ground that can be installed singularly, or in rows (walls), grids or blocks. When installed vertically the soft soil beneath the railway is stabilized. Vertical strengthening is normally used for newly constructed railways and there is an extensive experience with the application of deep mixed columns to support new railway embankments. Inclined lime cement columns for upgrading railway tracks have also been used (INNOTRACK, 2008). Both cases can be seen in Figure 4.1.18.

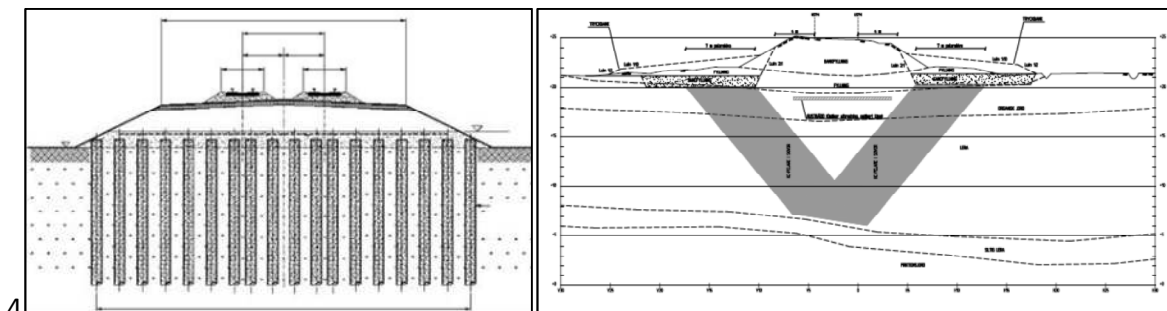


Figure 4.1.18. Design of strengthening of subsoil using vertical or inclined lime-cement column walls (SMARTRAIL, 2014)(INNOTRACK, 2008)

This method requires the following steps in the design, execution and control:

1. Geotechnical survey and site investigations
2. Design of strengthening with lime cement columns
3. Quality control and safety plan for the installation
4. Analysis of possible restriction on train speed and axle load during installation
5. Preparation of construction site
6. Transport of machinery, tools and materials for lime cement columns
7. Site supervision during construction (Figure 2.1.19)
8. Follow up after installation of strengthening
- 9.



Figure 2.1.19 Positioning of lime cement columns

Table 4.1.6 shows the advantages and drawback of this method in the upgrading of existing railway infrastructures:

Table 4.1.6. Advantages and drawbacks of deep mixing method

Advantages	Drawbacks
<ul style="list-style-type: none"> • Low vibrations during construction. • High attenuation of ground vibrations after installing. • High performance • Well known technology, with significant experience in new embankments (for vertical columns) • Technology available in most countries • Low impact on rail traffic (for inclined columns or vertical columns at the side of the track). 	<ul style="list-style-type: none"> • Not much experience in upgrading railways (for inclined columns) • The need of heavy machinery could be a problem in very soft soils.

4.1.2.3.2 Jet grouting

The jet grouting technology is very similar to the deep mixing technology except that very high-pressure fluids are used in jet grouting in order to erode subsurface soil particles and mix them with cement. The jet grouting system uses hydraulic energy. The jet erodes the soil and mixes it with an engineered grout of water and cement to form a solidified in situ element. Jet grouted elements can be installed in a variety of subsurface geometries. Columns with diameters of 0.6 to 2 m are common. 4.1.20 shows a scheme of the installation of the jet grouting system and a picture of the flow ejected by the machine.

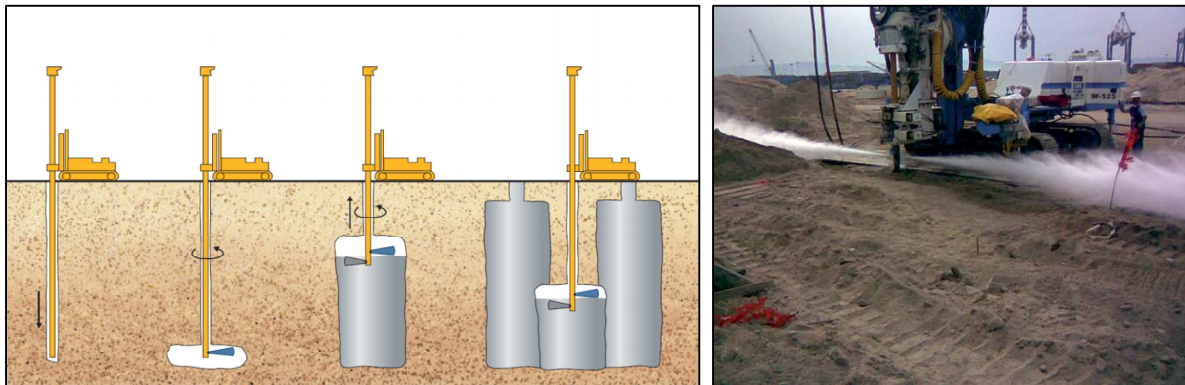


Figure 4.1.20. Scheme of jet grouting installation (left). Machinery used for construction (Right) (TRB, 2012)

This system can be installed horizontally or vertically, or even at an angle. There are three traditional jet-grouting techniques:

- Single-fluid jet grouting uses high pressure (400 to 500 bar) grout only
- Double fluid-jet grouting uses high pressure grout sheathed in compressed air
- Triple-fluid jet grouting uses high-pressure water sheathed in air followed by a second jet of high-pressure grout.

The equipment to perform jet grouting remains specialized but many contractors are available to carry out this technology. Drills range from relatively small electrically powered units up to the large crane supported systems. Some drills have the ability to create a wide variety of jet-grouted geometries. Theoretically, treatment depth is unlimited, but jet grouting has rarely been performed at depths below 50 m.

Table 4.1.7 shows the advantages and drawback of this method in the upgrading of existing railway infrastructures:

Table 4.1.7. Advantages and drawbacks of jet grouting

Advantages	Drawbacks
<ul style="list-style-type: none"> • Low vibration during construction. • Relatively small machinery, useful in small gauges. • Low surface settlements. • Suitable for many types of soil. • Different angles are possible for the columns. • Low impact on rail traffic (for inclined columns or installation at the side of the track). 	<ul style="list-style-type: none"> • Not effective in very plastic clays. • Complex construction procedure. • Technology not always available.

4.1.2.3.3 Stabilizing berms combined with Anchored walls

Stabilizing berms are often built adjacent to existing railway embankments to increase the bearing capacity of the embankment or to prevent instability. Stabilizing berms consist on a few meters of compacted material. Their thickness is about 1 to 2 meter. The use of loading berms is an effective way of increasing stability with costs that are quite low compared to other methods.

However, this method presents some disadvantages that are necessary to consider. The main disadvantage of the stabilizing berms is that it does not reduce settlements or vibrations. In fact, the settlements of the railway embankment can even increase when loading berms are used (INNOTRACK, 2008). The predicted total and differential settlements need therefore be evaluated in advance.

This negative effect can be reduced by combining the loading berms with walls installed along the railway embankment. Walls can be built for example as pile walls using concrete, steel, or wooden piles. Sheet piles as well as diaphragm walls have been used. Material used for supporting wall depends on the geotechnical conditions, the load and the geometrical situation of the particular project. On the other hand, it could be problematic to install driven elements, like for example sheet piles, through subsurface soils containing cobbles or boulders. In this case drilled or predrilled structures are more feasible.

If the walls are installed on both sides of the railway embankment, anchors can be installed to connect the walls on opposing sides of the embankment. The anchors are post-tensioned to reduce horizontal movements in the walls. Figure 4.1.21 shows an example of a sheet pile wall used for stabilizing existing embankments.

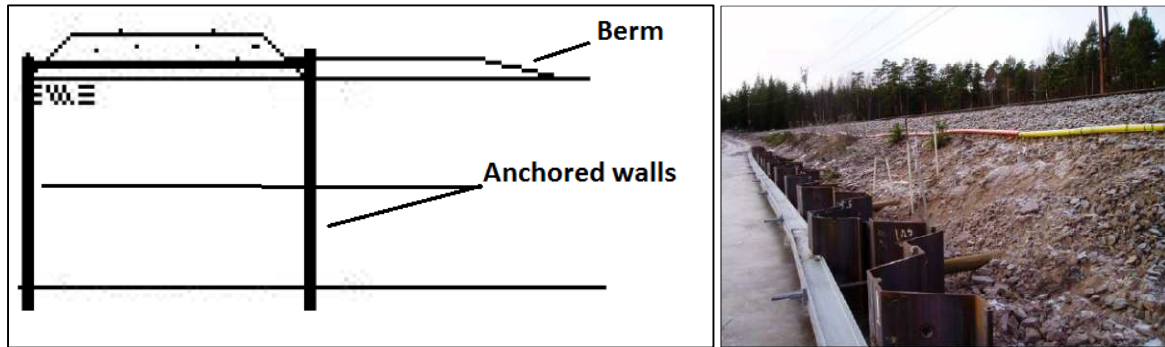


Fig.4.1.21. Left: Scheme of anchored walls with stabilizing berm in a railway embankment; Right: Example in Vitmossen (Sweden) (SUSTAINABLE BRIDGES, 2006)

Table 4.1.8 shows the advantages and drawback of this method in the upgrading of existing railway infrastructures:

Table 4.1.8. Advantages and drawbacks of stabilizing berms combined with anchored walls

Advantages	Drawbacks
<ul style="list-style-type: none"> • Low cost (if only stabilizing berm is used) • Low surface settlements (when using sheet pile walls). • Low impact on railway traffic 	<ul style="list-style-type: none"> • High vibrations during installation of sheet piles. • Complex and expensive construction procedures when using anchors. • Only suitable for soft clays. • No reduction in settlements, only improvement of stability.

4.1.2.3.4 Compaction grouting

The compaction grouting is a technology that injects a low slump grout into the ground to form a stiff, homogenous mass in the subsoil. The grout does not enter soil pores but remains in a stiff mass that gives controlled displacement in order to compact granular soils. The compaction grouting is commonly used to increase the density of soils. The grouted mass also adds vertical strength, since the grout compressive strength is much higher than that of native soil.

There are a number of applications of compaction grouting including remediation of embankment settlements, soil densification for site improvement, filling of voids in rock formation and liquefaction mitigation.

Compaction grouting is performed by injecting a very stiff grout through a casing at a high pump pressure; the casing is withdrawn incrementally and a column of interconnected grout bulbs is created. The grout is carried out through vertical or inclined grout pipes.

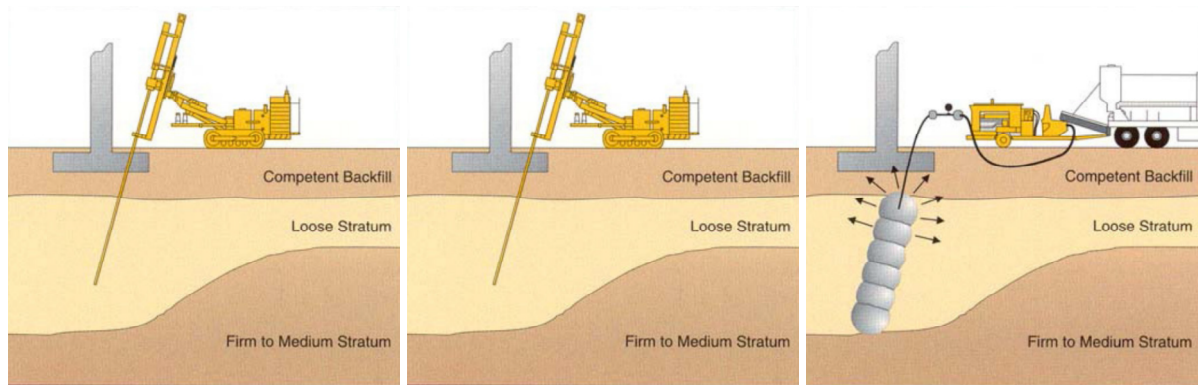


Figure 4.1.22. Installation procedure of compaction grouts below a shallow foundation (TRB, 2012)

Table 4.1.9 shows the advantages and drawback of this method in the upgrading of existing railway infrastructures:

Table 4.1.9. Advantages and drawbacks of compaction grouting

Advantages	Drawbacks
<ul style="list-style-type: none"> • Well known technology • Relatively small machinery, useful in small gauges. • Low impact on rail traffic. 	<ul style="list-style-type: none"> • Only suitable for granular soils. • In cohesive soils, the injection can cause high excess of pore pressure, with later consolidation settlements. • No standardized procedure • Long-term behavior of the cement mortar is unknown.

4.1.2.3.5 Precast concrete slab on piles

The main advantage of this method is that the piles and longitudinal beams are installed on both sides of the existing railway without any disturbance of traffic. However, the pre-cast slabs are place on piles after replacement of track in short traffic brakes. This operation can be carried out during the night and in this way, minimizing the interruption of the rail traffic in the rush hours. Figure 4.1.23 shows a scheme of this method.

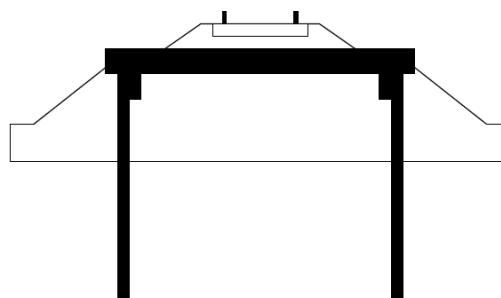


Figure 4.1.23. Scheme of precast concrete slab on piles

Table 4.1.10 shows the advantages and drawback of this method in the upgrading of existing railway infrastructures:

Table 4.1.10. Advantages and drawbacks of precast slabs on piles

Advantages	Drawbacks
<ul style="list-style-type: none"> • Excellent control of settlements. • Suitable for all types of soils. 	<ul style="list-style-type: none"> • Very expensive. • Low performance. • Moderate traffic disruption when installing the slabs. • Negative effects in the transmission of ground vibrations through the piles.

4.1.2.3.6 Soil nailing

Soil nailing is a strengthening technique that can be used to treat unstable natural soil slopes or as a construction technique that allows the safe over-steepening of new soil slopes. It can be also applied to existing railway embankments to prevent instability failures.

The technique consists on the insertion of relatively slender reinforcing elements into the slope, often general-purpose reinforcing rebar although proprietary solid or hollow-system bars are also available. Solid bars are usually installed into pre-drilled holes and then grouted into place using a separate grout line, whereas hollow bars may be drilled and grouted simultaneously by the use of a sacrificial drill bit and by pumping grout down the hollow bar as drilling progresses.

Kinetic methods of firing relatively short bars into soil slopes have also been developed. Bars installed using drilling techniques are usually fully grouted and installed at a slight downward inclination with bars installed at regularly spaced points across the slope face. Rigid facing or isolated soil nail head plates may be used at the surface. Alternatively, a flexible reinforcing mesh may be held against the soil face beneath the head plates. Rabbit proof wire mesh and environmental erosion control fabrics may be used in conjunction with flexible mesh facing where environmental conditions dictate.

The technique applied for strengthening of rail embankments is relatively cheap with high production in comparison to the other described methods. This technique can be applied from the track with minimal traffic disruption. Figure 4.1.24 4.1.24 shows an example of soil nailing driven from the track to the underground through the space between sleepers.

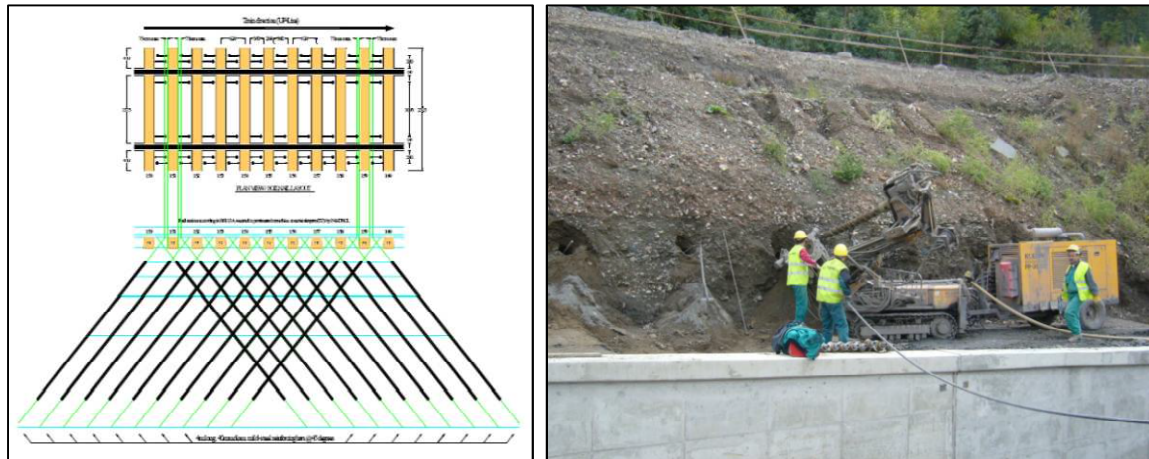


Figure 4.1.24. Left: An example of soil nailing executed from the track (Konstantelias, 2003); Right: Stabilization of a railway embankment with soil nailing (INNOTRACK, 2008)

Table 4.1.11 shows the advantages and drawbacks of this method in the upgrading of existing railway infrastructures:

Table 4.1.11. Advantages and drawbacks of soil nailing method

Advantages	Drawbacks
<ul style="list-style-type: none"> ● Low vibrations during construction. ● High performance. ● Good increase of stability. ● Well known technology. ● Low traffic disruptions. ● Suitable to install from the track, between sleepers. ● Relatively small machinery, useful in small gauges. 	<ul style="list-style-type: none"> ● No reduction of settlements, only improvement in stability. ● Low depth of strengthening.

4.1.2.3.7 Vibrocompaction

This is a suitable method for densification of non-cohesive soils like sand, gravel or crushed rocks. This method is based in the physical principle that vibration significantly reduces friction between soil grains and lets them rearrange to a denser structure. Subsequently, the pores content is reduced and the stiffness is increased.

Vibration is generated by cylindrical depth vibrators. The head of these vibrators carries unbalanced masses generating the vibration energy. Usually the vibrator penetrates to the designed depth and is then withdrawn in steps until the required compaction is achieved. During compaction additional material has to be added since the volume decreases.

This method has to be applied very carefully on in-service railway lines because the settlements of the track can be up to 15% of the treated volume. Additional ballast and sub-ballast as more friction material is recommended to be available for immediate levelling of the track.

Table 4.1.12 shows the advantages and drawbacks of this method in the upgrading of existing railway infrastructures:

Table 4.1.12. Advantages and drawbacks of vibrocompaction method

Advantages	Drawbacks
<ul style="list-style-type: none"> • High performance. • Well known technology. • Low cost. • No materials such as cement, water, concrete or steel are necessary. 	<ul style="list-style-type: none"> • High noise and vibrations during construction. • Only suitable for granular soils. • Possible settlements during construction, which motivates the line closure for safety reasons.

4.1.2.3.8 Vibroreplacement

This method is a combination of vibration and replacement of cohesive soils with granular material. The vibrator penetrates to a design depth and cavities are filled with coarse material (gravel, crushed stone or pebble) which is then compacted by the vibrator. In this way, soft soils are improved with well-graded load bearing columns (Figure 4.1.25).

There are two variants of this method: “Dry bottom feed technique” when using air pressure and “Wet bottom feed technique” when using pressurized water. The dry bottom feed technique is more suitable for railway embankments due to the lower risk for large settlements and changes in stiffness below the track.

The risk for excessive settlements during the installation is lower than for vibrocompaction. Nevertheless, it is recommended to have additional ballast or sub-ballast ready in case that levelling of the track is required.

The method consists in a steel casing pipe vibrated to the soil. When the pipe is withdrawn filling material is vibrated in, to create the stone or sand columns. Even a geotextile tube can be used to stabilize and improve performance of the columns. In this case the method is called “Geotextile Coated Columns (GCC)”.

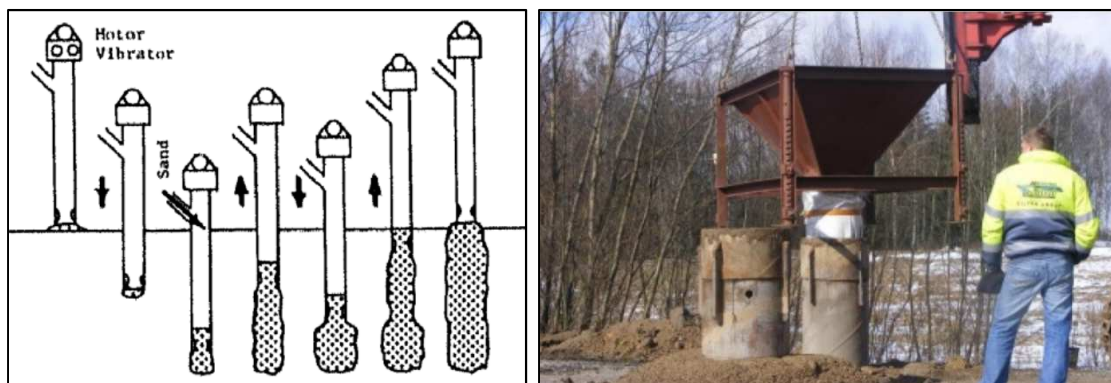


Figure 4.1.25 Scheme of vibro replacement procedure (left) (TRB, 2012); Installation of geotextile coated columns (right) (Geosintex, 2012)

Table 4.1.13 shows the advantages and drawbacks of this method in the upgrading of existing railway infrastructures:

Table 4.1.13. Advantages and drawbacks of vibroreplacement

Advantages	Drawbacks
<ul style="list-style-type: none"> • High performance. • Low settlements during construction compared to vibrocompacting. • Improvement of vertical drainage (pore pressure reduction). • Well known technology. 	<ul style="list-style-type: none"> • Relatively high cost. • In case of GCC (patented), there are royalty costs. • High noise and vibrations during construction. • Technology not always available. • Not much experiences in railways.

4.1.2.3.9 Grouted stone columns (GSC)

When using the vibroreplacement method in very soft clays or peat, lateral support is usually not sufficient for adequate stability of columns. In the previous description, it was mentioned that the application of geotextile improves the lateral support. Another way to guarantee the stability of the hole is to use a cement grout suspension together with gravel during the installation process. Cement grout suspension forms together with the coarse material a stiff body of the column. Of course, such columns show much higher bearing capacity than previously described methods.

Furthermore, the potential impact on the track during installation is lower but the time for hardening of the cement grout has to be considered before returning to normal train services.

Table 4.1.14 shows the advantages and drawbacks of this method in the upgrading of existing railway infrastructures:

Table 4.1.14. Advantages and drawbacks of Grouted stone columns

Advantages	Drawbacks
<ul style="list-style-type: none"> • High performance. • Low settlements during construction compared to vibrocompacting. • Excellent bearing capacity. 	<ul style="list-style-type: none"> • High cost. • High noise and vibration levels during construction. • Technology is not always available. • Not much experience in railways.

4.1.2.3.10 Vibroconcrete columns (VCC)

Pile-like bearing elements can be used for soils with very low bearing capacity, e.g. in an organic subsoil. These elements are inserted through the layers, which have a lower bearing capacity into the deeper and stiffer strata. In this case, after vibration the concrete is directly pumped from the bottom to the top of the column.

The earth structures above the concrete column can be reinforced horizontally with geotextiles (geogrids). This will improve the distribution of the pile loads and even out deformation. Under existing railways, the head of the columns can also be enlarged to obtain better support for sleepers. In these cases, it is recommended to place the column head deeper in the formation in order to avoid stress concentration in the ballast.

This method can be applied from the sides of the embankment or to reinforce the foundation of civil structures, as shown in Figure 4.1.26.



FIGURE 4.1.26. EXAMPLE OF VCC METHOD IN A RAILWAY BRIDGE

Table 4.1.15 shows the advantages and drawbacks of this method in the upgrading of existing railway infrastructures:

Table 4.1.15. Advantages and drawbacks of Grouted stone columns

Advantages	Drawbacks
<ul style="list-style-type: none"> • High performance. • Low settlements during construction compared to vibrocompacting. • Excellent bearing capacity achieved. 	<ul style="list-style-type: none"> • High cost. • High noise and vibration levels during construction. • Technology is not always available. • Not much experience in railways.

4.1.2.4 Other methods for subgrade improvement in railway infrastructure.

4.1.2.4.1 Hydraulic fracturing with injection of cement grout (Spain)

This use case consisted in the consolidation and stabilization of an embankment located in the railway line Calatayud–Valencia, at the section between kilometres 208/100 and 208/200. The transit in the line includes passenger and freight trains. The passenger trains included the Euromed, which operates at 200 km/h. The problem of the embankment was due to the low frequency vibrations transmitted by freight trains. This situation gave rise to sizeable deformations of the track, requiring very frequent tamping to correct the track geometry.

The adopted solution to this problem was the hydraulic fracture grouting technique through sleeve pipes (Figure 4.1.26). The works were carried out simultaneously with railway traffic operating at normal speed, while the substructure and the underlying embankment were being grouted down to a maximum depth of 7 m under strict requirements on keeping the track geometry (movements of rail less than 3 mm over a 5 m length).



Figure 4.1.26. Different phases in the construction (Lopez Moreno, 2013)



Figure 4.1.27. Execution of injections at the side of the in-service track (Lopez Moreno, 2013)

Other example of hydraulic fracture injection of stable cement-based mixed technique was carried out at the Southern embankment of the viaduct over the Ebro River, in the railway line Valencia–Barcelona of the Mediterranean Corridor (Oropesa–Vandellós line section).

The basic objectives to achieve were:

- Increasing and regulating the modulus of deformation of the embankment, in the zones where it appeared less compact according to the results of the previous geophysical explorations. It was understood that the lack of compaction of those embankment zones had caused the need for ballasting the track in this section almost continually.
- Making this soil improvement during normal service of the railway line.

For doing so, it was essential to carry out the improvement from a working platform independent from the railway line. This was achieved by building a berm adjoining the embankment and the transition zone.



Figure 4.1.28. general view of the embankment where the soil improvement was achieved (European Project SUPERTRACK, 2005)

On the other hand, it was considered that the grouting technique, applied by means of sleeve tubes, could provide:

- The possibility of improving a predefined volume or body of embankment, by means of fans of sleeve tubes, installed from the working platform and oriented in such a way as to cover the grouting of that volume, taking into account the reach of the grout placed from every sleeve.
- The application, at each grouting sleeve, of the necessary number of passes of treatment until the grouting pressure, adequate for the degree of improvement envisaged at this location of the transition zone, would be reached.
- Use, at each grouting pass, of a grouting mixture of the necessary characteristics (viscosity, setting time, ...) placed at the convenient rate and volume, in order not to move the rails in excess of the comfortable circulation deformation limit (a criterion more exigent than the safe circulation deformation limit).

- A safe criterion to correlate the grouting pressure and the final parameters of shear strength in the treated soil so that the process of grouting could be properly controlled, and the end of the treatment at each sleeve clearly defined.



Figure 4.1.29. grouting operations (European Project SUPERTRACK, 2005)

This action, performed at the Southern embankment of the railway viaduct over the Ebro River in Amposta, showed the feasibility of improving the condition of the transition zones without interference with the daily traffic passing during the improvement process.

It was verified that the modulus of deformation of the fill was increased by a factor of about 2.5, without exceeding the deformation limit for comfortable circulation on the track (3 mm maximum differential settlement between cross-sections 5 m apart).

The results obtained showed that the technique of hydraulic fracture by means of sleeve pipes might, if properly used, allow changing the conditions of any earth supporting structure of the track without interfering with the commercial traffic and the track. Consequently, its use can be extended to any type of transition zone and supporting ground if necessary. Since the differential movements involved in the applications are less than 1/1000 this is applicable to railways, highways and buildings and structures, irrespective of their nature and the initial conditions.

4.1.2.4.2 Injections of bentonite and cement mortar (Spain)

Another example of subgrade improvement in the rail embankment in Spain was the work carried out in the line Teruel–Sagunto (Figure 4.1.30). The degradation of some embankments in this line led to a limit on maximum speed to 30 km/h in several stretches ten years ago.

The process of strengthening was applied to embankment lengths of around 250 meters. In this case, a new technique based on injections of a special cement mortar with bentonite (type of clay) was used.



Figure 4.1.30 Treated railway section (Escudero, 2011)

This technique, as the example above, was applied from the sides of the embankment, avoiding traffic disruptions during the works.

4.1.2.4.3 Injections of cement mortar and polyurethane resin in karst (Spain)

In the high-speed line Madrid–Sevilla, next to the village of Yébenes, a karstified area in limestone was detected. In order to avoid a possible collapses of the track, cement mortar and polyurethane resin were injected in the cavities through boreholes drilled beside the track.

The improvement stretched up to seven meters below the track. The works were carried out in maintenance windows, without affecting train services at all. During the execution of works, the track geometry was controlled automatically to detect any movements.



Figure 4.1.31 Railway section treated (Rodio Kronsa, 1998)

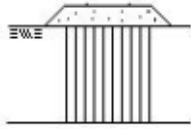
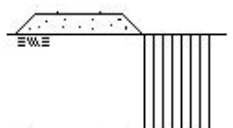
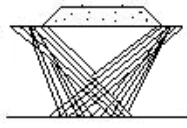
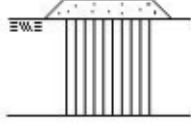
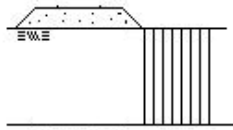
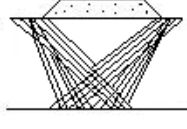
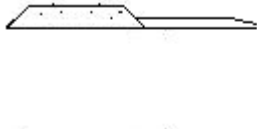
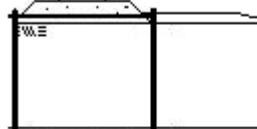
4.1.2.5 Comparison of improvement methods

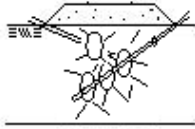

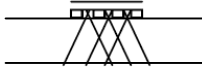
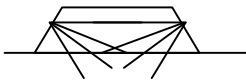
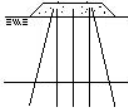
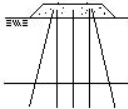
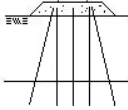
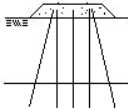
According to the previous descriptions, there are a number of advantages and drawbacks in each of the improvement methods. In order to facilitate the selection of the most suitable method from a technical point of view, table 4.1.16 shows a comparative assessment of six main characteristics:

1. Traffic allowance during execution
2. Type of soils where the method is applicable
3. Increase of stability against landslides
4. Effectiveness in the reduction of long-term settlements
5. Effectiveness in the reduction of vibration transmitted to neighboring buildings
6. Experience among European contractors and Infrastructure Managers.

Each criteria has three possible values: High (green), medium (yellow) and low (red).

Table 4.1.15 Compared advantages and drawbacks of soil improvement methods

Method	Scheme	ALLOW TRAFFIC	TYPE OF SOILS	INCREASE OF STABILITY	REDUCE SETTLEMENTS	REDUCE VIBRATIONS	EXPERIENCE
1 Deep Mixing		Red	Orange	Green	Green	Orange	Green
		Green	Orange	Green	Red	Orange	Green
		Green	Orange	Green	Orange	Orange	Green
2 Jet grouting		Red	Green	Green	Green	Orange	Green
		Green	Green	Green	Red	Orange	Green
		Green	Green	Green	Orange	Orange	Green
3 Stabilizing berms		Green	Orange	Green	Red	Red	Green
4 Stabilizing berms combined with anchored walls		Red	Orange	Green	Red	Red	Red

	Method	Scheme	ALLOW TRAFFIC	TYPE OF SOILS	INCREASE OF STABILITY	REDUCE SETTLEMENTS	REDUCE VIBRATIONS	EXPERIENCE
5	Compaction grouting		Green	Red	Orange	Orange	Red	Green
6	Precast concrete slab on piles		Red	Green	Green	Green	Red	Red
7	Soil nailing		Red	Green	Green	Red	Red	Orange
			Green	Green	Green	Red	Red	Orange
8	Vibrocompaction		Red	Red	Green	Orange	Red	Green
9	Vibroreplacement		Red	Orange	Green	Green	Red	Green
10	Grouted stone columns		Red	Orange	Green	Green	Red	Orange
11	Vibroconcrete columns		Red	Orange	Green	Green	Red	Red

4.1.3 MITIGATING GROUND VIBRATIONS

Several new traffic situations can imply an increase of ground vibration levels. Therefore, it is important to have a high understanding of ground vibrations. The topic of ground vibrations and how these should be mitigated has been the focus of the EU-project RIVAS – Railway Induced Vibration Abatement Solutions (see <http://www.rivas-project.eu>). A good and easy to read summary is presented in the final newsletter, see UIC (2013).

4.1.3.1 Methodology

RIVAS investigated a number of preventive and mitigating actions. To quantify the efficiency of these, vibration levels were measured in a test section and also in a reference section at a certain distance from the track before and after the installation of the mitigation measure. In addition, an analysis of vehicle, track and soil parameters that significantly influence vibration levels was carried out. To assess the annoyance inside buildings four descriptors (two vibration related and two vibration-induced noise related) were employed. The annoyance reduction was then presented with respect to the cost–benefit ratio.

