

Master's thesis

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# Track geometry degradation cause identification and trend analysis

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Norwegian University of  
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## PREFACE

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This master's thesis has been written as the final part of the Experience-based master's degree programme in Railway Engineering, at the Department of Civil and Environmental Engineering, NTNU. This thesis is a research assignment with a workload corresponding to 30 credits.

The main purpose of this research is to present decision makers how to get more benefit from track geometry measurement data. Root cause identification of track degradation, effectiveness of tamping actions and analysis of patterns of track degradation over time are some of the potential use of the measurement data.

First, I would like to express my sincere gratitude to my supervisor, Professor Elias Kassa, for his guidance and motivation. I also would like to thank my co-supervisor, Hallstein Gåsemyr at Bane NOR, for the constant support, enthusiasm, encouragement and for generously sharing his immense knowledge.

Very special thanks to Terje Vasset at Bane NOR for his help and interesting discussions.

Finally, I would like to express my gratitude to my husband, Håvard, and our sons Victor and Christian for their understanding that our free time together was less frequent than usual. I dedicate this thesis to them.

Adriana Fiedler Vikesland

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## ABSTRACT

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The Norwegian railway network is under pressure, following the worldwide trend towards heavier axle loads, increased speeds and greater traffic density. These factors will contribute to a more rapid degradation of the railway track, which in turn, lead to reduction in service life of track components and higher maintenance costs.

The track degradation has three different aspects: the sub-structural, the super-structural and the geometrical. As the track geometry begins to deteriorate, higher dynamic wheel-rail contact forces are being induced, resulting in wear on the various components of the track construction, plastic deformation and Rolling Contact Fatigue (RCF) of the rails.

Although safety is the most important reason why the track geometry is maintained, appropriate maintenance is also a precondition for the ride comfort. Track geometry faults cause poor ride quality and have a negative train performance impact. In extreme cases, these faults can lead to derailments. As a result, line closures and loss of public confidence are some of the potential consequences.

The track geometry is measured on the Norwegian railway network by a dedicated Track Recording Vehicle (TRV). Different alert limits are generated when predefined threshold levels are exceeded, imposing inspections, repair or corrective maintenance. However, a broad consensus at Bane NOR is that the data being collected are used to a very limited extent for further analysis.

A case study for the Norwegian heavy haul line was carried out to propose how decision makers can take more advantage of numerical data from track geometry measurements. Degradations trends and effectiveness of tamping were analysed.

Infrastructure Managers (IMs) need a better understanding of the infrastructure behavior over time and a greater control over the efficiency of its maintenance. A successful predictive maintenance strategy relies on the ability to observe track behaviour in the past and predict behaviour in the future, as well as the remaining service life of an asset. It means cost saving through planning of required maintenance aspects and applying for track possession time.

## SAMMENDRAG

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Det norske jernbanenettet er under press som følge av den globale utviklingen mot høyere aksellaster, økte hastigheter og større trafikk tetthet. Disse faktorene vil medvirke til raskere nedbrytning av overbygningen som i sin tur vil redusere levetiden for sporets komponenter og gi høyere vedlikeholdskostnader.

Nedbrytning av skinnegangen har tre ulike aspekter: underbygning, overbygning og geometri. Etter hvert som sporets geometri forringes, øker det dynamiske kontaktrykket mellom hjul og skinne, noe som fører til slitasje på de ulike komponentene i sporkonstruksjonen, plastisk deformasjon og rullende kontaktutmatting (RCF) av skinnene.

Selv om sikkerhet er den viktigste årsaken til at sporgeometrien vedlikeholdes, er korrekt vedlikehold også en forutsetning for komforten om bord. Feil i sporgeometrien kan gi ubehagelig gange og påvirke togets ytelse negativt. I ekstreme tilfeller kan slike feil føre til avsporing. Stenging av strekninger og tap av omdømme er noen av de potensielle konsekvensene.

På det norske jernbanenettet kontrolleres sporgeometrien med en egen målevogn. Ulike varsler utløses ved overskridelse av forhåndsdefinerte grenseverdier for inspeksjon, reparasjon eller feilretting. Det er imidlertid bred enighet i Bane NOR om at dataene som samles inn, brukes til videre analyse bare i svært begrenset grad.

En case-studie for den norske godslinjen ble gjennomført for å finne ut hvordan beslutningstakerne kan få større utbytte av numeriske data fra sporgeometrimålinger. Nedbrytningstrender og effektiviteten av pakking av sporet ble analysert.

Infrastrukturforvaltere trenger en bedre forståelse av infrastrukturens egenskaper over tid og bedre kontroll over vedlikeholdets effektivitet. En vellykket prediktiv vedlikeholdsstrategi er avhengig av at det er mulig å observere skinnegangen over tid og dermed kunne forutse både hvordan den vil utvikle seg fremover, og hvor lang levetid som gjenstår. Dette gir kostnadsbesparelser ved at nødvendige vedlikeholdsoppgaver og spordisponering kan planlegges bedre.





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## DEFINITIONS

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<b>Alert Limit</b>	Refers to the value which, if exceed, requires that track geometry condition is analysed and considered in the regularly planned maintenance operations
<b>Bane NOR</b>	A state-owned company responsible for the Norwegian national railway infrastructure
<b>Continuous welded rails</b>	Rails that are welded together to become long continuous track
<b>Corrective maintenance</b>	Maintenance performed after a failure has occurred
<b>Critical rail temperature</b>	The maximum rail temperature before measures to protect traffic should be taken
<b>Deterioration</b>	The process of declined condition
<b>Immediate Action Limit</b>	Refers to the value which, if exceed, requires imposing speed restrictions or immediate correction of track geometry
<b>Intervention Limit</b>	Refers to the value which, if exceeded, requires corrective maintenance before the immediate action limit is reached
<b>Jernbaneverket</b>	The former agency for the Norwegian National Rail Administration
<b>Main tracks</b>	Tracks between two station borders
<b>Pandrol</b>	A manufacturer and type of rail fastenings
<b>Predictive maintenance</b>	Maintenance performed to prevent failures



<b>Rail Neutral Temperature</b>	The temperature at which the rails experience zero stress
<b>Running tracks</b>	Tracks in station areas
<b>Sleeper</b>	A timber or concrete cross-member, supporting the rails of railway tracks
<b>Slow order</b>	A local speed restriction on a rail line which is set below the track's normal speed limit
<b>STRIX</b>	One of the recording cars which is used to measure track geometry quality in Sweden
<b>Superelevation</b>	The inclination of the rails in curves that makes the train tilt towards curve centre
<b>Tamping</b>	A maintenance activity performed to restore track irregularities by correcting the track geometry, by lifting and lining the track, while tamping the ballast

## LIST OF ABBREVIATIONS AND ACRONYMS

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AI	Artificial Intelligence
AL	Alert Limit
ANN	Artificial Neural Network
ARA	American Railway Association
ASCE	American Society of Civil Engineers
CWR	Continuous Welded Rail
D1	Wavelength range: 3 – 25 m
D2	Wavelength range: 25 – 70 m
D3	Wavelength range: 70 – 150 m (vertical) Wavelength range: 70 – 200 m (lateral)
EN	European Norm
GPR	Ground Penetrating Radar
GPS	Global Positioning System
GSM	Global System for Mobile Communication
IAL	Immediate Action Limit
IL	Intervention Limit
IM	Infrastructure Manager
JBV	Jernbaneverket
MDZ	Quality number adopted by TU Graz
MGT	Million Gross Tons (Traffic)
NSB	The Norwegian State Railways
RCF	Rolling Contact Fatigue
RNT	Rail Neutral Temperature
TQI	Track Quality Index
TRC	Track Recording Car

TRV	Track Recording Vehicle
TSI	Technical Specification for Interoperability
TU Graz	Graz University of Technology
UIC	(Union Internationale des Chemis de Fer) International Union of Railways

## LIST OF SYMBOLS

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$h_t$	Cant
$\theta_i$	Angle between the local $y_w$ - $z_w$ plane of the wheel and the radial plane passing through each point in the potential contact area (rad)
$Q_0$	Initial track quality
$R^2$	Goodness of a model
$z_p$	The deviation in z-direction of consecutive running table levels on any rail
$\delta_0$	Initial misalignment amplitude
$\sigma_H$	Standard deviation for longitudinal level
$\sigma_P$	Standard deviation for alignment
$\sigma_R$	Standard deviation for cant
$\sigma_S$	Standard deviation for cooperation
$2L_0$	Misalignment wavelength
$2L$	Buckling wavelength
$b$	Degradation rate
$B$	Basis for the measurement (twist)
$D$	Distance between the nominal wheel-rail contact points
$P$	Longitudinal rail force
$Q(t)$	Track quality at a certain point of time
$\alpha$	Difference between $\gamma$ and $\beta$ , which is converted to cant

- $\beta$  Angle between wagon body floor and wheelset
- $\gamma$  Angle between the wagon body floor and horizontal plane
- $\varphi$  Cant angle



# **1 INTRODUCTION**

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This chapter introduces the master's thesis, starting with a background, followed by the problem statement. Thereafter the research purpose, objectives, research questions and methodology are presented. Finally, the research scope and limitations are declared.

## **1.1 BACKGROUND**

In order to provide increased capacity in terms of both passengers and freight, the rail sector is moving towards higher train speeds, heavier axle loads and greater traffic density.

High-speed railway lines require precise track geometry and deviations from the design must be kept to a minimum. Heavier axle loads and dense traffic result in larger forces acting on wheels and tracks, leading to a more rapid degradation. Those factors demand high maintenance needs and costs.

Infrastructure managers (IMs) have less time to maintain the track. Available track maintenance windows are short, and work must be performed quickly. In addition, there is a considerable maintenance backlog on the Norwegian railway network at present.

Raising competence on track geometry deterioration mechanisms is therefore crucial to strengthen the basis for making decisions on priorities for maintenance and renewal work.

## **1.2 PROBLEM STATEMENT**

The track geometry is affected by several factors including the condition of the superstructure elements (rails, fastening system, rail pads and sleepers), the condition of the substructure (ballast, sub-ballast and subgrade), traffic density, speed, axle load, environment and current maintenance strategies, among others.

Track geometry has been inspected by a self-propelled diagnostic vehicle on the Norwegian railway network. Alert reports are generated when track geometry exceeds predefined threshold levels. However, the data collected are used to a very limited extent for analysis to find root causes of track deterioration and to monitor track behavior over time.

Detection of possible root causes of track degradation, effectiveness of tamping and analysis of patterns of track degradation over time are some of the potential use of the track geometry measurement data.

### **1.3 RESEARCH PURPOSE, OBJECTIVES AND QUESTIONS**

#### **1.3.1 Purpose**

The aim of this study is to suggest how raw data from track geometry measurements can be turned into information as a basis for maintenance decisions.

#### **1.3.2 Objectives**

More specifically, the objectives of this research are:

1. To perform root cause analysis by integrating data from different sources.
2. To find out the effectiveness of tamping.
3. To identify degradation trends by comparing several measurements.

#### **1.3.3 Research questions**

1. How to perform root cause analysis based on the sensor data from the geometry measurements?
2. How effective was the tamping action carried out on a specific track section?



3. What are the patterns of track geometry degradation on the Norwegian Heavy Haul Line?

## **1.4 METHODOLOGY**

The research methodology consisted of a literature study conducted to establish a knowledge base for this master's thesis, data collection, study visits, interviews with representatives from Bane NOR and a case study.

This study makes use of both quantitative and qualitative methods.

### **1.4.1 Literature study**

A comprehensive literature search was conducted using databases, including:

- The NTNU University Library's literature databases;
- Oria: Bane NOR's library. The search is extended to universities and university colleges in Norway;
- Brage: access to full-text versions of material published by Bane NOR and the former agency Jernbaneverket (the Norwegian National Rail Administration). Also contains older material from NSB (the Norwegian State Railways);
- E-books: access to Bane NOR's e-books;
- Railway technology textbooks: the textbooks cover various railway technology disciplines;
- ResearchGate: access to full-text versions of published material;
- Google Scholar;
- Standards, manuals and guidelines produced by Jernbaneverket and Bane NOR.

Internet search engines generated more references than the author was able to read. It was paramount to be selective by concentrating on references that were recommended by the author's supervisors or came from a trustworthy resource.

References that were cited in many other works and/or published in the last years were prioritized.

### **1.4.2 Interviews**

Informal conversational interviews have been held with key persons as a qualitative data collection method. Interviewees were selected based on their expertise or involvement in the decision-making process regarding maintenance at Bane NOR. It was crucial to select interviewees from different regions along the Norwegian railway network, to gain a broad perspective.

The result has indicated that track geometry measurement data is used to a very limited extent for further analysis. Root causes of track degradation have not been thoroughly investigated. It can lead to improper maintenance practices, as repeated tamping actions due to the underlying problems of insufficient drainage and/or bad soil conditions.

One of the identified problems is the lack of powerful computational tools which can help decision makers to take more advantage of the numerical data. InOffice is the software adopted to interpret the raw data on a user-friendly module, however it does not allow modelling track degradation based on consecutive measurements.

Finally, there was a broad consensus among the interviewees that it is crucial to verify the effectiveness of track maintenance. However, methods and tools to follow-up maintenance actions are unknown for most of the interviewees.

### **1.4.3 Data collection**

The following databases were used for data collection:

- BaneData: Bane NOR's system for rail infrastructure database and maintenance activity records;
- InOffice: a software to interpret the raw data from track geometry measurements on a user-friendly module;

- Pictures and video recordings from the external camera mounted on Roger 1000 Track and Overhead Line Recording Car;
- Accumulated tonnage (MGT) records.

The Analysis ToolPak in Microsoft Office Excel was utilized for statistical analysis of the raw measurement data from the Track Recording Vehicle (TRV), and MAPLE was utilized for calculations of the degradation rate.

#### **1.4.4 Study visits**

A site visit guided by a representative from Bane NOR has been conducted during this research in order to increase the author's knowledge on Ofotbanen. The study visit to this section of line has been important to get a general overview of the drainage conditions, since there was limited information available.

Although the substructure is beyond the scope of this thesis, there is a broad consensus among track engineers that polluted ballast, bad subsoil condition and insufficient drainage increases the rate of degradation of track geometry. This is supported by the literature review. The line has apparently good drainage conditions.

The author participated on a track measurement with Roger 1000 (Track and Overhead Line Recording Car) to gain more knowledge about the methods and procedures to measure track geometry.

#### **1.4.5 Case study**

Case study: the Ofoten Line (Norwegian: Ofotbanen) is a heavy-haul line with 30 tons maximum axle load and more than 30 million gross tons of traffic per annum. It results in a fast deterioration of the track, leading to high maintenance needs and costs.

Root cause analysis was performed by correlating information from many sources: track geometry measurements, track layouts, traffic loads and work history.

By utilizing data from the track geometry collected on Ofofbanen over 3 years, along with historical maintenance data during this period, track geometry degradation trends have been analysed. Track quality is given by the average standard deviation for longitudinal level for the left and right rails, with wavelength domain in the range of 3 – 25 m. This study analyzes trends for track geometry degradation for this specific line, as a function of traffic (MGT), and as a function of time.

## **1.5 RESEARCH SCOPE AND LIMITATIONS**

The track deterioration from geometric aspects, its influencing factors and degradation models have been studied.

Although track gauge, longitudinal level, alignment, cross level and twist are the principal track geometry parameters, the case study focuses on longitudinal level. This is the parameters which will be considered to assess the track quality and is often used for triggering preventive tamping.

Only track geometry irregularities with wavelength domain in the range of 3 – 25 m will be considered.

This master's thesis will focus on maintenance operations with tamping.

## **1.6 THESIS STRUCTURE**

This master's thesis is divided into 6 chapters, which are structured as follows:

**Chapter 1** introduces the master's thesis, starting with a background, followed by the problem statement. Thereafter the research purpose, objectives, research questions and methodology are presented. Finally, the research scope and limitations are declared.

**Chapter 2** provides the theoretical foundations on the basis of which this master's thesis is constructed.

**Chapter 3** presents a view of current methods and tools used to assess track geometry quality on the Norwegian railway network.

**Chapter 4** presents the case study carried out in this master's thesis and the discussion of the results.

**Chapter 5** provides the final conclusions of the research carried out in this master's thesis and proposals for future work.

**Chapter 6** presents a list of references.

## 2 LITERATURE REVIEW

---

The aim of this chapter is to provide an overview of the ballasted railway track structure, followed by track geometry issues.

### 2.1 OVERVIEW OF RAILWAY TRACK STRUCTURE

The ballasted railway track system is divided into two sections: superstructure and substructure. The superstructure consists of rails, fastening system, rail pads and sleepers. The substructure consists of ballast, sub-ballast and subgrade.

The ballast is used to provide stability, resilience and load distribution for the track superstructure. Further, it should allow for drainage and provide alleviation to frost, as well as easy adjustment of track geometry (Nielsen and Li, 2018).

The sub-ballast gives a solid support for the top ballast and reduces the seepage of water from the underlying ground. This layer is consisted of small crushed stones (Solomon, 2001).

The subgrade is particularly important in ensuring that the track quality reaches the standard necessary for the safe and comfortable operation of trains (Profillidis, 2006). Providing a stable foundation for the sub-ballast and ballast layers is the main function of the subgrade.

