MAINLINE

MAINtenance, renewal and Improvement of rail transport iNfrastructure to reduce Economic and environmental impacts

Collaborative project (Small or medium-scale focused research project)

Theme SST.2011.5.2-6.: Cost-effective improvement of rail transport infrastructure

Deliverable 2.1:
Degradation and performance specification for selected assets

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PP	Restricted to other programme participants (including the Commission Services)	
RE	Restricted to a group specified by the consortium (including the Commission Services)	
CO	Confidential, only for members of the consortium (including the Commission Services)	

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Glossary

Abbreviation / acronym	**Description**
DoW	Description of Work
WP	Work Package
IM	Infrastructure Manager
ASR	Alkali-Silica Reaction
LCC	Life Cycle Cost Study
LCA	Life Cycle Analysis
R&D	Research and Development
DoW	Description of Work
WP	Work Package

1. Executive Summary

1.1 Scope

The majority of existing European railway civil engineering infrastructure was designed and built long before the modern concept of 'design life' was developed in connection with limit state and reliability based codes. Earthworks in particular, are largely required to have virtually infinite life, as replacement without massive disruption is extremely difficult, if not impossible. It is a tribute to the 19th century engineers that a large proportion of this old infrastructure is still giving satisfactory performance, despite sometimes unsympathetic maintenance interventions. The reasons behind this will be explored as part of the work in Work Package 2 (WP2), which aims at the enhancement of degradation and performance models to provide a step change in the safe, cost effective management and/or life extension of these assets.

This deliverable specifies degradation scenarios and performance states for selected assets to be given special focus in the MAINLINE Project. **The aim is to select assets, corresponding degradation scenarios and performance states that will optimise the outcome of the MAINLINE Project taking into account the current 'state of the art' and the potential step change technologies.** This comprises the following tasks:

- Benchmarking the project focus area as defined in the Description of Work (DoW) through questionnaires to European Railway Infrastructure Managers (IM);
- Selection of specific assets and corresponding degradation scenarios;
- Definition of associated limit states throughout the asset life;
- Classification of degradation scenarios according to their time variability and expected consequences;
- Examination of limit states with respect to both extreme and cyclic load cases;
- Inclusion of effects from load evolution due to changes in rail traffic composition and frequency

The authors of this report have used their best endeavours to ensure that the information presented here is of the highest quality. However, no liability can be accepted by the authors for any loss caused by its use.

1.2 Basis

Three projects in particular constitute the baseline for this deliverable:

- INNOTRACK, Innovative Track Systems 2009 - a project within EU FP6, www.innotrack.eu;
- Sustainable Bridges - Assessment for Future Traffic Demands and Longer Lives 2007 - a project within EU FP6, www.sustainablebridges.net;
- UIC project on 'Monitoring track condition to improve asset management' 2010, ISBN 9778-2-7461-1871-3

Moreover, partners have drawn from their experience in undertaking various R&D projects and in managing a wide variety of railway assets in their respective countries.

1.3 Important definitions

The following definitions are highlighted:

Degradation is a general term covering the loss of performance of an asset, whether measured in terms of visual appearance or reduction of functionality (i.e. the ability to carry out its design requirements without restriction).

Deterioration is degradation caused by natural processes or through legitimate use of the asset. For example, this can take the from of rusting or fatigue[1] in metallic structures, rebar corrosion (whether carbonation or chloride induced) or spalling in reinforced concrete structures and loss of mortar or spalling in masonry structures.

Damage is degradation caused by accidental events such as collisions, overloading or design and construction errors.

Performance is the ability of an asset to meet its requirements over time. According to EN 1990 these requirements are related to safety, serviceability and durability.

Performance profile is a time dependent representation of the performance of an asset. Performance is described through performance indicators such as live load capacity, safety margin, reliability index etc. Performance profiles can be benchmarked against an acceptable level of performance.

Asset covers in this report all railway civil engineering structures, earthworks and tracks, i.e. all earthworks, bridges, tunnels, culverts, retaining walls and tracks (incl. rails, sleepers, ballast, switches and crossings).

Old vs. new assets - For bridges, traditional types of construction, utilising masonry, ferrous metals (now steel, but also including cast and wrought iron) and concrete are regarded as old whilst non ferrous metals (mainly aluminium) and fibre reinforced polymers (FRP) are regarded as new. Similar considerations apply to tunnels, where modern construction methods such as the New Austrian Tunnelling Method are considered as new. For plain line track, slab track is regarded as new, as are switch and crossing installations on concrete slabs.