The proposed methodology proved to be an efficient means of identifying the potential of different mitigating actions. One important finding was however that the measured efficiency depends significantly on the preload.

4.1.3.2 Generation of vibrations

Regarding the generation of vibrations in the wheel–rail interface, it was shown that wheel out-of-roundness (and in particular local tread defects such as wheel flats) could have a strong influence on the vibrations emitted by some categories of rolling stock. Track geometry defects with wavelengths shorter than some 10 m have some influence on the low frequency vibration, but the parametric excitation due to sleeper passing has to be considered in addition. Isolated defects shorter than 3 m may generate significantly high free-field vibrations. Finally, dipped rails at welds and insulated rail joints can generate high wheel–rail impact loads with broad frequency contents that lead to high vibration levels.

In summary, the analyses in RIVAS concluded that regarding track defects longitudinal level misalignments with wavelengths of 0.6 m seem to have the highest overall vibration impact, although the highest vibration levels are found near welds and insulation joints. Regarding wheels, out-of-roundness is the main influencing factor. To capture the essential track geometry characteristics, RIVAS proposed to introduce a new wavelength band containing wavelengths in the interval 0.5 – 3 m for the assessment of track geometry measurements.

Regarding vehicle characteristics, the influences of the carbody mass and the bogie mass are small on vibration that propagates to larger distances from the track. The secondary suspension comes into play at very low frequencies where other generation mechanisms related to the soil often are dominant. The primary suspension stiffness and in particular the unsprung mass are the two vehicle parameters that have the most significant influences on the vibration level. As an indication of the efficiency of mitigating actions, simulations estimated reductions in nearby houses to some 2–3 dB owing to a reduction of unsprung mass by 35%.

More information can be obtained from the RIVAS guideline D5.5.

4.1.3.3 Track-based mitigation

As for track-based mitigation, RIVAS evaluated the following measures:

Soft under sleeper pads coupled with heavy (wide) sleepers optimized for straight ballasted tracks

Soft rail pads optimized for straight ballasted tracks

Soft under sleeper pads in curves

Soft under sleeper pads in switches

Soft under sleeper pads optimized for the GETRAC slab track system

It was found that ground stiffness has a minor impact on the efficiency of both under sleeper pads and rail pads for sleeper systems.

Simulations indicated that sleeper pads on slab, would reduce ground-borne noise, but increase vibrations.

More information can be obtained from the RIVAS guideline D3.13.

4.1.3.4 Transmission reductions

Studied actions include trenches, buried wall barriers, subgrade stiffening, horizontally layered wave-impeding blocks, and heavy masses at the soil surface. Since the layered ground structure has a significant influence on the response, all options were studied numerically for different ground types to establish the circumstances in which they are efficient. In addition, three field tests were carried out: soil stiffening next to the track, a trench barrier, and a sheet piling wall.

As an example of results, a sheet piling wall in Furet (Sweden) with a length of 100 m and a depth of 12 m with every fourth pile extended to 18 m was installed. Measurements showed that maximum vibration levels are reduced by about 50% close to the wall. At 32 m from the centre the maximum levels are reduced by about 30% while at 64 m from track centre, no reduction in maximum vibration levels is seen. The sheet pile wall did not lead to any increase in vibration levels at the other side of the track.

A soft wave barrier, 6 m deep, 5 cm wide located at 8 m from the track also resulted in reduction of vibrations and ground-borne noise in three nearby buildings. The reductions varied between some 2 dB to some 15 dB depending on building and descriptor used.

The analysis further revealed the usefulness of numerical simulations in evaluating different solutions. In addition, the levels of reduction were found to be significantly influenced by ground characteristics. More information can be found in the RIVAS guideline D4.6.

4.1.3.5 References

The description above is mainly a summary of the overview information in

Dorothee Stiebel, Final evaluation of results, RIVAS Final Conference, November 21, 2013

UIC, RIVAS Vibrations: Ways out of the annoyance, 28 pp, 2013

For more in-depth information, the RIVAS project has authored a number of guidelines to aid in vibration reduction. These include:

- E. Bongini, R. Müller, R. Garburg and A. Pieringer, D3.13 – Guidelines for mitigation measures on ballasted track, curves, switches and slab track, 32 pp (and 3 annexes 6+8+4 pp), 2013
- G. Lombaert, A. Dijckmans, P. Coulier, G. Degrande, A. Smekal, A. Ekblad, D.J. Thompson, J. Jiang, M.G.R. Toward, W. Rücker, J. Keil, G. Vukotic, M. Rodríguez Plaza, Á. Andrés, V. Cuéllar, R. Garburg and R. Müller, D4.6 – Design guide and technology assessment of the transmission mitigation measures, 30 pp (and 4 annexes 4+3+3+2 pp), 2013
- Jens Nielsen, Adam Mirza, Philipp Ruest, Philipp Huber, Steven Cervello, Roger Müller and Brice Nelain, D5.5 – Guideline for design of vehicles generating reduced ground vibration, 43 pp (and 1 annex 3 pp), 2013

These reports can be downloaded from the RIVAS website (<http://www.rivas-project.eu>)

4.2 STRUCTURES

4.2.1 GENERAL

4.2.1.1 Influence of new traffic situations

Structures that are considered in this chapter are Bridges, Tunnels, Culverts and Retaining walls.

The new traffic situations described in chapter 3 have different impact on different types of structures as summarized in Table 4.2.1.

For bridges the highest impact comes from increased axle loads and meter loads. Longer trains and increased train weights have some impact, whereas higher speeds on freight trains has little impact and increased loading gauges usually has no impact. For tunnels the highest impact comes from increased loading gauges, whereas other changes usually have no impact. For culverts the greatest impacts are usually the same as for bridges. For common retaining walls that support masses beside the rail but not under it, the traffic situation has little or no impact. For the unusual case of retaining walls that support the track, the axle loads and meter loads may have great impact.

TABLE 4.2.1. IMPACT OF DIFFERENT TRAFFIC SITUATIONS ON DIFFERENT TYPES OF STRUCTURES.

Type of structure/ Traffic situation	Bridges	Tunnels	Culverts	Retaining walls
Longer trains	Some impact	No impact	Some impact	No impact
Increased trail weight	Some impact	No impact	Some impact	No impact
Increased axle loads and meter loads	Great impact	No impact	Great impact	Some impact
Higher speeds on freight trains	Little impact	No impact	Little impact	Little impact
Increased loading gauges	No impact	Great impact	No impact	No impact

In order to be able to successfully adjust existing structures to new traffic situations, codes and assessment methods have a vital importance. General principles will be commented on below before presenting specific conditions for different types of structures.

4.2.1.2 Differences in Codes for New and Existing Structures.

Today's codes are written for the design of new structures, e.g. the Eurocodes. EC0 – EC9 (2000-2009). An interesting account of the obstacles encountered in their development is given by Johnson (2009) who was involved in the first 40 years of the creation of the codes. Now work is going on to implement, revise and harmonize them, see e.g. EC Handbook 1-4 (2004, 2005) and JRC Bridge Examples (2012).

When building a new structure there are uncertainties regarding geometry, material properties, construction quality and loads. These uncertainties can be balanced by a reasonable safety margin in partial coefficients (γ), special load factors for railways (α) and reliability indices (β). The partial coefficients are safety factors used for loads and material properties, the α -factor adjusts for rail traffic that is heavier or lighter than normal traffic and the reliability indices indicate the accepted probability of failure. The (α -factor) is also used to meet future needs (see chapter 1 Background). Typical partial coefficients in the Eurocodes for permanent loads are $\gamma_G = 1.35$ and for live loads $\gamma_Q = 1.50$. For structural material properties, typical values of the partial coefficients are $\gamma_c = 1.50$ for concrete and $\gamma_s = 1.10$ for structural steel, see e.g. EC Handbook 1 (2004). The rail traffic α -factors may vary between 0.75 and 1.46 with $\alpha = 1.33$ recommended on lines for freight traffic and international lines, UIC 702 (2003). A probability of failure of 1 in 100 hundred, $P_f = 0.01$, corresponds to a reliability index $\beta = 2.3$, a probability $P_f = 10^{-5}$ corresponds to $\beta = 4.2$ and a probability $P_f = 10^{-7}$ corresponds to $\beta = 5.2$ see e.g. EC Handbook 2 (2005).

The Eurocodes indicate target reliability indices in relation to three classes of consequences (high, normal and low) and two reference periods T (1 and 50 years). For a failure with low consequences (e.g. a greenhouse) during a time period of $T = 50$ years the recommended reliability index is $\beta = 3.3$ corresponding to a failure probability of $P_f = 0.0005$ (5 in 10 000). For a failure with high consequences (e.g. a bridge) during a time period of $T = 1$ year the recommended reliability index is $\beta = 5.2$ corresponding to a failure probability of $P_f = 10^{-7}$ (1 in 10 millions). If the time period is increased to $T=100$ years, this increases the failure probability to $P_f = 10^{-5}$ (1 in 100 000) with $\beta = 4.2$.

As a comparison, the risk of death for a person during one year varies with where the person is living and what he/she is occupied with and it also changes during the life time. In Sweden the death probability during one year ($T=1$) for a 1 year old child is $P_f = 0.00012$ (1.2 in 10000), for a 40 year old person it is $P_f = 0,001$ (1 in 1000), for a 60 year old person it is $P_f = 0.01$ (1 in 100) and for a 100 year old person it is $P_f = 0.36$ (1 in 3). The death rate due to accidents and suicide is about $P_f = 0.001$, SCB (2007). The acceptable individual risk of death during a year due to an accident caused by a structural failure is in JRC Assessment (2015) proposed to be $P_f = 10^{-5}$ which is about 1 % of the general risk for death due to accident and suicide given above for Sweden.

When assessing the capacity of an existing structure many of the uncertainties that are present when building a new structure can be resolved. The codes for assessment therefore do not need to have the same high partial coefficients γ , α -factors or reliability indices β , see e.g. SB-LRF (2007), ML-D1.2 (2013) and JRC Assessment (2015). Also standard dynamic amplification factors for the influence of dynamic loads may be reduced after a study of a structure. Thus there is a need for special codes for upgrading of existing structures.

An early example of such codes was the Swedish BV Bärighet (1996). It has been updated several times and is presently divided in two parts, one with requirements TRVK Bärighet (2014) and one with recommendations TRVR Bärighet (2014).

One example of a new set of standards for assessment of existing structures is the Swiss code SIA 269/1/2/3 (2011). Brühwiler (2014, 2015) gives a description of the positive experiences with it. He highlights major principles and approaches, in particular those related to a stepwise procedure, see Figure 2.2.1, and by updating of action effects through monitoring, updating of structural resistance and novel technologies of intervention, such as strengthening. It is often beneficial to use site-specific live loads and dynamic amplification factors, see e.g. ML-D1.4 (2012) and Casas & Rodrigues (2015). In the Swiss codes the recommended reliability index for a reference period of $T = 1$ year varies from $\beta = 3.1$ for a minor consequence of a failure to $\beta = 4.7$ for a serious consequence, JRC Assessment (2015). This is lower than the factor $\beta = 5.2$ given above for new bridges.

Work has also started on a Eurocode for assessment of existing structures; see JRC Assessment (2015).

4.2.1.3 Assessment of Existing Structures.

In Figure 4.2.2 below, an assessment is divided into three phases: Initial, Intermediate and Enhanced. The initial first phase includes a site visit, study of documentation and a simple calculation. The intermediate second phase may include material investigations, detailed calculations and/or analyses and further inspections and monitoring. The enhanced third phase usually consists of refined calculations and analyses including statistical modelling and reliability based assessments, laboratory examinations and/or field-testing and economic decision analysis. Similar well-structured procedures are proposed in ISO 13822 (2010) and JRC Assessment (2015). An enhanced assessment is often the most cost effective way to increase the allowable load, as there is in most cases a considerable hidden reserve in existing bridges and culverts.

The value of a structured approach will be illustrated with examples below. A hidden strength has also been proven by full-scale tests of bridges to collapse, see e.g. Puurula et al (2015). In addition, there are many examples on successful strengthening of bridges that have given them the extra capacity needed to fulfil a new traffic situation, ML-D1.4 (2015), Nilimaa (2015). General information on upgrading and strengthening of structures are given in e.g. the guidelines and background documents from the European projects Sustainable Bridges (2007) and MAINLINE (2014).

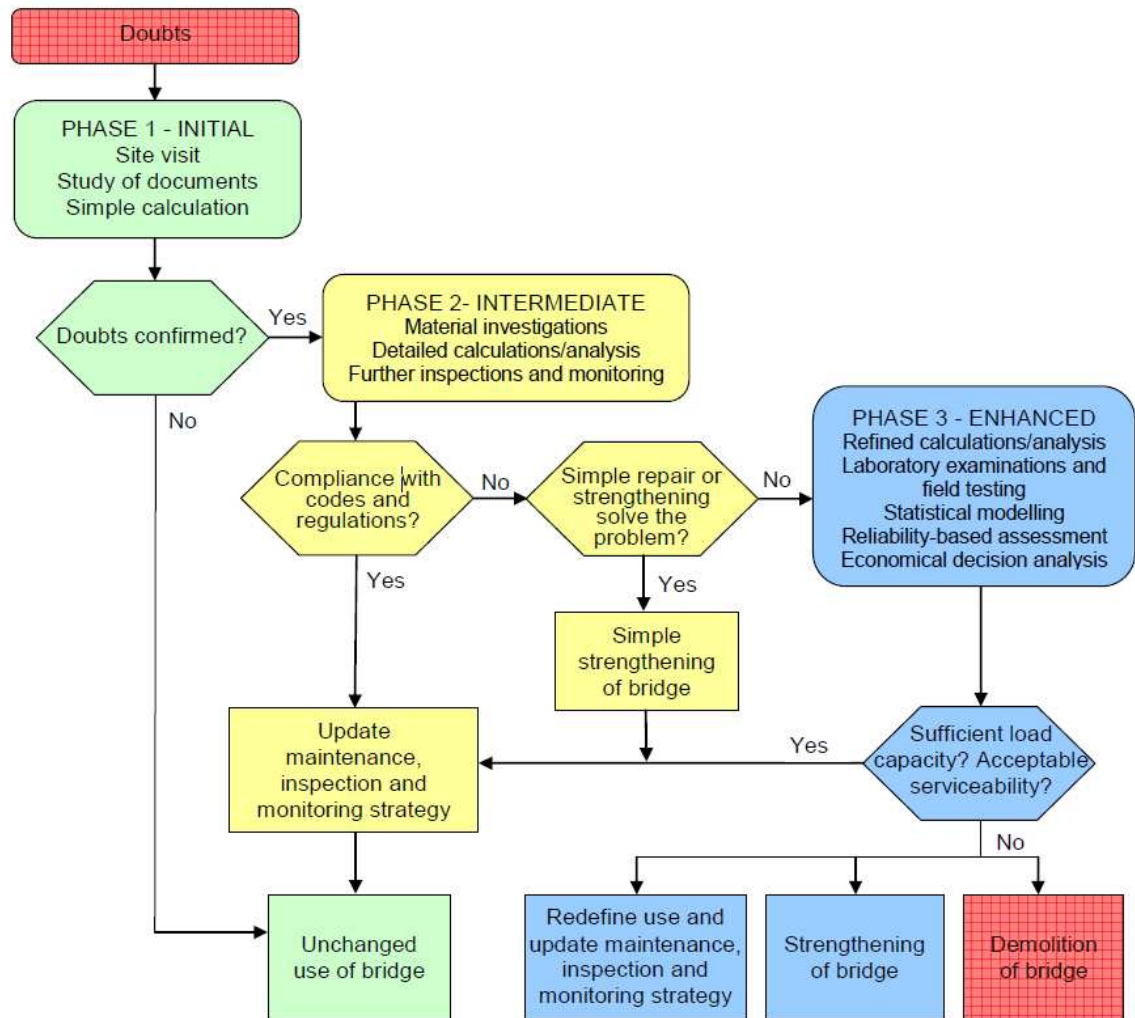


Figure 4.2.1 Flow chart for assessment of existing bridges and other structures. Three phases are identified: Initial, Intermediate and Enhanced depending on the complexity of the questions involved. SB-LRA (2007). UIC 778-4(2009), ML-D1.4 (2014).

4.2.2 BRIDGES

According to Table 4.2.1 it is mostly a higher axle load or meter load that influences the use of bridges.

There are at least 250,000 railway bridges in Europe and nearly 75 % of them are older than 50 years, with about 35 % being older than 100 years, SB-D1.2 (2004), SB-D1.4 (2005).

Most infrastructure managers have a bridge management system, BMS, with various levels of explicit details. Some examples are: Latvia (Lat Brutus), The Netherlands (DISK), Spain (SGP), Japan (JBMS), Ireland (Eirspan), Germany (GBMS), Finland (FBMS), Denmark (DANBRO), Switzerland (KUBA), and Sweden (BaTMan), see Mirzaei et al. (2012). The bridges are in most countries well documented and maintained in a professional way. This means that the challenge for bridge managers is how regular operation and maintenance shall be used as input when upgrading in an optimal way, ML-D1.4 (2014), Casas (2015).

In Sweden e.g. an internet based system is used since 2004, BaTMan (2015). There you can look up the structure you are interested in and see if enough data is given regarding drawings and design calculations or if you have to search manually in the archives.

In France partial safety factors have been studied for different variables, such as reinforcement area, yield strength, concrete density and fixed load moment. For road bridges the allowable reliability index could in some case be lowered with 7 % compared with the target reliability index, Cremona (2014).

When upgrading a system, five principle methods can be used, see Figure 4.2.2: (a) Refined calculation; (b) Increased cross section; (c) Change of static system; (d) External prestressing; and (e) Externally bonded reinforcement e.g. Fibre Reinforced Polymers (FRP).

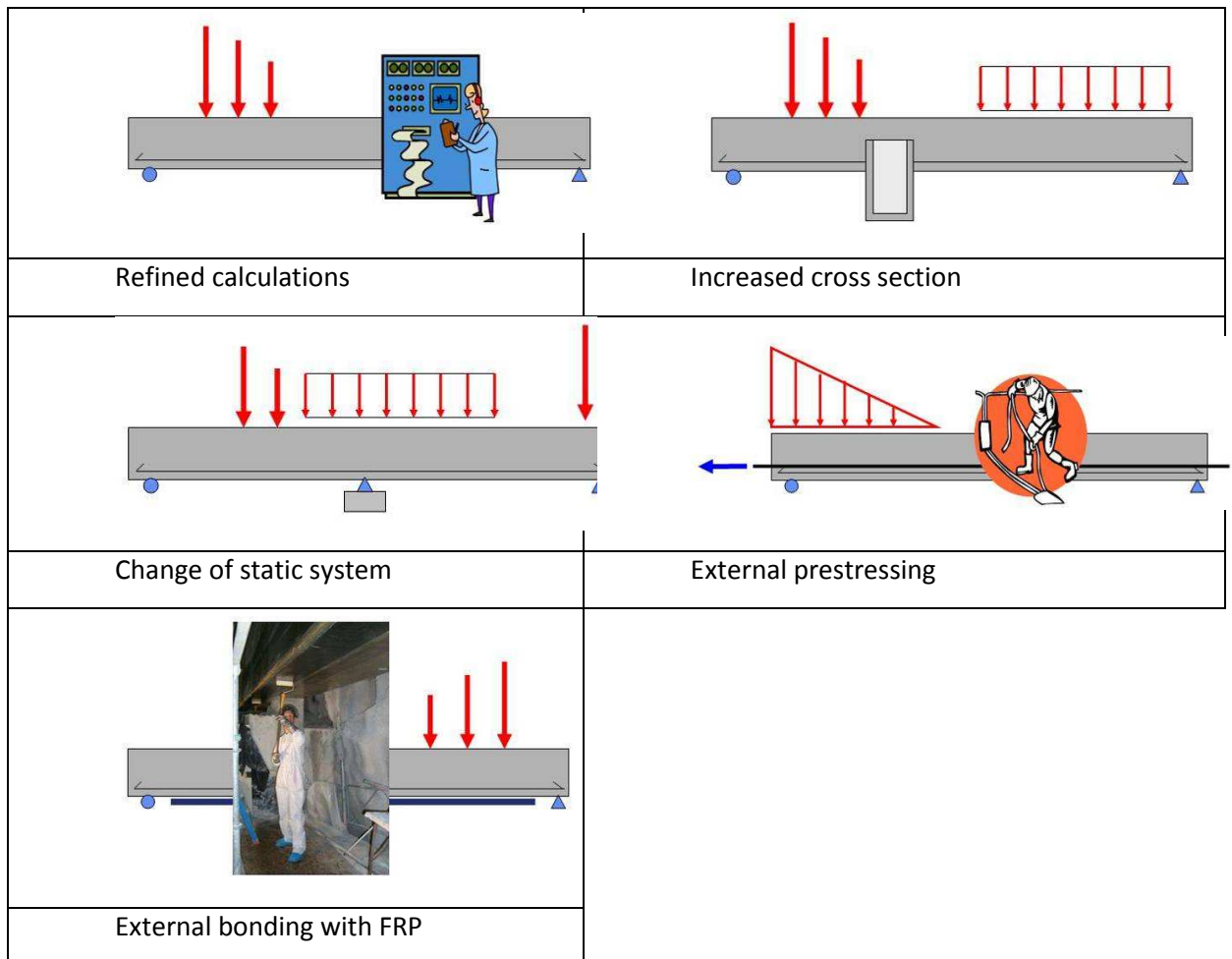


Figure 4.2.2. General upgrading methods, SB-STR (2007), ML-D1.4 (2014).

Upgrading and strengthening for the most common types of railway bridges will be treated in the following order: foundations, metallic bridges, masonry bridges, concrete bridges, prestressed bridges and composite bridges. For other types of bridges, such as suspension bridges, a special study has to be made for each individual bridge although the same principles can be applied as presented below.

4.2.2.1 Foundations

An example of how a foundation can be arranged is given in Figure 4.2.3. The subsoil conditions in the transition zones at the bridge abutments often govern the applicable strengthening methods.

Methods for strengthening and upgrading that can be used are e.g. deep mixing of the soil, jet grouting, sheet pile walls/stabilizing berms, compacting grouting and embankment piling. Examples and case studies are given in SB-STR (2007). The transition zones when the rails pass from the embankment to the bridge may cause “bump” problems due to different stiffness, see SB-LRA (2007) and Fara (2015).

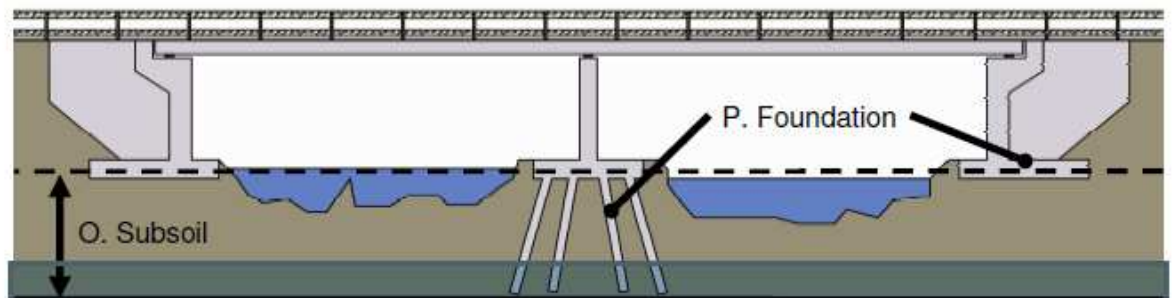


Figure 4.2.3. Example of a foundation with piles or slabs (P) resting on subsoil (o), SB-STR (2007), ML-D1.4 (2014).

Low stability of embankment in transition zones might be a problem, see Figure 4.2.4. Slip surfaces for stability calculations in Figure 2.2.4, may need refined methods in deformation-softening clays, see Bernander (2011).

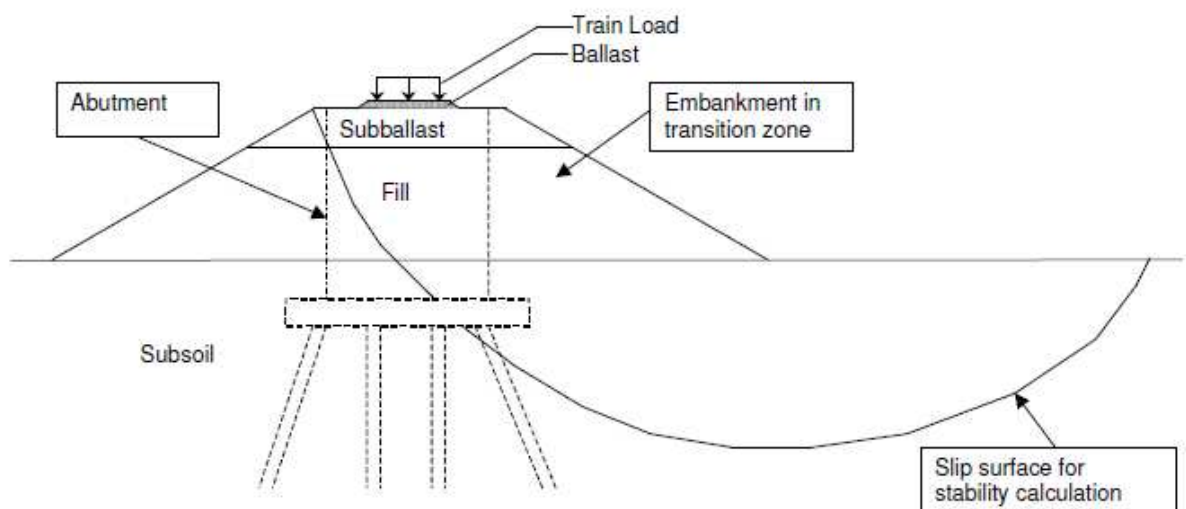


Figure 4.2.4. Principle for the analysis of stability of embankment in transition zones, SB-STR (2007), ML-D1.4 (2014).

4.2.2.2 Metallic bridges

Metallic bridges can be made of cast iron, wrought iron, steel, stainless steel or aluminium. They can have several different designs such as girder, truss and box girder bridges. In addition to this, metallic bridges can be found in almost any design with different material qualities for different structural members within the same structure, SB-LRA (2007).

4.2.2.2.1 Metallic Truss Bridge

The superstructure of a truss bridge commonly consists of two vertical trusses on each side of the track. The track can also be located above the truss beams. The trusses are often the primary structural elements of the bridge and they consist of many members, such as top and bottom elements, diagonal and vertical struts, and end struts. These members form the truss and are mainly subjected to tensile or compressive forces. A horizontal truss at the top or bottom of the bridge gives horizontal stability. The horizontal truss gives bearing capacity for horizontal loads from wind and horizontal accelerations of the train. Bolts or rivets mutually connect all truss members.

A schematic truss bridge is shown in Figure 4.2.5. This type is often used for bridges that were built before 1950s with medium span lengths.

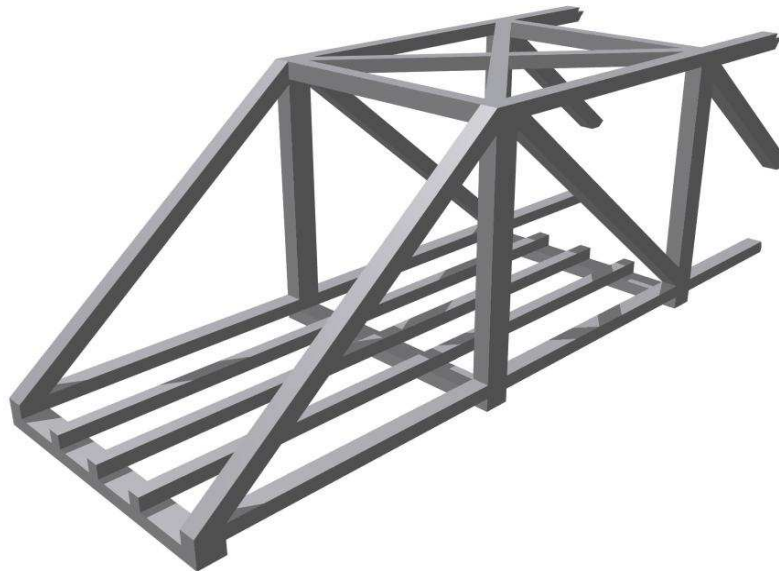


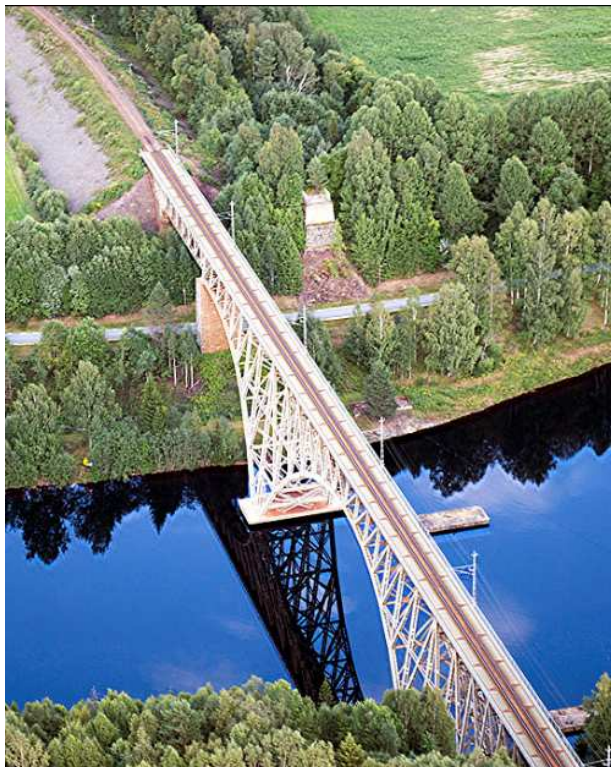
Figure 4.2.5. Metallic truss bridge, SB-STR (2007), ML-D1.4 (2014)

Typical reasons for upgrading are:

- Deficient global bearing capacity or stiffness, or deficient tension capacity in structural elements and joints. This can be remedied with external longitudinal pre-stressing
- Deficient compression bearing capacity of beam-like elements. Relatively many structures are slender and have more than sufficient compression capacity, hence global buckling limits the capacity. This can be remedied with external CFRP plates or sheets or by increase of the cross section.
- Deficient bearing capacity of joints including rivets. Cross-sections are often changed in connection to joints, which leads to additional stress concentrations, which may lead to fatigue problems. This can be remedied by external pre-stressing and/or replacement of components as rivets and bolts.
- Deficient shear bearing or buckling capacity of webs or flanges. Longitudinal and/or transverse stiffeners and/or external CFRP can remedy this.
- In conjunction to welds, cross-sections are changed, which causes additional stress concentration, which in turn may cause fatigue problems and possible crack initiation and growth. This can be remedied by repair welding of cracks, drilling of stop holes, and external prestressing or external CFRP.

Case studies and examples of upgraded structures are given in SB-STR (2007), ML-D1.3 (2015) and ML-D1.4 (2015).

After an enhanced assessment process, a partial replacement of the superstructure of an old truss bridge is often an economic way to upgrade the capacity and to prolong service life of the whole structure. Usually three critical constraints are present: (1) working with short possession times; (2) limited substructure capacity; and (3) limited budget.



An example of such a replacement is the exchange of the steel truss deck of the arch bridge at Forsmo over the Ångerman River in Northern Sweden; see Figure 4.2.6 and 4.2.7 Enevoldsen et al (2002), Collin et al (2010).

Another example is the strengthening by prestressing of a 85 year old four span truss bridge over Skellefteå River in 1996, see SB-STR (2007), Case study CS005:02.



Figure 4.2.6. Top: Forsmo Bridge over Ångerman River, Sweden, after strengthening; Bottom: Steel truss arch with replacement of top structure at the Forsmo Bridge. Collin et al (2010).

There may be considerable reserve strength in metallic bridges. One example is a 55-year-old bridge that was assessed and tested in the MAINLINE Project, see Figure 4.2.8. The regulated axle load on the route (25 tons) was exceeded approximately 4 times before a non-linear behaviour of the deflection the mid span occurred, and almost 6 times before the ultimate load was exceeded. The structural behaviour of the Åby Bridge remained ductile and the maximum load was reached in a nondramatic fashion without failures in any joints. No fatigue problems could be found and the final collapse was due to buckling of the top frame beam, ML-D1.3 (2015), Blanksvärd et al (2014), Häggström et al (2015).