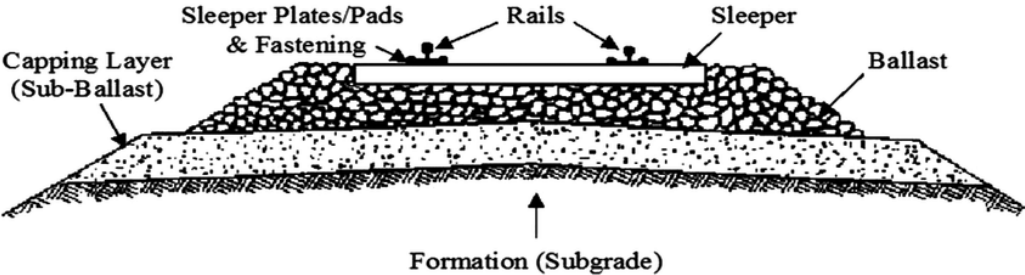


Figure 2-1 Cross-section of a railway track illustrating the main components (Hawari, 2018)

Rail is one of the most important and valuable components of the track structure. Figure 2-2 illustrates the terminology used for the common regions in rails:

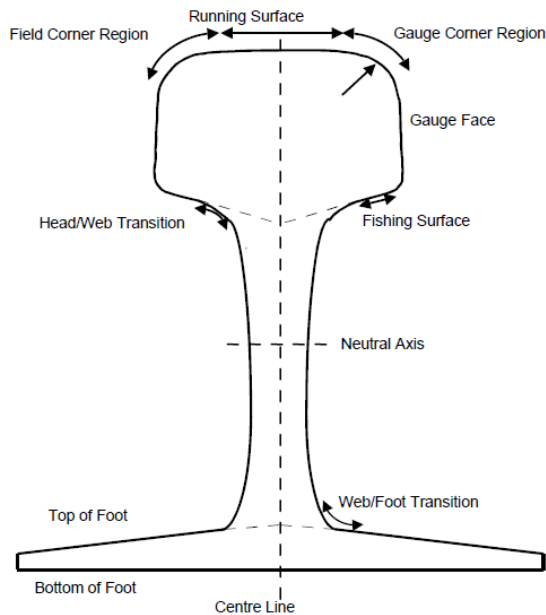


Figure 2-2 Terminology used for the common regions in rails (RailCorp Network, 2012)

Many standards are used for rail profiles, which includes:

- ASCE (American Society of Civil Engineers) standards: e.g. ASCE60, ASCE85
- ARA (American Railway Association) standards: e.g. ARA-A, 100ARA-B
- UIC (International Union of Railways) standards: e.g. 54E1 (UIC54), 54E3, 60E1

Other standards for rail profiles include the British standards (BS 80A, BS 90A, BS 100A), the Australian standards (D1, D2), the Chinese standards and the Indian standards.

In Norway, 54E3, 60E1 and 60E2 are the rail profiles to be used in new constructions (Bane NOR, 2018).

The rail fastening system connects the steel rails and sleepers together, preventing the horizontal and vertical movement of the rails.

Rail pads are resilient components installed on rail seats between the rail and sleeper in order to attenuate the impact loads and moderate track stiffness at the special locations (Ngamkhanong et al., 2018).

Sleepers are essentially beams that span across and tie together the two rails (Tzanakakis, 2013). Some of the main functions of the sleepers are to hold the rails to correct gauge, to maintain the alignment of the track and to transfer the load safely to the subgrade.

## **2.2 TRACK GEOMETRY PARAMETERS**

According to EN 13848-1 (European Standard, 2008), the principal track geometry parameters are:

- Track gauge
- Longitudinal level
- Alignment
- Cross level
- Twist

### ***Track Gauge***

Track gauge is the distance between the gauge faces of the two adjacent running rails (Al-Douri et al., 2016). In Norway, nominal track gauge is set at 1435 millimeters and is measured 14 millimeters down from the rail head (Bane NOR, 2018).

Tight or wide gauge are both a symptom of degradation and an indicator of the state of the track, and therefore causes further degradation of the track.

Changing the distance between the two rails usually modifies the position of the wheel-rail contact. Tight or wide gauge will adversely affect the ride of trains.



Especially in switches and crossings (S&C), variations in gauge trigger rough riding conditions (Civil Engineering Conference, 1999).

Tight gauge in tangent track promotes gauge corner contact, trucks hunting (the propensity for the bogie to oscillate from side to side on straight track) and RCF. At the nominal gauge, more of contacts will be carried towards the crown of the rail where contact conditions are usually less severe. In curves, controlling wide gauge is essential for mitigating low rail damage associated with hollow wheels. Wide gauge curves are also more susceptible to dynamic rail rotation, which often contributes to unfavourable contact geometry (Magel et al., 2004).

Correct track gauge extends the life of track components and train wheelsets, since the forces involved are minimized (Civil Engineering Conference, 1999).

According to (Wolf, 2015), the main cause of wide gauge is the excessive lateral pressure against the rail due to:

- Incorrect curve elevation: insufficient elevation causing pressure of high rail or excessive elevation causing pressure on low rail
- Pre-existing wide gage allowing greater wheelset angle of attack
- Horizontal alignment kinks causing increase in flanging force
- Lack of rail lubrication, particularly top of rail (extremely dry)
- Poor steering due to poor wheel-rail contact geometry

Various track conditions precipitate widening of the gage:

- Deformation of wood fibers holding spikes
- Broken screw spikes or cut spike fasteners
- Worn shoulders on tie plates
- Gage face wear on the rail
- Differential tie plate cutting
- Loose fasteners
- Poor wheel contact geometry toward field side of rail head
- Hollow worn wheels contacting field side of rail head

## ***Longitudinal Level***

According to EN 13848-1 (European Standard, 2008), longitudinal level is defined as the deviation  $Z_p'$  in z-direction of consecutive running table levels on any rail, expressed as an excursion from the mean vertical position (reference line), and it is calculated from successive measurements. Figure 2-3 illustrates the running table (1) and the reference line (2).



*Figure 2-3 Longitudinal level (European Standard, 2008)*

Longitudinal level is the geometrical parameter that most influences vehicles and track dynamics in the vertical direction (Vale et al., 2011), and it is measured separately for both rails of the track. This is the parameter that is most associated with substructure condition (Berggren, 2005).

The European Standard EN 13848-2 (European Standard, 2006) specifies that longitudinal level measurements shall either be made using an inertial system or by a versine system, or by a combination of both methods. It is also stipulated that the measurements should be performed under loaded condition. One important benefit of using a dedicated Track Recording Vehicle (TRV) or hauled Track Recording Car (TRC) is that the measured longitudinal level is a combination of contributions from irregularities in track geometry and track stiffness (Nielsen et al., 2013).

## ***Cant***

Cant (also referred as superelevation) is the height of the vertical side of the right-angled triangle related to nominal track gauge plus the width of the rail head (Al-Douri et al., 2016).

On curves, positive cant indicates that the outer rail is raised above the inner rail. Negative cant may be required in the diverging track near (or inside) canted turnouts. There are no sign rules for cant on a straight track.

The cant angle ( $\varphi$ ) can be determined by:

$$\varphi = \text{asin} \frac{h_t}{D} \quad (1)$$

where,

$D = 1.500$  m on standard track gauge;

$h_t = \text{cant}$ .

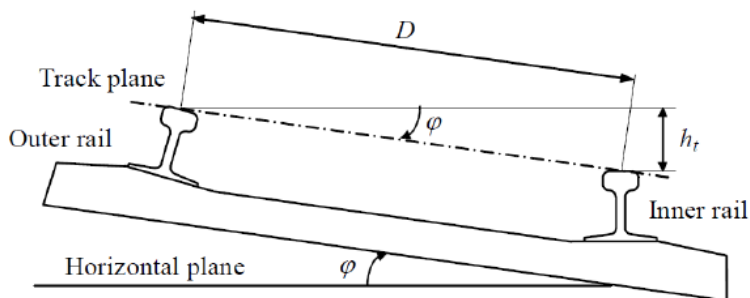


Figure 2-4 Cant and cant angle, adapted from (Pombo and Ambrósio, 2003)

## *Twist*

Twist is the term used to describe the variation in actual track cross level, i.e. the difference in level of the two rails, over a defined length.

Cant change in the transition of a curve is an example of a design twist and will have a maximum gradient of 1 in 400. Correctly maintained cant gradient is an acceptable form of twist that forms an essential part of the design of a railway curve. A twist fault is a condition where there is a difference in cross-levels between rails over a short distance (Civil Engineering Conference, 2002).

In Norway, a Track Recording Vehicle (TRV) measures dynamic twist, when the track is loaded, using two criteria: short twist is measured over 2 meters and long twist is measured over 9 meters. It corresponds roughly with the length of a bogie inner wheelbase and the length of a wagon inner wheelbase, respectively.

The twist is calculated according to the formula:

$$twist = \frac{(h2 - h1)}{B} \quad (2)$$

(‰ or mm / m)

where,

$h1$  and  $h2$  = cants;

$B$  = the basis for measurement.

Twist faults may cause unloading of one or more of the wheels causing them to lose contact with the rail. Once contact with the running surface is lost, the wheel can flange climb and derail the vehicle. Good consolidation of ballast during relaying activities is crucial to avoid twist faults, after the line has been opened to traffic (Civil Engineering Conference, 2002).

Figure 2-5 illustrates how flange climb derailment may occur.

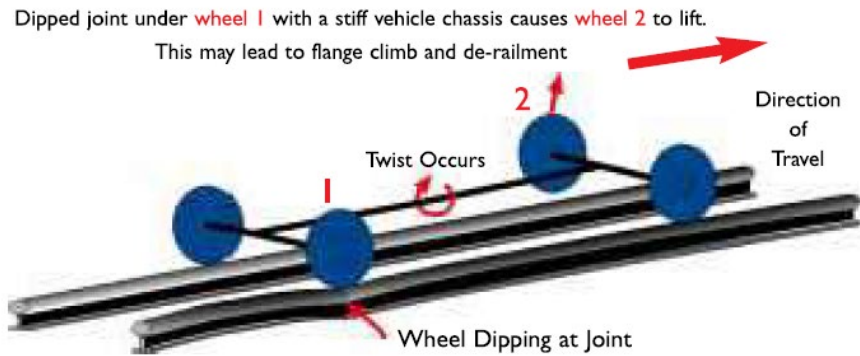


Figure 2-5 Figure showing how flange climb derailment may occur (Civil Engineering Conference, 2002)

### ***Alignment***

Alignment is the mean horizontal position covering the wavelength ranges stipulated, then calculated from successive measurements (Al-Douri et al., 2016).

### ***Cross Level***

Cross level is the difference in elevation between the top surfaces of the two rails at any point of railroad track (Attoh-Okine, 2017).

## **2.3 TRACK IRREGULARITIES**

Track irregularities are deviations from the design track geometry, generally within 1-200 m wavelength range.

Measurement of track geometry irregularities is the most automated condition monitoring technique in railway infrastructure maintenance. Most problems with the track are revealed as track geometry irregularities (Berggren, 2005).

For the detection of track geometry defects, measurements are traditionally assured using a dedicated track recording vehicle (TRV) or hauled track recording car (TRC) running around the rail network gathering track geometry data (Weston et al., 2015).

The amplitude and wavelength are identified as two major parameters describing track irregularities (Bian et al., 2011). There are three stipulated wavelength ranges for evaluation of track geometry, according to the standard EN 13848 - 1 (European Standard, 2008):

- D1 (3 – 25 m)
- D2 (25 – 70 m)
- D3 (70 – 150 m)

## **2.4 INFLUENCING FACTORS ON TRACK GEOMETRY DEGRADATION**

There are many variables affecting the degradation. Those variables can be divided into three categories: from the rolling stock, the track and the surroundings (Lyngby, 2009).

The degradation process due to the interaction between the track and the rolling stock (engines, bogies and wagons) is affected by the following variables: the annual tonnage passing over the track, speed, axle loads, locomotive traction forces, locomotive braking forces, wagons braking forces and the wheel condition.

According to (Greisen et al., 2009), high traffic volume, heavy axle loads and high train speed can produce large rail bending stresses which contribute to increased track deterioration.

The wheel condition plays also a role in degradation of track geometry. If wheels suffer from geometrical defects (e.g. wheel flats or out-of-roundness), it causes an additional dynamic axle load component (Steenbergen and de Jong, 2015).

### ***The substructure conditions***

Regarding the substructure, variations in layer thickness along the track and moisture content are useful information that allows a correlation between track geometry and substructure conditions. The moisture content indicates if there is a drainage problem.

### ***Ballast section and grading***

Ballast is an important component of the superstructure which provides the elastic support to track and sleepers. The thickness of the ballast and sub ballast layers should meet design criteria. The fine grade content indicates if the fouling limits are obeyed.

According to (Pen and Powrie, 2011), the lateral resistance is important to maintain a high track shift resistance of the railway track. This resistance is provided through ballast-sleeper contact at the sleeper base, sides (crib) and ends (shoulder). The ballast gradation, stone quality and compaction of the ballast are some of the factors that influence the lateral resistance. Calculations suggest that the shoulder extent has a greater impact on the lateral resistance than the height of the ballast shoulder.

### ***Fouled and deteriorated ballast***

Fouled and deteriorated ballast are caused by ballast breakdown and/or infiltration from outside the track (Li et al., 2010). These conditions reduce the strength and stiffness of the track substructure and their extent are difficult to determine by visual inspection. According to (Li and Read, 2013), the use of GPR (Ground Penetrating Radar) technologies has been demonstrated to be capable of inspecting ballast fouling and drainage conditions.

### ***Differential Track settlement***

An important contribution to track geometry degradation is differential track settlement, which corresponds to a space-variant reduction in horizontal level of the

ballast/substructure surface over time. Due to dynamic track loading and variations in support conditions along the track, the resulting differential track settlement leads to irregularities in track geometry (Nielsen and Li, 2018).

Settlement of ballasted track occurs in two phases. Immediately after the track construction, tamping or renewal work has been completed, a rapid settlement will arise until the ballast is consolidated. This first phase has an exponential relationship between degradation and load. The second phase is slower and there is more or less linear relationship between the degradation and load in the beginning. As the track degrades further, there is again an exponential relationship between degradation and load (Lyngby, 2009).

On the Norwegian railway network, depending on rail temperature and sleeper type, the speed must be reduced until the ballast is sufficiently stabilized. The track is considered completely stabilized after 100.000 gross tons (Bane NOR, 2018).

### ***Frost heave***

Frost heaving of soil is due to the development of ice lenses in the soil. Ice lenses form due to capillary rise of water (Li et al., 2002). When ice melts, an excess of water remains, which causes softening or loss of strength of the soil. During this period of thaw softening, severe plastic deformation can occur with resulting rapid loss of track geometry and accelerate damage to track components (Selig and Waters, 1994).

Frost susceptible subsoil, available water and subfreezing temperatures are the three prerequisites for the development of frost heave.

### ***Loss of neutral temperature***

Continuous welded rail (CWR) is laid at a Rail Neutral Temperature (RNT), the temperature at which the rails experience zero stress. In Norway, this temperature is set to 21°C (+/- 3°C) (Jernbaneverket, 2011).

Many factors can affect RNT changes in continuous welded rails: maintenance activities, train operation and environmental conditions (Sluz et al., 1999). The



change of RNT is usually toward a lower number (Thompson, 1991). Thermal buckling can become a problem if the neutral temperature falls too far below its target value. Low RNT results in high compressive forces in rail in hot weather. It must therefore be controlled during rail laying and track maintenance.

**Initial misalignments**

Track condition can be weakened by misalignments present in the track. As rail temperature increases, consequently the compressive force  $P$  increases. This may produce some growth in the initial misalignment. Several experiments and field observations have shown that as rail temperature increase to a maximum critical level, the initial misalignment will increase to  $w_B$ , as shown in Figure 2-6, which is an unstable equilibrium state (Kish and Samavedam, 2013). The track can suddenly buckle out into a new lateral position  $w_c$ , stretching over  $2L$ .

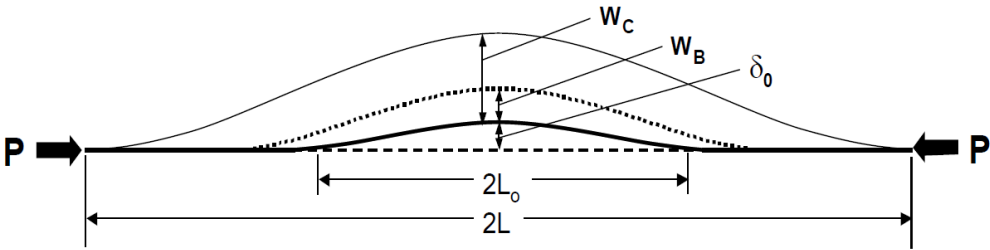


Figure 2-6 Pre-and post-buckled track configurations (Kish and Samavedam, 2013)

**The effects of tamping on degradation**

Maintenance also worsens the general condition of the ballast. The rate of track geometry deterioration tends to increase as the amount of maintenance performed to the ballast increase (Prescott and Andrews, 2013).

Tamping interventions are performed to restore track irregularities by correcting the track geometry by lifting and lining the track. This maintenance activity is carried out by two different methods. Tamping on relative base, which is typically used for shorter sector (less than 1 km) and generally for corrective, non-planned maintenance. The track is brought back to its right position related to the axle of the

tamping machine. The other method is tamping on absolute base, which is typically used for preventive interventions longer than 1 km. The track is brought back to its absolute optimal position, achieving a better track geometry quality (Czichos, 2013).

The ride comfort limits (Figure 2.7) was developed by UIC to define an intervention level for tamping based on the maximum allowable speed on the track and the standard deviation for the short wavelength (3 – 25 m) of longitudinal level defects.