Furthermore, overhead lines, signals and over passing road bridges are not part of the MAINLINE scope.

1.4 Target reader

The target reader of this deliverable is a civil/structural engineer experienced in assessment and/or inspection of railway infrastructure elements.

1.5 Results

First, this deliverable describes some key findings from the recently completed European projects regarding maintenance and renewal of both track (INNOTRACK) and bridge assets (Sustainable Bridges) that constitute the background and motivation of the MAINLINE project.

1 Fatigue induced cracking is commonly referred to as “fatigue damage”; however fatigue is a deterioration mechanism rather than a damage mechanism, hence its inclusion above as an example of deterioration.

Second, selection of assets according to the Description of Work is benchmarked through the response of Infrastructure Managers to a questionnaire that has been devised specifically for the purposes of this work package.

The following asset types are identified as focus areas due to a high probability for knowledge increase within a 3 year period and with useful validation data being available:

- Cuttings,
- Metallic bridges,
- Tunnels with concrete and masonry linings
- Plain line (total track superstructure) and switches and crossings[2]
- Retaining walls

Third, the above asset types, their degradation and associated performance and limit states as well as their temporal and spatial characteristics are addressed in individual chapters. Also, their sensitivity to load evolution and effects of climate change are addressed.

Other asset types, their degradation and performance and limit states are more briefly addressed in the individual chapters. However, for some of these other asset types quite extensive guidance is provided, e.g. concrete bridges where allocation of maintenance resources is an emerging challenge.

Each asset type is addressed in the chapters as listed below:

- Chapter 6, Earthworks with special focus on cuttings.
- Chapter 7, Bridges with special focus on riveted metallic bridges.
- Chapter 8, Tunnels with special focus on lined tunnels.
- Chapter 9, Track with special focus on plain track and switches and crossings
- Chapter 10, Other structures with special focus on retaining walls, coastal and river defences and culverts.

Appendix A compiles response to questionnaires divided into the above asset types.

[2] It seems that track degradation and performance models need further additions to answer main questions within the MAINLINE project. As track models are basis for switches and crossing models (incl. more components and parameters), we propose to start with track models and then concentrate on switches and crossings.

2. General remarks

The project 'MAINtenance, renewaL and Improvement of rail transport iNfrastructure to reduce Economic and environmental impacts' (in short MAINLINE) is an integrated project within the EU's 7th Framework Programme. It has been partly funded on the basis of the contract SST.2011.5.2-6 between the European Community represented by the Commission of the European Communities and International Union of Railways (UIC) acting as coordinator for the project.

The main objectives of the project are:

- Apply new technologies to extend the life of elderly infrastructure
- Improve degradation and structural models to develop more realistic life cycle cost and safety models
- Investigate new construction methods for the replacement of obsolete infrastructure
- Investigate monitoring techniques to complement or replace existing examination techniques
- Develop management tools to assess whole life environmental and economic impact

The present report D2.1 - Degradation and performance specification for selected assets has been prepared within the work package WP2 during the first 4 month period of the MAINLINE project. WP2 is named 'Degradation and structural models to develop realistic life cycle cost and safety models', one of the eight work packages (WP1-WP8) dealing with relevant tasks for maintenance, renewal and improvement of rail transport infrastructure to reduce economic and environmental impacts.

An overview of the general organization of the project is presented below together with the list of all the partners in work package WP2:

General organization of the project

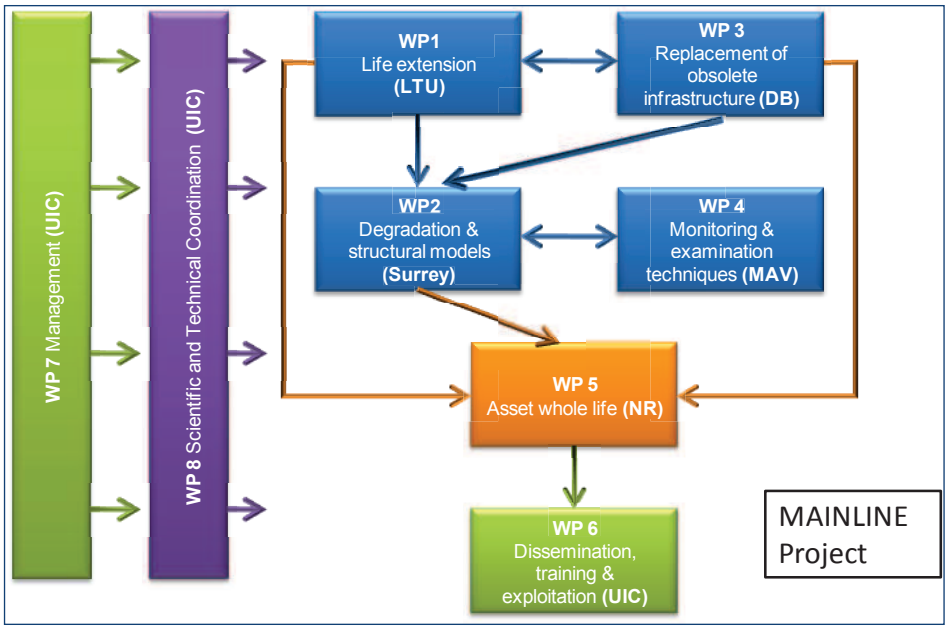

Figure 1: General organisation of the project.

UIC, Surrey, MAV and NR are listed below. DB is Deutche Bahn AG (Germany) and LTU is Luleå Tekniska Universitet (Sweden).

Part n°	WP2 Partners	Country
1	UNION INTERNATIONALE DES CHEMINS DE FER - UIC	FR
2	NETWORK RAIL INFRASTRUCTURE LTD	UK
3	COWI A/S	DK
4	MOUCHEL LIMITED	UK
5	UNIVERSITY OF SURREY	UK
6	TWI LIMITED	UK
10	MAV MAGYAR ALLAMVASUTAK ZARTKORUEN MUKODO RT	HU
12	TECHNISCHE UNIVERSITAET GRAZ	AT
17	SERVICE D'ETUDES TECHNIQUES DES ROUTES ET AUTOROUTES	FR

The main objective of work package WP2 'Degradation and structural models to develop realistic life cycle cost and safety models' is:

- To identify and model important degradation phenomena and processes for selected railway assets for the purpose of LCC and LCA analysis.
- To quantify the influence of intervention strategies on degradation time profiles.
- To develop performance time profiles for selected asset types.
- To validate the developed degradation and performance models through case studies.

WP2 interacts with WP1, WP4 and WP5. The interaction with WP1 consists of inputs for degradation and performance models that will be developed within WP1 and will also be utilised within WP2. The two-way interaction with WP4 is focused on identifying model parameters for the degradation and structural models that would benefit (in terms of the confidence with which they can be specified in models) through monitoring and examination. The main outputs from WP2 in terms of time-dependent performance profiles will be passed on to WP5.

3. Acknowledgments

This present report has been prepared within work package WP2 of the MAINLINE project by the following team of contractors with COWI as task leader and report coordinator:

- Union Internationale des Chemins de fer (UIC), France
- Network Rail (NR), United Kingdom
- COWI A/S (COWI), Denmark
- Mouchel Limited (Mouchel), United Kingdom
- University of Surrey (Surrey), United Kingdom
- TWI Limited (TWI), United Kingdom
- MAV Magyar Allamvasutak Zartkoruen Mukodo RT (MAV), Hungary
- Technische Universitaet Graz (TUGraz), Austria
- Service d'Etudes Techniques des Routes et Autoroutes (SETRA), France

Part of this report is based on the outcome of questionnaires sent out to Infrastructure Managers (IM's) within the project and through UIC. The following IM's have generously provided the project with some very useful information:

- Deutche Bahn AG (Germany)
- Mav Magyar Allamvasutak Zartkoruen Mukodo RT (Hungary)
- Network Rail Infrastructure Ltd (United Kingdom)
- Trafikverket (Sweden)

In addition, TU Graz has kindly provided input specifically on track assets.

Finally, the guidance of the scientific leader Professor Lennart Elfgren of Luleå University of Technology is greatly appreciated.

4. Introduction

4.1 Background and motivation

The demand for rail transportation across Europe continues to grow, and growth is predicted to continue unabated into the future. Midterm projections by the EC, COM (2006)[3], expect rail freight and passenger traffic to rise by around 13% and 19% respectively between 2000 and 2020. Whilst some of this growth will be met, particularly in Western Europe by the building of new high speed lines, much of it will have to be accommodated on existing lines which contain some of the oldest transportation infrastructure now in regular use for long distance journeys. For instance it is estimated that some 35% of the railway bridges in Europe are in excess of 100 years old and major earthworks (cuttings and embankments) and tunnels date back to the original construction of the route, possibly in excess of 150 years ago.

The MAINLINE project will draw on the considerable progress made regarding maintenance and renewal of both track (INNOTRACK) and bridge assets (Sustainable Bridges) within recently completed European projects. The following statements highlight selected background information from these projects as motivation.