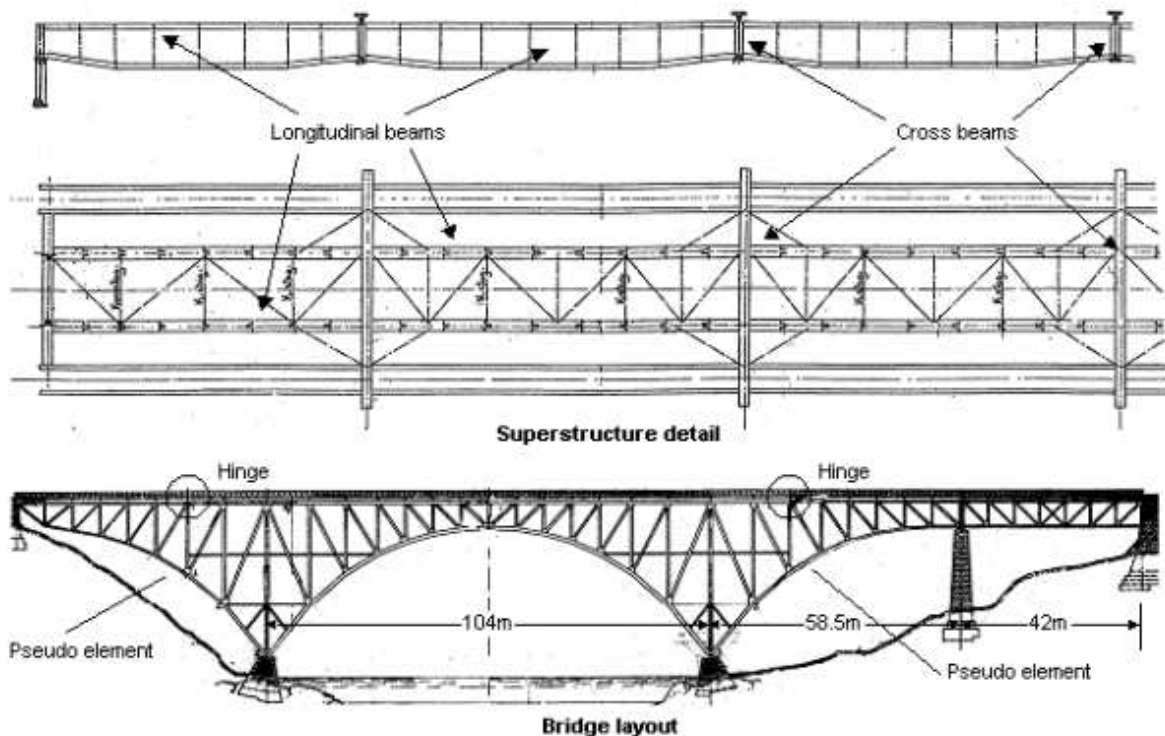


Figure 4.2.7. Change of superstructure of the Forsmo Bridge



Figure 4.2.8. Test of 55 year old 33 m long metallic bridge at Åby River in 2013. Overview of test site with two jacks anchored in the bed rock in the centre (left) and final buckling failure of top girder beam (right), ML-D1.3 (2015).

4.2.2.3 Masonry Bridges

It is vital from the outset that a holistic approach is taken to the assessment and upgrading/strengthening of masonry arch bridges. To change the nature and/or stiffness of one element of the bridge may result only in re-distribution of stresses, which may cause overstressing of another element.

Masonry arch bridges are usually defined by the material from which it is constructed, the shape of the arch, number of spans and its orientation; for example, a stone, segmental, 3-span, skewed masonry arch bridge.

Since most masonry arch bridges will be expected to remain in service for a considerable period in the future, it is important not to carry out works that might compromise their inherent durability, e.g. increase the working stress regime in the masonry and thus potentially reduce its fatigue life.

Masonry bridges can be found in a large variety of designs. The arch may have different numbers of rings and different formations. Some areas where upgrading/repairs may be needed are given in Figure 4.2.9, SB-STR (2007).

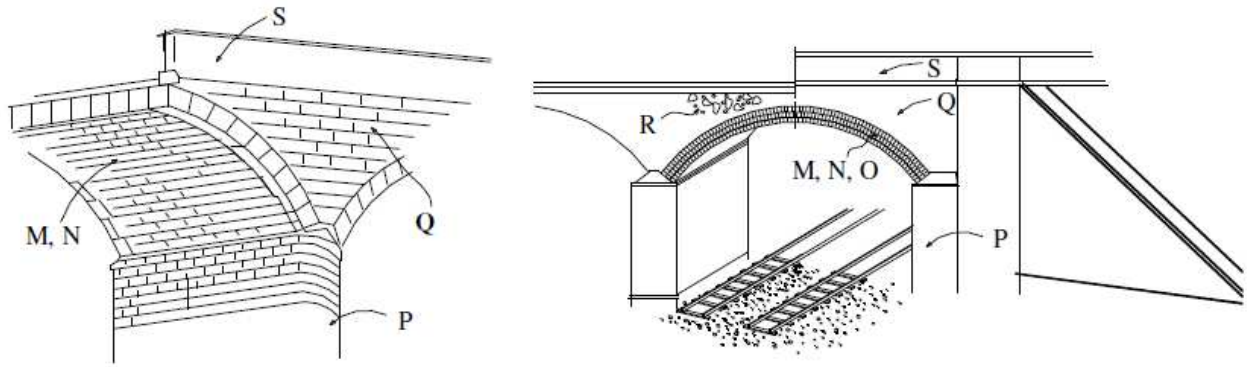


Figure 4.2.9. Masonry bridges with indications of different areas for upgrading/repair.

M – Deteriorating masonry; N – Inadequate load carrying capacity of arch or barrel; O – Separation between rings in multi-ring arch barrels (Delamination); P – Movements of piers and abutments; Q – Spandrel wall movement; R – Weak fill; S – Parapet damage

Example of upgrading and repair methods for the different areas in Figure 2.4.9 are:

- M Deteriorating masonry: Repointing, arch grouting, patch repairs, waterproofing and drainage
- N Inadequate load carrying capacity of arch or barrel:
Arch distortion remedial work, backfill replacement/reinforcement, concrete saddle strengthening, prefabricated liners, relieving slabs, retro-reinforcement and sprayed concrete lining
- O Separation between rings in multi-ring arch barrels (Delamination):
Arch grouting, patch repairs, through ring stitching.
- P Movements of piers and abutments:
Underpinning, patch repairs.
- Q Spandrel wall movement:
Backfill replacement/reinforcement, Spandrel strengthening
- R Weak fill:
Backfill replacement/reinforcement, grout fill if suitable.
- S Parapet damage:
Patch repairs, parapet upgrading.

Examples and methods are given in e.g. SB-LRA (2007) and SB-STR (2007). More material and backgrounds are given in e.g. Hendry (1998), UIC 778-3 (2014), SB-D4.7.1 (2007), SB-D4.7.4 (2007) and Harvey (2012).

4.2.2.4 Concrete Bridges

Relatively many concrete structures have a non-utilized in compression capacity and instead the amount of existing steel reinforcement limits the load carrying capacity. In cases when compressive strength limits the capacity, it is not possible to give general suggestions for improvement. In this section possible strengthening need will be given for; girder, trough, box and arch bridges.

4.2.2.5 Bridges with T-beams

In a beam bridge, the beams constitute the main load carrying structural element. They have to carry the loads from the traffic and the weight of the bridge deck as well as the self-weight and also to transfer the vertical and horizontal forces down to the substructure of the bridge. The beam bridge type of structure includes in situ casting and prefabricated beams and girders. The concrete beam bridge is usually constructed of two or more beams. The beams may be of T-section, rectangular or another shapes. In Figure 4.2.10 a beam bridge is shown with marked areas for possible upgrading. Example of upgrading and repair methods for the different areas in Figure 4.2.10 are given in Table 4.2.2.

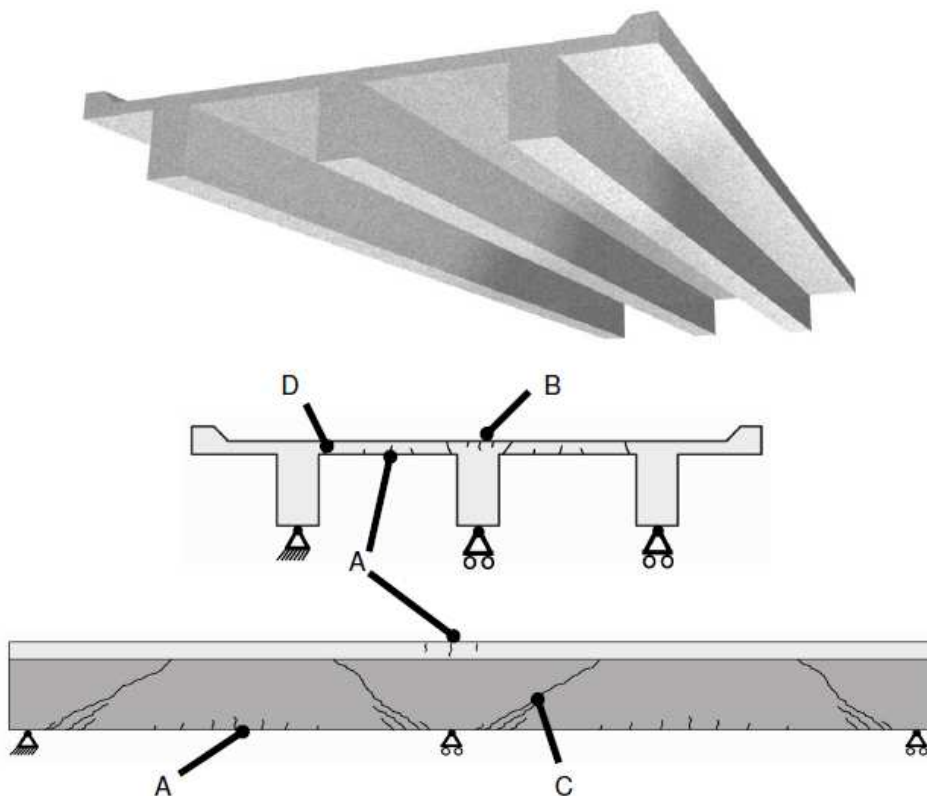


Figure 4.2.10. Beam bridge with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; B - Deficient flexural bearing capacity in areas hard to access; C - Deficient shear bearing capacity in beams; D - Deficient shear bearing capacity in slabs. SB-STR (2007).

4.2.2.6 Trough Bridges

The trough bridge shown in Figure 4.2.11 can be found in numerous alternative designs. Typically, two main girders, sometimes with a small flange at top, are connected with a slab at the lower part forming a trough. The trough is normally filled with ballast with one or two railway tracks at the same level as the upper part of the girders. The main girders can be different in size and shape with aligned or vertical sides. Trough bridges are often skewed.

Examples of upgrading and repair methods for the different areas in Figure 4.2.11 are given in Table 4.2.2.

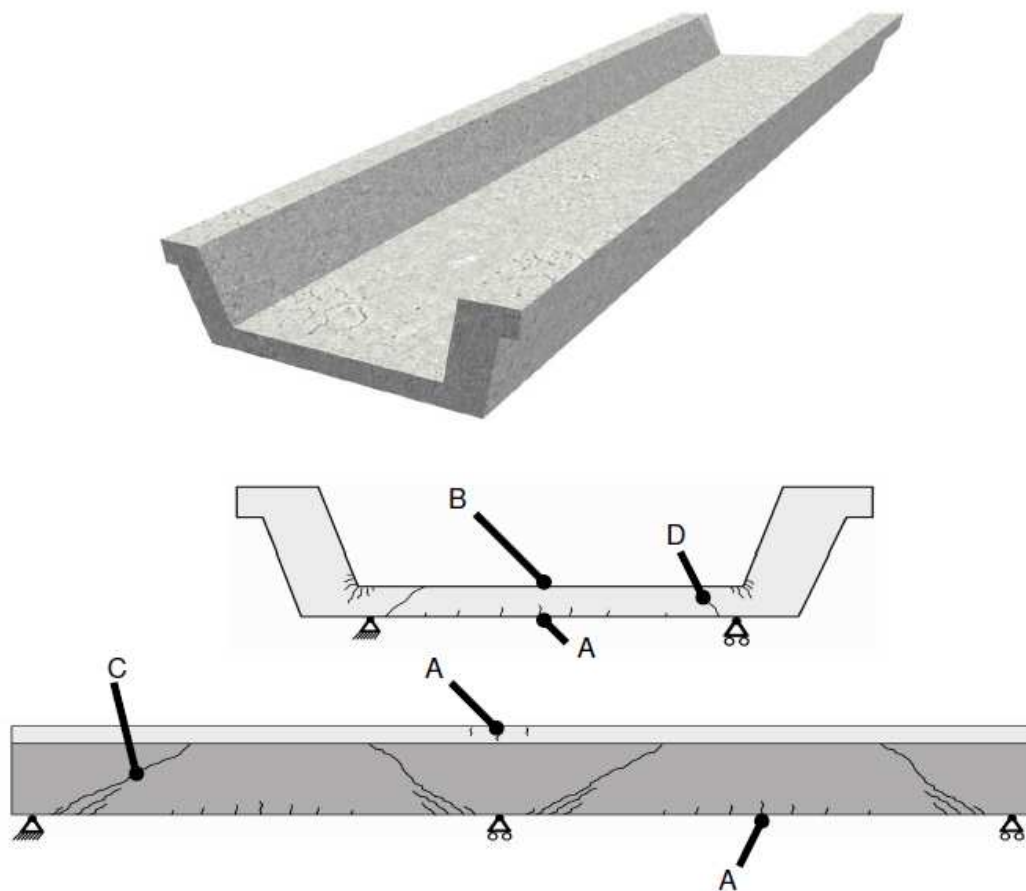


Figure 4.2.11. Trough bridge with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; B - Deficient flexural bearing capacity in areas hard access; C - Deficient shear bearing capacity in beams; D - Deficient shear bearing capacity in slabs. SB-STR (2007).

4.2.2.7 Box girder bridges

Compared to other beams, the deck of a box girder is identical to the top flange, the walls form the web and the bottom plate is similar to the bottom flange. The box girder bridge, shown in Figure 4.2.12 is often used for spans in the longer region. In such cases, the bridge carries a high amount of dead weight. Box girder bridges are often post tensioned and often carry two or more railway lines.

Examples of upgrading and repair methods for the different areas in Figure 4.2.12 are given in Table 4.2.2.

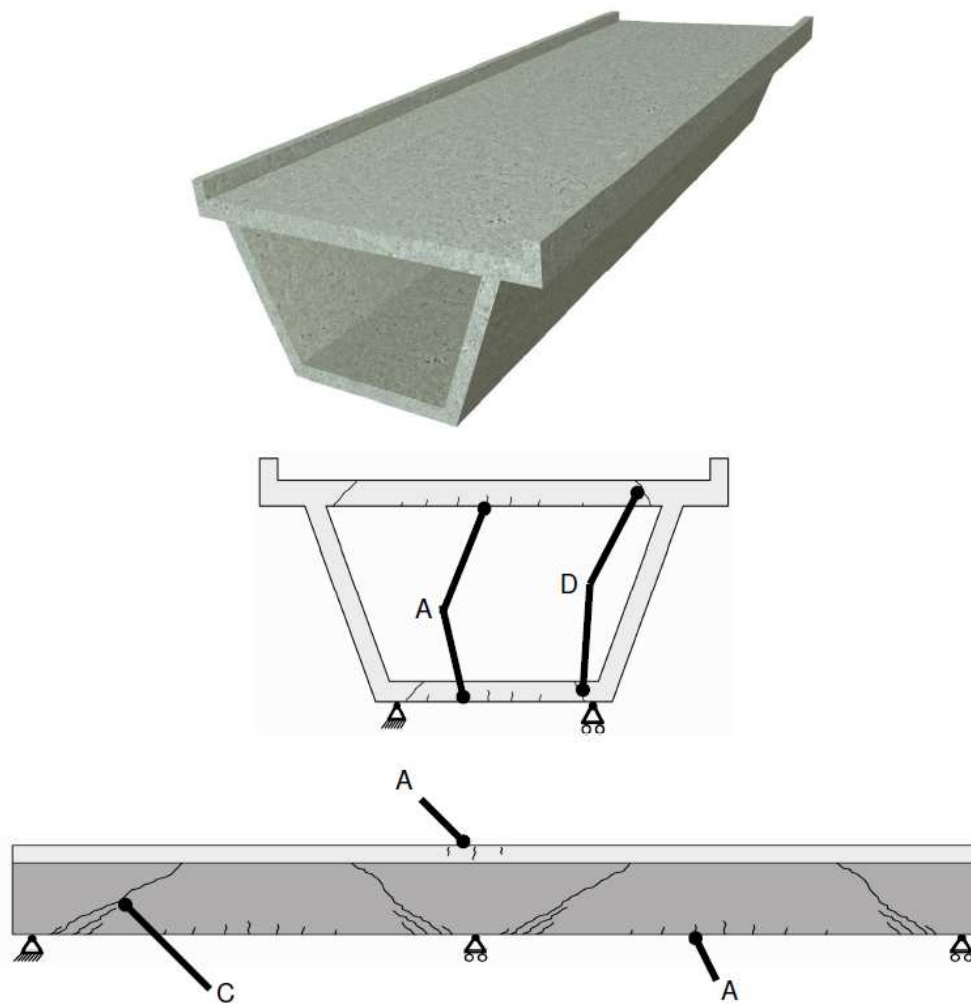


Figure 4.2.12. Box Girder Bridge with indication of possible areas for upgrading.
A – Deficient flexural bearing capacity; C - Deficient shear bearing capacity in beams;
D-Deficient shear capacity in slabs. SB-STR (2007).

4.2.2.8 Concrete arch bridges

The type of structure termed arch includes open and closed spandrel arch bridges as well as earth filled arch bridges. The bridge deck can be either above, between or underneath the arches. The concrete arch shown in Figure 4.2.13 can be found in several designs. Most arches are designed to carry loads through compressive forces. However, the arches often contain steel reinforcement and with the required loads of today, bending and tensile strains may be introduced in the arch. This means that arches once designed primarily for compressive forces, may also need to have sufficient shear and bending capacity.

Examples of upgrading and repair methods for the different areas in Figure 4.2.13 are given in Table 4.2.2

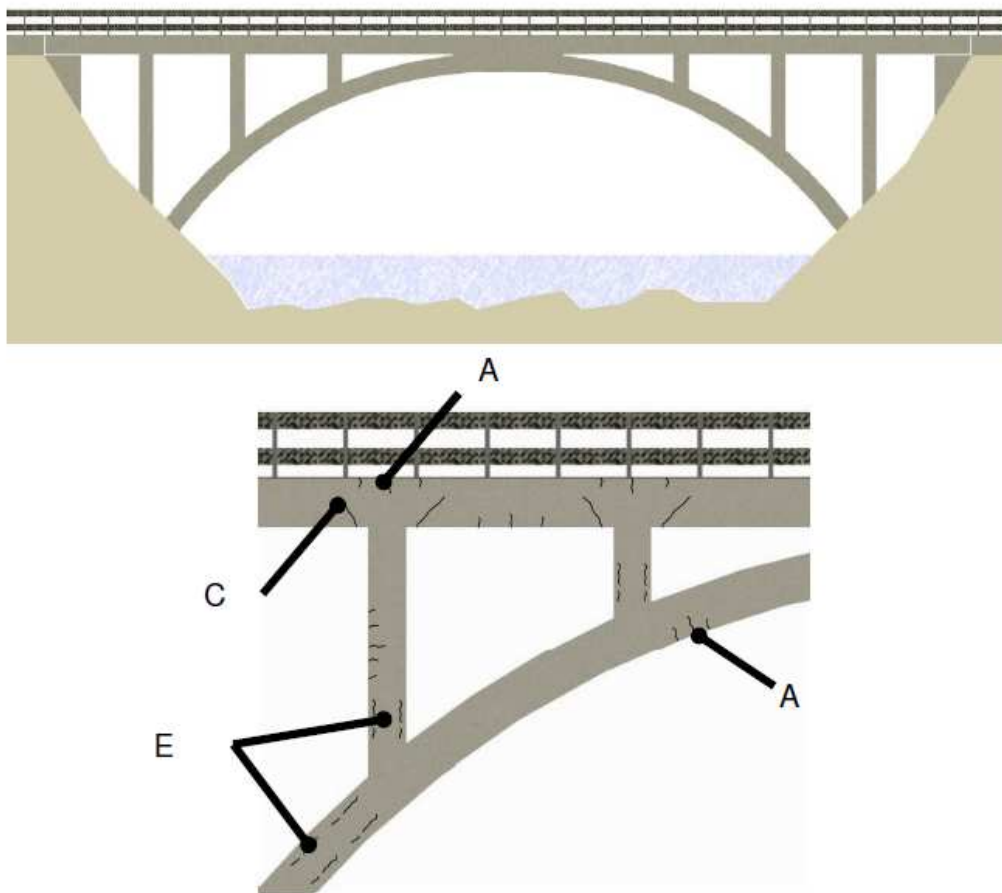


Figure 4.2.13. Concrete arch bridge with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; C - Deficient shear bearing capacity in beams; E – Deficient bearing capacity in columns and arches. SB-STR (2007).

4.2.2.9 Concrete structural components

There exist other bridges of reinforced concrete as well, not presented in this report. However, instead of aiming to present all the possible concrete bridges, structural components used in bridges will be presented below. Figure 4.2.14 shows a concrete column subjected to high compressive forces.

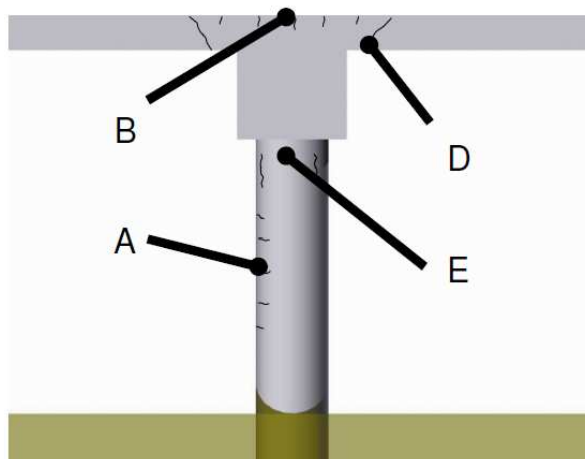


Figure 4.2.14. Concrete column and supporting slab with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; B - Deficient flexural bearing capacity in areas hard access; D - Deficient shear bearing capacity in slabs; E – Deficient bearing capacity in columns and arches. SB-STR (2007).

Figure 4.2.15 shows a concrete beam that can be loaded in any direction. However, most common for beam elements is that the element is loaded in a combination of shear and bending.

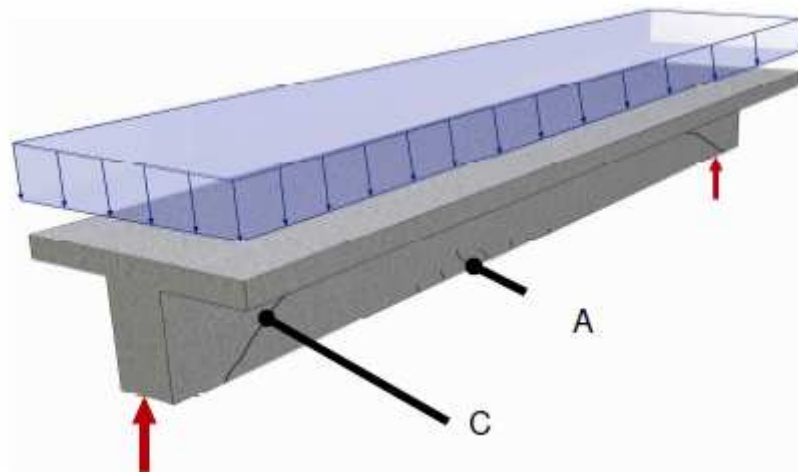


Figure 4.2.15. Typical concrete beam with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; C - Deficient shear bearing capacity in beams. SB-STR (2007).

Examples of upgrading and repair methods for the different areas in Figure 4.2.14 and 4.2.15 are given in Table 4.2.2.

Table 4.2.2. Examples of upgrading methods for concrete bridges, SB-STR (2007)

	Deficiency	Upgrading method
A	Flexural bearing capacity.	External Carbon Fibre Reinforced Polymer (CFRP) Plate or Sheet
		Mineral Based Composite (MBC)
		Near Surface Mounted Reinforcement (NSMR)
		External prestressing
		Increased cross section
B	Flexural bearing capacity hard to access	External prestressing
		Internal rods in drilled holes
C	Shear in beams	External Carbon Fibre Reinforced Polymer (CFRP) Plate or Sheet
		Mineral Based Composite (MBC)
		Near Surface Mounted Reinforcement (NSMR)
		External prestressing, longitudinal or transversal
		New stirrups and concrete casting
		Internal steel/CFRP rods
		Stitching
D	Shear in slabs	Internal steel/CFRP rods
		Internal or external prestressing
		Increase of load-transfer area
E	Columns	External Carbon Fibre Reinforced Polymer (CFRP) Plate or Sheet
		Steel jacket
		Increased reinforced cross section

Examples of methods with prestressing are given in e.g. Siwowski & Zóltowski, (2012) and Nilimaa (2015), see figure 2.4.16.

Methods for design of upgrading are given in e.g. SB-STR (2007), ML-D1.4 (2014), Täljsten et al (2011), fib B62 (2012), Reid (2009), ACI 440.4R-04 (2004) and ACI 440.1R-06 (2006). An example of the assessment of the effect of various strengthening techniques is given in Puurula et al (2015).

Noël & Souki (2013) have tested the effect of pre-stressing on the performance of glass fibre reinforced polymers (GFRP)-reinforced concrete slab bridge strips.



Figure 4.2.16. An upgraded concrete trough bridge built in 1959 in Haparanda in northern Sweden. In 2012 the bridge was strengthened with eight post-stressed bars installed in holes drilled through the slab (insert left). In this WAY, it was possible to increase the bending and shear capacity of the slab to enable an upgrading of the line from an axle load of 25 to 30 tonnes. ML-D1.1 (2013), Nilimaa et al (2013, 2014, 2015).

4.2.2.10 Prestressed Concrete Bridges

The difference between reinforced and prestressed bridges is that higher steel qualities are used that makes it possible to pre- or post-stress the concrete in compression so that tensile stresses and cracking can be avoided. Prestressed bridges can therefore often be longer than bridges without prestressing, see e.g. Collins & Mitchell (2001). A question that has to be clarified is the condition of the prestressing steel. Sometimes corrosion can be a problem, see e.g. Haixue (2014):

The strengthening methods given above for reinforced concrete can often also be used for pre-stressed concrete. However, in order to get enough effect, the strengthening materials also often needs to be pre-stressed. Below some recent examples are given.

Hassam & Rizkalla (2002) have tested half-scale models of a pre-stressed concrete bridge to failure. The test specimens consisted of one simple span and two overhanging cantilevers. Each specimen was tested three times using a different load location in each case. Five different strengthening techniques were investigated including near surface mounted Leadline bars, C-Bars, CFRP strips and externally bonded CFRP sheets and strips. Ultimate capacity, failure mechanism and cost analysis of various strengthening techniques for concrete bridges were presented.

Velerino et al (2006) have increased the shear capacity of a pre-stressed concrete bridge by using embedded Fibre Reinforced Polymer (FRP) bars.

Czaderski & Motavalli (2007) applied pre-stressed CFRP to strengthen a 17 m long pre-stressed I-beam that had been a part in the “Viadotto delle Cantine a Capolago” bridge constructed 40 years before the test, see Figure 4.2.17.

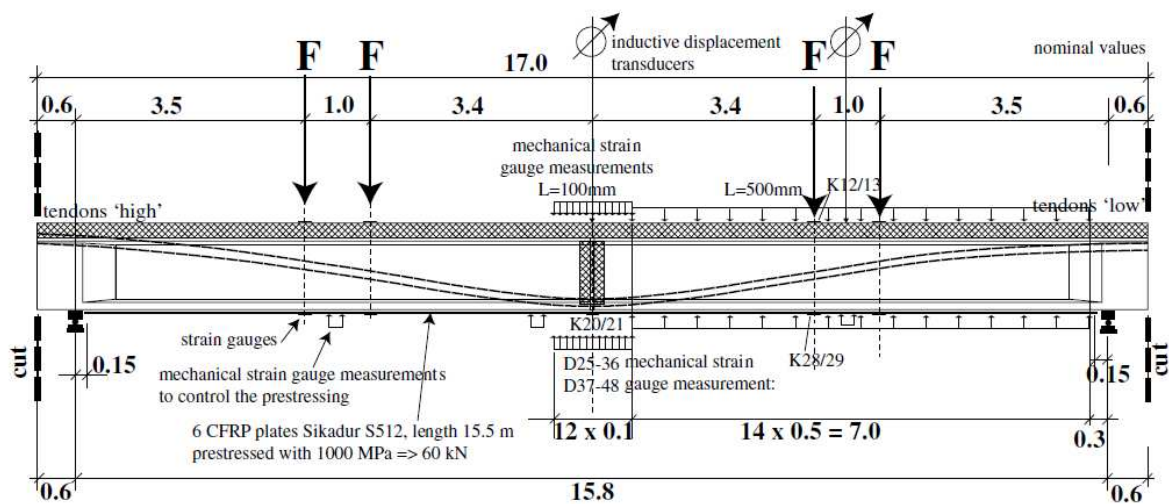


Figure 4.2.17. Test setup of a prestressed 40-year-old girder strengthened with prestressed CFRP plates, Czaderski & Motavalli (2007).

4.2.2.11 Composite Bridges

A structure in which the main bearing elements consist of parts of two different materials in structural cooperation is usually called a composite structure. The most common composite structures in bridges are those based on prefabricated steel girders onto which a slab of concrete is cast. In the same way, it is possible to think of pre-fabricated concrete beams in combination with an on-site cast slab. The slab can in turn be built up of elements of e.g. the so-called “filigree” type with an on-site cast concrete covering, see Figure 4.2.18.

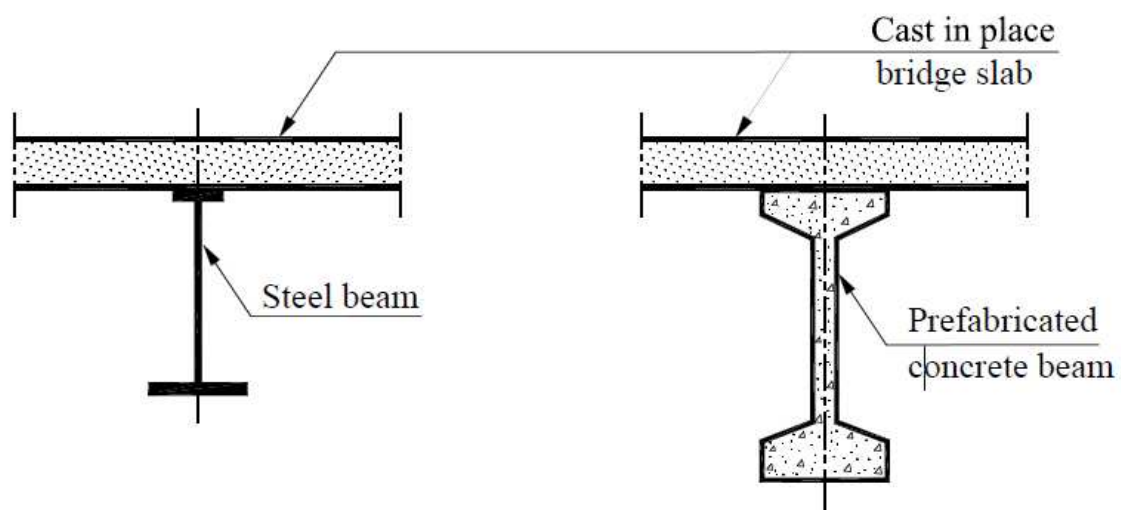


Figure 4.2.18 Examples of composite structures for a bridge. Collin et al (2012).

In order to achieve composite action or cooperation, some component that can transfer shear forces between the structural elements is required. As a result of this composite

action, a stronger and stiffer structure is obtained with a better utilization of the material. At the same time, it is an advantage that it is possible to prefabricate the main beams.

Bridges based on composite action between steel beams and a concrete slab have been used since the beginning of the 1970s. After the development of effective shear connectors in the form of shear studs this type of bridge has been widely used especially for spans in the order of 30 to 60 meters. Collin et al (2012), Collin (2013), El Sarraf et al. (2013)

Strengthening of composite bridges can be carried out by the principles presented earlier, i.e. prestressing or adding steel or CFRP on sections with too high stresses.

4.2.2.12 Examples of Assessment and Testing of bridges

The hidden capacity of bridges can be found by an enhanced assessment procedure. Several such assessments have been confirmed by load tests to failure of structures that have been – or were scheduled to be – taken out of service. The testing is also a perfect way to calibrate models for the load-carrying capacity. Important experiences in recent projects have been obtained in this way. Some examples are given below.

The possibilities were investigated to increase the axle load on Malmbanan (the Iron Ore Line in northern Sweden) from 25 to 30 ton. A full scale test was carried out in the laboratory at Luleå University of Technology of a 29-year-old trough bridge with a length of 7,2 m, see Figure 4.2.19, Paulsson and Töyrä (1996), Paulsson et al (1996, 1997), Thun et al (2000, 2006) and Elfgren (2015). Before the final test, the bridge had been loaded with 6 million load cycles with an axle load of $1,2 \times 30 = 36$ ton without any notable damage formation and with only hairline cracks in the bottom of the slab. The tests showed that the fatigue capacity of the bridge was much higher than what was predicted by the codes, Thun et al (2000, 2006).