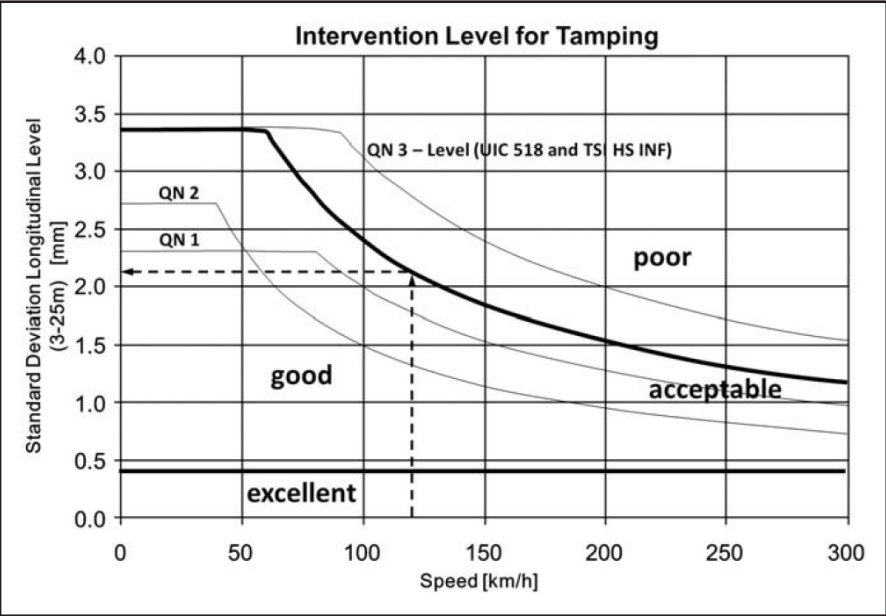


Figure 2-7 Intervention Level for Tamping (UIC - Infrastructure Department, 2008)

### 2.5 TRACK GEOMETRY QUALITY ASSESSMENT METHODS

According to (Sadeghi and Askarinejad, 2007), there are three aspects to track deterioration:

- The sub-structural aspect (i.e., degradation of the track sub-structure)
- The super-structural aspect (i.e., degradation of the track super-structure)
- The track geometrical aspect (i.e., degradation of the track geometry)

Track degradation models considered from the geometrical aspect use geometrical parameters as the main degradation criteria (Berawi et al., 2010). Typically, the

track is divided into several shorter sections and geometry statistics of the main geometry parameters are performed to each of these sections. Statistics are later summed up to give a measure of the overall track segment quality. This assessment technique provides the Track Quality Index (TQI).

Track quality index (TQI) is a numerical value that represents the relative condition of the track surface geometries (El-Sibaie and Zhang, 2004).

The standard deviation for the longitudinal level defects and the standard deviation for horizontal alignment defects are the main quality indicators related to railway track geometry degradation. For many European Infrastructure Managers, the standard deviation for the short wavelength (3 – 25 m) of longitudinal level defects is still recognized as the crucial indicator for planned maintenance actions (Andrade and Teixeira, 2015).

In Norway, combined standard deviation is the method adopted by Bane NOR to assess the track geometry quality. The track is divided into 1000 m sections, and geometry statistics are performed to each section.

Both the standards EN 13848-5 (European Standard, 2017) and EN 14363 (European Standard, 2016) for track geometry quality assessment uses standard deviations and maximum values of alignment and longitudinal level.

New and alternative methods for quality assessment were studied by a project called Dyno TRAIN, which is aimed to promote interoperable rail traffic in Europe by reducing costs of certification and closing “open points” in the TSI’s (TRIO TRAIN, 2013). One of the purposes of this project is to find out which track geometry description method gives the best correlation to the force reaction of typical vehicles. The test covered 7500 km of track and recorded 4,7 terabytes of data. The project was broken down into six technical work packages (WPs). The WP2 concerns track geometry quality.

The work package WP2 has studied a huge number of track quality assessment methods and concluded that the current geometric methods (i.e. standard deviations and maximum values of alignment and longitudinal level) are still superior. Only in a few cases, small improvements were found, e.g. if the wavelength ranges are changed or if more than one track geometry parameter is used -

alignment/longitudinal level with twist or cross level (European Commission, 2014), as indicated by the up arrows in Table 2-1. Up and down arrows simultaneously indicate the methods that show potential improvements, but up to now not for all vehicle assessment parameters.

<b>Alternative methods for track geometry quality assessment</b>	<b>Evaluation</b>
Combination of wavelength ranges D1&D2	↑
Combined standard deviation (EN 13848-6)	↑↓
Combination alignment/longitudinal level with cross level	↑
Combination alignment/longitudinal level with twist (2.5m / 14m)	↑
1. or 2. derivate of measured track geometry	↓
Point Mass Acceleration Method	↓
Wirkungsbezogene Gleislagebewertung	↓
Triangle Method	↓
Mexican hat wavelet	↓
Pupil (Assessment filters modelling the vehicle behavior)	↑↓
FIR filters derived by adaptive filtering	↑↓

Table 2-1 Alternative methods for track geometry quality assessment, adapted from (Haigermoser, 2013)

(Landgraf, 2016) presented an innovative approach to the evaluation of ballast and substructure conditions by fractal analysis of vertical alignment. The so-called Modified Divider Length Method enables to split vertical deflections into three different ranges of wavelengths: short-waved (0 m – 3 m), mid-waved (3 m – 25 m) and long-waved (25 m – 70 m) failures.

It is assumed that the short-waved range covers an error characteristic that describes the sleeper condition, as well as the interaction between sleeper and ballast bed. The medium wavelength dimension ought to be capable of quantifying the ballast condition. Deflections caused by insufficient substructure conditions are more likely to occur within the long-waved range.

Fractal analysis has been carried out to date on the main network of the Austrian Federal Railways, as well as within the network of Suisse Federal Railways (Landgraf and Hansmann, 2018). The methodology has shown good results for a condition evaluation of single track components.

## 2.6 TRACK DEGRADATION MODELS

Different degradation models have been developed to predict the future condition of railway tracks, by considering the influencing parameters. The deterioration models can be classified into four general approaches (Elkhoury et al., 2018).

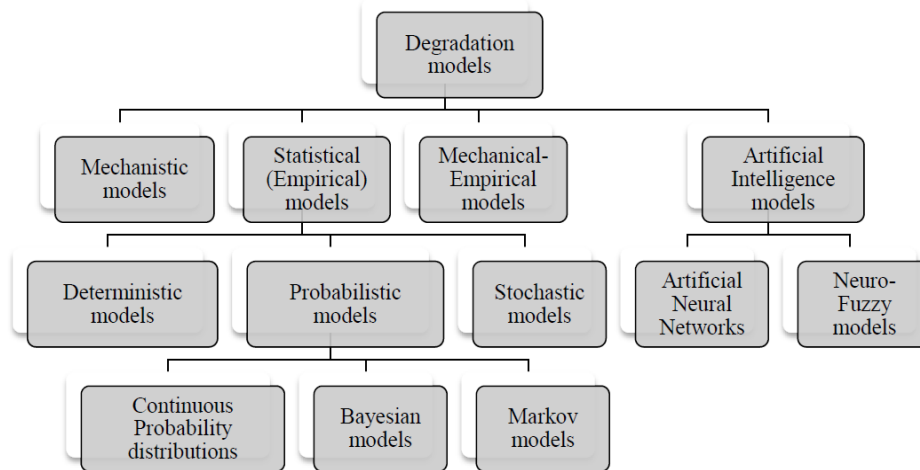


Figure 2-8 Classification of rail degradation models (Elkhoury et al., 2018)

### ***Mechanistic models***

The mechanistic models are considered the primary and traditional models to forecast the level of degradation of railways. This model type is based on mechanical characteristics of track components which result in rail degradation (Falamarzi et al., 2018).

(Elkhoury et al., 2018) highlighted that in general, mechanistic models reflect the actual physical interactions within materials or variables affecting the track structure that cause degradation. These variables may be difficult to quantify. Materials of the rail structure are not homogenous. Besides, this kind of model can be challenging, intensive and time consuming.

A mechanistic degradation model presented by TU Graz to predict railway track degradation applies a quality number named MDZ, which reflects the riding comfort. The MDZ number comprises both horizontal and vertical deviations in track together with speed and lack of superelevation (Hummitzsch, 2004).

According to (Elkhoury et al., 2018) this model analyses the development of track quality from a passenger’s point of view.

Experience indicates that the deterioration rate is lower when the initial quality is high. Nevertheless, comparative evaluation on different track with differing local factors shows wide variation in the rate of deterioration (b) (Veit and Lichtberger, 2007).

The analysis performed by TU Graz shows non-linear quality behaviour over time, caused by increasing dynamic forces due to the growth of track failures which is described by the following exponential function:

$$Q(t) = Q_0 \cdot e^{-b \cdot t} \tag{3}$$

where,

$Q(t)$ : track quality at a certain point of time;

$Q_0$ : initial track quality;

$b$ : deterioration rate over time;

$t$ : time.

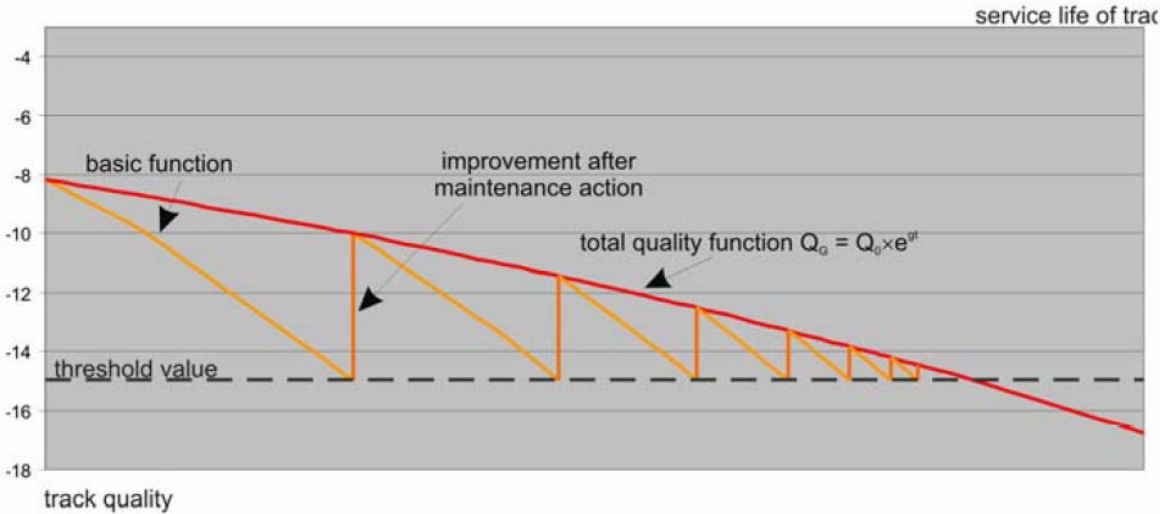


Figure 2-9 The non-linear quality behaviour over time (Veit and Lichtberger, 2007)

The deterioration rate  $b$  itself is a function, influenced by all boundary conditions of track, such as transport volume, superstructure and substructure.

The track quality can be estimated by the deterioration rate under given traffic load, by the loss of alignment parameters and track support modulus with the time under given traffic load. Quality is delivered on one hand by capital investment or renewal, and on the other hand by maintenance effort (Veit and Lichtberger, 2007).

### ***Statistical models***

According to (Elkhoury et al., 2018), statistical models are based on observations of the rail track structure and influencing factors, such as traffic, track components and maintenance variables. These models try to simulate real-life conditions with mathematical equation to predict how tracks will degrade in the future. Statistical models can be classified into deterministic, probabilistic and stochastic.

(M Quiroga and Schnieder, 2012) presented a stochastic model of track degradation using Monte Carlo simulation. The data source is from the French railway operator SNCF. The model assumes that track degradation occurs in two major phases: directly after a tamping activity, until the ballast is consolidated. This phase can be modelled with a lognormal function. The second phase is slower and can be modelled with an exponential function.

(Vale and M. Lurdes, 2013) presented a stochastic model for the geometrical railway track degradation process of the Portuguese railway Northern Line, focusing on the standard deviation for longitudinal level defects. The Dagum distribution which was adopted fitted well the track degradation behavior for the selected line.

(Spooner et al., 2015) have developed a model for the value of  $\sigma_H$  on a 200-meter track section, which accounts for the effect of tamping operations on the track. This model has successfully been implemented by Bane Danmark, together with a Tamping Planning Algorithm (TPA) developed by (Jensen, 2012) for planning preventive tamping.

### ***Mechanical–empirical models***

According to (Ahac and Lakušić, 2017), mechanical-empirical models are based on a combination of mechanistic and empirical modelling. This approach is considered the most effective track degradation modelling approach. These models are based on track segmentation, i.e. the linear rail infrastructure is divided into segments with homogeneous characteristics (traffic load, speed, sleeper type, rail type, among others). Track measurement data per each segment is collected and statistical regression analysis is performed using least square method. The regression model defines the degradation rate of the dependent variable (the observed track quality parameter) as a function of the independent variable (the exploitation period expressed as time or exploitation intensity).

### ***Artificial intelligence models***

Artificial intelligence (AI) is an algorithm, a math model or a software that can improve its own performance with time (Pires, 2018).

Artificial intelligence degradation models have been successfully used in civil engineering to predict degradation. They are becoming prominent among researchers from other disciplines, such as mechanical engineering (Elkhoury et al., 2018).

(Guler, 2014) presented an alternative method for predicting track geometry deterioration using Artificial Neural Networks (ANNs). Models were developed for the main track geometry parameters and produced significant relationships between the variables.



### **3 TRACK GEOMETRY MEASUREMENTS AT BANE NOR**

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Presently, Bane NOR uses a self-propelled diagnostic vehicle to measure geometrical irregularities.

#### **3.1 THE SELF-PROPELLED DIAGNOSTIC VEHICLE ROGER 1000**

Bane NOR (former Jernbaneverket) has developed with MERMEC S.p.A. of Italy a self-propelled track and overhead contact line diagnostic vehicle: Roger 1000. This was an important milestone for Bane NOR in the process of establishing a Recording Service.

Roger 1000 weights 60 tons and has a top speed of 160 km/h when self-propelled (hailed: 200 km/h). It was designed to operate at temperatures ranging from -40 °C to +40 °. This diagnostic vehicle is equipped with ATP (Automatic Train Protection), anti-skid brakes, cruise control and full GPS location equipment. A GSM modem allows remote diagnosis of all operating equipment. Helical spring primary and second suspensions, and active lateral suspension enables Roger 1000 to run through curves in the same condition as a passenger train (MERMEC, 2018).

The task of Roger 1000 is to collect, process and store all types of infrastructure data, and provide relevant and specific information for each user group.

All parameters are sampled at 500 mm intervals and the technique for track recording is based on contactless measurements methods. A high degree of automation enables Bane NOR to operate Roger 1000 with a crew of just two: a driver and a technician. However, it is expected that a representative from the track master is onboard in the track recording vehicle when measurements are being carried out on the respective track line.

Roger 1000 is a travelling laboratory with hardware and software able to monitor and analyze:

- Loaded Track Geometry
- Overhead Contact Line
- Rail Profile
- Integrated Track
- Integrated Overhead

Once these data are recorded, the next steps are pre-processing and full function, including integration with the driving controls. Local processing turns raw sensor data into information. Shortly after the recording, the results of measurements are normally available in a so-called *InOffice*. It is a software to interpret the data on a user-friendly module.



*Figure 3-1 Roger 1000, by Ingunn Halvorsen, 2017*

The measurement campaigns are defined for main tracks and running tracks. Those campaigns are centrally organized for the whole network. On high speed lines, measurements are carried out periodically 6 times a year. Due to the non-contact measurement system, snow in the winter period could cause a problem (Gåsemyr, 2018).

On normal speed lines, measurements are performed twice a year: from the spring, after snow has melted until late autumn, before the snowfalls. On the heavy haul line with 30 tons axle load track recording is presently also performed twice a year.

**3.2 TRACK GEOMETRY QUALITY ASSESSMENT AT BANE NOR**

The track recording system for measuring the track quality is based on eight decisive parameters. The sampling distance is 0,5 m. The following parameters are identified:

- Gauge
- Twist
- Cant
- Longitudinal Level concerning right and left rail
- Alignment concerning right and left rail
- Horizontal curvature

**3.3 DEFINITION OF SPEED QUALITY CLASSES**

Bane NOR defines a speed class regime based on track quality. Depending on the speed, six classes are defined:

Quality class	K0	K1	K2	K3	K4	K5
<b>Speed (km/h)</b>	V > 145	125 < V < 140	105 < V < 120	75 < V < 100	45 < V < 70	V < 40

*Table 3-1 Speed quality class dependency*

### 3.4 TRACK GAUGE MEASUREMENT

The gauge is measured by laser optic measuring system from the laser stations. The laser stations are located:

- In front of the car in driving direction.
- In the middle of the car (located under the car body).
- At the very back end of the car in driving direction.

This laser non-contact technique allows measurements at very high speeds, at very low speeds and without the effects produced by rapid accelerations/retardation of measuring car. Besides, no frequent re-calibration is necessary.

Roger 1000 measures the dynamic gauge, when the track is subject to train loading. It is an advantage when compared to devices that measure static gauge, which is without the influence of trains.

Figure 3-2 illustrates how gauge is measured by laser optic measuring system from the laser stations.

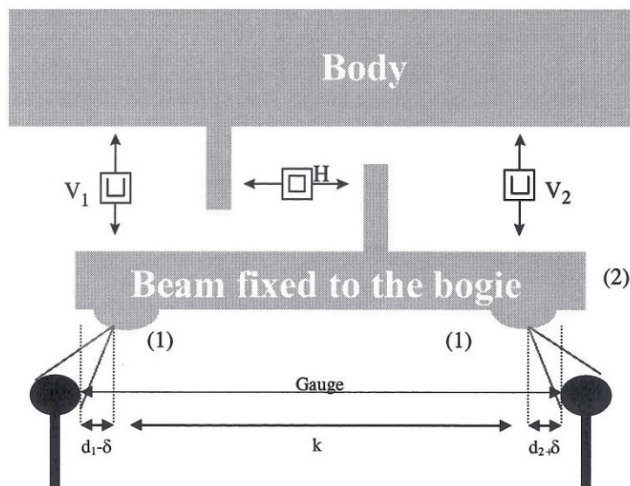


Figure 3-2 Gauge being measured using laser optic measurement system (MERMEC, 2001)

### 3.5 TWIST AND CANT MEASUREMENT

In Roger 1000, an inertial system is used for the measurement of twist and cant. Parts of the laser optic system are also applied in order to measure cant. The twist is then calculated as described in this chapter. In order to measure the cant, two angles are measured. The difference of the two angles is the inclination of the track plane with the horizontal plane (Gåsemyr, 2018).