Figure 2 presents the age distribution of railway bridges in 16 EU countries (Austria, Belgium, Czech Republic, Denmark, Ireland, Finland, France, Germany, Hungary, Italy, Poland, Portugal, Slovakia, Spain, Sweden, United Kingdom) and in Switzerland. This figure has been based on the results of a survey performed among the railway administrations in the above mentioned countries in the EU FP6 project Sustainable Bridges [2]. The result of the survey contains around two hundred and twenty thousand records corresponding to various types of bridges constructed from different materials, including concrete bridges, metal bridges, masonry arch bridges and composite (steel-concrete) bridges.

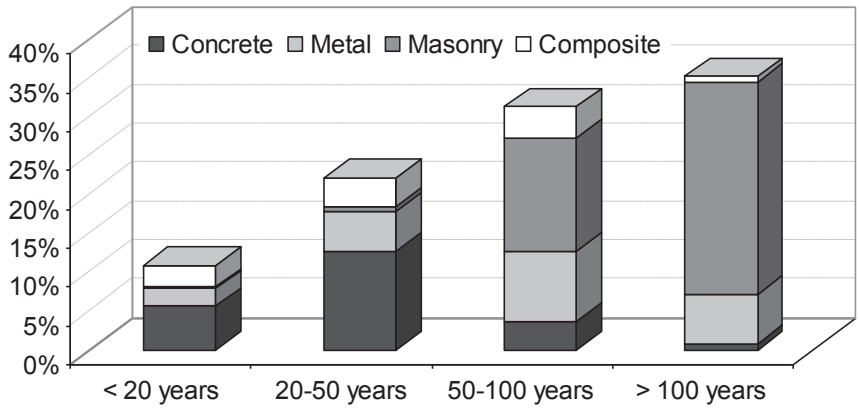

Figure 2: Age distribution of bridges considering bridge construction materials [2].

3 COM (2006): Keep Europe moving. Sustainable mobility for our continent. Midterm review of the European Commission´s 2001 White Paper, see: http://eur-lex.europa.eu/LexUriServ/LexUriServ.do?uri=COM:2006:0314:FIN:EN:PDF

The Sustainable Bridge project also addresses degradation mechanisms that are relevant to railway bridges, see [3].

For railway track components, infrastructure managers from eight EU countries (Spain, Austria, Czech Republic, France, Germany, Sweden, United Kingdom and Netherlands) have, as part of the EU FP6 project INNOTRACK [1], ranked track problems and causes based on their cost impact. They are listed below in descending order of importance:

Track problems	Causes
Rail: cracks and fatigue	creep forces
Rail: cracks and fatigue	bad wheel/rail interface
Track: bad track geometry	soft sub-structure/bad drainage
S+C: wear in switches	sub-structure
Rail: corrugations	vehicle/track interaction
S+C: cracked manganese crossings	weld quality
S+C: geometry maintenance	optimal maintenance regime?
Sub-structure: unstable	soft sub-structure/wet bed
Track: bad track geometry	sub-optimal maintenance
Track: bad track geometry	wrong/unknown stress free temperature

In the same project, the following were identified as the most important maintenance cost drivers: welding, grinding, inspection, rail lubrication, track renewal, sub-grade, geometry, drainage and minor periodic maintenance.

Furthermore, a UIC project on 'Monitoring track condition to improve asset management' [4] summarises input from Infrastructure Managers from nine EU countries (Portugal, Sweden, Norway, Czech Republic, United Kingdom, Italy, Ireland, Hungary and Belgium) with respect to which components and systems influence reliability, availability, maintainability, safety and cost. They are listed below in descending order of importance and allocated percentages ( ).

Reliability	Availability	Maintainability	Safety	Cost
Switch motor (25)	Switch, rail mechanics (25)	Switch, rail mechanics (40)	Switch, rail mechanics (35)	Rail head (40)
Switch, rail mechanics (20)	Insulation joints (20)	Ballast (25)	Insulation joints (20)	Switch, rail mechanics (15)
Insulation joints (15)	Rail head (15)	Rail joints and welds (15)	Rail head (15)	Ballast (15)
Rail head (10)	Switch motor (15)	-	Switch motor (10)	Switch motor (10)
Subgrade (10)	Rail joints and welds (10)	-	Subgrade (10)	-
Ballast (10)	Ballast (5)	-	Sleeper (10)	-

The predicted increase in traffic on existing elderly rail infrastructure across Europe will result in increased rates of degradation for the civil engineering and track assets concerned, which will then need to be maintained, and when necessary, replaced in as short a time as possible so that freight and passenger traffic is disrupted as little as possible.