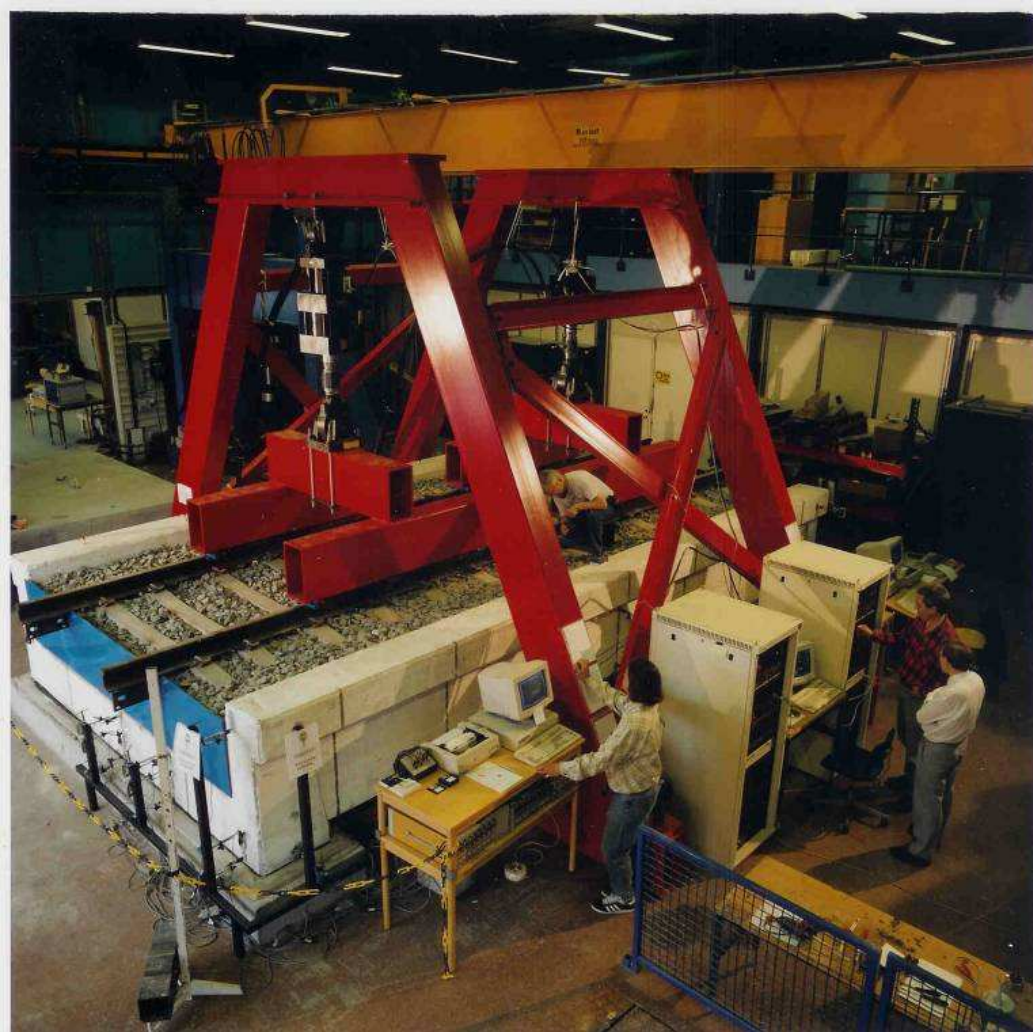


Figure 4.2.19. Full Scale Test of a 29-YEAR-OLD Railway Trough Bridge at Luleå University of Technology. Paulsson et al (1996, 1997), Elfgren (2015).

Another trough bridge on the Iron Ore Line at Luossajokk was in 2002 assessed with a reliability-based method to see if it could carry an increased axle load of 30 tonnes. It was shown that a satisfactory reliability index of $\beta > 4.75$ could be reached for reasonable combinations of loads, Enochsson et al (2002).

In the Sustainable Bridges Project a 51 year old, two-span trough bridge (12 + 12 m) was assessed and strengthened in bending with Near Surface Mounted Reinforcement (NSFR) consisting of rectangular Carbon Fibre Reinforced Polymer (CFRP) bars glued into grooves sawn into the bottom of the slab. The bridge was then tested to failure in 2006 and an interesting combined failure in bond, shear, bending and torsion was obtained for a load six times the design load, see Figure 4.2.20, SB-D7.3 (2008), Puurula et al (2012, 2013, 2015).



Figure 4.2.20. Test to failure of a strengthened trough in Örnsköldsvik, 2006. Top: Overview of bridge with East rail bank cut away and replaced by a temporary road to substitute the lane under the bridge span being tested. Below: Detail of failure in South beam caused by a combination of stresses due to bond, shear, bending and torsion. SB-D7.1 (2006), Puurula et al (2012, 2013, 2015).

In the MAINLINE project a 55-year-old 33 m long metallic truss bridge was assessed and tested in 2013 as presented in section 4.2.1.2. This bridge could also carry loads much higher than the design loads, ML-D1.3 (2015).

In 2014 a 55-year-old five span prestressed concrete beam bridge with a total length of 121.5 m was assessed and tested to failure in Kiruna in northern Sweden, see Figure 4.2.21. The final failure in bending and shear is still (2015) being analyzed but preliminary results are presented in Bagge et al (2014, 2015) and Nilimaa (2015).

The project provided a rare opportunity to monitor a post-tensioned concrete bridge during tests to failure using a wide array of instruments. The primary aim was to acquire relevant data for calibration and development of methods for assessing prestressed and posttensioned concrete structures. The test programme included evaluations of the super-structure's behaviour, two Carbon Fibre Reinforced Polymer (CFRP) strengthening systems and conditions of the post-tensioned cables.

The Near Surface Mounted (NSM) Carbon Fibre Reinforced Polymer (CFRP) systems used for strengthening did not exhibit debonding failure until a late loading stage of failure thus proving its reliability. The prestressed laminates debonded at rather early stages of loading. Further analysis of the recorded strains is needed to identify the utilization of the fibres and the contribution of this strengthening system to the capacity of the bridge.

Two finite element (FE) models showed good agreement with the test for loads up to approximately 9 MN. However, these models were based on characteristic material properties, with no bond models for the laminates and with assumed levels of post-tensioning force. Before the failure test, the bridge girders were already loaded to concrete cracking adjacent to the load application. This process was not taken into account in the FE analyses, hence the difference in stiffness, see Figure 4.2.21. Moreover, the FE analysis is based on equally loaded beams. That the results were still in good agreement confirms that even with a limited amount of information good predictions of the behaviour of bridges can be obtained using this method.

The results acquired during the investigations of the Kiruna Bridge suggest that the following aspects warrant further attention:

- Robustness, ductility and bridge behaviour;
- Shear force resistance of bridge girders;
- Shear force and punching resistance of bridge slabs;
- Condition assessment of post-tensioned steel cables;
- Temperature effects on deformations and strains;
- Strengthening methods using CFRP;
- Finite element model updating;
- Reliability-based analysis.

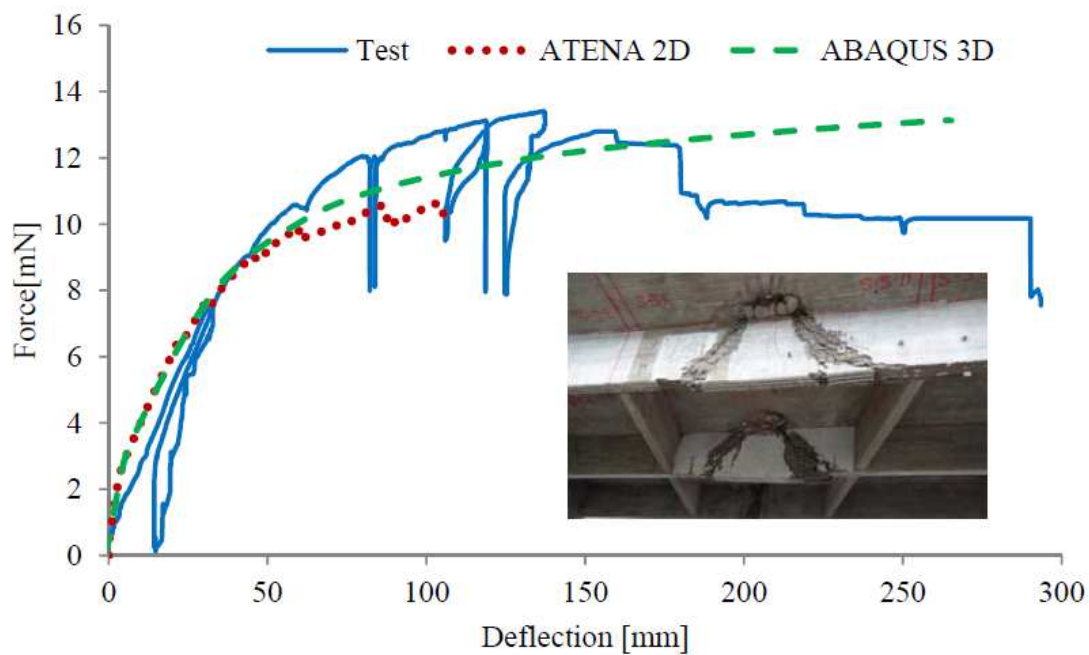


Figure 4.2.21. Test to failure of a strengthened prestressed five span bridge in Kiruna in 2014. Top: Overview from N-E with the LKAB iron ore mine in the background. Below: Tested and calculated load-displacement relationships for the failure test of the bridge girders and a photograph of the South and middle girders after failure. Bagge et al (2014, 2015).

4.2.3 TUNNELS

According to Table 4.2.1 it is mostly the need for a greater loading gauge that influences the upgrading of tunnels. The loading gauge defines the maximum height and width for railway vehicles and their loads to ensure safe passage through tunnels. A clearance is added to the loading gauge to obtain the structural gauge (also called the minimum clearance outline).

Below some guidelines are given. A general handbook on tunnels is Maidl et al (2013, 2014). More material can be found in ML-D1.2 (2014) and in UIC-779-9 (2003), UIC-779-10 (2011) and UIC 779-11 (2005). There is also a special working group on “Tunnel Redevelopment”, organized by STUVA (Studiengesellschaft für unterirdische Verkehrsanlagen mbH, Research Association for Underground Transportation Facilities Ltd), see Simon (2012).

4.2.3.1 Introduction

Basic tunnel sections for different subgrades are outlined in Figure 4.2.22.



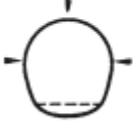
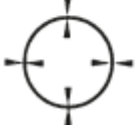
	<p>Rectangular</p> <p>Used when the external forces do not lead to any damaging movement of the rock mass into the tunnel.</p>
	<p>Semi-elliptical, parabolic or semi-circular</p> <p>Used when vertical forces act.</p>
	<p>Horseshoe, vaulted</p> <p>Used when horizontal and vertical forces act.</p>
	<p>Circular</p> <p>Used when forces act from all sides and particularly with internal water pressure.</p>

Figure 4.2.22. Basic tunnel sections for different subgrades. Maidl et al (2014).

Tunnels can be either lined or unlined and have a variety of shapes as illustrated in Figure 4.2.23.

The material used to line tunnels can be masonry (either stone or brick), in situ concrete, precast concrete segments or metallic (cast iron or steel) segments and each presents its own challenges when life extension is being considered.

As most railway tunnels date back to the 19th century they were inevitably constructed using hand tunnelling techniques, which usually required the construction of vertical access shafts from ground level. In some cases, these shafts were left open to provide ventilation but others were backfilled and are referred to as buried or hidden shafts whose location is often unknown. Shafts are discussed in ML-D1.3 (2015).

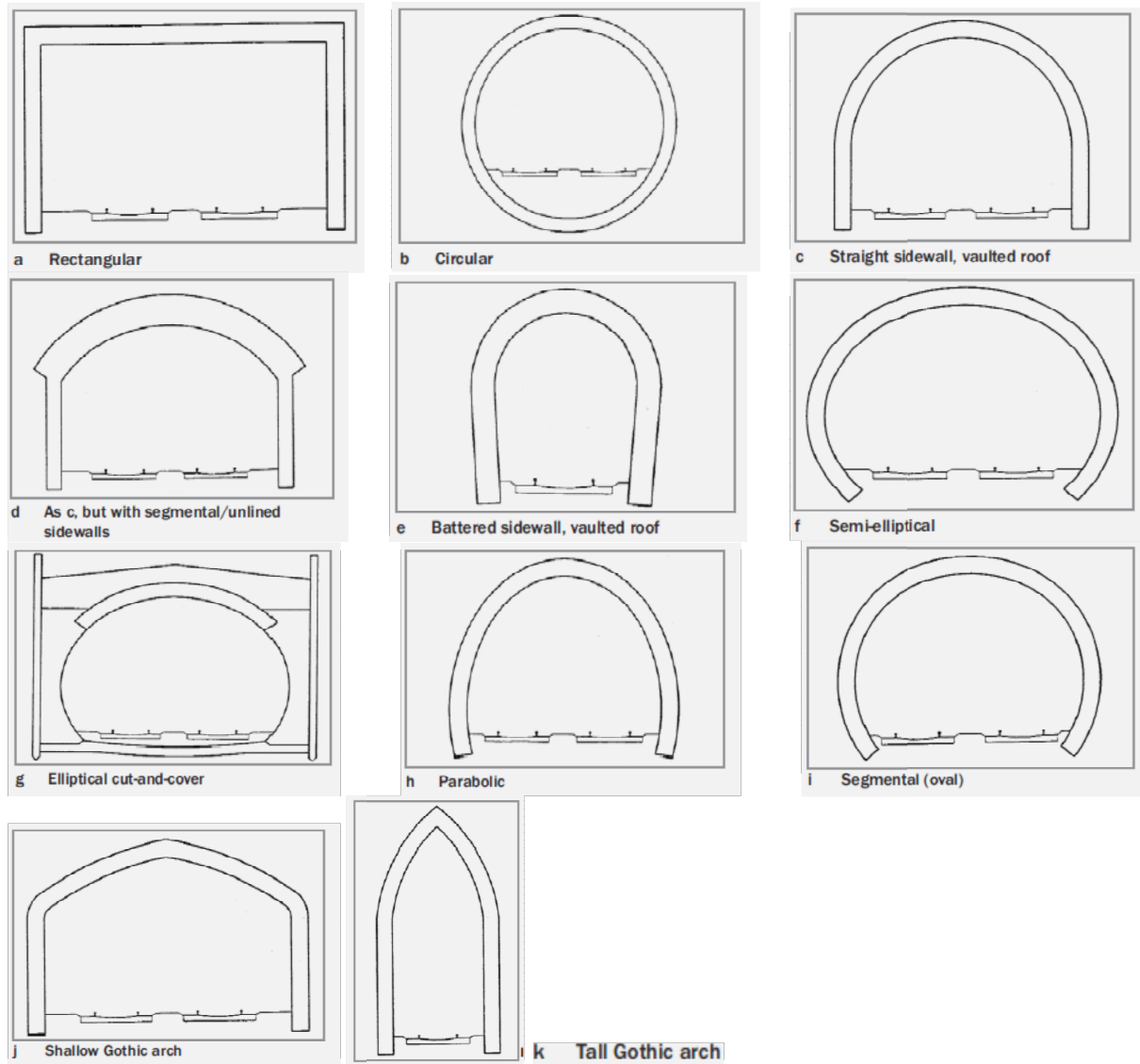


Figure 4.2.23. Typical UK Tunnel cross sections, Railtrack (1966)

The upgrading of a railway line to higher axle loads usually does not present any problems for tunnels as the sub ground mostly has a high load-carrying capacity. However, if there is a need to increase the tunnel profile, a major challenge is at hand. Then the present profile must be enlarged and this may lead to new drilling and blasting in the same way as when a new tunnel is constructed. Maidl et al (2013, 2014) give guidance for construction in the aforementioned handbooks.

Some standard loading gauges are shown in Figure 4.2.24. The European Rail Agency (ERA) in its Technical Specifications for Interoperability (TSI) has defined a number of recommendations to harmonize the train systems. The TSI Rolling Stock (2002/735/EC) has taken over the UIC Gauges definitions defining Kinematic Gauges with a reference profile such that Gauges GA and GB have a height of 4.35 m (they differ in shape) with Gauge GC rising to 4.70 m allowing for a width of 3.08 m of the flat roof. All cars must fall within an envelope of 3.15 m wide on a 250 m radius curve. The French high speed trains (TGVs), which

are 2.9 m wide, fall within this limit. The designation of a GB+ loading gauge refers to the plan to create a pan-European freight network for ISO containers and trailers with loaded ISO containers, UIC 506 (2008).

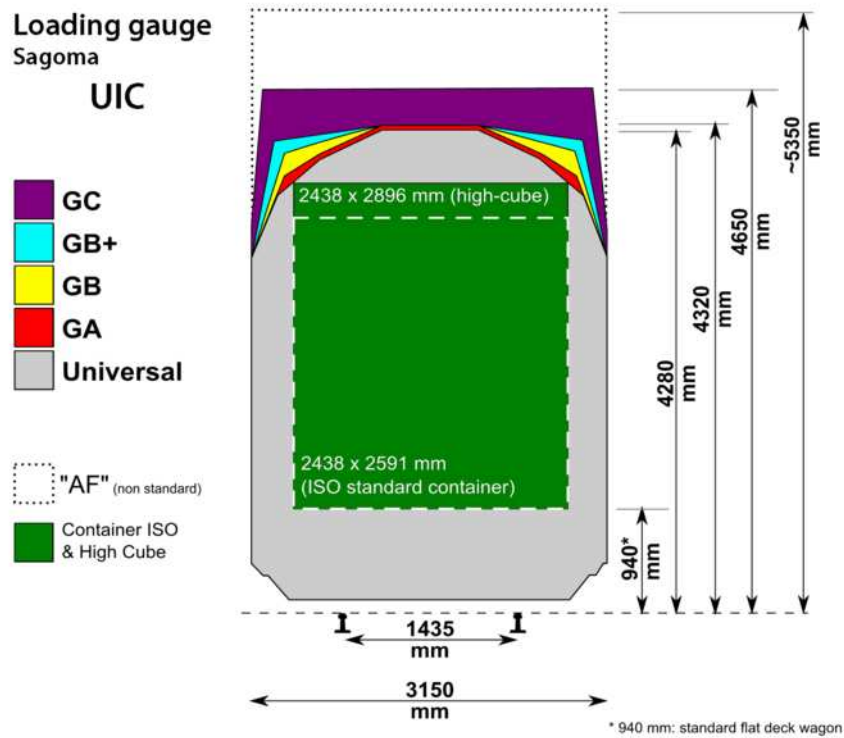


Figure 4.2.24. Different loading gauges and a standard ISO container. UIC 506 (2008).

Technical possibilities for the STUVA group on Tunnel Rehabilitation studies working in tunnels with rail services in operation, see Table 4.2.3

Table 4.2.3. Technical Possibilities for Working in Tunnels with Rail Services in Operation. Simon (2012)

Work in tunnel*	operational without closure times			operational with closure times		
	Single-track tunnel	2-track tunnel		Single-track tunnel	2-track tunnel	
		Alternating track operation	Conversion to 1-track in tunnel for construction period		Alternating track operation	Conversion to 1-track in tunnel for construction period
Vault support						
Mining means						
Support plate against caving						
Redeveloping vault and local enlargements						
Adapting clearance profile						
Improving the vault						
Vault enlargement						
Removing the vault						
Enlarging tunnel in rock						
Mining means possibly blasting			TIT			TIT
Vault support						
1-layer						
2-layer						
Segment lining						
State of technology/references available			Future application feasible			
*The restrictions and regulations in conjunction with the overhead line must be checked separately in the case of electrified routes!						

4.2.3.2 Conditions in different countries

4.2.3.2.1 Germany

In Germany, see Simon (2012), the DB AG is obliged to follow the data given by the EBO (ordinance regulating the construction and operation of the railways) to enlarge the track distance from in some cases only 3.5 m up to 4.0 m for new tracks and extensive upgrading projects. Widening the track distance results in part in protracted construction measures, which can be made even more difficult by having to renew the floor. As a result, the so-called tunnel-in-tunnel (TIT) method was applied for the first time when redeveloping the Jährod and Mausenmühlen tunnels, see Figure 4.2.25.

When renovating tunnel structures in excess of 1,000 m the new tunnel must be produced with 2 bores for such total redevelopments or renovations to comply with the regulations now laid down in the valid safety guidelines. This signifies the construction of a new single-track bore parallel to the existing one, which must be renewed through transforming it from a twin-track to a single-track tunnel. These 2 tunnels are then connected every 500 m with cross-passages for evacuation purposes.



Figure 4.2.25. Jährod Tunnel in Germany being enlarged by the Tunnel-in-Tunnel (TIT) method, Simon (2012).

4.2.3.2.2 Austria

There are numerous new tunnels being built or planned in Austria as well, Simon (2012). These include some 350 km of mainline tunnels including the Brenner Base Tunnel with a total driven length of roughly 110 km. Redevelopment measures devised to improve the substance such as in the Arlberg Tunnel or the Tauern Tunnel also take place against the background of improving safety technology. The measures to enhance “Tunnel Safety” were triggered some 10 years ago – owing to the fire incidents in the Mont Blanc road tunnel (March 1999), in the Tauern motorway tunnel (May 1999) and in the Channel Tunnel (November 1996).

Austria’s regulations designed to protect the workforce represent a further source for modifying and partial redevelopment of old tunnels.

4.2.3.2.3 Switzerland

In Switzerland the floor is frequently renovated on account of damage resulting from drainage and heaving rock formations, in many cases also in conjunction with lowering the floor to adjust the cross-section for future piggyback transportation. In the case of twin-track tunnels, it is also necessary to install a ballast retention system to secure the operational track. , Simon (2012).

4.2.3.2.4 Sweden

Sweden has since the seventies allowed special transports. These transports had both heavier loads and larger loading gauges than standard ones. In 1990, a special program was developed called LÖ90. To this end, positions and geometry of all obstacles were accurately measured according to a Swedish code called FOMUL-measuring. In the code all existing obstacles defined. It was also defined how often they should be controlled and measured. All special transports had to have a special permission. This was carried out during many years and gave SJ Freight good profits.

When discussion in 1998 started with Stora Enso they wanted to have both higher axleloads and larger loading gauges. Since Banverket had good control over the situation from working with LÖ90, Stora Enso was allowed to operate the “Stora Boxen” (the "big box") on some lines starting late 1999.

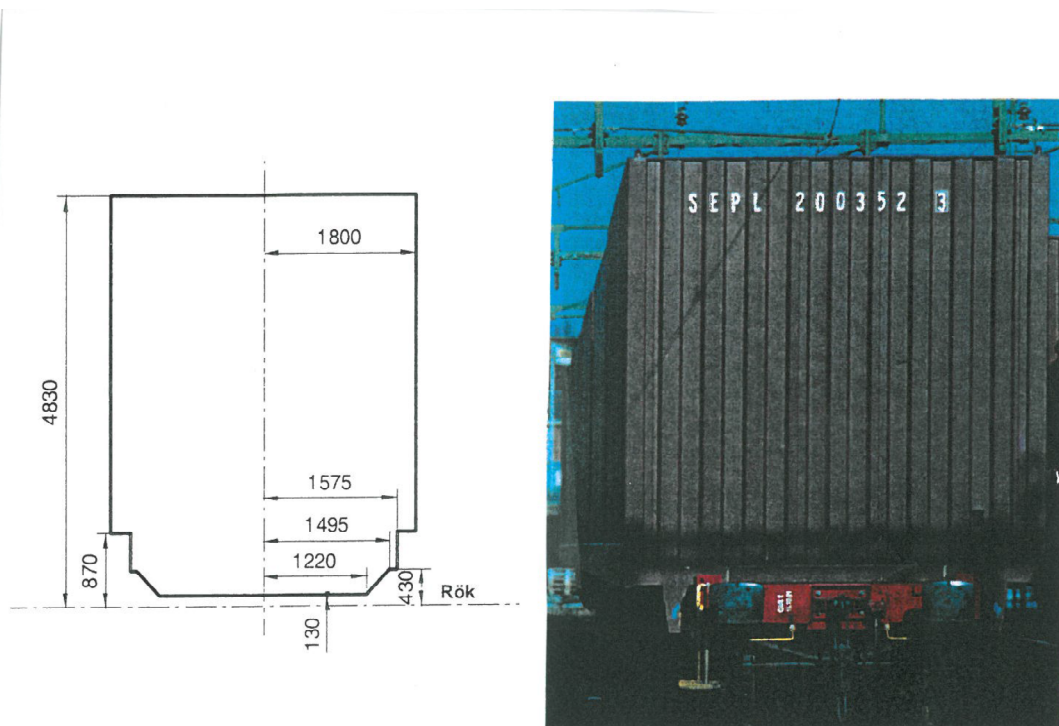


Figure 4.2.26. Swedish load profile C (left) AND a picture of the BIG box.

Today more than 25% of the net is upgraded with load profile C. The cost for upgrading is very dependent on the specific local circumstances. The costs to investigate and if necessary to move obstacles was on average 10 k€ per kilometre. The main problem was the clearance pole at meeting stations that was very expensive to move. This problem was therefore solved by putting an empty wagon with a standard load profile at the end of the train.

During the investigation and FOMUL-measuring, it was noticed that three tunnels were not within load profile C. For one of them it was enough with small measures and some upgrading of the catenary system. For the other two a lowering the level of the track was necessary. All work had to be carried out during vacation time in 1999. An important

experience was that logistics work has to be well planned. Additional costs compared with budget were 12% which is low in Swedish tunnel-contracts.

4.2.3.2.5 USA

One example of a cross section expansion is the Weehawken tunnel built 1881–1883 by the New York West Shore Railroad. The original rail freight tunnel had a width by height section of 8.4 x 6.4 m and it was in 2004–2006 enlarged to 19.7 x 10.4 m. The tunnel has a length of 1.25 km, had originally a semi-elliptical arched roof with nearly vertical walls, and was constructed for two tracks with a 4-m centerline distance. The majority of the tunnel was constructed in the rock of the Palisades Diabase. For approximately three-quarters of its length, the tunnel was unlined and the natural rock exposed. Eight sections of the tunnel were lined with brick masonry.

The geometrical requirements of the new tunnel cross-sections were established taking into consideration the dynamic clearance envelope of the light rail vehicle; system and catenary clearances; construction tolerances; a maintenance walkway; presence of the existing utility duct banks and need for their uninterrupted service in all phases of construction; requirements of minimizing the excavation quantities; and the tunnel ventilation requirements. Different “modified horse-shoe” shaped cross-sections were identified for the unlined sections and fully lined sections of the tunnel resulting in a new width by height section of 19.7 x 10.4 m, Berliner et al (2004). New concrete walls were added to decrease the risk of spalling, especially in the case of fire. A special mix was created with polypropylene fibers to form cast-in-place concrete walls in the station and in areas that did not already have the tunnel’s original brick arch and masonry sidewalls. Another mix was created for the drainage and waterproofing system. Here shotcrete (a high-powered spray application of concrete) was used for the liner at station-to-tunnel transitions and for the arched lining at the river portal. The new tunnel is shown in Figure 4.2.27.



Figure 4.2.27. The Weehawken Tunnel in New York was in 2006 widened from 8.4 x 6.4 m to 19.7 x 10.4 m, see Berliner et al (2004). Photos by John Papas 2004 and David Pirman 2006.

As part of the process of determining the need to undertake life extension of a tunnel, it is sometimes necessary to undertake a structural assessment. This will follow similar lines to

that adopted for bridges constructed of the same basic engineering material and can include the advanced techniques described in chapter 4.2.1 of this guideline.

4.2.3.3 Extension of unlined tunnels

The deterioration of unlined tunnels is usually limited to failures of the rock through which the tunnel is driven, often caused by ground water permeation or the effect of frost. This can vary from the regular loss of small pieces from friable rock to the dislodgement of blocks of stone, which could be large enough to derail a train as shown in Figure 4.2.28. The potential for the latter type of failure is largely dependent on the orientation of the bedding planes within the rock mass.

Life extension measures are largely traditional and can consist of (based on CIRIA C671 (2009));

- Scaling of loose material or the removal of loose blocks
- Provision of protection shelters
- Localised rock bolting
- Rock bolting and rock fall protection mesh
- Localised application of sprayed concrete to support weak areas of rock
- Installation of supporting ribs
- Installation of full lining, usually using sprayed concrete
-

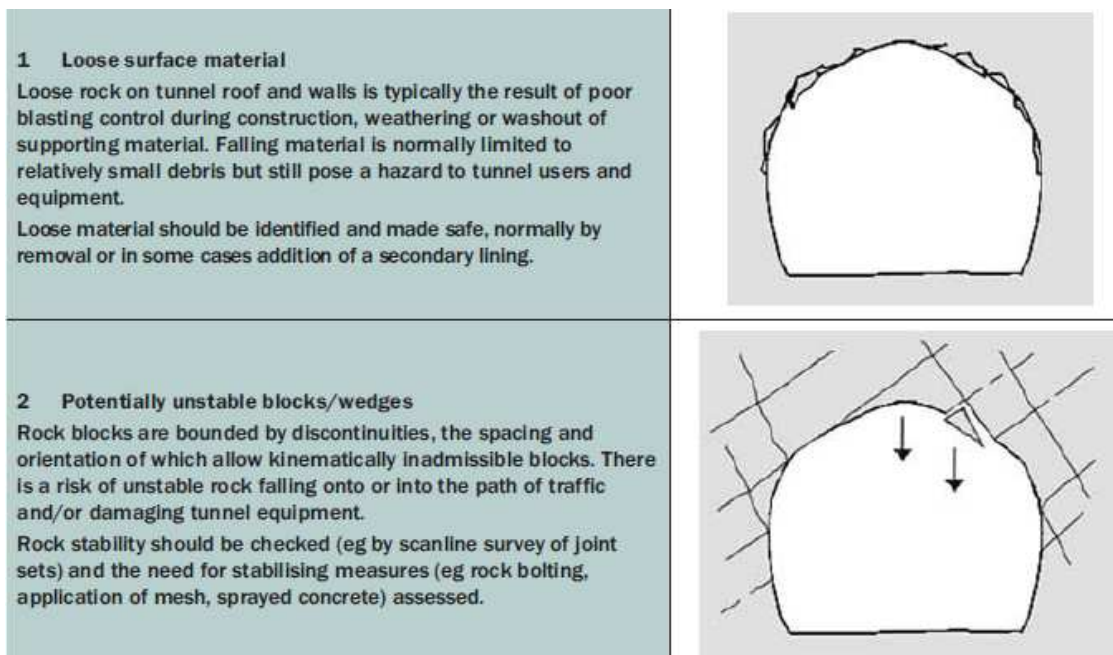


Figure 4.2.28. Defects in unlined tunnels .CIRIA C671 (2009)

4.2.3.4 Extension of lined tunnels

4.2.3.4.1 Masonry

The deterioration of masonry in tunnel linings is very similar to the deterioration of masonry in arch bridges and can be due to:

- Freeze thaw action
- Salt attack
- Sulphate attack
- Leaching of mortar
- Biological attack
- Use of unsympathetic repair material
- Thermal effects
- Saturation
- Ground movement
- Fatigue

These deterioration mechanisms usually manifest themselves as spalling of the masonry lining, cracking of the masonry lining or loose masonry caused by loss of mortar.

Traditional repair/life extension techniques usually consist of:

- Patch repairs
- Crack repairs
- Ring separation repair
- Grouting
- Underpinning
- Invert repair

New support systems have been developed, which can be left in situ permanently. One example is shown in case study 2 in Appendix C in ML-D1.3 (2014)

4.2.3.4.2 In situ concrete linings

In situ concrete linings usually suffer from similar deterioration mechanisms as other forms of concrete structure (principally carbonation or chloride-induced corrosion) and the repair methods generally follow traditional lines. Advice on how to ensure quality repairs can be found in numerous publications and research project outputs; one example is the manual from the European project, Rehabcon (2004).

More recently the use of both steel and carbon fibre to strengthen tunnel linings has started to become accepted, particularly in Japan. Details of one steel system can be found in Nippon (2005) and details of carbon fibre strengthening can be found in Karbhari (1998).

4.2.3.4.3 Concrete segments

Tunnels lined with concrete segments mostly have to be rebuilt. A detailed engineering assessment will be needed before deciding on the correct action.

A new lining will be necessary. This can be undertaken either by the installation of replacement segments or by the use of sprayed concrete and – although expensive – can be carried out in very short lengths during overnight closures of the tunnel.

4.2.3.4.4 Metallic segments

Metallic segments can be manufactured out of either steel or cast iron and are usually only to be found in circular tunnels. The most common defects are cracking (in general only at boltholes) and corrosion. Steel linings can be repaired by welding or bolting new steel into position but cracked cast iron should be repaired using stitching. Corrosion in general affects steel linings. They should therefore be treated using conventional surface coating techniques.

There is at least one example of the replacement of cast iron segments with cast stainless steel segments in an area where sulphuric acid was present in the ground water (reported in Appendix A1.9 of CIRIA C671). The solution has been designed for a 400-year life and, whilst expensive, was deemed to be the only practical solution in this particular case.

Three case studies are presented in Appendix C in ML-D1.3 (2015).

4.2.4 CULVERTS

According to Table 4.2.1 it is mostly a higher axle load that influences the use of culverts.

Culverts are sometimes neither well documented nor maintained in a professional way. The situation is different in different countries.