The principles for measuring cant with the inertial system are presented in Figure 3-3, and consist of:

- Measuring the absolute vehicle roll angle  $\gamma$  between wagon body floor and horizontal plane using an inertial system.
- Measuring the angle  $\beta$  between wagon body floor and wheelset which is parallel with the track plane.
- The angle  $\alpha$  is defined as  $(\gamma - \beta)$ , which is converted to cant due to calculations. Two cants report the twist.

Figure 3-3 illustrates the technique:

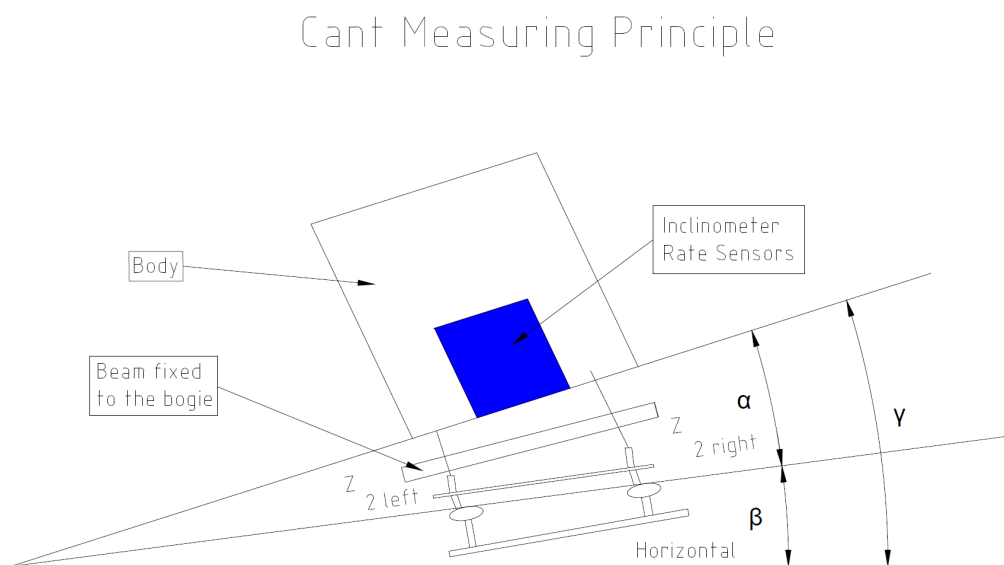


Figure 3-3 Principles for measuring cant with inertial system, adapted from (MERMEC, 2001)

A system consisting of the following equipment is required to record the angle at high speeds:

- An accelerometer, which is a compact device that measures non-gravitational acceleration.
- A gyroscope for measuring the rate of rotation around the vertical axis (yaw).
- A gyroscope for measuring the rate of rotation around the longitudinal axis of the wagon (roll).

The combination of these two devices (accelerometer and gyroscope) provides information on both acceleration and orientation.

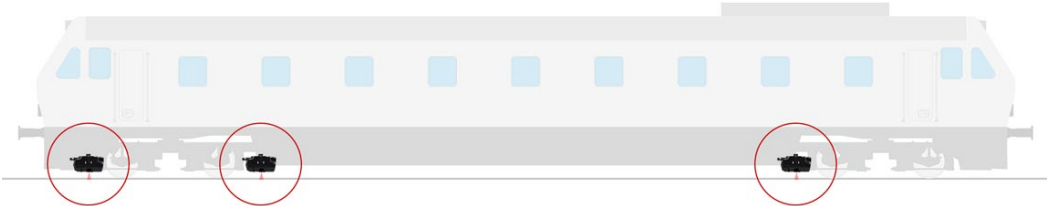


Figure 3-4 Measurement system typical installation (MERMEC, 2018)

A laser system illuminates the rail as video cameras capture full cross-sectional rail profiles. For Bane NOR's Roger 1000, a total of 20 PCs are used in network for processing, presentation and storage on board.



Figure 3-5 Non-contact optical technology (MERMEC, 2018)

### 3.6 MAINTENANCE REGIME ADOPTED BY BANE NOR

The maintenance regime adopted by Bane NOR is based on three limits: alert limits (AL), intervention limits (IL) and immediate action limits (IAL). Those limits depend on the maximum permissible train speed and are standardized in the EN 13848 series (European Standard, 2006).

Table 3-2 shows limit values for track gauge.

Quality classes	Speed (km/h) in accordance with quality class regime	Deviation in the track gauge (mm)			
		New track	AL – Alert Limit	IL – Intervention Limit	IAL – Immediate Action Limit
<b>K0</b>	145 -	+4/-0	+5/-3	+15/-5	+28/-7
<b>K1</b>	125 - 140	+4/-0	+7/-3	+20/-5	+35/-8
<b>K2</b>	105 - 120	+4/-0	+7/-3	+20/-5	+35/-9
<b>K3</b>	75 - 100	+4/-3	+15/-5	+30/-8	+35/-9
<b>K4</b>	45 - 70	+4/-4	+15/-5	+30/-8	+35/-9
<b>K5</b>	- 40	+5/-5	+15/-5	+30/-8	+35/-9

Table 3-2 Permitted gauge deviations from the nominal track gauge 1435 mm

Other limits are applied to Ofofbanen and the Dunderland line (Gullsmédvik – Ørtfjell) with custom rail inclination 1:30 on the rail head, axle load > 25 tons and maximum speed ≤ 50 km / h. The allowed deviation in the track gauge from the base 1435 mm given in Table 3-3.

Quality classes	Speed (km/h) in accordance with quality class regime	Deviation in the track gauge (mm)			
		New track	AL – Alert Limit	IL – Intervention Limit	IAL – Immediate Action Limit
<b>&gt; 25 tons</b>	≤ 50	+4/-3	+15/-5	+20/-8	+35/-9

Table 3-3 Permitted deviation in the track gauge (Ofofbanen and Dunderland line)

The deviation in the track gauge is based on a chord length of 10 m.

Quality classes	Speed (km/h) in accordance with quality class regime	Measured changes in gauge (mm) based on a chord length of 10 m	
		AL – Alert Limit	IL – Intervention Limit
<b>K0</b>	145 -	7	10
<b>K1</b>	125 - 140	8	12
<b>K2</b>	105 - 120	9	15
<b>K3</b>	75 - 100	10	18
<b>K4</b>	45 - 70	12	21
<b>K5</b>	- 40	15	25

Table 3-4 Measured changes in gauge based on a chord length of 10 m

Quality classes	Speed (km/h) in accordance with quality class regime	Unevenness in cant (+/- mm)		
		New tamped track	AL – Alert Limit	IL – Intervention Limit
<b>K0</b>	145 -	2	4	6
<b>K1</b>	125 - 140	2	4	7
<b>K2</b>	105 - 120	2	5	8
<b>K3</b>	75 - 100	3	7	10
<b>K4</b>	45 - 70	4	10	13
<b>K5</b>	- 40	5	12	16

Table 3-5 Unevenness in cant (+/- mm)

Table 3-6 shows limit values for short twist measured over 2 meters.

Quality classes	Speed (km/h) in accordance with quality class regime	Short twist (+/- mm)				
		New tamped track	AL – Alert Limit	IL – Intervention Limit	IAL – Immediate Action Limit	
					R ≥ 400 m	R < 400 m
<b>K0</b>	145 -	2	7	10	14	12
<b>K1</b>	125 - 140	2	7	10	14	12
<b>K2</b>	105 - 120	2	7	10	14	12
<b>K3</b>	75 - 100	3	7	10	14	12
<b>K4</b>	45 - 70	4	7	10	14	12
<b>K5</b>	- 40	5	7	10	14	12

Table 3-6 Limits values for twist measured over 2 meters (Bane NOR, 2018)



Table 3-7 shows limit values for long twist measured over 9 meters.

Quality classes	Speed (km/h) in accordance with quality class regime	Long twist (+/- mm)				
		New tamped track	AL – Alert Limit	IL – Intervention Limit	IAL – Immediate Action Limit	
					R ≥ 400 m	R < 400 m
<b>K0</b>	145 -	6	24	31	43	34
<b>K1</b>	125 - 140	6	24	31	43	34
<b>K2</b>	105 - 120	6	24	31	43	34
<b>K3</b>	75 - 100	9	24	31	43	34
<b>K4</b>	45 - 70	12	24	31	43	34
<b>K5</b>	- 40	15	24	31	43	34

Table 3-7 Limits values for twist measured over 9 meters (Bane NOR, 2018)

Table 3-8 gives an overview of maintenance regime related to isolated irregularities of longitudinal level with wavelength between 3 and 25 m (D1). The limits for maintenance depend on the maximum permissible train speed.

Quality classes	Speed (km/h) in accordance with quality class regime	Unevenness in level for each rail in the track (+/- mm)			
		New tamped track	AL – Alert Limit	IL – Intervention Limit	IAL – Immediate Action Limit
<b>K0</b>	145 -	2	6	9	16
<b>K1</b>	125 - 140	2	6	10	23
<b>K2</b>	105 - 120	2	7	12	26
<b>K3</b>	75 - 100	4	10	16	26
<b>K4</b>	45 - 70	5	13	21	28
<b>K5</b>	- 40	6	17	27	28

Table 3-8 Maintenance regime regarding unevenness in longitudinal level for each rail (Bane NOR, 2018b)

Track geometry is measured with the Track Recording Vehicle (TRV) every 0.5 meters. The calculation basis in a wavelength range of 3-25 m is 200 m, which slides along the track. Over 1000 m of track, approximately 2000 measurements are gathered, and a new value for the standard deviation is calculated. The standard deviation values for the individual quality classes are listed in Table 3-9.

The four parameters for which standard deviations are calculated are:

- Longitudinal level  $\sigma_H$  (mm)
- Variation of cant  $\sigma_R$  (mm)
- Alignment  $\sigma_P$  (mm)
- Cooperation  $\sigma_S$  (mm)

Cooperation is a parameter which reflects the comfort that can be experienced in a passenger car. It is calculated by summing alignment and variation of cant before calculating standard deviation. Standard deviation for alignment is calculated for outer rail in curves.

The following equation shows the standard deviation:

$$\sigma = \sqrt{\frac{1}{n-1} \cdot \sum_{i=1}^n (x_i - x_m)^2} \quad (4)$$

where,

$\sigma$  = standard deviation in mm;

$n$  = number of measurement values;

$x_i$  = the measurement value in mm;

$x_m$  = the mean of all measurements.

The standard deviation is usually considered over a stretch of 200 m or 1000 m segment. In Roger 1000, standard deviations will be calculated for different wavelength intervals, segment lengths and accuracy, given in Table 3-9.

Parameter	Wavelength interval	Measurement accuracy	Calculation basis
<b>Standard deviation for longitudinal level <math>\sigma_H</math></b>	3 – 25 m	$\pm 0.2$ mm	200 m
	25 – 70 m	$\pm 0.5$ mm	1000 m
	70 – 150 m	$\pm 1.5$ mm	1500 m
<b>Standard deviation for alignment <math>\sigma_P</math></b>	3 – 25 m	$\pm 0.2$ mm	200 m
	25 – 70 m	$\pm 0.5$ mm	1000 m
	70 – 150 m	$\pm 1.5$ mm	1500 m
<b>Standard deviation for variation for cant <math>\sigma_R</math></b>	3 – 25 m	$\pm 0.2$ mm	200 m
	25 – 70 m	$\pm 0.5$ mm	1000 m

Table 3-9 Standard deviation calculation for Roger 1000

### ***The so-called K-number***

Bane NOR uses the so-called K-number as a Track Quality Index (TQI) to calculate the track quality. The K-number can be used on longer track sections. In Roger 1000, this number is calculated every 1000 m. The K-number is not applicable for shorter track sections.

K-numbers are primarily a comfort figure which reflects the experience as train passenger in the form of irregularities in the track.

The K-number indicates how much of a stretch there all  $\sigma$ -values from Table 3-10 are within the tolerance limits, and can be expressed by the following equation:

$$K = \frac{\sum l}{L} \cdot 100 \% \quad (5)$$

where,

$\sum l$  = the sum of track lengths where all calculated standard deviations are within the quality limits;

$L$  = the track length inspected.

The K-number can vary from 0 to 100% and should be as high as possible.

Table 3-10 shows the quality limits for standard deviation, according to the quality classes.

Quality class	Speed (km/h)	Quality limits (mm)			
		Longitudinal level $\sigma_H$	Variation of cant $\sigma_R$	Alignment $\sigma_P$	Cooperation $\sigma_S$
<b>K0</b>	from 145	1,1	0,9	1,1	1,8
<b>K1</b>	125 - 140	1,3	1,0	1,2	1,7
<b>K2</b>	105 - 120	1,5	1,2	1,3	1,9
<b>K3</b>	75 - 100	1,9	1,4	1,7	2,4
<b>K4</b>	45 - 70	2,4	1,8	2,0	3,1
<b>K5</b>	up to 40	2,9	2,2	2,4	3,6

Table 3-10 Quality limits for standard deviation according to quality classes

The quality figure (K-number) should be as high as possible. Low quality figures will, in addition to reducing comfort, speed up the degradation of the track.

Quality classes	Speed (km/h) in accordance with quality class regime	K - number		
		New tamped track	AL – Alert Limit	IL – Intervention Limit
<b>K0</b>	145 -	90	90	50
<b>K1</b>	125 - 140	90	85	40
<b>K2</b>	105 - 120	90	80	30
<b>K3</b>	75 - 100	90	75	20
<b>K4</b>	45 - 70	90	70	20
<b>K5</b>	- 35	-	-	-

Table 3-11 K-number limits according to quality classes

## 4 CASE STUDY: OFOTBANEN

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For the purpose of this master's thesis, a case study for Ofotbanen is presented. The aim is to suggest how track geometry measurement data can be used for further analysis.

Presently, the so-called K-number, which is the Track Quality Index (TQI) adopted by Bane NOR, describes numerically the track geometry quality over 1000 m. This case study shows how the standard deviation for the short wavelength (3 – 25 m) of longitudinal level defects (right and left rail) could be adopted as an indicator for planned maintenance actions.



Figure 4-1 Ofotbanen (Bane NOR, 2019)

### 4.1 HISTORY

The Ofoten Line (Norwegian: Ofotbanen) is a 42 kilometres long railway, constructed from 1898 to 1902, which runs from the Port of Narvik to Riksgränsen on the Norway–Sweden border. The line continues as the Ore Line (Swedish: Malmbanen) from Riksgränsen to Port of Luleå.

For more than 100 years, rail traffic on Malmbanen and Ofotbanen has been of regional and national significance both in Sweden and Norway, given the enormous volumes of iron ore transported. The annual tonnage exceeds 31 million gross tons (MGT).

The problem of transporting the ore increased pressure for the line's construction. The ore was previously transported by reindeer or river boat in the 17<sup>th</sup>, 18<sup>th</sup> and 19<sup>th</sup> centuries. Ofotbanen joins not only ore deposits in Norway and iron ore fields in northern Sweden, but also is part of a supply chain that links a harbor on the Norwegian coast to overseas customers.

Ofotbanen is a railway of great historical significance. It demonstrates the technological and engineering achievements in railway construction from the turn of the century. Furthermore, the challenging track geometry is worthy of consideration. Railway engineers had to find means to overcome natural obstacles to build a railway through the Swedish-Norwegian mountains.

Supply roads were constructed to transport tools and building materials. On the Swedish side, these roads stretched for more than 60 km from east of Torneträsk to the border. On the Norwegian side, they were 20 km long and very steep in some places (Theander, 1996).

The line has contributed to the social and economic evolution of Narvik district.



*Figure 4-2 Ofotbanen, by Thor Brækkan*

## 4.2 FEATURES OF THE LINE

The track has a challenging geometry with small curve radii and steep gradients, from the Swedish border and down to the Port of Narvik. The vertical inclination downward is 14 – 15 mm/m (Gåsemyr, 2017).

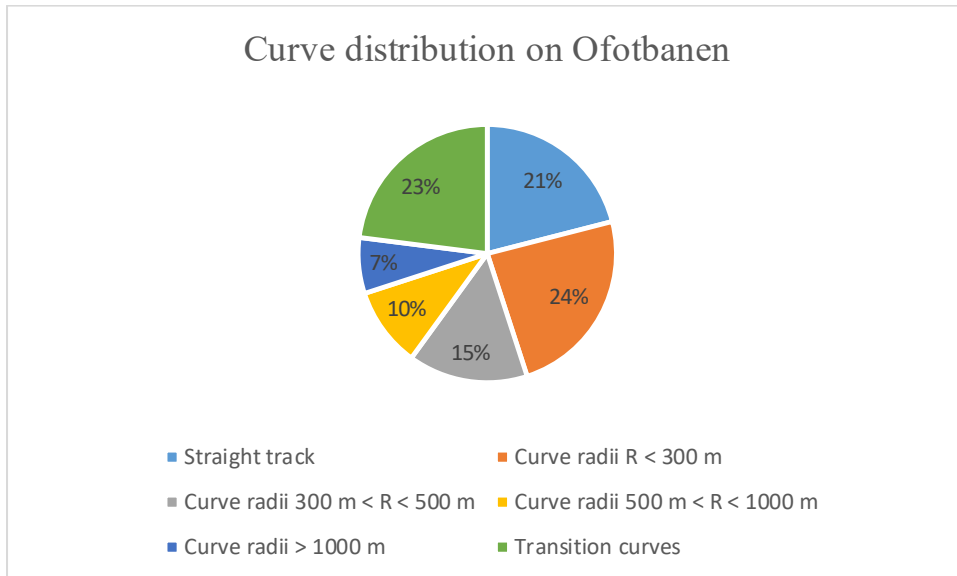


Figure 4-3 Curve distribution on Ofotbanen (Gåsemyr, 2017)

The line is equipped with 60E1, 60E2 and 54E3 rails. There are some sections of line equipped with wooden sleepers and other sections with concrete sleepers. At some locations the ballast profile is characterised by thin ballast layer. Table 4-1 shows the new and the old construction on Ofotbanen.

	<b>Old construction</b>	<b>New construction</b>
Rails	54E3	60E1/60E2
Rail fastenings	Pandrol (various types)	Pandrol Fastclip
Sleeper	Wooden; 520 mm	Concrete; 520 mm
Ballast fraction	Thin ballast height 25 mm – 50 mm	Sufficient ballast 25 mm -50 mm
Rails	Head to be ground for an inclination 1:30	
Turnouts	Application with movable frogs	

Table 4-1 Features of the Section of Line - Ofotbanen

Ofofbanen is a single-track line. Trains can only pass or overtake each other at sidings. The speed of loaded heavy haul trains is 50 km/h.

The line has been upgraded from axle loads of 25 to 30 tons and carries now much longer and heavier trains than it was designed to carry. However, many of the original features of the railway remain in use.

The market forces Bane NOR to find ways to expand Ofofbanen's carrying capacity. The forecast for rail traffic shows an increase that will exceed the maximum achievable capacity for this line. Today's traffic consists of freight trains and passenger trains. Within both categories, volumes are expected to increase steadily from about 2020 up to 2040. In particular, in the mining industry and ore transport, there are high expectations for increased production (Jernbaneverket, 2013).

Capacity increasing measures, such as double-tracks, additional crossing loops and higher axle loads, have been considered and analyzed.

One of the strategies that Bane NOR considers is to raise the allowable axle loads for freight cars on Ofofbanen. A research project has been initiated to determine if increasing from 30 tons to 32,5 tons axle load is technically feasible and economically desirable. On a trial bases, one 32,5 tons test train per day ran along the line. The remaining trains continued operating with axle loads of 30 tons (Einås, 2017).

The axle loads have most influence on the track deterioration on the Norwegian heavy haul line Ofofbanen. This parameter must be added to the greatest importance in weighted assessment. The low operational train speed of 60 km/h (loaded 50 km/h) has less influence. Due to this fact, balance speed in all curves is achieved.

The subarctic climate characterized by long and cold winters, imposes its own constraints, reducing the size of maintenance windows.



### 4.3 DATA COLLECTION AND ANALYSIS

One of the purposes of this quantitative analysis is to correlated track geometry data from the measurement campaigns with operating conditions, weather, superstructure elements and maintenance activity history.

Data were collected with the Roger 1000 Track and Overhead Line Recording Vehicle and IMV 200 Track Recording Train. This last one is operated by Infranord in Sweden. Track geometry measurements are performed twice a year on Ofofbanen. Since the laser system on Roger 1000 was replaced in 2013, all data recorded from 2014 is assumed to be more accurate.

The results presented are based on the data collected from two measurements per year during the last five years, from 2014 to 2018.

The InOffice software interprets the raw data from track geometry measurements on a user-friendly module. The track geometry measurements are gauge, twist, cant, longitudinal level (right and left rail) and alignment (right and left rail). The standard deviation for longitudinal level  $\sigma_H$ , standard deviation for alignment  $\sigma_P$ , standard deviation for variation of cant  $\sigma_r$  and standard deviation for cooperation  $\sigma_s$  are calculated according to the EN 13848 series (European Standard, 2006) on a 200 m long segment. The wavelength domain is in the range of 3 – 25 m. The train quality class is K3.

Mutual data alignment was possible by integrating Banedata and InOffice. This case study focuses on the main line.

The K-numbers are derived from the data collected by Roger 1000 and were calculated for each measurement campaign. Figure 4-4 illustrates the variation of the K-number over time.

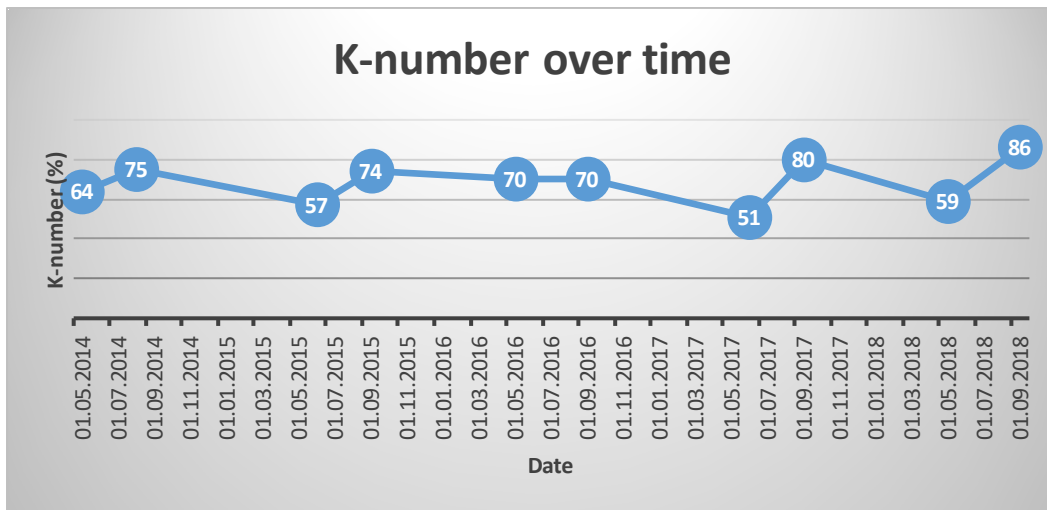


Figure 4-4 K-number variations for Ofofbanen over time

Figure 4-4 shows that the K-number is usually lower for the measurements carried out in the spring than measurements carried out in autumn. These numbers reflect the impact of the extreme weather conditions during the winter on railways.

The degradation of track geometry is distinguished by the increase of the standard deviation for longitudinal level defects ( $\sigma_H$ ) for the wavelength range D1 (3 - 25 m), as shown in Figure 4-5 and 4-6.

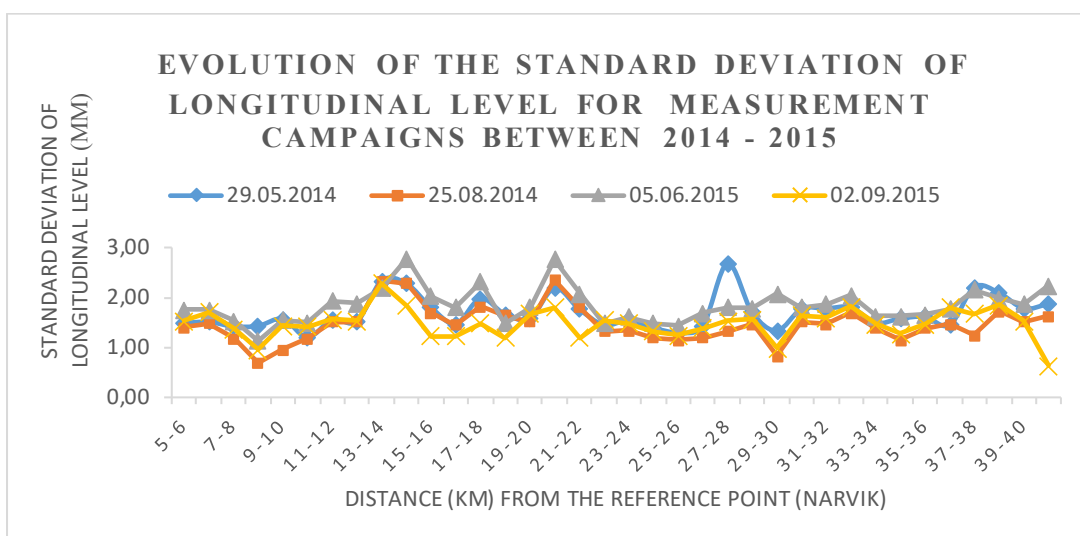


Figure 4-5 Evolution of standard deviation of longitudinal level between 2014-2015

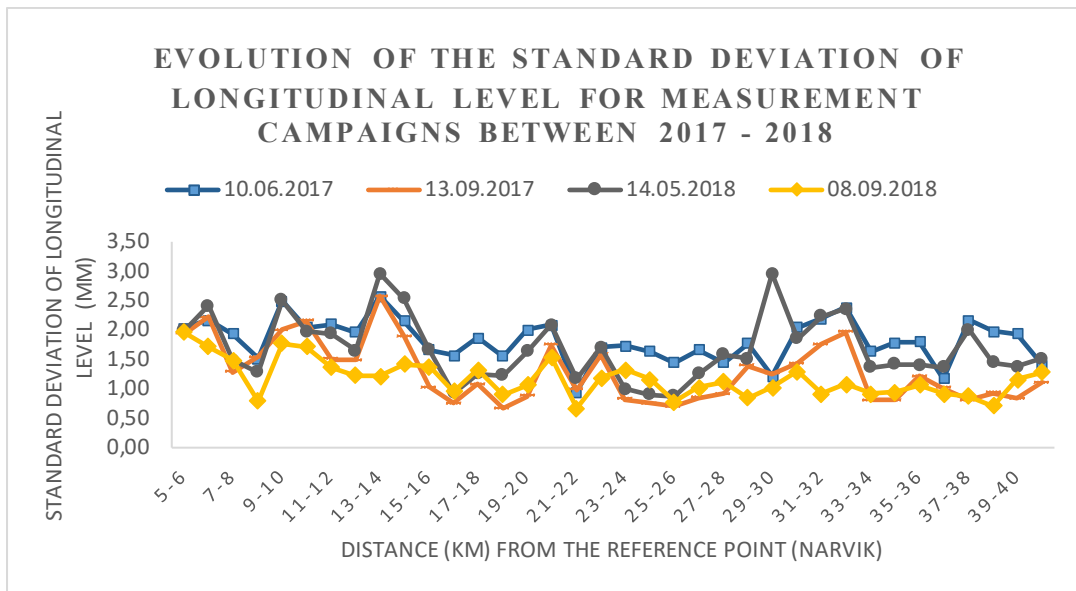


Figure 4-6 Evolution of standard deviation of longitudinal level between 2017-2018

In Norway, mechanized track maintenance (rail grinding, tamping, ballast cleaning, ballast plough and ballast addition) is commonly carried out after the snow has melted in spring and during the summer. It means that the effect of maintenance is shown on measurements taken in autumn (Figures 4-5 and 4-6). Problems due to frost heave can also give rise to increased standard deviation for longitudinal level ( $\sigma_H$ ) on measurements taken in the spring.

Figure 4-7 shows the distribution of the K-number along the line for a measurement campaign carried out on 13 September 2017.

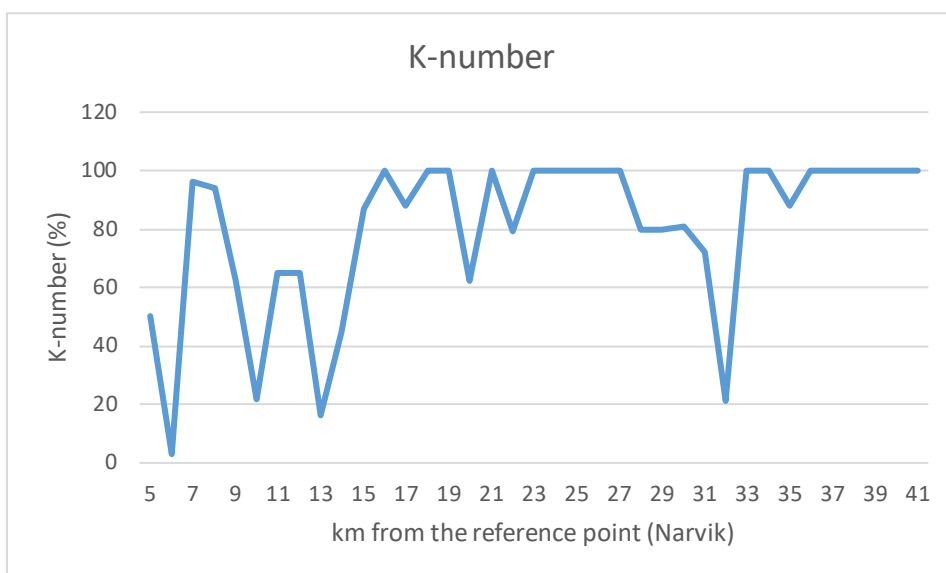


Figure 4-7 K-number variations for Ofofbanen for a specific measurement campaign

The K-number alone may not provide enough information that leads to maintenance actions or strategy. It is important to identify which parameters have a negative effect on the K-number. The quality limits are indicated in Table 4-2.

Quality class	Speed (km/h)	Quality limits (mm)			
		Longitudinal level $\sigma_H$	Variation of cant $\sigma_R$	Alignment $\sigma_P$	Cooperation $\sigma_S$
<b>K3</b>	75 - 100	1.9	1,4	1,7	2,4

Table 4-2 Quality limits for Ofoibanen

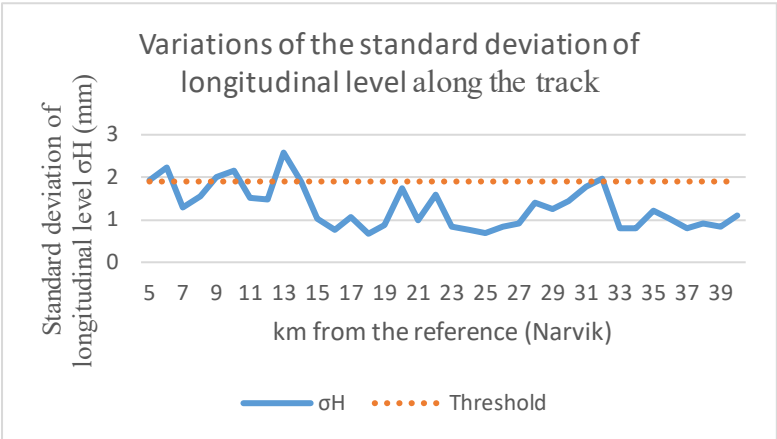


Figure 4-8 Variations of the standard deviation for the longitudinal level

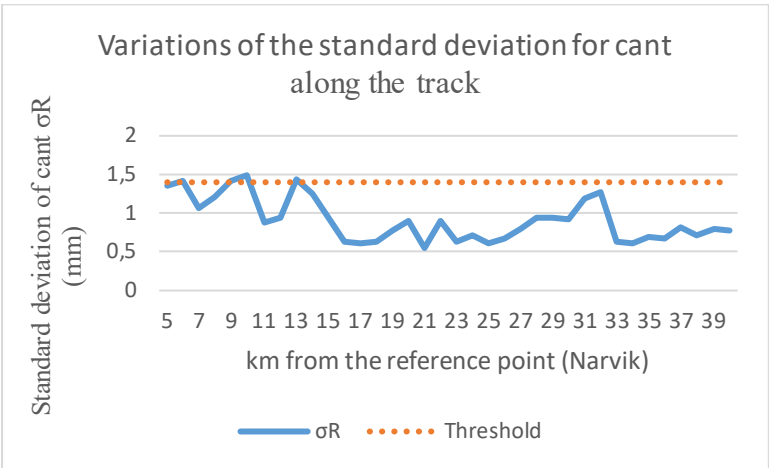


Figure 4-9 Variations of the standard deviation for cant

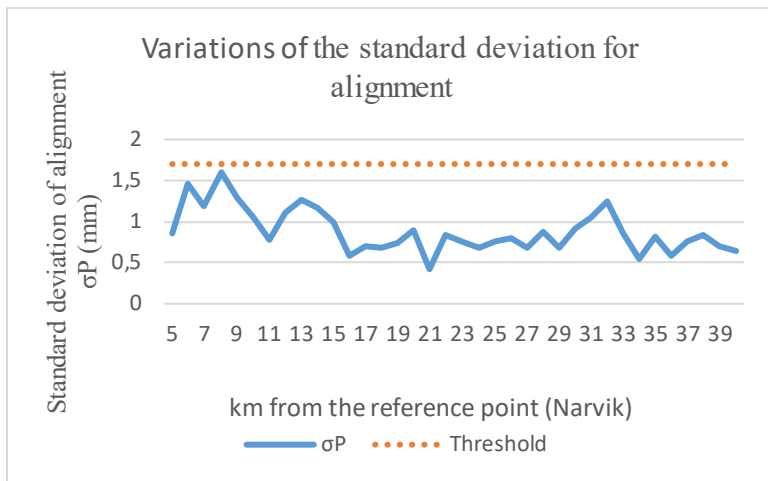


Figure 4-10 Variations of the standard deviation for alignment

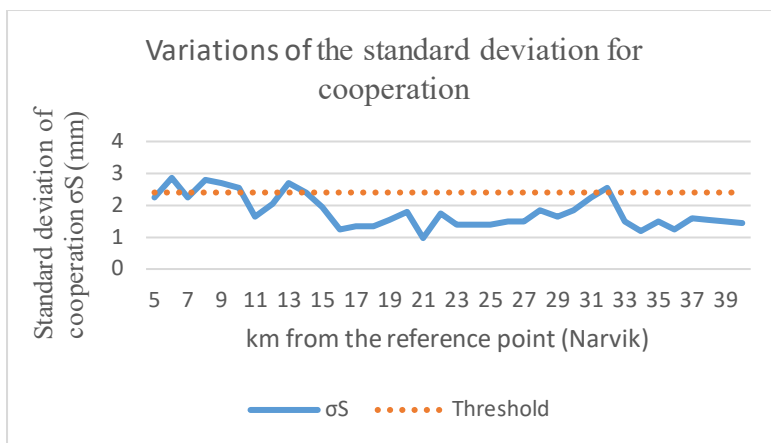


Figure 4-11 Variations of the standard deviation for cooperation

From figures 4-8, 4-9, 4-10 and 4-11, one could conclude that the standard deviation for longitudinal level and the standard deviation for cooperation have the most negative effect on the K-number.