Guidelines and a handbook for assessment and repair of culverts have been issued by UIC and Trafikverket, see Södergren (2011) and TRVH Avvattning (2011). General design guides for culverts have been published by e.g. FHWA (2000) and Balkham (2010).

4.2.4.1 Current Situation

Pipes and culverts have different cross sections and can be built up in various ways. They can for instance be square, rectangular, circular, arch shaped and be made of separate parts or just one unit, see Figure 4.2.29



Figure 4.2.29. Different types of culverts, Södergren (2011)

When the railway was new, many culverts were made with a square or rectangular cross section piled up with blocks of rock like masonry. Today's culverts are often pipes with a circular cross section and often made in sections, although they can also be in a single unit. Common materials for circular culverts made of sections are concrete, steel or plastic while those of one unit normally are made of plastic or steel. In recent years arch shaped culverts made of precast concrete with an interlocking effect have been introduced on the market.

Depending on how the pipes and culverts are designed and what materials they consist of they are susceptible to different types of damage.

Each buried pipe or culvert is subjected to loads that cause stresses in the walls. Some of these loads can be calculated. Others that can cause problem are not easy to calculate.

Loads that can be calculated

- Earth fill above;
- Traffic load.

Loads that cannot be calculated easily

- Elongation of pipes and culverts under deep fill,
- Settlement in the foundation,
- Frost heave,
- Lack of lateral support due to insufficient compaction in the surrounding soil,
- Restraints caused by for example manholes,
- Thermal effects.

The UIC group Panel of Structural Experts (PoSE) discussed the use of Pipes and Culverts and the requirements concerning design and installation methods and made a survey in 2009. Eighteen Rail Administrations answered a questionnaire and the result showed that there is a great difference in practices, Södergren (2011).

Some examples of the result are summarized below

- Material accepted

Reinforced concrete is accepted in all countries, steel in all except one, concrete in 10 and fibre glass in 7 countries. Corrugated steel pipes are used in 9 countries. Other materials mentioned are plastics and stone.

- Minimum depth under track

The minimum depth varies between 0.55 m and 2.2 m

- Angle between track and pipe

7 countries require 90°, other minimum angles mentioned are 80° – 30°, 4 countries have not required any minimum angle.

- Maximum diameter of pipe allowed without a static calculation

10 countries require calculation in all cases; the other 8 countries have given a diameter above which calculation is needed.

- Traffic load

Dynamic factor applied to the classic Load Model UIC 71 varies between 1.0 and 1.67

- Fatigue

4 countries are requiring fatigue checks.

- Foundation

6 countries have a special regulation for the foundation.

- Back fill

15 countries have regulation for back fill and compaction work.

4.2.4.2 Damage and malfunction

Many old culverts have barrels built by boulders forming both walls and roof. With increased axleloads, the boulders may separate, which causes sand from the bank on top of the culvert to pour down into the culvert. This and other examples of damage are illustrated in Figure 4.2.30.



Figure 4.2.30. Examples of damages in culverts (from top left clockwise): (a) Cracking and downfall of bank material; (b) Ice clogging the culvert; (c) Crater in bank due to downfall of material into culvert; (d) Erosion along bank. TRVH Avvattning (2011).

4.2.4.3 Measures

Examples on measures to restore culverts are given in Figure 4.2.31. If only small deformations appear, a possibility is also to protect the culvert with a fiberglass lining. After cleaning the culvert, the lining is inserted and hardened by ultra violet light.



Figure 4.2.31. Examples of measures to restore culverts (from top left clockwise): (a) Lining with plastic material; (b) Repair of joints; (c) Extension of culvert; (d) Corrugated steel culvert repaired with glass fibre lining. The lining expands at the ends of the culvert and stabilizes the embankment. Södergren (2011), Railcare (2012), ML-D1.1 (2013).



Figure 4.2.32. Left: Old stone culvert where the boulders have separated and sand and mud have poured down. Right: A culvert is kept together by post-tensioning of steel bars Sjöberg et al (1998), TRVH Avvattning (2011).

The problem with separation of blocks has been analysed by Sjöberg et al (1998). The recommendation was to keep the culvert together with post tensioning with steel bars, see Figure 4.2.32.

4.2.5 RETAINING WALLS

Retaining walls are structures designed to restrain soil to unnatural slopes. They are used to bound soils between two different elevations often in areas of terrain possessing undesirable slopes or in areas where the landscape needs to be shaped severely and engineered for more specific purposes like hillside farming or roadway overpasses. Various types of retaining walls are illustrated in Figure 4.2.33, see e.g. Terzaghi (1943). Table 4.2:1 indicates that a change in traffic situation it is mostly a higher axle load that influences retaining walls.

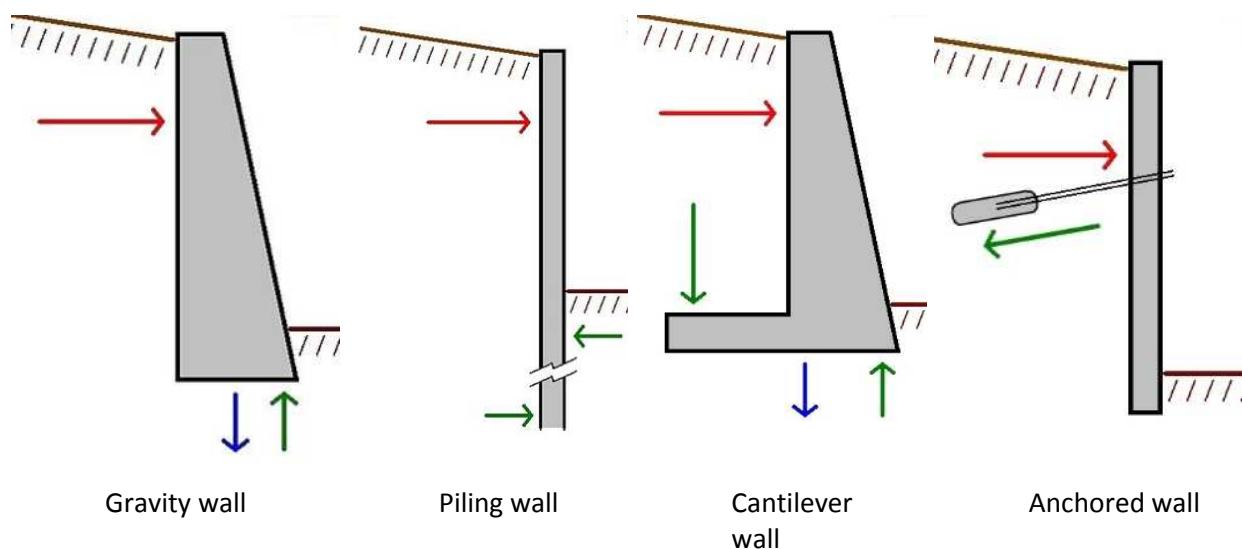


Figure 4.2.33. Various types of retaining walls, Ingolfson (2007)

Problems related to degradation of Earthwork and retaining walls are treated in ML-D1.4 (2014). In SR-D3.2 (2014) tests with geotextiles are presented to improve the subgrade of a railway line in Slovenia.

Retaining walls made of cages of stones, so called gabion retaining walls, are techniques that are common for highways and also used for railways, see Figure 4.2.34. A *gabion* (from Italian *gabbione* meaning "big cage"; from Italian *gabbia* and Latin *cavea* meaning "cage") is a cage, cylinder, or box filled with rocks, concrete, or sometimes sand and soil for use in civil engineering, see e.g. Freeman & Fischenich (2000).

An example of retaining wall made of prefabricated concrete panels to prevent debris from landslides is shown in Figure 4.2.35, Laboe (2015).



Figure 4.2.34. Gabion retaining wall at the new Airdrie to Bathgate passenger rail link in the central belt of Scotland is a landmark project by Network Rail to widen and re-open a 24km line closed over 50 years ago. Gabion (2015).



FIGURE 4.2.35. EXAMPLE OF RETAINING WALL TO PREVENT DEBRIS FROM LANDSLIDES, LABOE (2015).

4.3 TRACK AND SWITCHES & CROSSINGS

The text below mainly mentions track issues. The reasons is that to a large extent, the approaches, consequences etc are similar for switches & crossings. Particular issues related to crossings are discussed in a separate subsection in section Section 4.3.4.

4.3.1 INTRODUCTION AND BACKGROUND

The proposed methodology is based on a three-stage approach as outlined in Figure 4.3.1. The three stages are:

1. Low-resolution analysis:

In this stage the effect of the intended upgrading on degradation, safety, costs etc. is assessed in an overview manner. This stage is concluded by an evaluation of whether refined analyses can be motivated based on the uncertainties/risks regarding future degradation and/or safety levels. Such a low-resolution analysis is commonly a first step in the planning for a major upgrade as a mean to assure feasibility and make preliminary cost estimations.

2. Medium resolution analysis:

In this stage cost estimations are refined and cost drivers are identified in a very overview manner. Note that this stage basically does not provide any improvements aimed at decreasing degradation and/or increasing safety¹³, but solely improves the resolution of the cost analysis. The stage is concluded by an evaluation of whether potential cost savings of further investigations exceeds the costs of these studies.

3. High-resolution analyses:

This final stage targets previously identified potential cost drivers and safety issues. The analysis investigates in detail degradation and safety impact of intended upgrades. Further, alternative solutions and/or mitigating actions can be evaluated and the effect of these be quantified.

The outcome of this three-stage chain of analyses is a sufficiently detailed cost estimation with uncertainties identified and quantified (with sufficient accuracy). Further, if also the high-resolution analysis is employed the impact of alternative solutions and/or mitigating actions have been investigated so that an optimum approach can be employed.

Note that a typical analysis does not imply all three steps. As an example, a small upgrade, i.e. a slight increase in operating frequency on a line with moderate capacity utilization would typically only require the first step to provide a rough idea on increasing deterioration rates as a basis for future maintenance budgets. A second case would be e.g. the introduction of a new vehicle type. Here the first step could be left out and the second step could be employed to provide an idea on how the new vehicles would deteriorate the track in comparison to existing vehicles. A third case would be when a detailed analysis is employed directly to analyse known problematic sections. An example could be a switch with a high frequency of faults. Here the detailed analysis is intended to provide a good base for an improved design.

However, there are also cases when a full three-step analysis is performed. An example would be a larger upgrading of a line where a first low-resolution analysis is made as a basis

¹³ Naturally, such investigations can be performed in parallel.

for a long-term budget. Then a medium-resolution analysis is performed to refine the budget estimates and identify potential problems. Finally, a high-resolution analysis is carried out on identified critical sections/components.

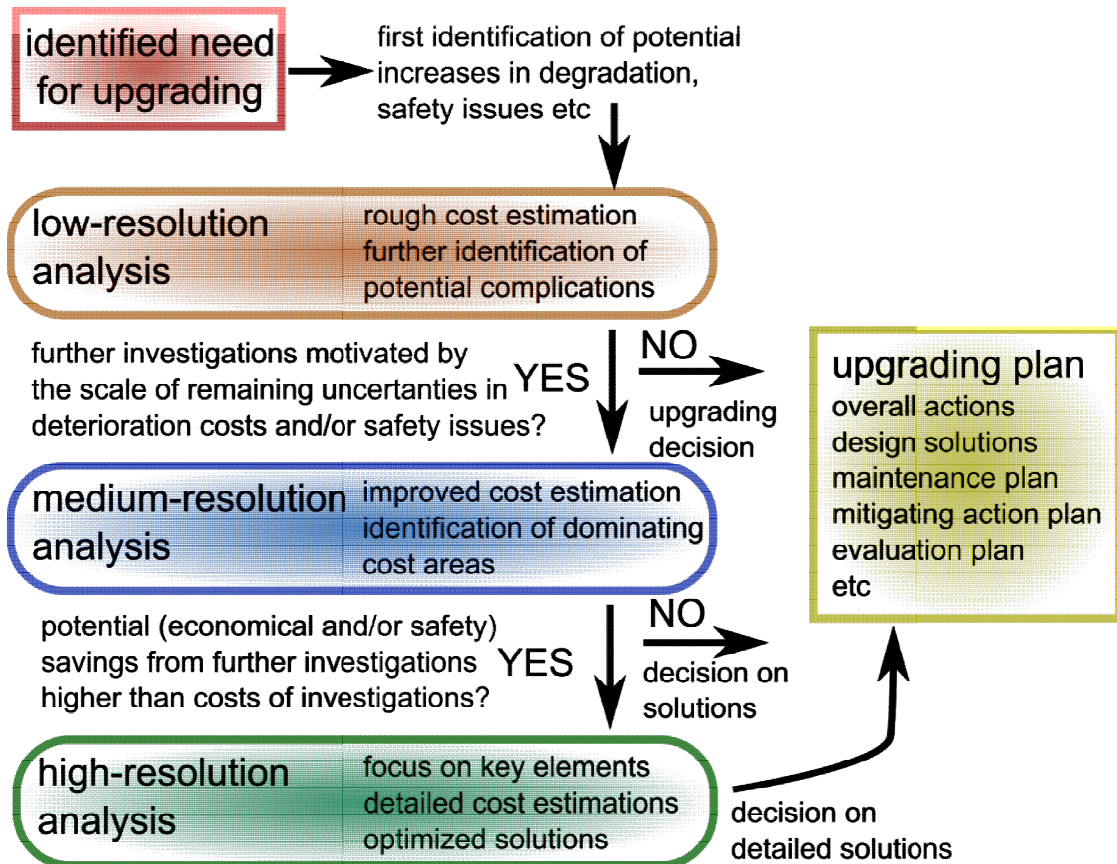


Figure 4.3.1. Flow-chart for investigations and decisions related to track upgrading

4.3.1.1 Effect of different types of operational upgrades

Depending on the nature of the upgrade, different consequences may be expected. When evaluating such consequences of upgrading, three important facts should be remembered:

- A new piece of infrastructure is always “better” (in terms of fewer failures) during the first years of operations. This is a consequence of the lack of initial defects. Thus, to get a fair evaluation of the effectiveness of the upgraded infrastructure, such an evaluation should be carried out after the “run-in” period.
- Further, it takes time for altered operational conditions to give an effect on quality indicators. In particular, it often takes a long time before changes are sufficiently statistically significant that they can be distinguished from random scatter and/or seasonal changes. Consequently, evaluations must be carried out over a fairly long period of time.

- The railway is a tightly integrated system. For example, an increased axle load will affect track settlements. If these settlements cause higher dynamic loads, settlements will be further magnified. The result may be damage e.g. to abutments and other stiff sections. Consequently, not only the part of the track directly affected by upgrading, but also components indirectly affected need to be assessed and monitored.

In order to prevent detrimental consequences to an as large extent as possible, and also to be able to plan mitigating actions towards remaining issues, a thorough understanding is required. To this end, common upgrading scenarios are listed below together with likely consequences regarding the mechanical loading and subsequent deterioration.

Upgrading through new vehicles often results in better running performance due to the improved characteristics of the new vehicles. This generally reduces deterioration (or can allow for higher loading while keeping degradation levels constant). Note however that the new vehicles require (potentially new) maintenance procedures to keep them at a high quality.

An increased axle load will increase the (average) contact load. This will result in increased propensity for wear, rolling contact fatigue, track settlements etc. In addition, a higher axle load will commonly require higher levels of traction and braking that will add to the problem.

An increase in speed will lead to a larger scatter in contact load magnitudes, which may cause more damage on wheels and rails (and in some cases also a different nature of damage). Operations at higher speeds will also be more sensitive to (track geometry, rail/wheel profile and material) defects – in other words, a given defect will have a larger impact. This usually requires stricter maintenance requirements. Note also that operations at higher speeds commonly result also in higher magnitudes of traction and braking. These will result in a higher propensity mainly for crack formation and wear.

An increased capacity outtake with existing train fleet will result in an increased loading per time unit even though the loading of the individual vehicles does not increase. This will result in a faster degradation. At the same time, there is less time for maintenance and repair. Further, faults will become more costly since they will have a higher influence on operations. The same effects are also partly valid in cases of upgrading with **longer trains**. In this case there may be additional complications by longitudinal vibrations of trains, need for longer meeting tracks etc.

Upgraded maintenance procedures will, if properly employed, decrease operational loads etc. This will lead to a longer operational life of the infrastructure. Note that increased operational frequencies often require a shift towards planned (preventive) maintenance with less time to carry out the maintenance. In such a scenario, upgraded maintenance practices may instead be used to keep maintenance at levels that preserve the current status of the track while decreasing time in track.

Upgrading through improvement of the track structure will in general decrease loads (in a broad sense e.g. due to decreased "dynamic effects") and also increases the operational life of the track. The latter is both a consequence of the decreased operational loading and of the longer residual life of the new track structure.

4.3.1.2 A brief overview of damage phenomena and influential parameters

A good illustration of how the operational load level influences (mechanical) track degradation is given by the Wöhler curve, depicted in figure 4.3.2. Although the slope of the curve varies, the general trend that the logarithm of the operational life is proportional to the (logarithm of the) applied loading is valid for a broad range of deterioration phenomena. As an illustration, the mechanical fatigue life of a component may decrease by 70% if the load magnitude is increased 10%.

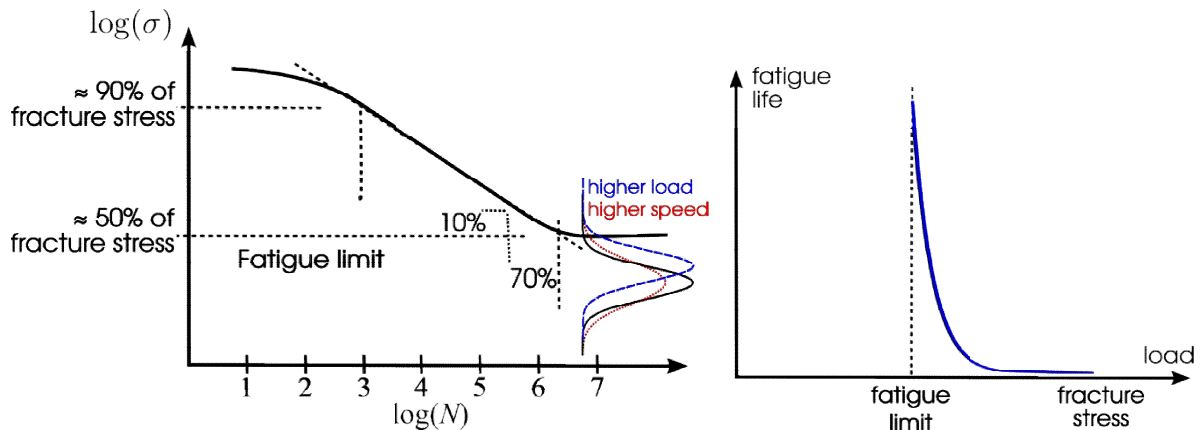


Figure 4.3.2. The Wöhler curve in logarithmic scale (left) with consequences of higher load / higher speed on the load spectrum indicated. The right picture shows the relation between applied load and resulting fatigue life in a linear scale. Note the drastic decrease in fatigue life when the operational loading exceeds the fatigue limit.

Track degradation involves several damage and deterioration phenomena. Some of these and the pertinent main influencing parameters are listed in table 4.3.1. Note that new damage phenomena may occur as a consequence of upgrading. This can be related to *increased loading* due to the upgrading. Typical examples are increased rolling contact fatigue due to higher axle loads, and squat formation (tentatively) due to higher traction vehicles [Grassie 2012]. Altered damage patterns could also be a consequence of the altered – *although not more severe per se* – operational conditions. Examples are the altered forms of corrugation that may form depending on the operational conditions [Grassie and Kalousek 1993, Grassie 2009]. Increased corrugation formation due to higher speeds may then in turn give rise to higher dynamic loads that promotes subsurface RCF [Nielsen et al 2005, Ekberg 2007]. Finally, it may be noted that *improved* operational conditions may shift the mode of damage so that a slower deterioration phenomenon will have time to occur before the track is replaced. One example is that decreased wear rates may give time for surface cracks to form on wheels and rails. Note however that a return to higher wear rates is often a sub-optimum solution. The reason is that the increased wear rate is not controlled (as it is in the case of "artificial wear", e.g. by grinding) and may cause a shorter overall operational life of the track.

Table 4.3.1. Some track deterioration phenomena

Deterioration phenomenon	Main influencing load parameters	Important material/component parameters	Time to deterioration
Track settlements	Track forces (vertical and lateral)	Ballast (and sub-ballast) resistance	Medium
Sleeper damage	Vertical (impact) forces, support conditions	Reinforcement, sleeper design, sleeper support	Short if overloaded
Plastic flow	(Effective) stress magnitude	Yield limit, wheel/rail surface profiles	Short
Wear	Contact load (pressure and frictional), sliding distance	Hardness / yield limit	Medium
Plain fatigue	Stress amplitude	Fatigue limit, surface roughness, corrosion	Medium – long
Surface initiated rolling contact fatigue	Frictional contact stresses	Hardness, ductility	Medium – long
Subsurface initiated rolling contact fatigue	Contact pressure	Material defects, fatigue limit	Long

Below a brief overview of potential consequences of upgrading is provided. A more thorough overview of potential consequences of different types of upgrading including predictive and mitigating means is provided in Section 4.3.4.

4.3.1.3 Influence of higher loading on deterioration

“Higher loading” is here used in a very broad sense that includes operational upgrades such as increased axle loads, increased speed, but also increased operational frequency.

4.3.1.4 Influence on track support

Ballast subjected to increased loading will experience more deterioration of the ballast stones. This will influence the ballast stiffness and the resistance towards track shift and settlements. The result will be a more severely deteriorated track geometry. Note that wear particles may also decrease the frost resistance of the ballast, see [Dahlberg 1997].

In EUROBALT II the main influencing parameters on the ballasted track deterioration was investigated. Since ballast degradation is a slow process, a follow-up was performed by long term measurements at Swedish test sites. In total 13 test sites situated along four different Swedish railway lines during 2000–2005. Only draft reports have been written. Therefore, since this study is unique some work have to be carried out and put into D1.1.5, see also [Olsson E-L, 2007]

The main influencing parameter for sleepers is often not the average load, but extreme impact load magnitudes that can be expected under different operational conditions. To complicate matters, the resistance of a sleeper against impact loads will depend greatly on the support conditions. Further, a sleeper designed to sustain the most extreme impact loads foreseeable will be highly non-optimal since it will increase loading of other parts of the system. Consequently, design of sleepers should be based on a probabilistic approach where the risk of high impact loads and subsequent sleeper failures is analysed. Further, a well-tamped and compacted track will significantly decrease the risk of a sleeper fracture, see e.g. [Bolmsvik, R and Nielsen, J.C.N. 2006].

Fastenings are subjected to fatigue due to rail and sleeper vibrations. Detachment of fastenings due to fatigue failure or other causes will generally worsen the situation since vibrations will increase. This is a typical example of the negative "deterioration spiral" that is very common: Deterioration increases loading that increases the rate of deterioration that increase the loading further, etc.

4.3.1.5 Influence on rails

The "fastest" damage phenomenon for a rail is plastic deformation that occurs if contact stress magnitudes are too high. In extreme cases, plastic deformation decreases with the number of load cycles due to the formation of residual stresses and also "run-in" where the wheel and rail contact profiles adopt to each other (to some extent) through plastic deformations.

When plastic deformation decreases, wear may take over as the dominating form of deterioration [Johansson et al 2011]. Wear can be a fairly fast process that may be self-stabilizing in the sense that stress magnitudes are decreased as the rail is worn-in towards (an envelope of passing) wheel profiles. However, wear may also increase the propensity for additional damage (e.g. rolling contact fatigue) due to degrading surface conditions. One example of this is the adverse effect of hollow wear [Fröhling et al 2008, Karttunen et al 2014]. Another example is the influence of periodic wear that may result in the formation of (different forms of) corrugation and wheel out-of-roundness. Periodic wear will increase dynamic loads that may cause additional deterioration.

If operational loads are high, and wear rates are kept at reasonable levels, surface initiated rolling contact fatigue will often be the next problem to occur [Tournay and Mulder 1996]. Although there are many different types (and also classifications) of surface initiated rolling contact fatigue, the most common tend to be headchecks that occur at the gauge corner in medium to shallow curves [UIC 2002], and squats/studs [Grassie et al 2012] that occur more stochastic on the rail head.

Subsurface initiated rolling contact fatigue [Ekberg and Kabo 2005] may occur because of high dynamic contact pressures and material defects both in wheels and in rails; in the latter case often in welds. This is a relatively slow growing type of fatigue (at least if material defects and tensile residual stresses are moderate) that, especially in the wheel case, may cause severe damage.

Plain fatigue may occur in the foot and web of a rail. In particular, this may be the case in connection to bolt holes e.g. for insulated joints. The result may be a complete rail break. In the case of a foot crack the critical flaw size that may initiate a rail break is small (in the order of centimetres) [Ekberg et al]. In the case of a crack setting out from a hole, the crack may

break off a considerable length of the railhead. It may also detach fishplates, which may have adverse effects, as manifested e.g. by the accident at Brétigny-sur-Orge outside Paris in 2013. It should here be noted that holes at insulated joints are especially affected since the joint typically introduces an impact load and also provides a free surface that aids plastic deformation and formation and growth of fatigue cracks [Sandström and Ekberg 2009].

Finally, it should be noted that all phenomena mentioned above might occur also in switches & crossings. Here the situation is aggravated by the geometry of a switch that introduces geometrical irregularities etc. How large these effects are, can be appreciated by a comparison between forces induced during switch negotiation, and forces induced during the negotiation of a sharp curve with the same radius as the switch, see [Braghin et al 2013].

4.3.1.6 Influence on vehicles

Track upgrading will influence also the vehicles. Generally, this may be beneficial: A new track has less track faults that cause high loads; new vehicles perform better etc. This may initially be manifested as a decreased deterioration. However, if the overall load magnitudes (in a general sense) are increased, deterioration will eventually increase. Depending on the type of upgrading this may be manifested in different ways.

Increased speed and increased axleloads will increase vertical loading (in the case of higher speed due to the increased influence of track irregularities), and often need to increased brake and traction demands. The result will generally be higher risks of rolling contact fatigue and track settlements, which need to be counter-acted by more strict maintenance demands.

Another scenario is an increased capacity outtake that may, depending on the operational scenario, increase the operational outtake of the vehicle, which increases the deterioration per time unit and leaves less time for maintenance.

Note that this brief summary focuses on degradation related to wheel/rail interaction. The rate of degradation is often very dependent on the climate (e.g. winter condition), which complicates a post-evaluation of the predictions. In addition, an upgrading will also demand vehicles to fulfil general demands such as stability, structural integrity, engines, aerodynamics.

4.3.1.7 Influence on noise, vibration and other environmental factors

There are cases when noise emissions may be critical for an upgrading – in other words; if noise emissions cannot be managed, the upgrading will not be allowed. To deal with this is complicated by the fact that the influence of upgraded operations on noise emissions is very dependent on the operational conditions.

Noise emissions may be estimated by numerical simulations, and measured and controlled. There are however complications in that some forms of emissions (e.g. squeal) are notoriously difficult to predict. Further to obtain comparable demands on measurements, the measurement sites need to be harmonized, see e.g. [UIC 2014]. In addition, vibrations may add to the problem.

For mitigating actions against noise pollution and vibrations, see e.g. the outcome of the EU-funded projects SilentTrack, SilentFreight, QCITY and RIVAS. It may be noted that simple

solutions to noise reductions have already often been implemented on sensitive sites. This calls for tailored solutions to target critical components (track sections and/or vehicles) if conditions are to be further upgraded.

Additional environmental factors that may require investigations include particle emissions (e.g. due to mechanical braking or tunnel operations), and transportation of hazardous goods.

4.3.1.8 Influence on safety

To maintain safety levels in an upgraded condition is usually a strict requirement. Often this is not a major problem: If upgrading is carried out in a structured manner with suitable analyses and implementation of counter-measures, safety may actually increase. However, safety assessments are often required to be assessed using strict (and often national) protocols.

General statements about these protocols are difficult to make. However, the D-rail project [D-RAIL 2015] provides an overall assessment of freight operations and tools for risk, LCC and RAMS assessment of safety promoting actions. It could here be noted that even though safety levels typically increase as tracks are upgraded, it may be cumbersome to obtain a formal validation/approval of this. To appreciate why, recall that safety levels are very high in most European countries. Consequently, it is very difficult (not to say impossible in most cases) to obtain a statistical proof of unaltered or improved safety levels. On the other hand, the risk is never zero, and accidents do occur. So, if a personal responsibility is attached to the statement that safety levels are unaltered or improved, the lack of statistical "hard evidence" and the potential risk of personal repercussions leads to a very low incitement to make such a statement.

It could finally be noted that rail operations are about 50–100 times safer than road [SIKA 2009]. Thus, solutions that drive operations to road (i.e. are too costly and/or too limiting) are suboptimal. On the other hand, severe accidents have severe secondary effects (in addition to potentially tragic primary effects) in terms of both traffic disruptions and long-term reduced willingness to use train services.

4.3.1.9 4.3.1.4 Mitigating actions

There is a multitude of actions to target issues related to upgrading. This is further discussed below. Note that if the upgrading is handled properly, the deterioration may actually decrease in the upgraded state. As an example, we can look at upgrading by increased axle loads. Here the frictional stresses will generally increase, which will increase the risk of surface initiated rolling contact fatigue. However, better steering vehicles and better control of braking and traction may reduce this effect. In addition, a better match of wheel and rail profiles will decrease the effect even further. In this way, the result may actually be a reduced deterioration even if the operational performance has increased.

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4.3.2 LOW-RESOLUTION ANALYSES – ESTIMATING OVERALL DETERIORATION FROM INCREASED LOAD

In this section we consider very rough methods to predict consequences of operational upgrades. These methods are intended to provide a first budget estimate of consequences related to upgraded operational conditions. Depending on the outcome, the analysis can then be refined if it is clear that such refined analyses are needed to provide the required accuracy of cost estimates and/or can provide cost savings that exceeds the additional costs of the refined analysis. Methods for refined analyses are presented in later sections.

4.3.2.1 Estimations of degradation and cost

The simplest estimation of costs is to presume that deterioration rates are proportional to the load (typically measured in MGT). This results in a relationship between load and track quality degradation on the form

$$DQ = k_1 \cdot P \quad (1)$$

Here DQ is the degradation in track quality (in general terms) due to the loading P (e.g. measured in MGT over the time that DQ is evaluated) and k_1 is a coefficient that would depend on the basic conditions of the track, the state of the operating vehicles etc. An increase in loading with ΔP would then give a decrease in track quality that can be expressed as

$$\frac{DQ + \Delta DQ}{DQ} = \frac{k_1 \cdot (P + \Delta P)}{k_1 \cdot P} \Leftrightarrow \frac{\Delta DQ}{DQ} = \frac{\Delta P}{P} \quad (2)$$

In other words, the relative increase in degradation is **under these presumptions** equal to the relative increase in loading. To be useful in investment decision, the increased degradation then needs to be related to an increased life cycle cost. In its simplest form, this can also be presumed as a linear function.

A more refined approach is to employ the fact that damage typically increases exponentially with an increased loading. The overall track quality can then be expressed on the form [Veit and Marschnig 2011]

$$Q(t) = Q_0 e^{bt} \quad (3)$$

where Q_0 is the initial track quality (in [Veit and Marschnig 2011] defined by a so-called MDZ index), b is the deterioration rate and t the elapsed time.

4.3.2.2 Practical implications

For the simplified models described above, the effects of all different influencing parameters need to be combined into one measure that is related to the expected increase/decrease in deterioration due to the upgraded conditions. However, in reality the different parameters will influence the degradation in different manners.

For increased (axle) load, an exponential relationship as above can be adopted. Note however that different track components will respond differently to the increased load. As an example, for rails increased wear and/or rolling contact fatigue may result. Which of these two damage phenomena that will dominate will depend on the operational conditions. It may even be that an increased loading will shift the dominating mode of damage from one form to the other. Especially in the case that the upgrading includes updated running gear this may be the case. The practical consequence is that not only may deterioration *rates* change due to the upgrading; the (dominating) *form* of deterioration may also shift.

Upgrading through increased speed is somewhat more cumbersome since the nominal, static load does not change. The peak loading will however still increase due to dynamic effects (basically the increased speed will lead to a broadening of the load spectrum so that while the average load remains constant, the magnitude of the maximum load will increase and the magnitude of the minimum load will decrease, see figure 4.3.2). This increase may be estimated by more or less sophisticated means ranging from dynamic load factors (see e.g. Esveld 2015) to multibody dynamics simulations (which will be discussed at length in section 4.3.4). It should also be noted here that higher speeds increase the need to limit track geometry irregularities etc. Consequently, increases in speed will typically not only increase deterioration rates, but also increase demands on track quality.

4.3.2.2.1 An example from 25 and 30 tonnes investigations in Sweden

In connection to upgrading of maximum axle load from 25 tonnes to 30 tonnes on the Iron Ore line in northern Sweden, and from 22.5 to 25 tonnes on main freight lines in Sweden investigations were made on the consequences on increased deterioration, see e.g.

[Banverket 1996, Banverket 1996b and Banverket 2001]. In short, the increase in maintenance costs was in [Banverket 2001] estimated to some 1.4–4.8 %. The increase will depend on the line properties (curve radius etc.), freight / passenger traffic ratio, normal / high speed, as well as the current maintenance costs. The model employed to estimate the costs includes track –mainly rail – deterioration described by two mechanisms:

Fatigue (both at the surface, and due to internal defects e.g. due to welding). Static, quasi-static and dynamic loads are considered.