Figure 4-8 shows that the weakest point along the track, i.e. the point with highest standard deviation for longitudinal level, is between km 13 and 14.

Presently, InOffice provides only the K-number for 1000-meter sections. As a means to assess track geometry quality for shorter track sections, i.e. 100- or 200-meter sections, the raw data from the track measurements were retrieved. In this case, the parameter of interest was the amplitude of the longitudinal level defects. The

standard deviation ( $\sigma_H$ ) was then calculated for 200-meter track sections. The wavelength domain is in the range of 3 – 25 m, and the train quality class is K3.

Table 4-3 shows a sample of the calculations based on the track geometry measurement carried out on 13 September 2017.

Track segment	Standard deviation of longitudinal level ( $\sigma_H$ )
km 13,0 - 13,2	3,03
km 13,2 - 13,4	2,37
km 13,4 - 13,6	4,03
km 13,6 - 13,8	2,04
km 13,8 - 14,0	1,45

Table 4-3 Sample of calculations for the track segment km 13,0 - 14,0

By correlating the track geometry measurement data and video recordings from the external camera mounted on the TRV, one could conclude that the railroad switch gives rise to the increased standard deviation for longitudinal level ( $\sigma_H$ ).



Figure 4-12 Ofoibanen km 13,04 (Bane NOR, 2017)

At switches and turnouts, there is an abrupt change of vertical track stiffness, implying irregular wheel-rail contact forces. Such locations are more prone to develop track settlements due to the permanent deformation of the ballast and the substructure.

The next weakest point of the track, i.e. the track segment where the predefined threshold level for  $\sigma_H$  is also exceeded, is between km 6 and 7.

Track segment	Standard deviation of longitudinal level ( $\sigma_H$ )
km 6,0 - 6,2	2,27
km 6,2 - 6,4	2,11
km 6,4 - 6,6	2,40
km 6,6 - 6,8	2,30
km 6,8 - 7,0	2,00

Table 4-4 Calculations for the track segment km 6,0 - 7,0



Figure 4-13 Ofoibanen km 6,47 (Bane NOR, 2017)

Figure 4-13 shows a view from the external camera mounted on the TRV at km 6,47, right before the tunnel opening. Old tunnel constructions give rise to increased standard deviation for longitudinal level ( $\sigma_H$ ). This is mainly due to the fact that tunnels on older track sections are not deeper as they should to get sufficient total height up to the tunnel top, and to be able to accommodate a ballast layer under the sleepers, as well as sleepers and rails.

Underneath the ballast layer, it will then be a bedrock that is to be considered as hard ground, in the same way as bridges/culverts of concrete. In this case, sufficient spring and damping properties concerning the superstructure are not achieved, due to the very hard stiffness of the substructure. In normal conditions, correct elasticity as for a substructure of clay is achieved.

These conditions together often give rise to increasing standard deviations in the vertical position. A greater stress is induced on track components and ballast layer, which over time reduces the ballast quality in the form of crush down of the stones and the occurrence of fine materials.

Another weak point of the track, i.e. a track segment where the predefined threshold level for  $\sigma_H$  is also exceeded, is between km 9 and 10.

Track segment	Standard deviation of longitudinal level ( $\sigma_H$ )
km 9,0 - 9,2	1,50
km 9,2 - 9,4	1,31
km 9,4 - 9,6	1,81
km 9,6 - 9,8	2,53
km 9,8 - 10,0	3,06

Table 4-5 Calculations for the track segment km 9,0 - 10,0



Figure 4-14 Ofofbanen km 9,8 (Bane NOR, 2017)

Records from the external camera mounted on the TRV, between km 9,8 and 10 did not reveal any apparent drainage problems, although the quality limits for standard deviation for longitudinal level ( $\sigma_H$ ) were exceeded. According to Banedata, this is



a track segment with small curve radii (< 350 m). A more rapid degradation of the track is expected.

**4.4 EFFECTIVENESS OF TAMPING**

For the purpose of this study case, some track segments were selected, and the effectiveness of tamping was evaluated based on the graph developed by UIC (Figure 4-15). The maintenance history for tamping actions was collected from BaneData.

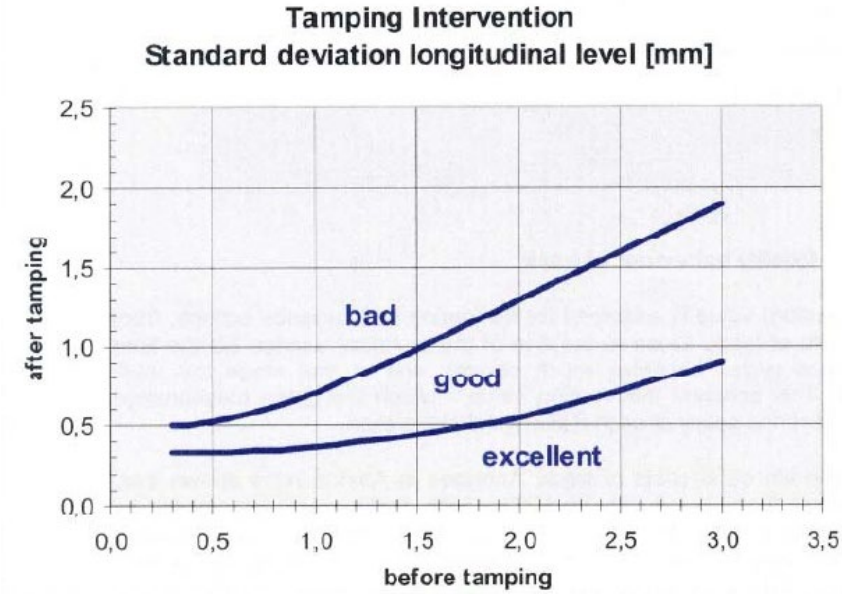


Figure 4-15 Effectiveness of tamping (UIC - Infrastructure Department, 2008)

Maintenance Activity	From km	To km	Date	Length (m)
Tamping	11,27	12,52	02.08.2017	1250

Table 4-6 Maintenance activity record for the track segments km 11,27 - 12,52 (Bane NOR)

Track segment	Standard deviation of longitudinal level $\sigma_H$ (mm) before tamping 10.06.2017	Standard deviation of longitudinal level $\sigma_H$ (mm) after tamping 13.09.2017	Effectiveness of tamping
km 11,2 – 11,4	2,51	2,15	bad
km 11,4 – 11,6	2,10	1,23	good
km 11,6 – 11,8	1,98	0,95	good
km 11,8 – 12,0	1,84	1,06	good

Table 4-7 Evaluation of the effectiveness of tamping, km 11,2 – 12,0

Table 4-7 shows an evaluation of the effectiveness of tamping carried out on 2 August 2017. One could conclude that the desired performance level after tamping was not achieved for the track segment km 11,2 – 11,4, according to the graph developed by UIC (Figure 4-15).

Another track segment was randomly selected, and the effect of the tamping was evaluated.

Maintenance Activity	From km	To km	Date	Length (m)
<b>Tamping</b>	14,65	15,28	05.08.2016	632

Table 4-8 Maintenance activity record for the track segments km 14,65 - 15,28 (Bane NOR)

Track segment	Standard deviation of longitudinal level $\sigma_H$ (mm) before tamping 09.06.2016	Standard deviation of longitudinal level $\sigma_H$ (mm) after tamping 01.09.2016	Effectiveness of tamping
km 14,6 – 14,8	1,70	1,03	good
km 14,8 – 15,0	1,95	1,10	good
km 15,0 – 15,2	1,80	1,72	bad

Table 4-9 Evaluation of the effectiveness of tamping, km 14,6 - 15,2

Table 4-9 shows an evaluation of the effectiveness of tamping carried out on 5 August 2016. One could conclude that desired performance level after tamping was not achieved for the track segment km 15,0 – 15,2.

The quality limit for the standard deviation of longitudinal level  $\sigma_H$  shown on Table 3-10 not always has been maintained. This is probably due to budget constraints.

Figure 4-16 shows the evolution of  $\sigma_H$  along the time for the track segment km 9,8 – 10,0. Although the threshold for the quality limit was exceeded, the values for  $\sigma_H$  were within an acceptable level, according to the UIC tamping intervention graph shown on Figure 2-7.

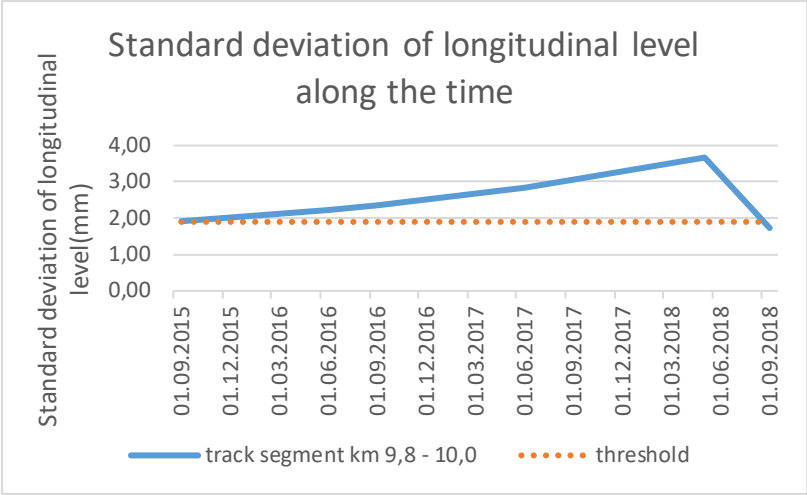


Figure 4-16 Variations of  $\sigma_H$  along the time for track segment km 9,8 – 10,0

Measurement date	02.09.15	09.06.16	01.09.16	10.06.17	13.09.17	14.05.2018	08.09.18
<b>Standard deviations of longitudinal level <math>\sigma_H</math> (mm)</b>							
<b>km. 9,8 – 10,0</b>	1,91	2,23	2,36	2,83	3,06	3,67	1.73

Table 4-10 Values of  $\sigma_H$  along the time for track segment km 9,8 – 10,0

Maintenance Activity	From km	To km	Date	Length (m)
<b>Tamping</b>	9,91	10,42	29.08.2018	510

Table 4-11 Maintenance activity record for the track segments km 9,91 – 10,42 (Bane NOR)

Tamping was carried out on 29 August 2018 to restore the track geometry in this segment of track, as the value of the standard deviation of longitudinal level  $\sigma_H$  exceeded the acceptable limit.

#### 4.5 PREDICTING THE PROGRESSION OF THE STANDARD DEVIATION FOR LONGITUDINAL LEVEL DEFECTS

One of the objectives of this case study is to determine empirically how the standard deviation for longitudinal level (right and left rail) defects behaves over time. This is the only parameters which will be considered to assess the track geometry degradation and is often used for triggering preventive tamping. Deterioration rates can be estimated based on several consecutive measurements over time, taken at the same position.

The amplitude for the longitudinal level defects (wavelength range 3 – 25 m), derived directly from the raw signal, were collected from each measurement campaign. The quality signal is calculated by a moving average, within a 200-meter track segments. These data, together with the dates when the measurements were performed, were consolidated in Excel.

Records of the annual tonnage for Ofofbanen were collected and organized, as shown in Table 4-12.

<i>Annual gross tonnage for Ofofbanen</i>					
	<b>2014</b>	<b>2015</b>	<b>2016</b>	<b>2017</b>	<b>2018</b>
<b>January</b>	3 016 331	2 237 748	2 397 563	2 715 863	2 865 497
<b>February</b>	2 983 975	2 213 338	2 481 744	2 624 183	2 596 848
<b>March</b>	2 572 216	2 357 088	2 861 763	2 451 794	2 772 384
<b>April</b>	3 031 601	2 260 304	2 934 167	2 710 857	2 460 156
<b>May</b>	3 012 793	2 381 403	2 782 777	2 998 246	2 721 616
<b>June</b>	2 390 040	2 180 108	2 434 453	2 182 481	2 500 888
<b>July</b>	2 844 487	2 327 129	2 317 968	2 878 568	2 215 455
<b>August</b>	2 909 273	2 374 883	2 474 039	2 414 705	2 429 061
<b>September</b>	2 460 406	1 851 197	2 823 299	2 736 889	3 072 069
<b>October</b>	2 856 668	2 363 249	2 482 465	2 611 935	2 398 176
<b>November</b>	2 299 523	2 374 411	2 680 224	2 705 059	2 556 783
<b>December</b>	2 535 880	2 407 064	2 611 437	2 875 653	2 975 110
	32 913 193	27 327 922	31 281 899	31 906 233	31 564 043

Table 4-12 Annual gross tonnage for Ofofbanen between 2014-2018

The accumulated tonnage between two consecutive track geometry measurements was calculated, as shown in Table 4-13.

<i>Measurement date</i>	<i>Days between two consecutive measurements</i>	<i>Accumulated tonnage (gross tons)</i>
<b>29.05.2014</b>	N/A	N/A
<b>25.08.2014</b>	88	7 775 089
<b>05.06.2015</b>	284	22 524 645
<b>02.09.2015</b>	89	6 627 279
<b>09.06.2016</b>	281	23 063 771
<b>01.09.2016</b>	115	9 295 444
<b>10.06.2017</b>	282	24 731 752
<b>13.09.2017</b>	95	7 934 246
<b>14.05.2018</b>	243	21 667 553
<b>08.09.2018</b>	117	9 457 121

Table 4-13 Accumulated tonnage between two consecutive measurements

The track geometry measurements carried out on 29 Mai 2014 and on 5 June 2015 were carried out by Infranord. Track geometry data from those two measurements campaigns were not considered for the degradation trend analysis.

Statistical regression analysis has been performed to identify significant variables influencing the track degradation.

A track segment was chosen to illustrate how regression analysis was performed for each track segment. In this case, no tamping actions were carried out between track geometry measurements. The accumulated tonnage was the first variable analyzed.

<b>Track segment km. 6,0 – 6,2</b>		
<b>Accumulated tonnage (MGT)</b>	$\sigma H$ (mm)	Measurement date
<b>0</b>	1,89	02.09.2015
<b>23,06</b>	1,98	09.06.2016
<b>32,36</b>	2,02	01.09.2016
<b>57,09</b>	2,20	10.06.2017
<b>65,02</b>	2,27	13.09.2017
<b>86,69</b>	2,34	14.05.2018
<b>96,15</b>	2,40	08.09.2018

Table 4-14 Accumulated tonnage vs. sigma H for track segment km 6,0 – 6,2

Both linear and exponential regression were performed to find a line that best fits the data points.

A linear trendline typically describes a continuous rise or fall over time, by the following equation:

$$y = b \cdot x + a \quad (6)$$

where,

$b$  = the slope of the trend line;

$a$  = the y-intercept.

Figure 4-17 shows the linear regression for accumulated tonnage vs.  $\sigma H$ . The  $R^2$  represents the goodness of the model.  $R^2 = 0$  means that the regression line does not fit the data at all.  $R^2 = 1$  means that the regression line fits the data absolutely. The  $R^2 = 0,985$  is a very good fit.

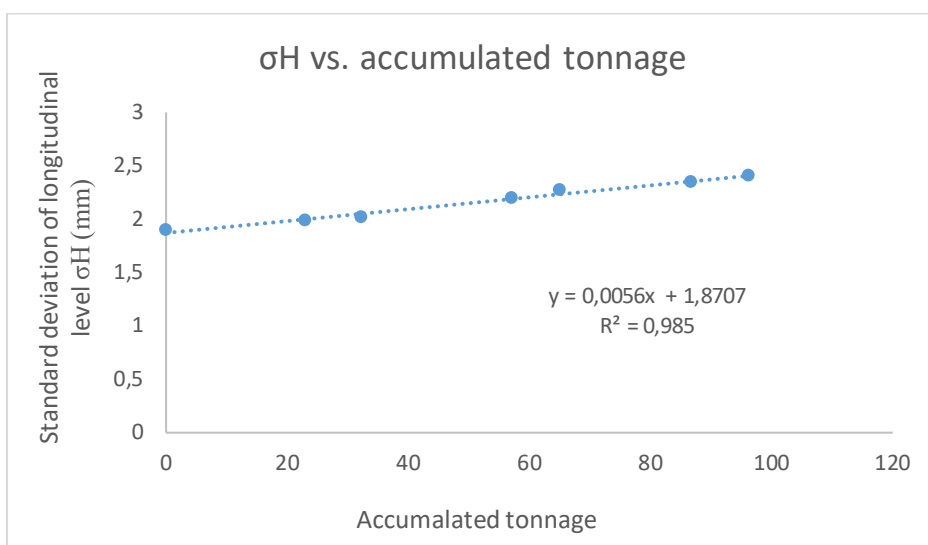


Figure 4-17 Accumulated tonnage vs. sigma H, linear regression, for track segment km 6,0 – 6,2

The exponential regression was then performed, represented by the following equation:

$$y = a \cdot e^{b \cdot x} \tag{7}$$

where,

$a, b$  = calculated coefficients;

$e$  = the mathematical constant  $e$ .

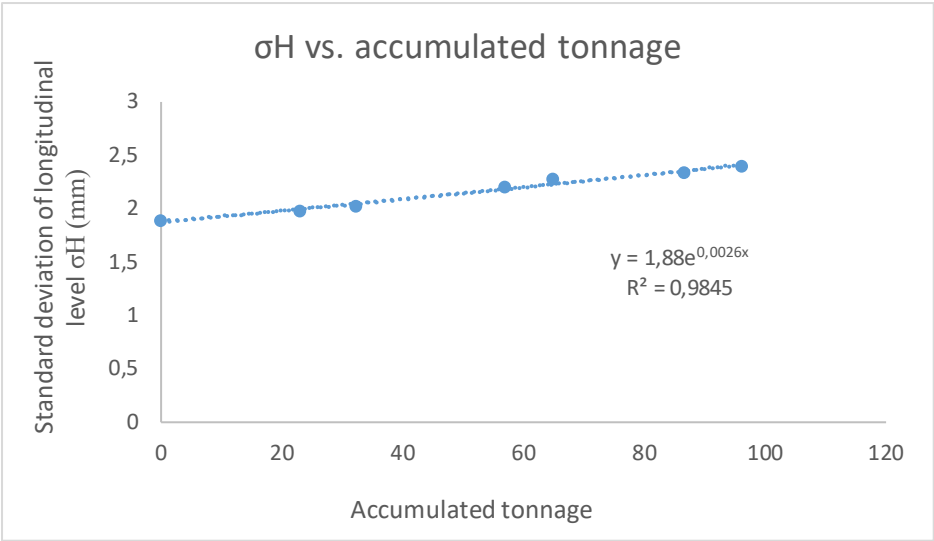


Figure 4-18 Accumulated tonnage vs. sigma H, exponential regression, for track segment km 6,0 – 6,2

Figure 4-18 shows the exponential regression for accumulated tonnage vs.  $\sigma H$ . The  $R^2 = 0,985$  is also fitting very well.

The second variable analyzed is time (days), as shown in Table 4-15.

Track segment km. 6,0 – 6,2		
Time (days)	$\sigma H$ (mm)	Measurement date
0	1,89	02.09.2015
281	1,98	09.06.2016
396	2,02	01.09.2016
678	2,20	10.06.2017
773	2,27	13.09.2017
1016	2,34	14.05.2018
1133	2,40	08.09.2018

Table 4-15 Time vs. sigma H for track segment km 6,0 – 6,2

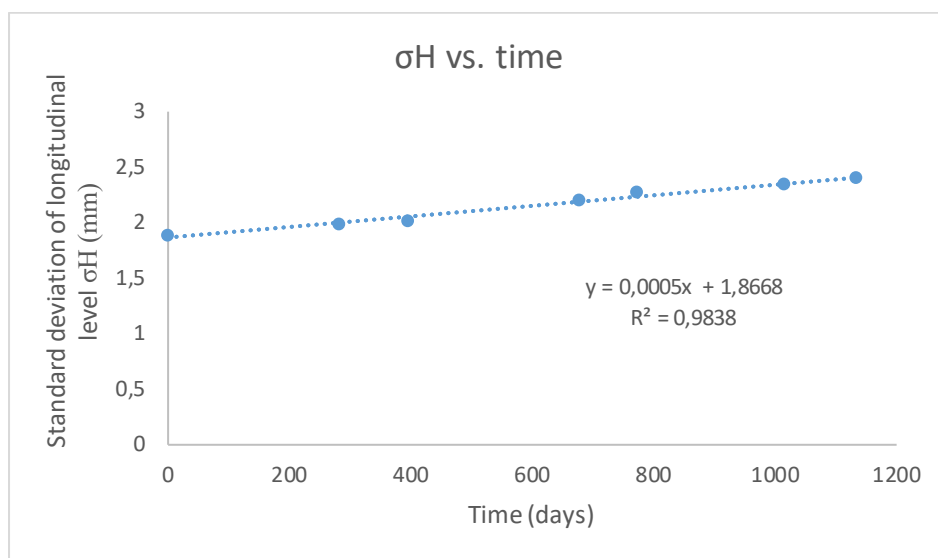


Figure 4-19 Time vs. sigma H, linear regression, for track segment km 6,0 – 6,2

Figure 4-19 shows the linear regression for time vs.  $\sigma H$ . The  $R^2 = 0,9838$  is a very good fit.

The exponential regression was then performed, as shown on Figure 4-20. The  $R^2 = 0,9839$  indicates that the exponential model also fits the data points.



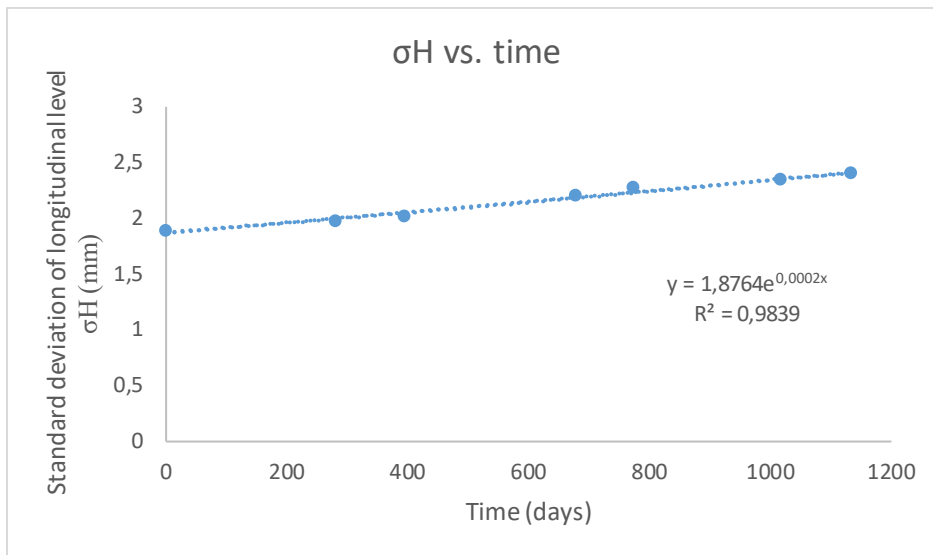


Figure 4-20 Time vs. sigma H, exponential regression, for track segment km 6,0 – 6,2

Another track segment was arbitrarily selected, regression analysis was performed, and the results were compared.

Track segment km 14,4 – 14,6		
Accumulated tonnage (MGT)	$\sigma_H$ (mm)	Measurement date
0	1,77	02.09.2015
23,06	1,88	09.06.2016
32,36	2,01	01.09.2016
57,09	2,11	10.06.2017
65,02	2,26	13.09.2017
86,69	2,33	14.05.2018

Table 4-16 Accumulated tonnage vs. sigma H for track segment km 14,4 – 14,6

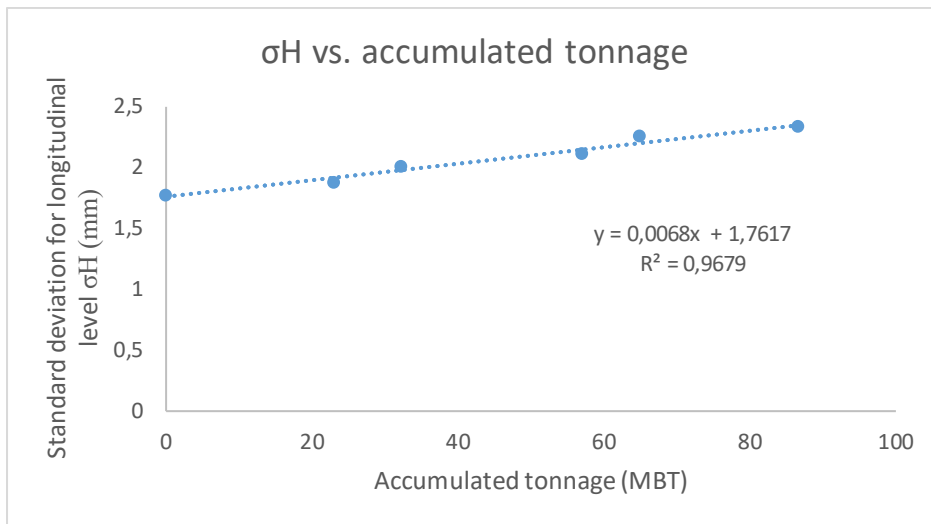


Figure 4-21 Accumulated tonnage vs. sigma H, linear regression, for track segment km 14,4 – 14,6

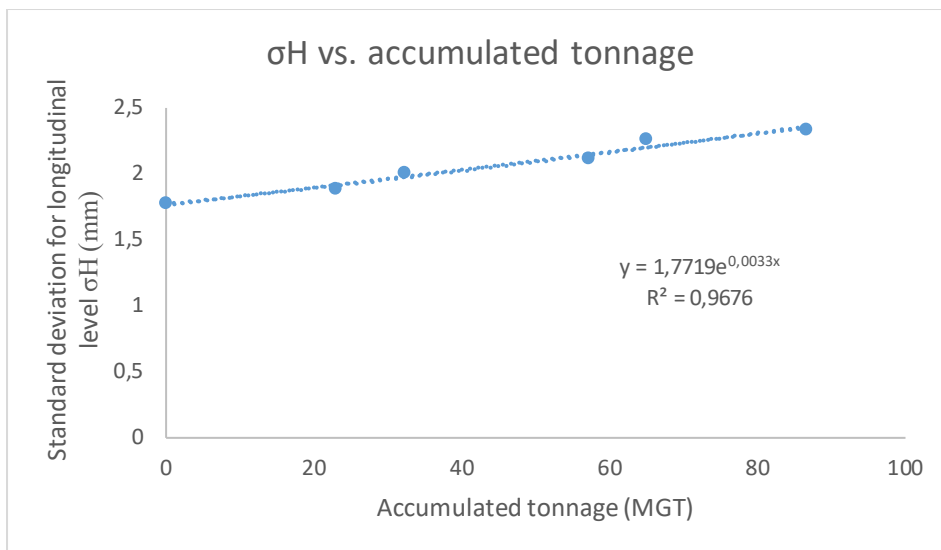


Figure 4-22 Accumulated tonnage vs. sigma H, exponential regression, for track segment km 14,4 – 14,6

Track segment km 14,4 – 14,6		
Time (days)	σH (mm)	Measurement date
0	1,77	02.09.2015
281	1,88	09.06.2016
396	2,01	01.09.2016
678	2,11	10.06.2017
773	2,26	13.09.2017
1016	2,33	14.05.2018

Table 4-17 Time vs. sigma H for track segment km 14,4 – 14,6

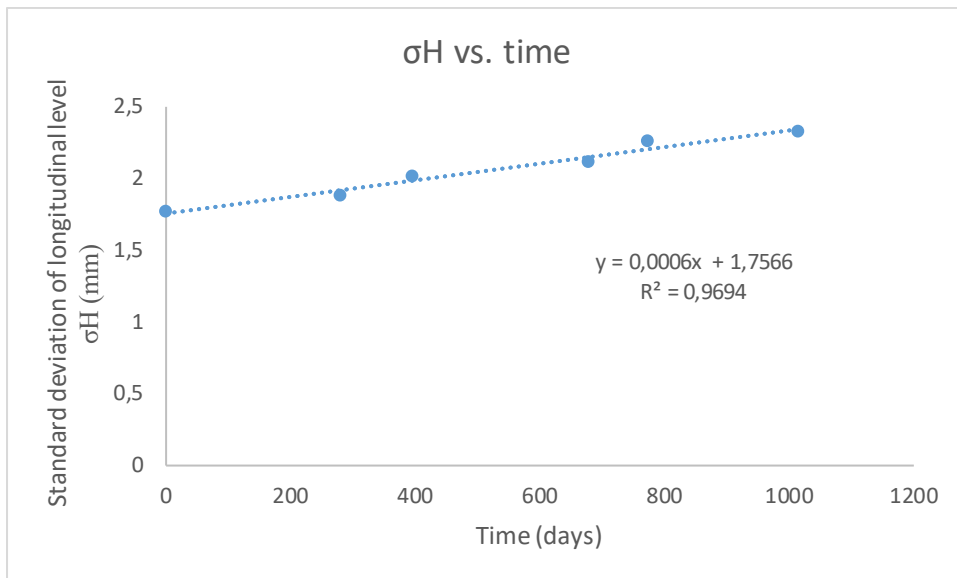


Figure 4-23 Time vs. sigma H, linear regression, for track segment km 14,4 – 14,6

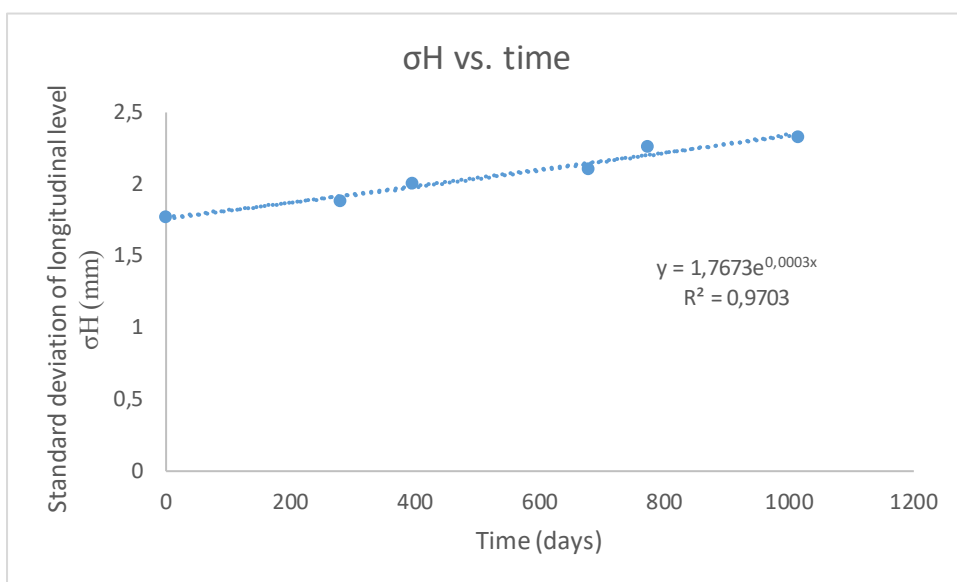


Figure 4-24 Time vs. sigma H, exponential regression, for track segment km 14,4 – 14,6

From the regression analysis performed for track segments km 6,0 – 6,2 and 14,4 – 14,6, it was concluded that both linear and exponential models fit the data points. Another conclusion is that the track segments have different degradation rates, although submitted to the same traffic conditions.

The next stage in the data processing is calculating the degradation rate (b) for the other track segments. As supported by the literature review, there is an exponential

relationship between degradation and load. The exponential model was therefore chosen for further analysis.

Tables 4-18 shows a sample of the data processing for the calculation of the degradation rate.

<i>Measurement date</i>	02.09.15	09.06.16	01.09.16	10.06.17	13.09.17	14.05.2018	08.09.18
Accumulated tonnage (MGT) between two measurements		23,06	9,30	24,73	7,93	21,67	9,46
<b>Standard deviations of longitudinal level <math>\sigma H</math> (mm)</b>							
<b>km 9,0 – 9,2</b>	1,90	1,50	1,99	3,04	1,50	1,67	1,33
<b>km 9,2 – 9,4</b>	1,20	1,38	1,49	2,27	1,31	1,51	1,62
<b>km 9,4 – 9,6</b>	1,19	1,43	1,48	1,84	1,81	2,14	2,27
<b>km 9,6 – 9,8</b>	1,64	1,91	2,04	2,51	2,53	3,35	1,91
<b>km 9,8 – 10,0</b>	1,91	2,23	2,36	2,83	3,06	3,67	1,73

Table 4-18 Standard deviation for longitudinal level ( $\sigma H$ ) calculated for different track segments

BaneData provided the maintenance history for all the mechanized track maintenance which was carried out between 2015 and 2018. Track segments where tamping have been carried out between two consecutive measurements, commonly show a decrease in the standard deviation for the longitudinal level. These values were eliminated from the data, i.e. not considered when calculating the degradation rate for 200-meters track segments.

The degradation rate (b) was calculated for 200-meters track segments, as a function of tonnage. Table 4-19 shows a sample of the data processing.

<b>Track segment</b>	Standard deviations of longitudinal level $\sigma H$ (mm)							<b>b (mm/MGT)</b>
<b>km 9,0 – 9,2</b>	1,90	1,50	1,99	3,04	1,50	1,67	1,33	0,0171
<b>km 9,2 – 9,4</b>	1,20	1,38	1,49	2,27	1,31	1,51	1,62	0,0074
<b>km 9,4 – 9,6</b>	1,19	1,43	1,48	1,84	1,81	2,14	2,27	0,0077
<b>km 9,6 – 9,8</b>	1,64	1,91	2,04	2,51	2,53	3,35	1,91	0,0071
<b>km 9,8 – 10,0</b>	1,91	2,23	2,36	2,83	3,06	3,67	1,73	0,0073

Table 4-19 Sample of the data processing

Based on the calculations shown on Table 4-19, one could conclude that the track segment km 9,0 – 9,2 has a high degradation rate  $b$  (mm / MGT). A report generated from BaneData shows that a tunnel ends at km 9,04 and a turnout starts at km 9,19. Thus, there is an abrupt change of vertical track stiffness. This track segment is more prone to develop track settlements.