Wear with influence of rail lubrication, rail curve radius as well as car type considered

The relative track deterioration is the estimated by a formula of the type:

$$E = k.T^{\alpha}.P^{\beta} \quad (4)$$

where k is a constant specific for each section of a railway line, T is total accumulated tonnage (since track construction), and P is the axleload. Exponents α and β have different values depending on the type of deterioration considered. The employed model can also consider that operational data vary in time.

4.3.2.3 Conclusions

The low-resolution models presented in this section are simple and easy to use. They are however very limited and should mainly be used to obtain a rough estimation of expected changes in deterioration due to upgraded operational conditions. The two main drawbacks of the low-resolution models are:

The models rely on a number of empirical parameters. These need to be estimated and preferably calibrated. Usually this is made through historic experience. However, the obvious question is how reliable such an estimation is if the aim is to investigate changes in degradation due to “new” operational conditions (new vehicles, track components etc). Another option is to rely on international experience, but also there an obvious question is how much that can be translated to the current local condition. The calibration is further complicated by the fact that the increase in deterioration is not in general a linear function of the load (where “load” is used in a broad sense). Provided there is sufficient data to calibrate parameters, the precision can to some extent be improved by calibrating and evaluating degradation in shorter sections of the track.

The models are limited in considering the degradation of the entire track system. This can be overcome by dividing the track into subsystems (e.g. rail, sleepers, switches & crossings etc.). This was in fact (to some extent) the approach adopted in the above example on upgrading to 25 and 30 tonnes axle load in Sweden. Such models will be discussed more in depth in section 4.3.3.

The main benefit of the low-resolution models is the low requirement for input data. This means that the models can be used as a first, rough estimation of changes in deterioration rates and costs. Based on this evaluation it can then be decided if more refined analyses are worthwhile; in other words:

Is the cost of more refined cost analyses balanced by the increased value of less uncertainties regarding future costs and deterioration levels an improved basis for finding mitigating solutions.

The latter item relates to the fact that an increased deterioration due to upgrading can (almost) always be (partly) mitigated through improving the operational characteristics. However, exactly which the required improvements are depends on the type of upgrading and the type of deterioration considered. As a simple example, increasing the speed will increase wear and rolling contact fatigue of rails. These adverse effects can be mitigated by better running gear, improved wheel and rail profiles etc. However, the simple models described in this section have a very limited (if any) capability of identifying which mitigating actions that would be most efficient. Often such an identification is based on experience. However, to obtain a better quantification more refined predictive models need to be used. These will be discussed in sections 4.3.3 and 4.3.4.

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4.3.3 MEDIUM RESOLUTION ANALYSES – ESTIMATION OF DETERIORATION ON A SUB-SYSTEM LEVEL

Medium resolution analysis is an extension to low-resolution analysis. Here simplified track deterioration models that employ e.g. individual Wöhler curves for the different track components are considered. Different deterioration mechanisms are assessed with more relevant and accurate vehicle and track data.

Over 20 track deterioration models are found in the literature [Ö2009]. Two examples of these are briefly described here: the model from [Ö2009], which was developed in 2006–2008 for the needs of Banverket (Swedish Rail Administration, now Trafikverket) for a new track access charging system, and a second model from [VUC] on Variable Usage Charge (VUC) rates for Network Rail in Great Britain.

4.3.3.1 Example 1 – Track deterioration model for Swedish Railways

The majority of the track deterioration models are based on only one or a few track deterioration mechanisms. The track deterioration model proposed in [Ö2009] considers four mechanisms:

track settlements (due to non-uniform movements in the ballast),
component fatigue (including switches & crossings, internal fatigue of rails, fastenings, ballast, rail pads, etc.),
abrasive wear
(surface initiated) rolling contact fatigue of rails.

The model is developed as a basis for a track access charging system for former Banverket. It is able to include vehicle characteristics and relate these to track deterioration. This track deterioration model is implemented in a software package called DeCAyS (**D**eterioration **C**ost **A**ssociated with the **R**ailway **S**uperstructure), which is designed in an Excel environment.

4.3.3.1.1 Model framework

Axleload, un-sprung mass and wheelset steering capability are decisive for track deterioration and considered in the model. The model uses the dynamic vertical wheel load instead of the axle load. This makes it possible to account for the quasi-static force contributions due to curving. The influence of speed and un-sprung mass on the high-frequency part (up to some 100 Hz) of the dynamic vertical wheel load is also considered.

In this model, settlement (a) and component fatigue (b) are taken as proportional to the cube of the wheel load. The deterioration therefore increases progressively with increased axle load.

The wear analysis (c) in the model is captured using the wear number concept, which is often referred to as T-gamma (i.e. product of total creep force and total creepage) and is an appropriate measure of the frictional energy dissipation in the wheel-rail contact. Note that energy dissipation due to rotation (spin) is not included in T-gamma.

The RCF (d) and wear (c) are competitive mechanisms: For small curve radii where wear is the dominating mechanism, small initiated RCF cracks are worn off. Thus, the total deterioration from wear and RCF in [Ö2009] is divided in two regimes: one where the RCF is dominating and another where wear is dominating. Following [Burstow] when the wear number ($T\gamma$) is between 0 and 15 N, it is considered that no RCF or wear will occur. RCF will occur for $15\text{ N} < T\gamma < 65\text{ N}$ and should be ground off by maintenance actions. RCF will be partly removed by wear in the range of $65\text{ N} < T\gamma < 175\text{ N}$. If the wear number exceeds 175 N wear will be dominating.

The deterioration should be evaluated for different curve zones since track forces and frictional energy dissipation depend on the curve radii. Thus, the total track deterioration depends not only on vehicle characteristics, but also on curve radii and tonnage.

The analyses of the total accumulated annual deterioration cost for the whole railway network with Z vehicles accounting for j curve zones, is based on the formula proposed in [Ö2009]:

$$\Delta E_a \approx \sum_Z \left[k_1 \cdot \sum_j T_Z \cdot \frac{L_{R_j}}{L_t} \cdot Q_{tot_j}^3 + k_2 \cdot \sum_j T_Z \cdot \frac{L_{R_j}}{L_t} \left[\sqrt{Q_{tot}^2 + Y_{qst}^2} \right]_j^3 + k_{34} \cdot \sum_j T_Z \cdot \frac{L_{R_j}}{L_t} \cdot \frac{\sum_{i=1}^{n_z} [f(\bar{F}_v \bar{v})_i]}{m_z} \right]$$

The formula consists of 3 terms, each including the marginal average cost coefficients k_1 (for settlement), k_2 (for component fatigue), and k_{34} (wear and RCF of rails); T_Z that is tonnage or traffic volume for vehicle type Z during the year in gross tonne-km; L_{R_j} that is length of curve zone with mean radius R_j [m], and L_t which is total length of the track section or the network [m]. Further, Q_{tot} is total vertical wheel load (usually low pass filtered at 90 Hz [kN]); Y_{qst} is quasi-static lateral wheel load or “guiding force” [kN]; $f(\bar{F}_v \bar{v})$ is a function relating wear and RCF to the friction energy dissipation for the outer wheels, see Figure 4.3.3.; \bar{F}_v is total creep (friction) force [N] and \bar{v} is total creepage [-]; m_z is mass of vehicle type in tonnes (the total weight of each freight wagon and an additional 50 per cent to account for passengers is employed for passenger wagons); subscript i relates to axle i .

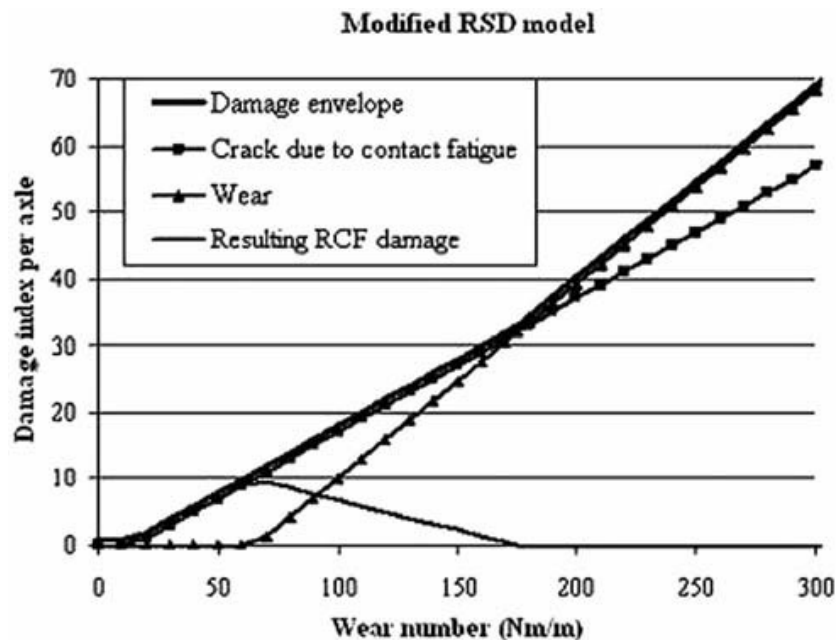


Figure 4.3.3. The damage function to quantify wear and RCF of rails as adopted in [Ö2009]

Some important conclusions are drawn from simulation results based on railway traffic in Sweden [Ö2009]: Wheelset steering capability is crucial for track deterioration and influences wear and RCF. If a stiff bogie is replaced by a flexible bogie, the predicted saving in cost could be in the order of 2–10 times per gross tonne-km. The un-sprung mass and the height of the centre of gravity have some influence on track deterioration. The track deterioration predicted by the model is strongly dependent on vehicle type. Heavy freight wagons with link suspensions are predicted to cause mainly settlement and component fatigue. Y25 bogies are predicted to produce, among other things, wear and RCF deterioration. The latter is also verified in more detailed analysis in [KK] where track irregularities are accounted for. Freight wagons with three-piece bogies of standard type are predicted to produce the highest deterioration of all freight wagons.

4.3.3.1.2 Calibration of the model

The model was applied to Swedish mainline traffic, but is according to [Ö2009] believed to predict realistic results also for heavy-haul rail operations. The deterioration model was calibrated against total annual marginal cost for the Banverket's track maintenance together with known and estimated traffic data per vehicle type for the Swedish rail network in 2001. The values of an average marginal cost (excluding reinvestments) as 0.0003 Euro/gross ton-km, and the total annual traffic volume of 64 000 Mega gross ton-km are used in the analyses. Track settlement is assumed to be responsible for 25 % of total marginal cost, component fatigue for 35 % and wear and RCF take the remaining 40 %.

4.3.3.2 Example 2 – Variable usage charge model for Network Rail

The variable track usage charge model from [VUC, VUC_ppt] determines track access charges to be paid by train operators to Network Rail for use of its infrastructure. The price lists in [VUC_ppt] are in 2012/2013 prices. The purpose of the charge is to recover Network Rail's operating, maintenance and renewal costs that vary with traffic, so called "wear and tear" costs. The charge is designed to be cost reflective, e.g. "track friendly" vehicles, which cause less "wear and tear" to the network, are subject to lower variable usage charge (VUC) rates than vehicles, which are more damaging.

4.3.3.2.1 Methodology

The VUC recovers following costs depending on traffic [VUC_ppt]:

- track costs (85%), including maintenance and renewal costs, 70% of costs relate to vertical rail forces and 30% of costs relate to horizontal rail forces;
- signalling (5%) including maintenance costs and points renewal costs;
- civil structures (10%) including costs associated with underbridges, embankments and culverts.

The Network-wide VUC costs defined as the product of "efficient average VUC rate" and the "Network-wide traffic" give an assessment of a vehicle's "track friendliness". There are four key vehicle characteristics that determine the "track friendliness score" [VUC_ppt]:

- operating speed,
- axle load,
- un-sprung mass,
- bogie primary yaw stiffness (indicative of curving capability).

The higher these values are, the worse the vehicle's "track friendliness score" and the higher the vehicle's VUC rate. Variable usage costs are translated into VUC rates using a cost allocation model. Track damage costs are estimated by surface damage components:

- horizontal track damage,
- vertical track damage,
- RCF / Wear (referred to as T-gamma).

T-gamma (see also above) depends on vehicle suspension type, curve radius and cant deficiency and can be evaluated by detailed vehicle dynamics simulations.

4.3.3.2.2 Required input data and precision

In order to calculate VUC rates for different vehicle types a special VUC calculator was developed [VUC]. The calculator (in the form of a spreadsheet) is a tool that calculates a VUC rate based on relevant information on vehicle characteristics.

- **Vehicle type** (passenger or freight).
- **Motor / trailer.** This does not impact the level of VUC rate but clarifies whether the proposed rate will apply to the motor or trailer cars.
- **Locomotive, coach or multiple units.** All other things being equal, coaches and multiple units pay lower VUC rates than locomotives. This reflects the lower traction forces that each wheel imposes on the track.
- **Number of axels** used (together with vehicle weight) to calculate the average axle load of the vehicle.
- **Vehicle weight in tonnes.** Tare weight of the vehicle in serviceable condition (i.e. weight of the vehicle with no passengers).
- **Total number of (passenger) seats.** For multiple units and coaches the VUC is calculated for the weight of the vehicle with 50% of seats presumed full, and assuming 75 kg per passenger.
- **Un-sprung mass** (kg per axel). This comprises the masses of the wheelset, axel boxes, brake gear, axle-mounted gearbox or final drive, and any other equipment that is mounted on the wheelset or axleboxes. An average value should be taken if the un-sprung mass is not the same for every axle.
- **Maximum speed** (in miles per hour).
- **User calculated operating speed.** The operating speed can be estimated from timetable excluding stopping time at stations. If the “user calculated” speed is omitted, the operating speed will be estimated by the standard formula: $\text{Operating speed} = 0.021 * (\text{max speed})^{1.71}$
- **Curving class.** The specified “curving class” is a way of categorising vehicles according to the rail surface damage in terms of wear and rolling contact fatigue that they generate.
- **User defined T-gamma table.** T-gamma values could be determined from vehicle dynamics simulations for the vehicle over a specified range of curves. In this case it is not necessary to select one of the generic curving classes.

The output from the VUC calculator sets out the vehicle characteristics and the calculated VUC rate, broken down by asset type (track, structures, signals (fixed), signals (variable), and surface damage). The VUC rates increase annually with inflation.

- To calculate a new freight VUC rate it is necessary to define additional information on vehicle characteristics such as:
- **Suspension band.** Wagons with “track friendly” suspensions/bogies will pay a lower VUC than wagons with suspensions/bogies that impose higher forces on the track.
- **Ride Force Count (RFC) value.** The RFC is estimated to provide a quantitative assessment of the “track friendliness” of a wagon’s suspension/bogie type, following vehicle dynamics simulation.
- **Freight commodity.** There is an option to select all commodities or several commodities from the list (for example, Iron Ore, Domestic Waste, Petroleum, Industrial Minerals, Chemicals, European Conventional etc.). The VUC rate will be calculated for each commodity.

4.3.3.2.3 The use of “track friendliness” formulae and limitations

VUC rates are designed to be cost reflective. For each vehicle type a “track friendliness score” can be calculated and used to apportion scores. To assess “track friendliness” one of the following empirical formulae for track friendliness scores [VUC_ppt] may be used depending on the VUC cost category:

$$C_t \cdot (0.473 e^{0.133 A} + 0.015 S \cdot U - 0.009 S - 0.284 U - 0.442) \cdot GTM \cdot axels$$

‘track friendliness’ score – vertical track loads

$$C_t \cdot A^{3.00} \cdot S^{1.52} (\text{per tonne} \cdot \text{mile}) \cdot GT$$

‘track friendliness’ score – civil structures

Where

$$C_t = \begin{cases} 0.89 & \text{for loco – hauled passenger stock and multiple units} \\ 1.0 & \text{for all others} \end{cases}$$

for vertical track

$$C_t = \begin{cases} 1.2 & \text{for 2 – axle freight wagons} \\ 1.0 & \text{for all others} \end{cases}$$

for civil structures

A – axle load (tonnes) within the range of 5 to 25 tonnes,

S – operating speed (miles/hour) within the range of 25 to 100 mph,

U – un-sprung mass (tonnes/axle) within the range of 1 to 3 tonnes,

GTM – gross tonne miles.

Track friendliness scores for horizontal track loads is allocated using the “curving class” methodology, i.e. with user inputs of T-gamma with curvature for required vehicle(s) and converting T-gamma to wear and RCF damage for each radius, then weighting damage by population of curve radii in network and finally converting damage to cost for each curve and summing to get total cost [VUC_ppt]. 50 % of signalling costs are assumed to be load-related allocated using the track (vertical) formula and 50% of costs assumed to be non-load-related allocated based on vehicle miles.

4.3.3.3 References

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M C Burstow. Whole life rail model application and development for RSSB – continued development of an RCF damage parameter. Report no. AEATR-ES-2004- 880 Issue 2, 2004, 74pp. London: Rail Safety and Standards Board (<http://www.rssb.co.uk>) [Burstow]

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Ben Worley and Mark Burstow. The variable usage charge (VUC) in CP5, 29 pp [VUC_ppt]

K Karttunen, E Kabo, and A Ekberg. The influence of track geometry irregularities on rolling contact fatigue. Wear, 2014, vol 314, pp 78-86 [KK].

4.3.4 DETAILED ANALYSES – ESTIMATION OF DETERIORATION ON A COMPONENT LEVEL

In this section we outline possibilities for detailed analyses of consequences of operational upgrades on track and switches & crossings. This includes a thorough outline of consequences of different operational types of upgrading, methods and modelling to estimate the effects in detail, and potential strategies to minimize the consequences.

Note that these investigations are basically the last steps in the upgrade investigation chain. This does of course not imply that all previous analysis levels have to be investigated – in many cases it is very clear already at the onset that detailed investigations will be required. However if the previous low and/or medium resolution analyses as outlined in sections 4.3.2 and 4.3.3 have been carried out, these should have resulted in a fairly good overview of which topics that need further evaluation, and also the order of magnitude of the consequences of an upgrade.

This section sets out by identifying the consequences of increased axle loads and speeds. These upgrades will increase the actual load imposed on the track and switches & crossings, however in very different manners – a higher payload will increase the average load level, whereas an increased speed will increase the scatter in load magnitudes around an average that is not altered.

The focus is then turned towards the influence of increased load gauges, operational frequencies and track lengths. These do not in themselves increase the imposed load per wheelset on the track, but will nevertheless have consequences on requirements and maintenance needs for the track structure.

It should here be noted that the study focuses on requirements to allow for the upgrading in terms of infrastructure capabilities and (increased needs for) maintenance. Some other aspects are not included in this overview. This includes topics such as

- noise emission and ground vibration
- logistics needs
- general safety issues
- additional environmental issues

These topics are discussed briefly in section 4.3.1.3. These issues are not unimportant – they should be considered in every upgrading plan. However, the analysis of these topics is highly dependent on local conditions. Further, the excluded topics are of general concern and should therefore not be confined to an assessment of track and switches & crossings.

Finally, it should be noted that analyses described in this section require high quality in input data, employed tools (including calibration), and staff carrying out the analyses. We are here dealing with analyses that are on the engineering and research forefront and errors in analysis assumptions, input data, and result evaluations will have dire consequences on the outcome. Such errors run the risk of being very costly.

Finally, it can be noticed that the described analyses also can be used to optimise choices by adding LCC and RAMS evaluation of different operational scenarios to the analysis. The interested reader is referred to Ekberg & Paulsson (2010).

4.3.4.1 Consequences of heavier and/or faster operations

As mentioned above a higher payload will increase the average load level, whereas an increased speed will increase the scatter in load magnitudes around an average that is not altered. This will correspond to differences in needs for the technical performance of the track structure, different characters of the increased deterioration, and different key influencing parameters for the different phenomena.

In particular note that “heavier” in the context of this paper implies increased load per wheel (either due to an increase in the allowable axle load or that trains are loaded higher within existing load limits). The cases where loading is increased through the use of longer, or more frequent trains are dealt with in subsequent sections.

In general, heavier load will – as far as track and switches & crossings are concerned – correspond to an increase in low frequency (including static) loads, whereas a higher speed will correspond mainly to an influence at higher frequencies (up to some kHz in the case of rail corrugation). This also translates e.g. to the influence of vibration and noise: For example, a higher speed shifts the main emission source from sleeper, to rail, to wheel and finally to air streams. The interested reader is referred to e.g. Thompson and Jones (2009).

Below, the influence on different components of the track – and on switches and crossings – will be discussed in detail. The different phenomena will be described, possibilities of predicting upgrading consequences will be outlined, and means of limiting the negative consequences will be discussed.

4.3.4.1.1 Rails

Phenomena

For **heavier** operations, a detailed, knowledgeable and very up-to-date practical treatise is given in Leeper and Allen (2015).

For faster operations, a good first overview can be obtained from UICs information on the topic, see <http://uic.org/spip.php?rubrique865> .

The phenomena resulting from axle load upgrade vary depending on operational specifics. Mechanical deterioration phenomena related to the rail (and wheel) can be ordered in a “hierarchy” depending on the time to damage formation as follows:

Plastic deformation – Plastic deformation basically occurs always in the wheel/rail interface. However, in most cases the plastic deformation decreases with time. The plastic deformation referred to here implies large-scale plastic flow that results e.g. in lipping (material roll-out on the field corner) of the railhead, heavy deformation of insulated joints etc. Such large-scale plastic deformation is typically a sign of very high dynamic loads and/or poor contact geometry.

Wear including periodic wear – Wear is a result of high interfacial shear combined with a large relative motion between wheel and rail (see figure 4.3.4). Wear will modify wheel and rail profiles, which will influence contact stresses. This may result in more conformal contact leading to more evenly distributed contact stresses (so-called wear-in). This may decrease wheel and rail deterioration. However, wear may also lead to poorer steering of the vehicles, which may increase subsequent deterioration. In addition, wear may form in periodic patterns, which results in corrugation of the railhead and/or out-of-roundness of the wheel.

This phenomenon will increase (vertical) dynamic loads, which is highly detrimental especially in fast operations where it may promote the formation of subsurface initiated rolling contact fatigue. Wear will also remove small defects and cracks on the surface of wheels and rails. To retain the positive effects of wear while avoiding the drawbacks, artificial wear in the form of rail grinding and wheel reprofiling is generally adopted.

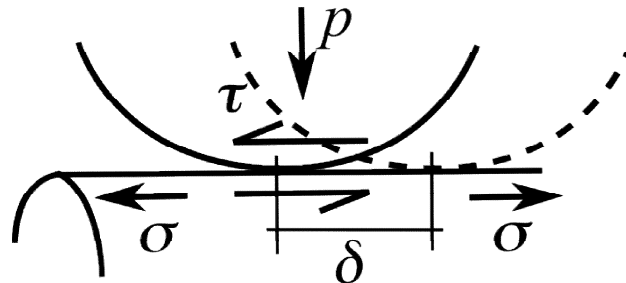


Figure 4.3.4. Definition of contact pressure (p), interfacial shear stress (τ) and relative sliding distance.

- Surface initiated rolling contact fatigue – This form of rolling contact fatigue relates to the formation of surface cracks, typically as a consequence of long-term accumulation of plastic deformations (so-called ratcheting) or low-cycle fatigue damage accumulation. Once cracks are initiated they will grow as a consequence mainly of shear deformation of the crack faces and under the influence of the deformed material texture. Longer cracks may deviate to a transverse growth, which may cause rail breaks. More details on the formation and growth of surface initiated rolling contact fatigue may be obtained from Ekberg and Kabo (2005).
- Subsurface initiated rolling contact fatigue – This form of rolling contact fatigue relates to the formation of fatigue cracks below the running surface of a wheel or a rail. The crack will typically propagate below the surface until final fracture occurs either towards the surface – typically breaking off a large part of the surface material – or transversely into the wheel or rail – causing a catastrophic failure. More information can be found from Ekberg and Kabo (2005).
- Plain fatigue – Typical results of plain fatigue are foot cracks in rails and disc cracks in wheels. A special form related to high axle loads and a severely worn railhead is fatigue crack formation on the underhead radius of the railhead, see e.g. Magel et al (2015). It should be noted that the critical size of a rail foot crack (i.e. the size of the crack that corresponds to fracture) is on the order of a thumbnail, see e.g. Ekberg, Kabo and Nielsen (2015).

In addition to the general description of the problems given above, it should be noted that a railway track consists of rail sections that are especially prone to deterioration. Such constructions include insulated joints (see e.g. Sandström and Ekberg (2009); and Sandström et al (2012)), welds (see e.g. ch 5.6 of Ekberg and Paulsson, (2010)) and sections with reductions in the load carrying capacity of the rail, e.g. due to holes.

In particular, upgraded operations will affect switches & crossings. In principle a switch & crossing construction can be considered to be a sharp curve with interface gaps (an actual

comparison between contact forces in a sharp curve and a switch & crossing can be found in Braghin et al (2013)). This will require consideration of all problems related to a sharp curve (plastic deformation, wear, rolling contact fatigue), but also additional considerations of the influence of the interface gaps. These include plastic deformation, wear and RCF of crossing components.

Means of prediction

A standard way of analysing the influence of upgrades is to couple an analysis of (upgraded) train/track interaction to a subsequent damage evaluation. To be able to predict likely changes in the deterioration pattern, an analysis should also be carried out for the existing operational conditions. For the train/track interaction analysis, commercial codes (Gensys, Simpack, Vampire, Nucars etc) can usually be employed.

The different deterioration phenomena typically require different approaches to prediction. A brief overview of options is presented below:

In the case of **plastic deformation**, there is generally a need to investigate the amount of plastic deformation and whether the plastic deformation is confined to some initial load cycles (i.e. if shakedown will occur). This typically requires finite element simulations. Further, these simulations need to feature an elastoplastic material model with non-linear kinematic hardening that has to be calibrated towards the cyclic response of the rail material. Note that the simulations typically need to be three-dimensional, which increases the simulation times significantly.

If the analysis of train/track interaction is carried out using commercial codes, these typically include modules for evaluation of **wear**. A common method of predicting wear are Archard's wear law, see Archard (1953)

$$V_{\text{wear}} = k \frac{P \cdot \delta}{H} \quad (5)$$

Here V_{wear} is the volume of wear, P is the normal force, δ is the sliding distance (see), H is the hardness of the softer material and k is a wear coefficient that will depend on the contacting materials, sliding speed contact pressure, etc. The Archard wear equation gives a quantitative measure of the wear volume and requires two material parameters: the hardness and the wear coefficient.

Another option is to employ the $T\gamma$ -model, see e.g. Burstow (2004). Here wear is related to a wear number, $T\gamma$, which is the product of interfacial shear force (T) and the creepage in the wheel/rail interface (γ). Wear (and RCF) is then assessed through a damage function. In essence, the damage function states that for low $T\gamma$ magnitudes, no damage will occur. As the magnitude of $T\gamma$ increases, rolling contact fatigue will increase up to a peak where wear will start to have an effect in reducing the apparent amount of RCF (by wearing off initiated RCF cracks). For sufficiently high $T\gamma$ magnitudes wear will dominate completely and there will be no RCF. In addition to the operating conditions, the $T\gamma$ -model requires calibration of the damage function, which requires four material parameters. For a more in-depth description, please refer to Burstow (2004).

As for **surface initiated rolling contact fatigue** (RCF) the previously mentioned $T\gamma$ criterion can be used. It should here be noted that a low damage evaluated by the $T\gamma$ -criterion could correspond either to a low $T\gamma$ magnitude (implying no damage) or a very high $T\gamma$ magnitude

(implying gross wear). Note that the original formulation of the $T\gamma$ -criterion does not account for creep due to spin. An extension where this is incorporated can be found in Dirks and Enblom (2011).

Alternatively, a shakedown based approach can be taken. The original version according to Johnson (see e.g. Johnson (1989), employs a shakedown map see Figure 4.3.5. Here the condition in the wheel/rail interface (at any given time) is characterized by a work point (WP in Figure) which is given by the normalized vertical load (peak contact pressure divided by the yield limit in shear) and the traction coefficient (utilized interfacial shear force divided by the normal contact force). If the work point is outside the boundary line (BC in Figure 4.3.5) plastic deformation will occur on the contacting surface and rolling contact fatigue cracks will (eventually) initiate.

The shakedown approach was extended in Ekberg et al (2002) with the derivation of a “fatigue index” for surface initiated rolling contact fatigue. This index (which is the horizontal distance from WP to BC in Figure 4.3.5) can be expressed as

$$FI_{\text{surf}} = f - \frac{2\pi abk}{3P} \quad (6)$$

where the traction coefficient $f = F_{\text{lat}}/P$ where $F_{\text{lat}} = \sqrt{F_x^2 + F_y^2}/P$ where F_x and F_y are the lateral forces and P is the normal contact force (positive in compression), a and b are semi-axes of the hertzian contact patch, and k the yield limit in cyclic shear.

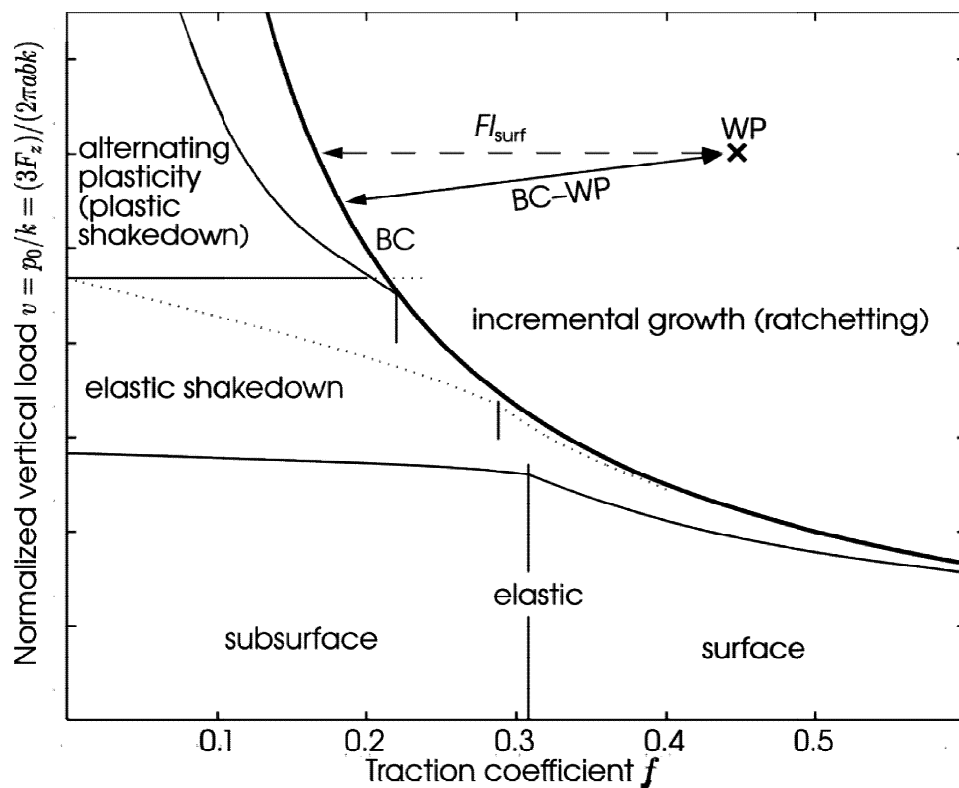


Figure 4.3.5. Shakedown map with indication of the surface fatigue index FI_{surf}

Note that shakedown-based approaches presume hertzian contact conditions¹⁴, and full slip¹⁵. Extensions to account for full slip by adopting the local traction can be found in the literature, see Dirks and Enblom (2011). It should be noted that this is a conservative assumption, as quantified in Ekberg et al 2015.

The only material parameter in the shakedown diagram is the yield limit in cyclic shear. It should be noted that this is not really a material constant, but evolves during loading. In addition, the influence of temperature may be significant, see Ahlström et al (2015).

Note that the methods to predict surface initiated rolling contact fatigue presented here do not really provide an absolute life unless the output is calibrated towards evaluated RCF lives. For the fatigue index, a tentative calibration of a Wöhler type fatigue life equation yields, see Kabo et al, 2010

$$FI_{\text{surf}} = 1.78(N_f)^{0.25} \Leftrightarrow D = \frac{(FI_{\text{surf}})^4}{10} \quad (7)$$

Here N_f is the fatigue life and D the fatigue damage. Note however that for comparative purposes there is less need to evaluate absolute fatigue lives.

In contrast to surface initiated rolling contact fatigue, **subsurface initiated rolling contact fatigue** (RCF) is more of a high-cycle fatigue phenomenon. In other words, it is not so much related to the global plasticity of wheel and rail, as to the damage formation in the presence of material defects located some 2–20 mm below the wheel tread. Employing a multiaxial fatigue criterion according to Dang Van, subsurface initiated rolling contact fatigue can be considered a risk if the following inequality is fulfilled (see Ekberg et al (2002))

$$FI_{\text{sub}} = \frac{F}{4\pi ab} (1 + f^2) + c_{dv} \sigma_{h,\text{res}} > \tau_{e,\text{red}} \quad (8)$$

Here c_{dv} is a material parameter (for “normal steels” a reasonable estimation is $c_{dv} \approx 1/3$). Further, $\sigma_{h,\text{res}}$ is the hydrostatic part of the residual stress. Note however that employing the theoretical benefit of a compressive residual stress (i.e. $\sigma_{h,\text{res}} < 0$ is likely to be non-conservative). Further, $\tau_{e,\text{red}}$ is the reduced fatigue limit in shear, which can be estimated as

$$\tau_{e,\text{red}} \approx \tau_e (d/d_0)^{-1/6} \quad (9)$$

Here d is a “worst case defect”, τ_e the nominal fatigue limit, and d_0 the corresponding defect size (typically in the range 10 μm to 20 μm).