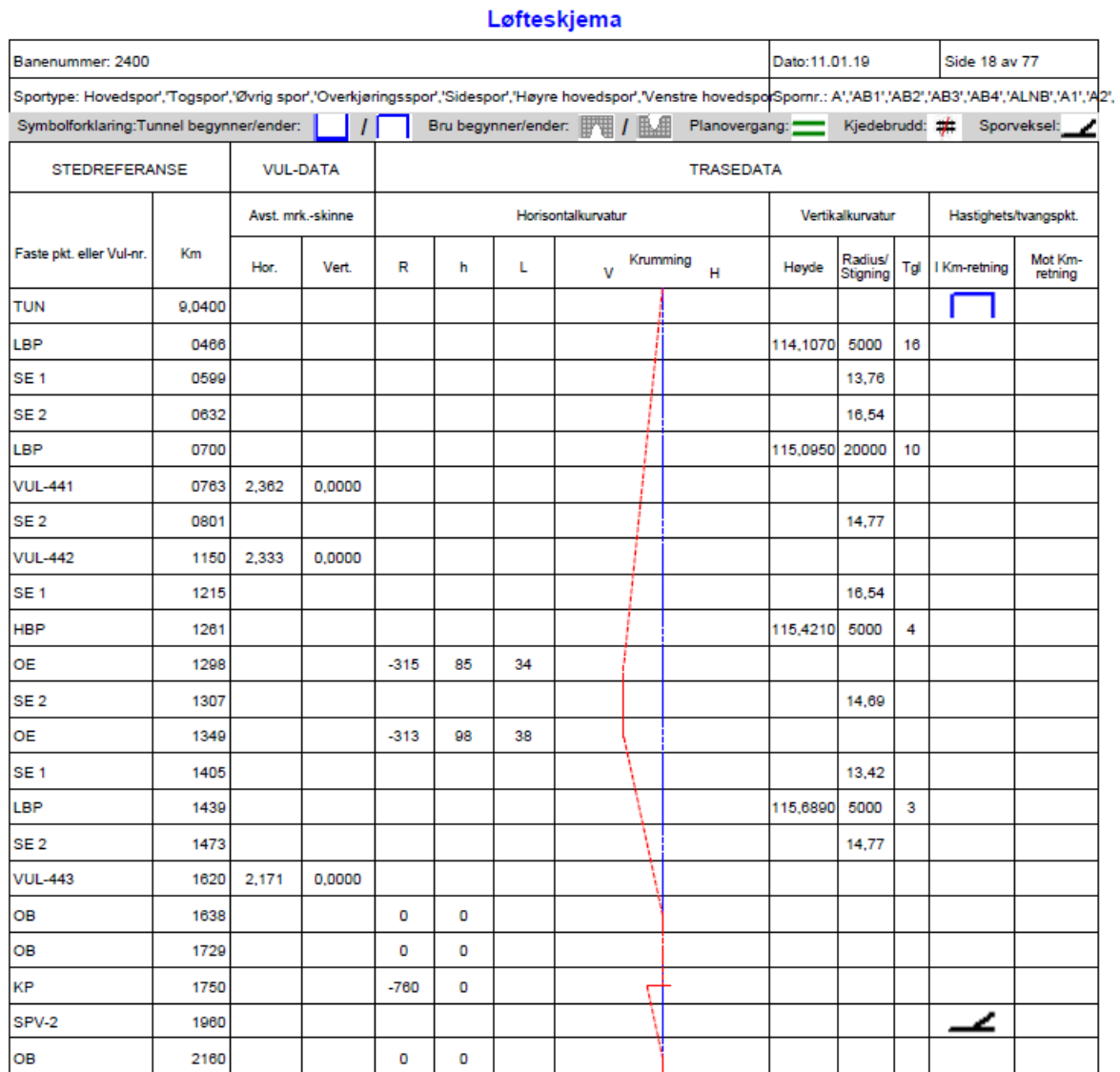


Figure 4-25 Description of Ofotbanen given in Løfteskjema (Bane NOR, 2019)

The same data processing was then conducted for a thirteen kilometres pilot section of the Ofofbanen, from km 6,0 to km 19,0. The pilot section was divided into 200-meter track segments. Table 4-20 shows the descriptive analysis of the degradation rates (mm / MGT) for Ofofbanen.

Descriptive analysis of the degradation rate (mm/MGT) for Ofofbanen	
Mean	0,0079
Standard Error	0,00043
Median	0,0061
Mode	0,0041
Standard Deviation	0,0071
Sample Variance	5,01E-05
Kurtosis	7,03
Skewness	2,48
Range	0,040
Minimum	0,0006
Maximum	0,040
Sum	2,098
Count	265

Table 4-20 Descriptive analysis of the degradation rate as a function of tonnage

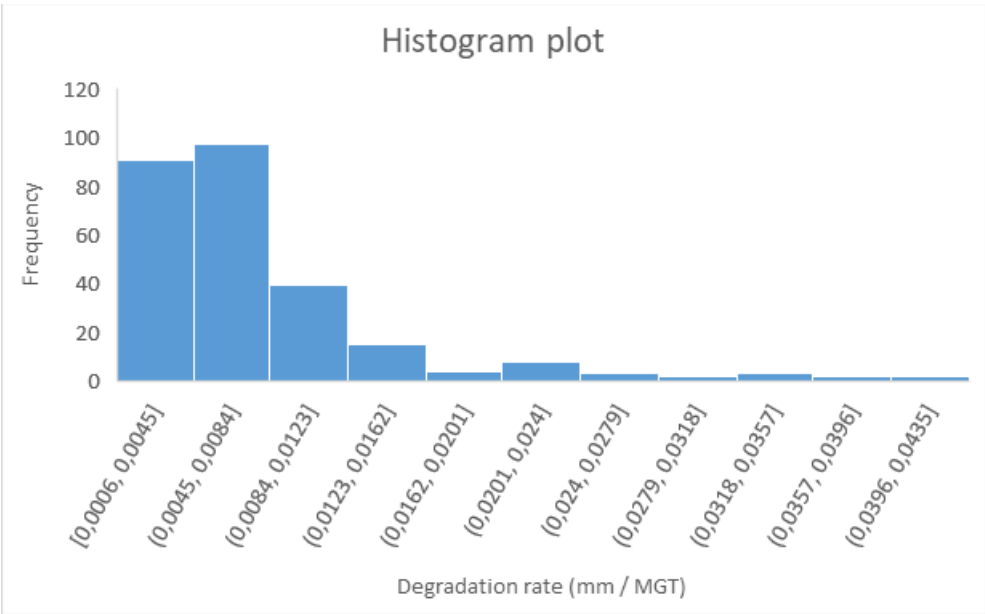


Figure 4-26 Histogram plot for the degradation rate as a function of accumulated tonnage

The degradation rate (b) was also calculated as a function of time. In this case, the value chosen for the variable time (t) was 30 days.

Descriptive analysis of the degradation rate (mm/30 days) for Ofofbanen	
Mean	0,020
Standard Error	0,0011
Median	0,015
Mode	0,012
Standard Deviation	0,018
Sample Variance	0,00033
Kurtosis	6,61
Skewness	2,42
Range	0,103
Minimum	0,002
Maximum	0,105
Sum	5,35
Count	265

Table 4-21 Descriptive analysis of the degradation rate as a function of time

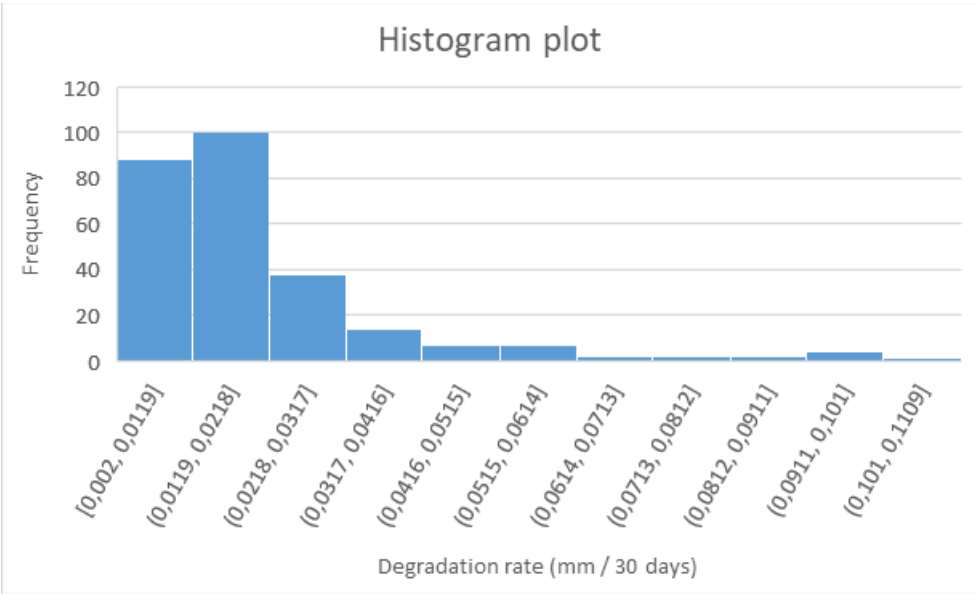


Figure 4-27 Histogram plot for the degradation rate as function of time

From Figures 4-26 and 4-27, one could conclude that the distribution is right-skewed. The target numbers in Tables 4-20 and 4-21 show that the median and mode values are lower than the means values. The median yields a more appropriated idea

of the data distribution, while the mean is influenced by the extreme values. If data were normally distributed, it would have been reasonable to use the mean value of the degradation rate as a function of time (or accumulated tonnage). However, in this case, this strategy would overestimate the degradation.

A sample of the calculated values of the degradation rate (b) for each track segment is shown on Table 4-22. The track segments have small curve radii (300 m < R < 500 m).

Track segment	Initial $\sigma H$ (mm) 08.09.2018	Degradation rate b as a function of accumulated tonnage (mm / MGT)
km 7,4 – 7,6	1,10	0,0043
km 11,6 – 11,8	1,28	0,0094
km 12,8 – 13,0	1,21	0,0038
km 15,4 – 15,6	1,09	0,0057
km 16,0 – 16,2	1,27	0,0051

Table 4-22 Sample of the calculated values of the degradation rate for each track segment with small curve radii

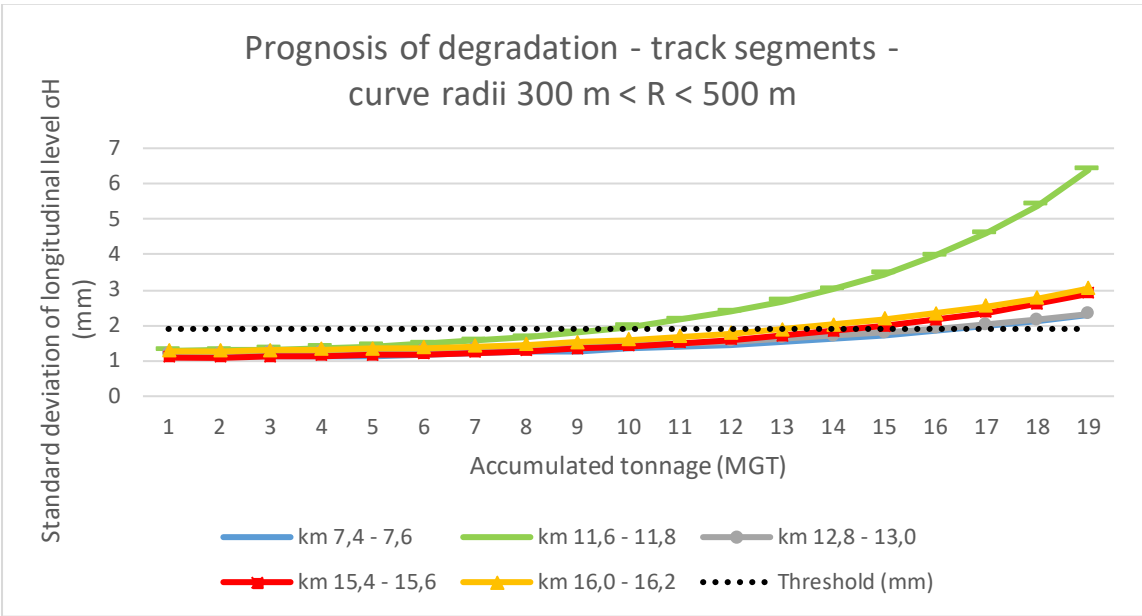


Figure 4-28 Prognosis of degradation per track segment, small curve radii

Figure 4-28 shows the prognosis of degradation for five track segments with small curve radii (300 m < R < 500 m). The graph indicates when the standard deviation



of longitudinal level  $\sigma H$  will exceed the threshold value (1,9 mm). The track segment km 11,6 – 11,8 has an initial value for  $\sigma H = 1,28$  mm. The graph indicates a demand of preventive tamping action after approximately 10 MGT. The track segment km 7,4 – 7,6 has an initial value for  $\sigma H = 1,1$  mm and a demand of preventive tamping action after approximately 17 MGT.

The values of the degradation rate (b) were also compared for straight track segments, as shown on Table 4-23.

Track segment	Initial $\sigma H$ (mm) 08.09.2018	Degradation rate b as a function of accumulated tonnage (mm / MGT)
km 6,0 – 6,2	2,40	0,0024
km 6,8 – 7,0	1,21	0,0055
km 12,2 – 12,4	1,34	0,0073
km 16,4 – 16,6	0,84	0,0070

Table 4-23 Sample of the calculated values of the degradation rate for straight track segments

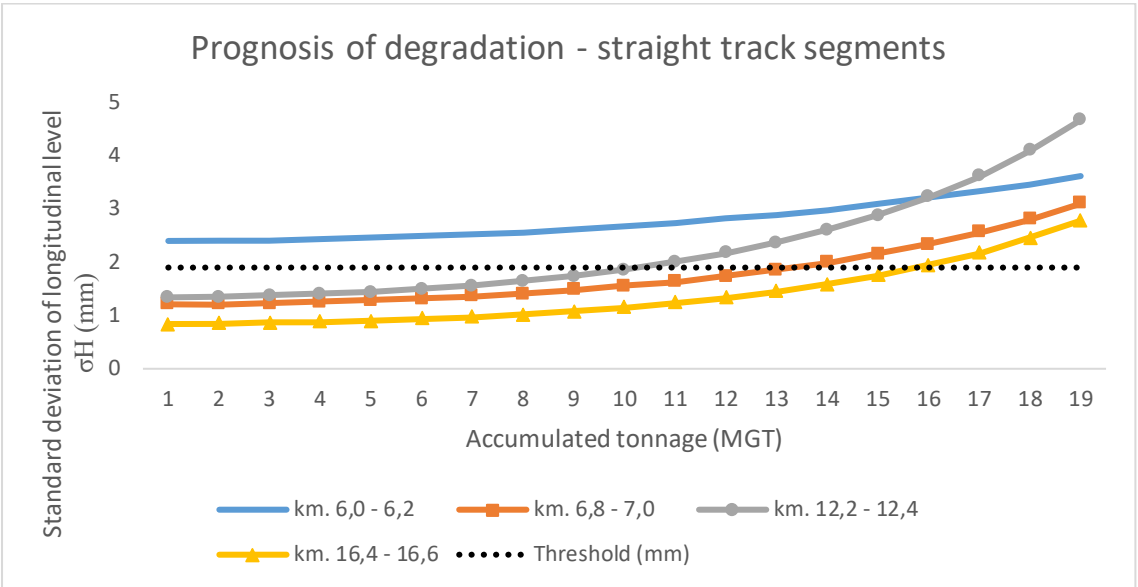


Figure 4-29 Prognosis of degradation per track segment, straight tracks

Figure 4-29 shows the degradation trends for four straight track segments. The graph indicates when the standard deviation of longitudinal level  $\sigma H$  will exceed the

threshold value (1,9 mm). The value for  $\sigma_H$  have already been exceeded for the track segment km 6,0 – 6,2.

As supported by the literature review, other factors than traffic conditions and curvature affect the degradation rate, causing some track segments to deteriorate significantly faster than others. Different superstructure types (e.g. rail profile, sleeper types, rail type), superstructure age, soil type, ballast condition and number of tamping actions performed since last renewal are some of the factors that additionally affect the infrastructure behaviour.

The tale of the distribution (Figures 4-26 and 4-27) indicates that many track segments have a very high degradation rate. These segments require a more frequent monitoring, since measurement campaigns for Ofofbanen are only carried out twice a year, with intervals up to 283 days (or approximately 24 MGT).

A more advanced degradation model could be developed using multiple regression analysis. This technique requires two or more independent variables (or explanatory variables). The literature review supports that tamping actions damage the track ballast. It would therefore be advisable to analyze the relationship between the number of tamping actions carried out on each track segment and the standard deviation of longitudinal level  $\sigma_H$ .

## 5 CONCLUSION AND FUTURE WORK

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The rail industry has experienced an important technological breakthrough the last few years. The Norwegian Railway Infrastructure Manager could benefit even more from the sensor technology and advanced analytics solution available at the time.

Data collected from the track recording vehicle (TRV) have mainly been used to perform corrective maintenance when predefined thresholds levels are exceeded.

A case study for the Norwegian heavy haul line was carried out to propose how decision makers can take more advantage of numerical data from track geometry measurements. By analysing records from different measurement campaigns together with maintenance history, one could conclude that some track segments require repeated tamping operations. Abrupt change of vertical track stiffness, insufficient thickness of the substructure and existing frost insulations are some of the root causes of repeated track geometry problems.

A method to assess the effectiveness of tamping operations based on numerical data from track geometry measurements reveals that the desired performance levels not always have been achieved. One possible explanation is that the track does initially achieve the desired performance level after the tamping action, but rapidly deteriorates again due to bad ballast or soil conditions.

Another finding from the case study is that track segments with similar superstructure conditions and traffic load have different degradation rates. This might be explained by the unknown ballast, drainage and soil conditions.

Predictive maintenance requires monitoring not only track geometry, but also the ballast and subsoil condition. GPR (Ground Penetrating Radar) technologies have been successfully used by many European Infrastructure Managers to inspect ballast fouling and drainage conditions. Thus, a more reliable condition assessment can be achieved.

Handling and processing data from different sources together (track geometry, traffic density, maintenance history, subsoil and ballast condition, among others) require advanced methods and tools to manage large amounts of data.

Bane NOR should reassess its method of accessing track geometry quality. It should be an important aspect to reflect the latest research within the field. Methods that combine alignment, longitudinal level and twist (or cross level) have shown promising results. An innovative approach to the evaluation of ballast and substructure conditions by fractal analysis of vertical alignment implemented by the Austrian Federal Railways is also worthy of consideration.

Finally, raising competence in track degradation mechanisms, together with advanced analytics capability will remain important elements. Collecting even more data is useless without the skills to interpret them.

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