It should be noted that subsurface initiated rolling contact fatigue are related to vertical contact stresses. Usually high magnitudes of vertical contact stresses are induced by high-frequency dynamic forces (see e.g. Nielsen et al 2005). This calls for numerical analyses (see e.g. Ekberg et al 2007), and track force measurements (see e.g. Gullers et al 2011) of train–track interaction that can incorporate high-frequency loads – typically up to some kHz.

¹⁴ Hertzian contact conditions imply linear elastic material, a small contact area compared to body dimensions and contact radii, and smooth surface geometries on the macro and micro scale. If these presumptions are fulfilled, the resulting contact patch will be elliptic.

¹⁵ Full slip implies that the interfacial friction is proportional to the normal pressure, i.e. $\tau = f\sigma$ with f being the traction coefficient.

Finally, the case of **plain fatigue** can be analysed using standard methods of fatigue and fracture analysis. Note that plain fatigue approaches also are valid for the growth of long rolling contact fatigue cracks that have deviated into a transverse growth and can be presumed to be driven by rail bending, see Ekberg et al (2015b).

Means of influencing

Perhaps most efficient in mitigating the consequences of heavier and faster freight operations is to decrease contact stress magnitudes. Reduced interfacial shear forces can be obtained by improved steering of the vehicles, but also to improved control of braking and traction. Reductions in contact stress magnitudes can be obtained through better control of wheel and rail profiles. In particular, the possibility of evaluating the detrimental influence of worn profiles – and thereby mitigating the most detrimental profiles – should be noted. This relates both to hollow worn wheels, see Karttunen et al (2014), and to matching gauge corner / flange root geometries; see Karttunen et al (2015).

Further, the track geometry plays an important role. Regarding interfacial shear, the main factor is the lateral track geometry. Increased curve radii will decrease interfacial shear, but also shift the main deterioration mechanism away from wear towards RCF. Further, lateral track irregularities will have a significant influence in both increasing deterioration magnitudes locally and in introducing a more mixed mode of wear and RCF deterioration, see Karttunen et al (2012) and Karttunen et al (2014b).

Normal contact forces will influence the risk of transverse crack growth and rail breaks, in particular in cold climate. The influence of out-of-round wheels should be especially noted, see Ekberg et al (2015b). Here impact load detectors are useful in detecting out-of-round wheels for mitigating actions as noted in UIC (2014). In addition, rail defects will induce higher loads. In particular, for operations at higher speeds it is crucial to manage rail surface defects and wheel out-of-roundness to limit the dynamic loads. Note also that high traction/braking efforts in combination with surface defects (in different forms) seem to be a major contributing factor to the formation of so-called squats and similar RCF phenomena, cf Grassie 2011.

In the context of vertical forces, also the track stiffness should be considered. Especially stiffness transitions will be of higher importance the higher the loading is. This concerns “large-scale” transitions, e.g. in the entrance to a bridge, but also “small-scale” deviations such as hanging sleepers. In the latter case, it has been shown (see Segond and Wibeaux) that there is an increased risk for railhead cracks in an interval of some four sleeper spans in both directions from a hanging sleeper. Improving transition zone characteristics may yield significant benefits, as demonstrated in Ekberg and Paulsson (2009).

Similar as for the case of interfacial shear stresses, the wheel and rail profile matching will have a profound influence on normal stress magnitudes. It should here be noted that wear and RCF patterns on operating wheels and rails provide valuable information on the actual profile matching. In combination with an analysis of seasonal trends in wheel/rail damage, root causes of increased deterioration resulting from upgraded operational conditions can usually be identified, see e.g. Ekberg et al (2014).

The influence of wheel and rail material is an important – and very complex – topic. Depending on the main mode of deterioration, different characteristics of the material will be of more importance, cf INNTRACK (2009). In the case of plastic deformation, a higher yield limit is of course beneficial. However, it is not only the initial yield limit that is of

importance, but also the ability of the material to harden under cyclic loading. This fact is used to the extreme in the case of explosion hardened manganese steel crossings. As for wear, the wear resistance typically increases with increased hardness. Note however that if the hardness is obtained through an increased carbon content, the drawback may be a decreased thermal capability (i.e. a higher risk to form martensite). As for surface initiated RCF, premium grade rails have been shown to have a better resistance, see INNOTRACK (2009). However, wear and RCF are interacting phenomena where small surface cracks may be worn off. Since premium grade rails have less wear, this effect is less. Consequently, artificial wear in the form of controlled grinding is especially needed for premium grade rails. Regarding subsurface initiated rolling contact fatigue, a major factor regarding material selection is the material cleanliness. As for plain fatigue, standard fatigue strength parameters (fatigue limit including influence of environmental factors, crack growth characteristics and fracture toughness) are decisive.

To control the effects of upgraded operational conditions, increased monitoring can be adopted. Regarding safety, it has been shown (see D-RAIL (2014)) that the most beneficial monitoring systems are axle load checkpoints, track geometry measurement systems and hot axle box detectors. In addition, crack detection systems and profile measurement systems are useful in controlling and limiting deterioration, and to analyze deterioration trends. In this context it should be noted that the influence of upgraded conditions should be monitored over a longer period – typically some years. The reason is that it takes time for deterioration patterns to develop and stabilize. Further, seasonal effects may be significant.

As for switches and crossings, basically the same means of influencing exists as for rails. In addition, there are more possibilities of influencing the switch geometry and stiffness to reduce dynamic forces and contact stress magnitudes, see e.g. examples in Ekberg and Paulsson (2009). If the needs to reduce loading are significant, there are also possibilities of very efficient – but more costly – solutions such as longer switchblades and moveable frogs.

4.3.4.1.2 Fastening systems, sleeper / slab and ballast

Phenomena

Increased loads on fastenings and pads may exacerbate wear and fatigue of fastenings. This may trigger a vicious circle in which detaching fastenings and worn out rail pads increase dynamic loads, which results in even higher deterioration etc.

In addition to increased dynamic loads, deteriorated/loose fastenings and rail pads may result in loss of track stability. The consequences may be excessive gauge with rail rollover being the extreme example. During the warm season, also the risk of sun-kinks (lateral buckling of the track owing to thermally induced compressive stresses) will increase on all-welded tracks. In cases of high braking /tractive efforts of heavy haul trains, also rotation of sleepers may occur.

As for sleeper and slab, increased loading will of course increase the risk of fatigue and fracture. At high speeds on tracks with stiff rail pads, this relates e.g. to increased dynamic loads due to out-of-round wheels and transition zones. For higher axleloads these issues are also relevant, but in addition the higher axle load also imposes a higher nominal load. In this context also the effect of hanging sleepers in increasing the loading should be noted. This may cause damage to nearby sleepers, and also to the ballast (so-called "whitening") due to the "pumping effect" of a hanging sleeper.

Increased loads will also affect ballast deterioration e.g. in the form of settlements and ballast stone degradation. Note here that (if and) how an increased dynamic wheel/rail contact load is transferred to an increased loading on sleeper and ballast is complex: Generally, if the pad stiffness is soft the increased load is confined to the rail, whereas a stiffer pad will increase load transfer into sleeper/slab and ballast, see e.g. Bolmsvik and Nielsen (2014).

Regarding track settlements, it should be noted that tamping operations generally cannot restore the track geometry to the original condition, see e.g. Arasteh Khouy et al (2014). In addition, locations of high settlement typically refer to a localised condition on the track, see e.g. Ekberg and Paulsson (2010). This localised condition can be either a location where loading is increased (e.g. a transition zone) or where the resistance is decreased. Regardless, settlements are likely to reappear soon after mitigating actions if root causes for the settlement are not mitigated – a phenomenon commonly referred to as “track memory”. Operations at higher speeds and/or heavier axle load are likely to further reinforce these problems. Consequently, known problematic zones should be charted and mitigated before upgrading. Alternatively they may be kept under tighter inspections during the introduction of upgraded operations.

Ballast stone degradation will generally influence the internal friction of the ballast, which will decrease the ballast’s deformation resistance. In addition, the increased amount of fine material may have a detrimental effect on frost resistance and drainage capability of the track.

A decrease the lateral track resistance due to decreased internal friction of the ballast will also increase the risk of sun kink formation. In heavier operations, the risk of sun kink formation is also increased by the fact that curve and braking loads typically are higher as loads increase. Also in faster operations braking (and acceleration) loads may be higher to reduce travel times. In addition, consequences of sun kink formations are generally worse under highspeed operations. For this reason, the risk of sun kink formation should be considered in the assessment of consequences of upgraded operations.

Means of prediction

Numerical simulations of train–track interaction can be carried out to evaluate the influence of increased axle loads and/or train speed on the loading of fastenings and pads. This can include assessing the influence of deteriorated fastening systems. Note however that a thorough evaluation may require high-frequency load content to be accounted for.

Regarding the risk of sun-kinks, the influence of loose fastenings and ballast resistance can be assessed through FE-simulations. Note however that such an analysis is complex. In particular, the issue of how to define instability needs some consideration. Parts of the issue are scheduled for investigations in the EU-project In2Rail. Note in this context that also other factors such as ballast resistance will have a crucial effect; see Kabo (2006).

As for the risk of sleeper damage, the loading of sleeper (and slab) is very dependent on the support conditions; see e.g. UIC (2014). Track stiffness measurements can be used to identify sections where the support conditions may be insufficient.

Sleepers and slab should be designed to account for the increased loading. However, a design following too conservative design criteria regarding the risk of concrete cracking in sleepers may be sub-optimal. The reason is that the sleeper then becomes very stiff and heavy, which will affect the overall dynamic behaviour of the track. A more suitable option may therefore

be to design the sleeper based on a statistical analysis of the risk of sleeper damage; see Bolmsvik and Nielsen (2006).

Increased ballast settlements can be predicted from evaluated contact pressure distributions between sleepers (or slab) and ballast. An example of how numerically evaluated dynamic contact forces in a turnout can be used to predict resulting settlements is given in Li et al 2014. Note that variations in track stiffness etc. along the track will lead to differential settlements. The consequence will be track irregularities that induce higher dynamic wheel–rail contact forces, which leads to further track settlement, but also to higher risks of other types of damage phenomena to the track.

Means of influencing

In order to reduce (and/or control) operating loads, axle load checkpoints can be employed to identify and mitigate vehicles inducing excessive loads. This should be supported by inspection of fastening systems, and track stiffness. Here stiffness measurement cars give an objective and well-documented report on the current status. In the case of sleeper tracks, these can be complemented by visual inspections where "whitening" of ballast due to hanging sleepers can be identified. Note that mitigation of poorly supported sleepers/slabs should include inspection to identify potential cracks in nearby sleepers and rail (some four sleeper spans in both directions from the hanging sleeper). In addition, the inspections should assure that the poor support has not resulted in any damage to the fastening system.

To improve the ability to withstanding the increased load levels, the sleeper/slab and rail pad should be designed accordingly. As noted above, a statistically based design methodology here has significant benefits. In addition, sleeper/slab support conditions may be improved. This includes correct tamping, but also potential use of (correctly tuned) under sleeper pads.

As for higher speed, this generally calls for more stable track support, especially if the speeds are becoming very high as in future scenarios of high-value freight operations. There are in essence two ways of achieving this:

1. Sleeper track on compacted and homogenous support where under sleeper pads etc. can be employed to ensure sufficient track flexibility and load distribution.
2. Slab-track solutions of varying designs. Also here ensuring suitable support conditions is crucial.

Both solutions have benefits and drawbacks. These include the long-term stability of the track geometry, as well as costs and complications when the geometry eventually needs to be mitigated. Slab track solutions essentially eliminate the (admittedly low on high-quality tracks) risk of sun-kinks and ballast spray. Further, the risk of rail breaks is lower on slab tracks due to the decreased bending of rail on slab track.

In both sleeper and slab tracks, the risk of hanging sleepers / poor support of track sections needs to be accounted for. This includes requirements for drainage that can handle flooding events and also possible solutions to mitigate track settlements. Further aspects to be considered when designing the track solution include the risk of rail corrugation, noise emissions etc. and also environmental, safety and economic consequences.

To balance these different factors is complex. The task is not made easier by the fact that neither the "sleeper track", nor the "slab track" is a well-defined construction: There exists a multitude of solutions, and the characteristics of the different track solutions vary significantly also within each category. Ultimately, the choice of a suitable track support for

the upgraded conditions therefor comes down to a thorough analysis to establish relevant, realistic and well-defined technical requirements for the track.

To mitigate the effect of high dynamic loads, correctly tuned rail pads and under sleeper pads can be used. In addition, there is a major potential in designing transition zones as to minimize dynamic loading. This topic will be investigated in the currently initiated European project In2Rail.

4.3.4.1.3 Switches & crossings

As noted above, phenomena occurring on plain track are also found in switches & crossings (S&C), often to a higher degree. There are several reasons that S&Cs are over-represented in failure statistics. Some of these are:

1. The construction imposes a stiffness transition in the track
2. The complicated geometry gives rise to high contact pressures due to contact point transitions, contact close to free edges, etc.
3. High loading at fairly small cross-sections, e.g. in switch-blades
4. Impact loads due to traversals of gaps
5. High frictional loads due to sharp curving in diverging mode
6. The switch contains moving parts
7. The switch interacts closely with the signalling system

The influence of these (and other) effects can be analysed using methods and tools discussed above. It could be noted that the complex geometry naturally complicates such analyses. Further, there are four modes of traversing a switch (facing/trailing move, through/diverging route). Actions that improve the characteristics in one of the modes does not necessarily benefit the other modes – it may even be that they are detrimental.

As for the items in the list above, these can be mitigated to some extent in different manners:

1. Stiffness transitions lead to high vertical (and potentially lateral) loads. This may cause settlements, rolling contact fatigue (RCF) and plastic deformations. The influence can be mitigated by designing the support structure in an appropriate manner. Here the use of under sleeper pads and ballast mats are of use.
2. High contact pressures can cause RCF and plastic deformation. The high contact pressures can be targeted by adapted profiles. A complication is that these have to be able to handle all passing wheel profiles. In general, the result is that the profiles have to be adopted so that the majority of the wheel profiles give rise to (relatively) low contact pressures, whereas some outlier wheel profiles may cause high contact pressures, but do not cause a safety risk.
3. The loading of switch blades can cause plain fatigue and plastic deformation of the switch blades. The amount of loading can be controlled by the switch geometry. This is however a complicated issue since a late transition to the switch blade (i.e. when the switch blade cross-section is larger) has negative effects on the curving capability of the switch.

4. Impact loads may result in settlements, plastic deformations, wear and RCF. The impact loads can be decreased by designing profiles to allow for a smooth passage over the discontinuities in the running band, especially at the crossing. Also here it is a matter of compromise since the profiles also need to ensure a safe and not too severely loaded passage also of poor wheel profiles (e.g. with significant hollow wear). A further extension in reducing impact loads is the use of moving frogs. Drawback with these are the cost and that they introduce an additional moving part.
5. The high frictional loads can cause wear and RCF. They can be targeted by better curving through more track-friendly vehicles and/or a tailored gauge widening (see INNTRACK Deliverables D3.1.5 and D3.1.6 for details). Further the use of longer switches (with more shallow curving) and lubrication are potential mitigating actions.
6. The moving parts in a switch introduces additional modes of failure. The movement can be jammed by objects (ballast stones, ice etc.), the driving and locking devices may fail etc. To minimize the risk of failure it is important to make the design as fail-proof and robust as possible. This also includes aspects such as a precise evaluation of allowable closure gaps. Further, there is a need for well-planned monitoring and maintenance to identify and mitigate potential problems before they cause
7. The close interaction with the signalling system means that failures on the S&C will be reflected in the signalling system and vice versa. Normally one branch of maintenance staff handles the signalling system, whereas the S&C typically is included in the track maintenance. It is therefore important to be able to locate malfunctions as precisely as possible. This also relates to the strive to have clear interfaces between the S&C and the signalling system. Note also that a similar interface exists between the S&C and power support. This is sometimes not restricted to power to run the driving and locking device, but also e.g. for defrosting of the S&C in cold climate.

In addition to the description above, it should be noted that operations in a diverging route are often the most detrimental for an S&C. Consequently, not only the operational frequency, but also even more the frequency of operations in a diverging route will govern the deterioration of a switch.

Further, S&Cs are used to give flexibility in operations and are therefore usually placed at key locations in a system. In addition, an S&C involves at least two tracks. As a result, the failure of an S&C will often cause more operational disturbances than a line failure.

In general, upgraded conditions will influence the discussed phenomena in a manner similar to what was discussed mainly from a track perspective above. These issues should therefore be handled in a general upgrading plan. However, due to the higher sensitivity of S&C there are often reasons to study these separately in a more detailed analysis. In particular, there are often some key S&Cs in a network (e.g. at the entrance to major stations and shunting yards). To investigate these in detail and adopt strategies to mitigate detrimental effects of upgraded conditions is generally a good idea.

4.3.4.2 Consequences of longer trains

4.3.4.2.1 Track deterioration

In contrast to higher or faster operations, operations with longer trains need not necessarily increase the load magnitude on individual axles; whether this is the case will depend on the train configuration as discussed below. If the loading on the individual vehicle axles is not increased (see chapter 3), the mechanical consequences of the altered operational loading (as related to the track) are confined to the influence of the increased number of passing axles on a track section. Before a discussion on the consequences of longer trains, it is vital to understand the difference between increasing the load magnitude on (a number of) axles and increasing the number of passing axles: In general, mechanical damage (be it rolling contact fatigue, settlements etc) can often be approximated as proportional to the number of load cycles (passing wheels) and to an exponential function of the load, i.e.

$$D \equiv 1/L \propto N \cdot P^m \quad (10)$$

Here D is the damage, L is the remaining operational life, N the number of load cycles (passing wheels), P the load and m and exponent, typically in the order of 3. From equation (10) it is clear that a doubling of the load per axle has far more detrimental consequences than a doubling of the number of passing axles. Naturally equation (10) is a very rough approximation – e.g. loads under a certain threshold magnitude are not considered to have any (or only a very small) influence. Still it shows the qualitative differences between the two ways of doubling the loading.

In the case of vertical loading, a longer train generally will not influence the load on individual axles. Consequently, the influence on increased deterioration will be through the increased number of axles. Thus the theoretical decrease in track life should be proportional to the number of axles.

A longer train generally requires higher tractive and braking efforts. If these efforts are distributed per wagon this will not increase the braking and tractive efforts per axle. For braking this might be the case if distributed tread braking is employed. However, there is a current trend towards more regenerative braking, which will concentrate the increased braking efforts on the locos. For traction, the usual scenario in freight operations is that this is provided by locos.

If locos are employed for higher traction and braking efforts, the results will be more wear and rolling contact fatigue on loco wheels. On the rail, this will typically correspond to an increased occurrence of gauge corner wear, and headchecks. In the case of increased traction, there is also a growing agreement in the research community that this is a likely cause for increased squat formation.

In a scenario where increased braking/tractive efforts are delivered by the loco there will also be an increased requirement for sufficient friction in operations with steeper slopes and/or required stops.

There are also a number of more vehicle related issues with operations of longer trains. One such issue is the increased loading of couplings. A common way of mitigation is the use of distributed locos in the train, see e.g. Govender and Pillay (2015). An additional phenomenon that may occur is longitudinal vibrations within the train, see e.g. Muginshtein and Yabko (2015). These issues and mitigating actions can be investigated by numerical simulations and further assessed through field measurements e.g. of stresses in couplings.

Finally, it should be noted that the derailment risks of long trains might increase if unloaded (or lightly loaded) wagons are placed in the front part of the train.

4.3.4.2.2 Infrastructure needs

Longer trains may also require modifications of the infrastructure. Such modifications include:

Meeting stations	that may need to be extended to allow the longer trains to utilise the station.
Level crossings	that may need to be modified to ensure that road signals etc. operate for a sufficiently long time. In addition, there may be needs for actions to avoid too long road closures if the long trains are operating in parallel with shorter (and faster) trains.
Signalling sections	which may need to be adjusted to account for the longer trains.
Signals	that may require repositioning e.g. so that stopped trains do not block a level crossing or intrude on the loading gauge of a nearby track at meeting stations. This also includes repositioning to ensure that slopes at a stop signal are sufficiently low also for the longer trains.

4.3.4.3 Consequences of increased operational frequency

If the operational frequency is increased without any modifications to the vehicles, the increased deterioration of the track will – in theory – increase in direct proportion to the number of trains (see the elaboration on this in the previous section on longer trains). There are however still significant practical complications that will arise. In short these can be summarized as

- There will be shorter intervals (in wall-clock time) between required maintenance actions.
- There will be less time (and shorter intervals) available in track for the required maintenance actions.
- There will be less room for, and worse consequences of unplanned events such as vehicle breakdowns.

These consequences interact and together significantly increase costs for maintenance. In the extension, this requires a shift from an “infrastructure approach” – where faults are dealt with as they occur – to a “process approach” – where 24/7 operations are required and potential disruptions need to be handled before they interfere with operations. Some means of targeting this are provided below. Note also that chapter 5 focuses on maintenance. For this reason, the description below approaches the topic from a technological point-of-view based on the discussion in other parts of section 4.3.

4.3.4.3.1 How to reduce deterioration – general principles

As discussed above an increased operational frequency often leads to a requirement that deterioration levels (per "unit load") need to be decreased if maintenance should be economically and logistically feasible. Decreased deterioration can be obtained by decreasing the loading and/or increasing the strength of a component. However, this is far more complicated than it sounds. The reason is first that the definition of "load" and "strength" varies dramatically between components and deterioration phenomena – an indication of the complexity is given by the different damage modes for rails discussed above.

In addition, there are large statistical uncertainties and variations. This is especially cumbersome for threshold phenomena that do not pose any problem until the load/strength ratio reaches a certain threshold. An illustrative example is the case of sun kinks (lateral track buckling). In this case the "loading" is the stress in the rail and the "strength" is the resistance of the rail to lateral buckling. If the load/strength ratio is below a critical value, no sun kink will form. This sounds simple enough, but as often the devil is in the details: The stress in the rail will in this case mainly be governed by the rail temperature in relation to the stress free temperature¹⁶. However, it will also be influenced by the track geometry (e.g. curves and lateral track geometry irregularities). To establish the stress free temperature is generally related to large uncertainties. Also the resistance of the rail to lateral buckling will be significantly more complicated when examined in detail: It will be influenced by (among other things) how the rail is supported to the sleeper, the friction between sleeper and ballast, and by the ballast geometry. These factors interact in a very complex manner, cf Kabo 2006, and are all encumbered with uncertainties. Add to this that sun kinks will only occur on the worst location(s) along the track. The consequence is that a design to completely avoid any sun kinks will be extremely over-designed and expensive (if it would even be possible to obtain at all). Instead, a (implicit or explicit) risk-based analysis in combination with periodic inspections is required to reduce the risk of sun kinks to acceptable levels.

Sun kink formation is in essence a "true" threshold problem – either you have a sun kink or you do not – however the main features of the sun kink example are valid also for phenomena where deterioration is a more gradual process, e.g. wear, rolling contact fatigue, settlements etc. For all of these the main characteristics are valid:

- The deterioration phenomena are complex with a multitude of influencing and interacting parameters.
- Many of the involved parameters are encumbered with uncertainties.
- Designing the system to completely avoid the deterioration phenomena is unrealistic from technical, economical, operational and safety¹⁷ perspectives.

Consequently, deterioration needs to be dealt with in a "risk aware" and pro-active manner. Here the consequences of any deterioration (and in particular deterioration that causes operational disturbances) needs to be considered and contrasted to costs of preventive actions. In this analysis it should be considered that deterioration typically is an accelerating phenomenon: Early interventions are typically less costly than delayed interventions even if

¹⁶ The stress free temperature is the temperature at which there are no thermal stresses in the rail.

¹⁷ Recall that train operations are some 50–100 times safer than road transportations. Add to this the very low likelihood that people and freight agencies would use train operations if costs were tripled and/or speeds/axle loads halved.

additional costs for unplanned interventions (in contrast to planned interventions) are not accounted for.

Note that such an analysis will result in different preventive actions to be considered profitable for different scenarios: Such preventive actions may be preventive maintenance, but also e.g. redundant systems in trains.

4.3.4.3.2 Faster maintenance and inspection (automated monitoring)

As mentioned above increased operational frequencies generally result in less time (and shorter time slots) for maintenance. This calls for faster maintenance and inspection routines.

There are basically two ways of achieving this:

1. Focus inspection and maintenance to where it is most needed.
2. Carry out required inspections and maintenance with less time in track.

The first item is often overlooked, but carries extreme potential: Many inspection and maintenance routines are based on tradition; with development in knowledge and technology it is not certain that these are still the optimum. Further, possibilities for analysis of inspection data have improved significantly in later years. This will also be discussed below.

As for faster inspection and maintenance, this includes topics such as modularisation and “plug-and-play” replacements. This will be further discussed in chapter 5.

An aspect that could not be overemphasized neither when it comes to inspection/monitoring, nor when it comes to modularised replacements is the definition of standardized interfaces. Here it is important to define exactly where the interfaces are (in terms of data transfer or physical contacts) and which aspects that need to be standardized. Among other benefits, a proper definition of standardized interfaces will also aid the industry in focusing on development within given frames. This should benefit both efficiency and competition.

4.3.4.3.3 Reduced allowance for / increased consequences of unplanned actions and failures

As mentioned, increased operational frequencies decrease the room for – and increase the consequences of – unplanned disruptions. This means that “trial-and-error” is no longer an option; solutions must – as far as possible – be validated before introduction in operations. The means to provide a validation beforehand are basically:

Simulations – an efficient way to evaluate a number of “what if” scenarios with varying complexities.

Test track operations – although typically limited in what can be tested (and costly) such tests provide a controlled environment that can e.g. be used to validate/calibrate simulations.

Operations on low-frequency lines and/or operations in other networks – in some sense as close to “real life” as can be, but differences between test conditions and future operational conditions must be identified and quantified.

In addition to validating solutions as far as possible beforehand, there is also a need to mitigate any operational complications (especially during the “run-in” period) as fast and efficient as possible. Ideally this calls for a pre-operations risk analysis before the start of upgraded operations not only from a logistics/operational point-of-view, but also from a technical point-of-view.

4.3.4.3.4 Improved predictive capabilities

In the previous sections “simulations” were highlighted as a means to validate solutions before introduction into operations. In addition, simulations have a vital role also in predicting deterioration and the risk of failure. This is critical if it should be possible to plan maintenance and inspections. It also has the additional benefit of minimizing (in the sense that they can aid in optimizing what and when) inspections and maintenance actions. As discussed above this reduces needs for track possessions and also lowers LCC costs.

Possibilities and challenges in prediction naturally depend on the involved component / mode of deterioration, but also on the required level of detail in the predictions. An overview of possibilities and challenges for a wide range of components/modes of deterioration was given above.

It should be noted that it is important to continuously calibrate and validate predictive models towards actual deterioration patterns. Simple regression analysis of measured data (big data analysis) may help, but caution should be taken since data analysis with no underlying physical model can be very imprecise and deteriorate over time, see e.g. Lazer et al (2014). To this end, the aim should be to have as precise physical models as possible and tune predictions from these towards observed behaviour. In addition to allowing extrapolations of current operational characteristics, such an approach will also provide the massive benefits of being able to analyse “what if scenarios” and to investigate likely consequences of planned changes in operations.

4.3.4.4 Consequences of increased load gauge

Unless an increased load gauge also affects the load of the vehicles, it will not lead to any increased deterioration of the track. It will however have consequences on the track construction that need to be considered. This will be described in overview below.

4.3.4.4.1 Wider load gauge

Operations with a wider loading gauge require a general gauge widening on the parts of the lines to be operated with the wider gauge. Naturally this will cause logistic complications if wider gauge is allowed only on parts of a network, or even in cases where different gauges are allowed in connected countries with international operations.

As for curves the required load gauge will not only depend on the nominal gauge of the vehicles, but also of their curving characteristics, e.g. how much additional space the vehicles require to negotiate curves of different radii.

Note also the particular case of stations where the required stop point before a signal is depending on the vehicles load gauge. Consequently, upgrading to a wider gauge may require such stop signals to be relocated.

Tunnels are especially complicated to upgrade to a wider gauge (unless the initial design was sufficient visionary to foresee such an upgrade). Requirements to widen a tunnel will have consequences on installations, existing reinforcements etc, and may also require additional reinforcement due to a reduced load carrying capacity of the tunnel structure.

4.3.4.4.2 Higher gauges

Allowing a higher gauge in a tunnel is generally easier than adopting to a wider load gauge, since the case of a higher gauge can be accomplished by increasing the depth of the tunnel through bench blasting. In general, this has much less effect on the bearing capacity of the tunnel structure than an increased width.

On the line outside of tunnels an increased height of the gauge should give no complication as long as it is within the catenary tolerances. If not, either the track section need to be lowered or the catenary heightened. In the former case, special care needs to be taken regarding culverts etc, but also regarding signalling installations in the track bed.

4.3.4.4.3 Means of prediction / influencing

Typically, consequences of an increased gauge are evaluated through (static or quasi-static) geometrical analyses. To improve the ability to account for the dynamic motions of the train, these can be complemented with multi-body simulations of train–track interaction. In special cases (e.g. tunnel entrances) also aerodynamic characteristics may affect the required geometry.

The analytical and/or numerical analyses can be complemented by track load gauge measurements from an operating vehicle and by track based vehicle load gauge measurements.

Basically the only means of influencing the effect of a certain load gauge is through the bogie design. Through the bogie design the height and the lateral deflection of the wagon can be controlled to some extent. Note however that all wider vehicles on the line need to be mitigated if this should be an efficient approach.

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Mallorie Segond and Quentin Wibeaux, The influence of track stiffness on rail crack occurrence, *Projet de Fin d'Études en Génie Mécanique, Chalmers Applied Mechanics*, 56 pp, 2014

David Thompson and Chris Jones, Noise and vibration from the wheel–rail interface, in *Wheel-rail interface handbook* (eds. Roger Lewis and Ulf Olofsson), Woodhead Publishing, Cambridge (UK), ISBN 978-1-84569-412-8, pp 477-509, 2009

UIC, HRMS: Harmonization – Running behaviour and noise on measurement sites, 153 pp, 2014.

4.4 ENERGY SUPPLY

Energy supply will not be dealt with due to budget limitations. If the Executive Board decides, it can be included in D1.1.5.

4.5 SIGNALLING

Also signalling will not be included due to budget limitations. If the Executive Board decides, it can be included in D1.1.5.

4.6 STATIONS

Stations have an important impact on some of the traffic situations. Examples on this is longer trains and increased loading gauges. However, stations will not be included in D1.1.4 due to budget limitations. If the Executive Board decides, it can be included in D1.1.5.

5 MAINTENANCE ROUTINES

A railway track is a complex system, which comprises different subsystems, namely rails, fastenings, sleepers, ballast, slabs, track formation and subsoil and assets like bridges, switches and crossings or level crossings. The maintenance of all these subsystems is a challenging issue influenced by parameters such as geographical and geological features, topography or climate conditions. This needs to be taken into account during maintenance planning¹⁸. The required performance and reliability of the whole system depends on the performance and reliability of each of these heterogeneous subsystems. Any type of fault/failure in any of these components might result in a system fault/failure or higher degradation rate(s) in the other subsystem(s). To achieve the desired overall system performance, it is necessary to attain synergy and account for interaction between the subsystems including the interaction with operating vehicles.

Manual and automated inspection methods are applied to measure the track geometry and condition of track components. To evaluate the track condition efficiently and effectively, it is essential to determine optimal inspection intervals for each subsystem through a systematic deterioration / risk / RAMS / LCC assessment. Insufficient inspections (too long inspection intervals) may result in higher risk of accidents, lower track availability and greater maintenance cost; while, on the other hand, too frequent inspections can cause increased traffic disturbances and consume an unnecessary amount of resources.

Thereafter, the collected data is processed to identify faults that need to be restored either by preventive or corrective maintenance actions. Critical failures, which may cause derailments, are reported immediately to the operation control centre and the infrastructure manager.

The maintenance intervals (limits) should in the optimum case be specified by applying an efficient and effective decision support system which not only assures the safety but also make operation and maintenance procedures cost-effective. Extra or inadequate maintenance may result in higher degradation of the assets. For instance in tamping, too frequent tamping can result in higher degradation of the ballast. On the other hand, allowing the track quality to deteriorate to higher levels by postponing maintenance would require more than one tamping operation since the initial track quality cannot be achieved by normal tamping; consequently, it leads – depending on the quality of ballast stones – to higher degradation rate of ballast¹⁹.

¹⁸ Jimenez-Redondo N., et al. (2012) Automated and Cost Effective Maintenance for Railway (ACEM-Rail), Procedia – Social and Behavioral Sciences 48, 1058-1067.

¹⁹ UIC (2008) Best practice guide for optimum track geometry durability, Paris, France: ETF - Railway Technical Publications.

Table 5.1 The main maintenance routines for each subsystem are stated in the following²⁰.

Track subsystems	Maintenance routines
Rails	<ul style="list-style-type: none"> • Ultrasonic / eddy current inspection • Welding • Grinding / milling • Joint replacement • Replacement
Fastenings	<ul style="list-style-type: none"> • Elastic pads renewal • Gauge correction • Re-tensioning • Replacement
Sleepers – in case of ballasted track	<ul style="list-style-type: none"> • Tamping • Replacement • Implementation of Under Sleeper Pads
Ballast – in case of ballasted track	<ul style="list-style-type: none"> • Tamping • Stabilising • Cleaning • Refilling • Wild plants removal
Track formation	<ul style="list-style-type: none"> • Draining • Insertion of frost protective layer • Insertion of geotextile / ballast mats • Completing and compacting • Insertion of frost protective layer
Slab in case of ballast-less tracks	<ul style="list-style-type: none"> • Grout injection of crack • Adjustment of slab by injection (soil / HBL)
Subsoil	<ul style="list-style-type: none"> • Draining • Examination of defective locations • Soil replacement • Soil improvement

5.1 RAIL AND WHEEL GRADE SELECTION

Rail grade selection is recommended to follow UIC Code 721 R²¹. This code was finalised in October 2013. The code is considering increase in railway traffic, heavier axle loads, higher speeds and the introduction of new generation of rolling stock. It is largely based on the

²⁰ Lichtberger, B. (2005) Track compendium: Formation, Permanent Way, Maintenance, Economics, Eurailpress, pp. 98.

²¹ **UIC 721 R (2013)**: Recommendation for use of rail steel grades. UIC Code 721 R, 2013, International Union of Railways, Paris

findings from the EU-project INNOTRACK²². Regarding (heavy) freight traffic, the topic is also discussed in the IHHA Best practice handbook published in June 2015²³.

The selection of wheel and steel grades is governed by the operational loading. In very general terms a premium grade steel shows less wear and less propensity for headcheck formation. In contrast, a softer rail will be more "forgiving" in that wear-in of the rail profile towards (an envelope of passing) wheel profiles is a fairly fast process. With less natural wear, premium rail grades require grinding to assure that profile matching between rail and wheel is good. The trade off is a more stable profile as well as lower risk of headcheck formation and slower crack growth when these occur. Note that this implies that benefits of premium grades are generally confined to curves. This is also noted and quantified in the UIC Leaflet.

An interesting new development in steel grades for rails and wheels is the use of bainitic steels. These generally have a very high resistance against rolling contact fatigue, but to obtain equally high wear resistance as for pearlitic steels there is a need for a strictly controlled manufacturing process²⁴.

5.2 GRINDING

Modern railway traffic operations provokes in many places (depending on local, operational conditions) a rail surface fatigue phenomenon referred to as rolling contact fatigue (RCF). Head checks, rail corrugation and similar RCF defects develop sooner or later. The rail steel quality plays a determining role, but there is no material available at present, which fully can prevent the development of RCF cracks. Furthermore, the majority of rails in track today, with adequate but lower fatigue resistance, have a residual life span, which makes it much more economical to maintain them in an appropriate manner rather than to replace them. Thus, rail maintenance is an inevitable must. The best way to enhance rail life if fatigue cracks have formed is through grinding.

In INNOTRACK rail grinding was investigated, resulting in four deliverables and a UIC/UNIFE Technical recommendation (TecRec). This work represented the latest findings at the time. However, the work in INNOTRACK did not include High Speed Grinding, nor milling.

To increase the lifespan of the rails and to decrease the life cycle costs preventive rail grinding is used more and more often in the last years. Typical preventive grinding strategies are; Two Pass Grinding (TPG) or High Speed Grinding (HSG).

In contrast to TPG, where conventional grinder with 48 or 64 stones are used, HSG requires a special high speed grinder that runs at a speed of approximately 80km/h.

²² Anders Ekberg & Björn Paulsson (eds), INNOTRACK--Concluding technical report, International Union of Railways (UIC), ISBN 978-2-7461-1850-8, 288 pp, 2010
INNOTRACK, Deliverable D4.1.5 – Definitive guidelines on the use of different rail grades according to duty conditions and based on RAMS and LCC principles.

²³ <http://ihha.net/resources>

²⁴ Andrea Gianni, Andrea Ghidini, Tord Karlsson & Anders Ekberg, Bainitic steel grade for solid wheels: metallurgical, mechanical, and in-service testing, IMechE Journal of Rail and Rapid Transit, vol 223, no 2, pp 163-171, 2009

The higher speed is the major advantage of HSG as the grinding train operates like a “normal” train without causing major disturbance to operational traffic. Especially at lines with passenger trains at day and freight trains at night HSG facilitates the maintenance activities.

But due to the high speed the grinder only removes a small amount of material. Therefore (at least) three passes are necessary during one grinding activity. Starting with a failure free rail surface, which could be achieved by conventional grinding, HSG is used to remove cracks and other failures in an initial state. This means HSG behaves like a controlled “natural” wear. Depending on the boundary conditions (traffic, rail steel, track layout, routing) 3 to 6 grinding activities per year are necessary to maintain the rail surface free of failures. Practical experiences show that HSG is insufficient to re-profile the rail and to perform grinding in curves with high side wear. In these cases, conventional grinding is necessary.

The right mixture of HSG and conventional grinding or milling increases the lifespan of the rail and reduces maintenance costs.

The need for grinding is determined by profile geometry (which requires means to quantify the quality of the profile) and/or the existence of rail cracks. In addition, grinding can also be employed to decrease surface roughness (corrugation) and resulting rolling noise and dynamic wheel–rail contact forces.

As for rail defects, these are categorised in the UIC Leaflet 712²⁵. Defect levels that can be accepted depend on the operational conditions at hand.

The relation between surface roughness spectra, rolling noise and dynamic wheel–rail contact forces has been quantified in published studies²⁶. Also in this case, the operational conditions (e.g. if the railway is operating in populated areas) will decide suitable limit values.

Finally, it should be noted that if the traffic situation is altered it is important to re-examine the grinding strategy.

5.3 WHEEL RE-PROFILING

The most expensive component on a wagon that needs periodic maintenance and replacement is the wheel. In addition to result in costly reprofiling, poorly maintained wheels also increase track wear and tear. Hence appropriate tolerances for wheel wear are highly important. Accurate predictions of wheel status enable more cost-effective maintenance operation. Primarily, the following four factors usually drive the life expectancy of wheel maintenance; (a) available rim material; (b) dominant wear pattern; (c) wear rate; (d) work rate of the rolling stock. These factors have a significant influence on how much material is available to be removed either by normal operational wear or by re-profiling. In terms of normal wheel wear, it is important to know whether to expect predominantly flange wear or hollow wear as the influence on wheel life is significant. Hollow worn wheels and wheels with uniform tread wear require relatively little material removal, whereas wheels with flange wear require significant material removal to restore the flange.

²⁵ UIC Code 712CR, Rail defects, 4th edition, January 2002.

²⁶ Jens C O Nielsen & Anders Ekberg, Acceptance criterion for rail roughness level spectrum based on assessment of rolling contact fatigue and rolling noise, *Wear*, vol 271, no 1–2, pp 319–327, 2011

The relationships of contact mechanics to wheel/rail performance involves accounting for factors such as contact stress, creepage, conicity, conformity and curving, in order to optimise wheel and rail profiles. Maintaining a compatible wheel profile is critical – low contact stresses, favourable steering, good wheel set stability and low creepage all prolong rail life. Railways are exploring the benefits of the latest lubrication technologies, bogie types and metallurgies. However, wheel/rail contact mechanics phenomena are often overlooked or poorly controlled. The need to quantify the performance of worn wheel profiles is important for controlling and understanding the fundamentals of the wheel/rail interface.

In general wheel reprofiling is carried out to ensure a correct wheel profile, a correct diameter, and to remove severe defects.

From a general maintenance perspective, the wheel geometry is currently defined by the flange thickness, slope and height. The former two are basically safety measures, whereas the third is an indirect measure of tread wear. Although wheel reprofiling is generally carried out towards limit values related to these measures, they provide only a rough picture of the overall wheel geometry and how this geometry affects the rail. More refined methods are being developed that employ the entire wheel profile geometry to quantify its impact on further imposing deterioration²⁷.

Categorisation of wheel defects have been compiled in defect catalogues²⁸. However, in general reporting of wheel damage is usually much rougher in descriptions. This has implication since accurate maintenance strategies need to be based on a good understanding of the deterioration status of the fleet. To this end, there are attempts to refine reporting procedures²⁹.

It should also be noticed that most vehicles have restrictions on wheel diameter differences between wheels on the same axle or in the same bogie. The consequence is that it is not unusual that most reprofilings occur as a secondary consequence of the need to reprofile one damaged wheel.

5.4 LUBRICATION

Due to increased stress levels, in the wheel/rail interface due to altered loading conditions (increased load cycles, higher speed and dynamic forces, higher axle loads, higher acceleration/degradation) there are needs for new methods and maintenance procedures to counter these stress levels. Wheel/rail friction management, using gauge lubrication, Top of Rail friction modifiers, and/or flange lubrication etc can, if properly employed, achieve this aim

From a scientific and technological standpoint there is a need to understand and develop the “Top of Rail” and “Flange” lubricating systems (way side and/or on board) and to quantify the influence of friction control and management (lubrication and/or friction modifiers) on the

²⁷ Kalle Karttunen, Elena Kabo & Anders Ekberg, Numerical assessment of the influence of worn wheel tread geometry on rail and wheel deterioration, *Wear*, vol 317, no 1–2, pp 77–91, 2014

²⁸ Deuce R. Wheel tread damage – an elementary guide. Report no. 100115000, 2007, 38pp. Netphen, Germany: Bombardier Transportation GmbH.

²⁹ Anders Ekberg & Elena Kabo, Classification of wheel damage, Chalmers Applied Mechanics, Research report 2011:13, 7 pp, 2011

development and evolution of damage in wheels and rail (wear, fatigue cracks, contact fatigue damages, corrugations) and also on energy consumption of locomotives. Largely this requires enhanced means of collecting and analysing data.

A large amount of Top of Rail (TOR) related research regarding friction at the wheel/rail interface and corresponding energy savings has been produced recently (around 2011–2013). Often these investigations are using simplified small scale lab apparatus to estimate friction properties. One example of these types of research contributors regarding friction and energy savings, which also propose a related research methodology, is performed in Australia [³⁰]. In general the study reports possible energy savings (up to 30 % reduction of the electric power to the locomotives) if the friction can be hold at optimum values. This is most likely unrealistic values. More important is that there have been relatively limited investigations of the impact of friction modifiers (FM) on RCF. However, regarding RCF, Reiff [³¹] reported in 2007, reduced RCF generation and grinding requirements with FM application in heavy haul applications compared to a control zone and D.T Eadie et al [³²] were in 2008 using a field study to investigate the effect of FM on short pitch corrugation generation in curves. Increased scientific knowledge is thus needed, both laboratory based and in the field to better understand this area. One example of a full scale test rig for this purpose is the rail wheel test rig at voestalpine Schienen GmbH. Regarding RCF crack formation and propagation, the test result from this rig are not clear so far³³, indicating that more research is needed and also suggested by several researchers³⁴ that entrapped lubricant can lead to conditions that favour accelerated crack growth due to the effects of the entrapped incompressible fluid and corresponding hydro pressurization. Eadie et.al. also performed a comprehensive field test and related lab tests on a heavy haul freight railroad in the western United states where two test zones were established for testing with and without FM using TOR lubrication. The two 10-mile test zones were separated by a 5-mile buffer zone. It is interesting to note the good agreement between the lab tests and the field test in this particular study. The main reason is most likely the large full-scale size of the lab tests. To conclude, more research is needed on the topic of how TOR will affect wear and crack propagations as well as on how TOR influences friction (safety reasons) compared with ordinary (no TOR) condition. Also more research is needed regarding the total life cycle costs of TOR from the maintenance and life cycle cost perspectives and regarding “best practice” of TOR application, since almost no earlier research has been performed on these topics.

Friction coefficients between the rails and the wheels on a 360t and 10,800 kW IORE locomotive has been measured using the locomotive's built-in traction force measurement system. The locomotive hauled a fully loaded set of 68 ore wagons (120t/wagon). The most important results are that the friction coefficient is definitely not constant and surprisingly low (0.10–0.25) when the ToR friction modifier is used. It is also significantly dependent on

³⁰ .D.Raman, G. Chattopadhyay, M. Spirigian, M.Sajjad, Research methodology for evaluation of top of rail friction management in Australian Heavy Haul Networks, Proceedings from 10 th International Heavy haul Conference 2013, New Delhi, India.

³¹ R.Reiff, Top of rail friction control on rail surface performance and grinding, TTCI Technology Digest TD-07-039 (2007)

³² D.T Eadie, D. Elvidge, K.Oldknow, R.Stock, P.Pointner, J. Kalousek, P.Klauser, The effect of top of rail friction modifier on wear and rolling contact fatigue:full scale rail-wheel test rig evaluation, analysis and modeling, Wear 265 (2008) 1222-1230

³³ S. Bogdanski, Liquid-solid interaction at opening in rolling contact fatigue cracks, Wear (2005)1273-1279

³⁴ S. Bogdanski, P.Lewicki, m. Szymaniak, Experimental and theoretical investigation of the phenomenon of filling the RCF crack with liquid, Wear 258 (2005) 1280-1287

the amount of ToR friction modifier present. Furthermore, it is concluded that operational train/track friction coefficients are in general lower than the friction coefficients measured with the hand-operated tribometer. A final remark is thus that the use of a water-based ToR friction modifier can give excessively low friction, which can result in unacceptably long braking distances³⁵.

5.5 CRACK DETECTION

To detect cracks on wheels and rails, a broad range of inspection methods are employed. These include ocular inspection, ultrasonic testing, eddy-current testing and even magnetic particle inspection at special occasions.

The inspection interval is often regulated in infrastructure owner's safety regulations. These regulations are usually developed during, and relies on, years of practical experience. A change in operational conditions (change of cars, wheel and rail profiles, increased axle load etc.), or in maintenance strategies will have an effect on the formation and development of RCF. Consequently, empirically decided inspection intervals may no longer be relevant.

Ultrasonic inspection of the rail is a common method to detect visible and non-visible RCF formation such as severe head-checks and development of shelling. The detection is usually carried out using ultrasonic train inspection. Hi-rail test vehicles are often used for ultrasonic rail flaw detection by light rail, commuter and heavy haul railroads. A contractor can supply a fleet that includes vehicles for stop or non-stop high speed testing. Stop testing gives immediate verification of defects while non-stop testing allows for a chase vehicle to verify defects at indicated locations.

For the rails there is still problems to solve in order to detect cracks close to the surface by using eddy-current methods or ultrasonic testing (UT) for deeper positioned cracks. There is also a potential to use the Barkhausen effect in in order to detect early material changes in the rail due to RCF, which later can result in crack initiation. Some commercial measurement equipment is today available for this purpose

Crack detection challenges today mainly concern high-manganese crossings. The advantages of high –manganese steel have been studied by several researchers, see for example the works by Smith et.al [³⁶. Unfortunately there is one evident disadvantage with high manganese steel as compared to conventional steel and that is that the coarse grained structure of the material makes it difficult to measure internal flaws and cracks by using UT. Typical measurement results with conventional UT will be full of scatter and background noise, see for instance the work from 2011 by Lundberg et al^{37, 38}. It is thus a big challenge to

³⁵ J. Lundberg, M. Rantatalo, C. Wanhainen, J. Casselgren, "Measurements of friction coefficients between rails lubricated with a friction modifier and the wheels of an IORE locomotive during real working conditions", *Wear* 324-325, pp 109–117, (2015)

³⁶ Smith, R.W., DeMonte, T.N., Mackay, W.B.F., Development of High-Manganese Steels for Heavy Duty Cast-To-Shape Applications, *Journal of Materials Processing Technology* 153-155, 2004

³⁷ Lundberg, J., Bohlin, A., Syk, M., Blindfold Tests on Manganese Crossing In Railway Application, *International Journal of System Assurance Engineering and Management*, Vol 73, 2011

³⁸ Lundberg, J., Bohlin, A., Syk, M., Capacity Test of ultrasonic equipment Used for Crack Detection in Railway Application, *International Journal of System Assurance Engineering and Management*, Vol 73, 2011

use conventional UT in this application, since the propagation medium is anisotropic and nonhomogeneous in nature. Some railway infrastructure owners like DB are for this reason not using frogs made of high manganese casting steel at all. Obviously, there is a need of finding non-destructive methods that are capable of measuring the condition of the frogs made of this material. SAFT (Synthetic Aperture Focus Technology) have theoretically a potential to handle coarse grained structure, in order to achieve a better detection of internal flaws. Many researchers have studied SAFT for many applications; however only a few have studied SAFT for measuring on high manganese steel for railway frogs and especially in the combination of Phased Array, which will increase the efficiency of SAFT. The amount of available research results regarding SAFT measurement with or without phased array and with the application on railway high manganese components is very limited. Diaz performed the first significant important contribution in this subject in 2004. et al³⁹. They did SAFT measurement without phased array probes on high manganese steel from a railway switch frog and reported that the tests in the Laboratory of Batelle were successful, demonstrating in principle an effective means to detect and localize anomalies introduced as a function of size and depth from the top of the frog rail. Using non-optimal, commercially available transduced coupled with the low-frequency/SAFT-UT approach, Batelle staff conducted preliminary evaluations of the effects of the material microstructure on ultrasonic propagation, sensitivity and resolution in thick section frog components with side-drilled holes. They also concluded that much work still needs to be conducted in order to generate the specifications for a pre-service inspection system for these types of material. In 2008, another application of SAFT was demonstrated by Vopilkin et al⁴⁰. A mobile system with a scanning device, using acoustical holography with FT-SAFT processing was used for rails inspection, however, the tested material was conventional rail steel with unacceptable flaws (not manganese steel). They report that comparisons between acoustical images and fractographic analyses show good agreement (flaws size and position measurement accuracy is 5–10 %). In 2008 also significant research was performed in Japan. Recently, the EU project SAFT Inspect will be finalized (April 2015) regarding the use of SAFT for crossings, but unfortunately they did not use Phased Array in combination with UT, but some other interesting results were found, such as a method describing the cracks in a defined measurement volume.

In conclusion, more research is needed regarding of how to use Phased Array in combination with UT for high manganese crossings and regarding measurements of the early signs of crack in ordinary rail steel by using measurement devices based on the promising Barkhausen effect.

³⁹ Diaz, A., Andersen, E., Samuel, T., Low Frequency-SAFT Inspection Methodology for Coarse-Grained Steel Rail Components (Manganese Steel Frogs), Proceedings of the 16 th Conference on Non Destructive testing, Canadian Institute for Non Destructive Evaluation, Hamilton, ON, Canada, 2004

⁴⁰ Vopilkin, A., Kh., Kononov, D., A., Tikhonov, S., D., Acoustical Holography Implementation for Defected Rails Inspection, 17 th world Conference on Nondestructive Testing, Shanghai, China, 25-28 Oct, 2008

5.6 WELDING

Welding is a necessary method to join rails. There are different methods to weld rails, among which flash-butt and thermite welding are two common techniques.

A lot of work was carried out in the European research project INNOTRACK SP4 and the result is possible to use. Also the open points are now addressed and will be further investigated in the European research projects In2Rail and Wrist.

In short the main challenge with welding is to join the rail ends while causing as little disturbances to the rail as possible. This includes obtaining a narrow heat-affected zone, minimizing hardness variations over the weld, achieving uniform wear resistance. Failure of obtaining any of these objectives may result in excessive dynamic loads. In addition, a weld will normally introduce a detrimental state of residual stress. Together with material defects that may be results of poor weld quality this will also increase the propensity for crack formation.

For these reasons welds are often over-represented in damage levels. This situation is not likely to improve with current demands for faster and more cost-efficient welding procedures. The latter includes repair welds covering large portions of the railhead to avoid rail replacement.

5.7 STRATEGIES FOR MAINTENANCE

As a general rule, maintenance strategies will differ for different operational situations. This includes all operational variables, such as type and status of track, type and status of vehicles, operational patterns etc.

The formulation and implementation of courses of action to achieve an intended set of maintenance objectives is a key aspect in developing guidelines for sustainable and competitive railway transport. An effective strategy addresses cost, risk and performance factors by making optimal trade-off between them⁴¹. In the presence of adequate information, simulation of possible scenarios is possible in a cyclic process that continues until a suitable balance with acceptable trade-off is reached.

The present guiding principle in the formulation of maintenance strategies among infrastructure managers in EU is to minimize corrective maintenance by transforming reactive actions to proactive interventions. This principle also aims at replacing existing cyclic maintenance with condition-based maintenance to a great extent. There is a need to improve the present fundamental principle for strategy formulation such that the following issues are covered:

- balancing cost, risk and technical performance
- transforming reactive maintenance to proactive
- integrating data and experience driven approaches for cyclic maintenance
- refocussing condition monitoring to prognostic and health monitoring

⁴¹ Kumar S, Espling U, Kumar U. Holistic procedure for rail maintenance in Sweden. Proc Inst Mech Eng Pt F: J Rail Rapid Transit 2008; 222(4):331-344.

- adjusting inspection strategy to fit into RAMS based inspection schemes
- motivation of action steps at the strategic, tactical and operational level

These overall strategic priorities will serve as a guideline for low-level strategy to be used at the tactical and operational level where each maintenance and renewal task is addressed. This overall strategy can be cascaded from network level to maintainable item level, including manual works (small-scale such as surface welding and fastener re-tensioning) and mechanical works (large scale tasks such as grinding and tamping). In essence, an effective strategy aims to balance different maintenance actions, policies and concepts as shown in figure 5.1.

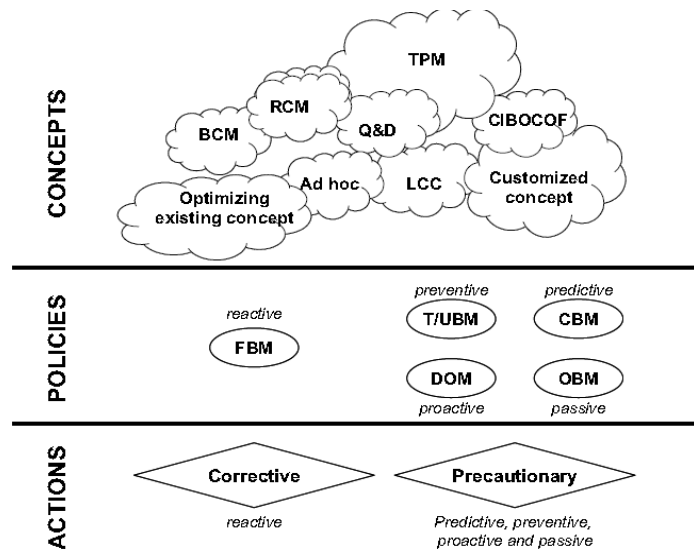


Figure 5.1: Actions, policies and concepts in maintenance strategy⁴²

5.8 AUTOMATION OF MAINTENANCE

The operational maintenance of track can be more or less automated by the adoption of machines that are autonomous to a lower or higher degree. Difficulties and suitability for automation varies between applications: In the case of switch failure occurrences (e.g. track geometry failure over short length of track, surface welding and switches), manual maintenance, supported by small machines, is usually performed and (so far) beneficial. However, for other maintenance actions (e.g. tamping, ballast stabilizing and grinding) maintenance actions are becoming more automated.

Because of decreased deterioration, the required intervention interval increases due to greater mean time between failures. This will result in that the track availability will be higher and cause less traffic disruption. Nevertheless, on the other hand, it requires extra planning time due to some factors such as logistic and allocation of maintenance time to minimize the effect on the track availability.

However, it is not only the operational maintenance that is being automated. As data harvesting increases, so does the potential for automated maintenance decisions and prioritization. This requires relevant measurement data that can be stored in a fairly compact

⁴² Murthy DNP, Kobbacy KAH. Complex system maintenance handbook. London: Springer; 2008.

form and can be translated to maintenance decisions. An example⁴³ of this trend is an automated categorization of wheel profiles based on their impact on wear and rolling contact fatigue formation. This automated procedure parameterizes (e.g. laser) measured wheel profiles, which allows for compact storage. The parameterized are then evaluated with respect to their impact on wear and rolling contact fatigue formation using meta-models. Such an automated analysis can for example be used to derive a prioritization list for wheel reprofiling.

By means of the new automated measurement system and development of decision support tools, future track degradation and potential failures can be predicted in advance. This provides a time horizon for planning for logistics and for allocating a maintenance possession time prior to failure.

Through optimizing maintenance processes such as tamping and grinding, in terms of reducing the required maintenance possession time, the capacity of the European rail network can be increased to allow increased freight traffic. For this purpose, Key Performance Indicators (KPIs) and categories of performance killers/drivers for RAMS and LCC need to be identified, developed and transformed into a railway user environment for adaption and implementation⁴⁴.

5.9 USE OF NEW INNOVATIVE MAINTENANCE CONCEPTS

An effective guideline for best maintenance practice requires the application of innovative maintenance concepts to enhance the performance of railways. Rule of the thumb and conventional maintenance concepts are almost reaching their performance threshold due to emerging technologies and operational demands, boundary conditions and defined targets. Innovative decision structures or logic for exploring maintenance and renewal tasks and scenarios before taking decisions are therefore required for managing a high performing railway infrastructure. These concepts include:

- **Streamlined reliability centered maintenance:** This involves the adaptation of a reliability centered maintenance approach for a linear infrastructure with long life span such as railways. This concept includes additional features that will overcome the complexity of the conventional approach and also support the use of computerized maintenance management system to achieve a more efficient maintenance decision structure and logic.
- **e-maintenance concepts:** The concept of applying Information and Communication Technologies (ICT) as an enabler in maintenance management process is a promising future solution of infrastructure maintenance. This concept uses ICT “e-technologies” for such as e-monitoring, e-diagnosis, e-prognosis to support maintenance decision

⁴³ Kalle Karttunen, Elena Kabo & Anders Ekberg, Numerical assessment of the influence of worn wheel tread geometry on rail and wheel deterioration, *Wear*, vol 317, no 1–2, pp 77–91, 2014

⁴⁴ AUTOMAIN (2013) Optimised maintenance activities like, grinding, tamping and other maintenance processes, Project Reference no. 265722, Deliverable D4.2.

and actions⁴⁵. This concept is the required solution to support implementation of real time, remote, prognostic and collaborative maintenance.

- **RAMS and LCC based maintenance concepts:** The integration of LCC and RAMS concepts as philosophies in maintenance practices will help to fine-tune maintenance decisions. These concepts will help to improve infrastructure uptime, maintain infrastructure integrity, and assure safe working conditions and environment, with limited maintenance budgets. Seamless integration of these two concepts as provided in European standards^{46,47} will give some innovative perspectives that will guide effective maintenance practice.
- **RAM4S:** Traditional practice of RAMS (Reliability, Availability, Maintainability and Safety) is a characteristic of a system's long term operation and can be characterized as a qualitative and quantitative indicator of system performance and technical integrity for the fulfilment of Quality of Service (QoS) requirements in a life cycle perspective. These RAMS characteristics for a railway system can describe the confidence with which it can guarantee the achievement of a defined level of rail traffic in a given time. Therefore, cost effective environment friendly design, deployment and use of railway infrastructure can be realized through development and application of integrated RAM4S (Reliability, Availability, Maintainability, Safety, Security, Sustainability and Supportability) models, technologies and methodologies needed for smart/Intelligent use of resources, for operation and maintenance of infrastructure in a sustainable way. Safety, security and availability are inter-linked in the sense that a weakness in either, or mismanagement of conflicts between safety, security and availability requirements, may prevent the achievement of a dependable and robust system. RAM4S models, technologies and methodologies will facilitate development of integrated asset life cycle management models considering or taking into account RAM characteristics of components, systems and Systems of Systems (SoS) thinking and types of support needed to fulfil the requirement of stakeholders.
- **Bridge concept:** Another innovative and emerging trend in maintenance is the idea of “cherry picking” or building bridge between useful aspects of commonly used maintenance concepts and philosophies. For example value driven maintenance (VDM), risk based centered maintenance (RBCM), business centered maintenance (BCM), lean-RCM and constrain based RCM⁴⁸.

45 Karim R. A service-oriented approach to e-maintenance of complex technical systems. Luleå: Luleå tekniska universitet, 2008. 91 s. (Doctoral thesis / Luleå University of Technology; Tidskriftsnummer 2008:58).

46 EN 50126. Railway applications - The specification and demonstration of Reliability, Availability, Maintainability and Safety (RAMS). European Standard 1999

47 EN 60300-3-11. Dependability Management Part 3-11: Application guide- Reliability Centered Maintenance. European Standard 1999

48 Murthy DNP, Kobbacy KAH. Complex system maintenance handbook. London: Springer; 2008.

Focus should be on developing and adapting methodologies, technologies and models to decide on the RAM4S requirements of railway infrastructure based on the capacity requirements and

1. to identify factors that act as bottlenecks to achieve required QoS levels for infrastructure, e.g. regarding capacity
2. to link these factors to technical functions and items of the infrastructure to define the critical functions and items to be addressed
3. to link the critical functions and items to system requirements
4. to define engineering configuration of the technical system that will meet RAM4S requirements
5. to define an integrated asset life cycle management model based on systems engineering practices and that supports maintenance decisions based on RAM4S principles

6 CONCLUSIONS

Section 4.1.1 was dedicated to the assessment of substructure conditions in order to identify current state and possible improving needs. The interpretation and comparison of results obtained by different measurement techniques is critical for the recommendation of suitable repair and/or reinforcement solutions for the railway substructure. Several techniques including rolling stiffness measurements, various geophysical methods and geomechanical investigation procedures which allow obtaining an extensive knowledge regarding the substructure condition were discussed.

In section 4.1.2 the existing methods for subgrade improvement are presented. There are factors other than technical that influence in the decision about the proper method in each case, such as the allowance of traffic during the repair, the type of soil suitable for the technique, the effectiveness in increasing each stability aspect and the experience in the use of this particular technique. All the explored improvement methods are assessed against these criteria.

In section 4.1.3 (ground vibrations) some key results on reducing ground bourn vibrations from the concluded EU-project RIVAS were presented. These included reductions in the generation of vibrations where wheel flats and track point sources were the main noise generators. For vehicles especially the unsprung mass is important. Several methods for track based mitigations and transmission reductions have been investigated in the project. Some results were briefly introduced and references to additional information – in particular to guidelines from the RIVAS project – were given.

In section 4.2 (structures) the impact of different traffic situations is first given for different types of structures. For bridges the highest impact comes from increased axle loads and meter loads. Longer trains and increased trail weight have some impact whereas higher speeds on freight trains has little impact and increased loading gauges usually have no impact. For tunnels the highest impact comes from increased loading gauges, whereas other changes usually have no impact. For culverts the greatest impacts are usually the same as for bridges. For common retaining walls the traffic situation has little or no impact.

Differences in the safety concept in codes for new and existing structures are outlined. Many of the uncertainties that are present when building a new structure, requiring a high safety factor, can be resolved when you are dealing with an existing structure, where actual loads, geometry and material properties can be determined with a high accuracy.

An assessment of an existing structure is recommended to be carried out in three phases: Initial, Intermediate and Enhanced. The initial first phase may include a site visit, study of documentation and a simple calculation. The intermediate second phase may include material investigations, detailed calculations and/or analyses and further inspections and monitoring. The enhanced third phase usually consists of refined calculations and analyses including statistical modelling and reliability based assessments, laboratory examinations and/or field testing and economic decision analysis. A good assessment is often the most cost

effective way to upgrade a structure. Examples are given on how assessments indicating a high hidden capacity have been confirmed by full-scale tests to failure.

For different kinds of bridges, specific areas for upgrading/repair is indicated. Examples are also given on methods to use. These include refined calculations, increased cross sections, change of static system, external prestressing and use of externally bonded fibre reinforced polymers (FRP).

For tunnels different cross sections and typical loading gauges are presented as well as possibilities for working in tunnels with rail service in operation. Examples are also given on the conditions in countries like Germany, Austria, Switzerland, Sweden and the U.S.

For culverts different types of damage and malfunctions are described and examples are given on measures that can be taken. For retaining walls a few examples are given on how they can be constructed.

In section 4.3 (track and switches & crossings) consequences of upgraded operational conditions were investigated. An overview of different deterioration phenomena, and descriptions of potential consequences of different types of upgrading was presented. A three-level approach to successively refined investigations was then outlined and exemplified. In the third level detailed analyses are performed. For upgrading actions that tend to increase deterioration, phenomena affecting different parts of the track structure are presented together with means for predictions of potential consequences. This is accompanied with possible actions to reduce the consequences. It is found that most of the detrimental consequences can be predicted and mitigated. However, this requires sophisticated analyses and correct measures. Also upgrading consequences that do not mainly involve increased deterioration are investigated. In particular, the case of increased operational frequencies is discussed and it is demonstrated how such an upgrade increases the need for high-precision monitoring, prediction and maintenance.

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