Balancing the economic and environmental costs generated by the maintenance and renewal of existing infrastructure is far more difficult than it is when building new especially as it is necessary to minimise the risks to the continued safe operation of the transport network. The residual life of each historic asset will vary and is difficult to predict. It is also necessary to balance the natural desire for lower first cost solutions, particularly in the current economic downturn, with the real need to achieve the lowest life cycle cost. The development of a tool to assist railway administrations to balance these various considerations is the principal objective of the MAINLINE project.

The safe adoption of LCC and LCA analyses must be underpinned by a time-variant approach that quantifies the potential performance loss in the asset over a relevant time period. In turn, performance loss (typically captured through reduction in limit state margins such as durability, serviceability and structural strength/capacity), is determined by the manner (e.g. local or global) and the severity in which degradation processes impact on railway assets. These processes are typically caused through a combination of environmental aggressors and various forms of loading acting on the structures; their effects can be mitigated through intervention actions which take place at different points in time during the life span of the asset. The overall aim of this WP is to develop and validate the required degradation and performance time-dependent models, including appropriate intervention strategies for realistic LCC and LCA analysis to be carried out in WP5.

4.2 Objective and scope

The objective of this deliverable is to determine the specific railway assets and operational scenarios that will be analysed in WP2, i.e. it will review the original project focus areas as described in the Description of Work and ensure that assets dealt with in detail have a high probability for knowledge increase within a 3 year period and that useful validation data is also available. This includes the following tasks:

- Benchmarking the original project focus area as defined in Description of Work through questionnaires to European railway Infrastructure Managers;
- Selection of specific assets and corresponding degradation scenarios;
- Definition of associated limit states throughout the asset life;
- Classification of degradation scenarios according to their time variability and expected consequences;
- Examination of limit states with both extreme and cyclic load cases;
- Inclusion of effects of load evolution due to changes in rail traffic composition and frequency

The above tasks are performed with due attention to the overall aim of WP2 that is to develop and validate the required degradation and performance time-dependent models, including appropriate intervention strategies for realistic LCC and LCA analysis to be carried out in WP5.

As indicated above, asset selection will be aided by the response of Infrastructure Managers to a questionnaire that has been devised specifically for the purposes of this work package. Where possible, references will be given for assets that have not been selected for detailed examination within this work package.

To meet the objective, in most of the topics related to degradation and performance of railway infrastructure the use of the current state-of-the-art knowledge and the presently best practice is sufficient. Therefore, some parts of the report may reference the best practical experience and know-how of all the partners involved. In other areas there is a need for innovative methods, models and tools as highlighted in this report.

4.3 References

[1] INNOTRACK, Innovative Track Systems (2009) - D1.4.6: A report providing detailed analysis of the key railway infrastructure problems and recommendation as to how appropriate existing cost categories are for future data collection - a project within EU FP6. www.innotrack.eu

[2] Sustainable Bridges (2004), European railway bridge demography, Background document D1.2. Prepared by Sustainable Bridges - a project within EU FP6 www.sustainablebridges.net

[3] Guideline for Load and Resistance Assessment of Existing European Railway Bridges (2007), Sustainable Bridges www.sustainablebridges.net

[4] UIC (2010), Monitoring track condition to improve asset management, ISBN 978-2-7461-1871-3

5. Selection of assets

5.1 Project focus area according to Description of Work (DoW)

5.1.1 Classification of assets

Railway civil engineering structures and track can be broadly classified as:

- earthworks (embankments, cuttings, etc),
- bridges,
- tunnels,
- other civil structures (culverts, retaining walls and coastal/river defences),
- track (rail, sleepers, ballast, switches and crossings etc)

Each of these structures can be further sub divided in a number of different ways. In considering the state of the art it has been necessary to decide on which sub divisions should be regarded as “old” and hence relevant to the MAINLINE scope, and those which are “new” as described below. This distinction has been made largely on the basis of the small numbers of “new” assets actually in service, which means that the level of confidence in deterioration data is very low and little information relating to maintenance and repair will be available.

For bridges, traditional types of construction, utilising masonry, ferrous metals (now steel, but also including cast and wrought iron) and concrete are regarded as old whilst non ferrous metals (mainly aluminium) and fibre reinforced polymers (FRP) are regarded as new. Similar considerations apply to tunnels, where modern construction methods such as the New Austrian Tunnelling Method are considered as new. For plain line track, slab track is regarded as new, as are switch and crossing installations on concrete slabs.

5.1.2 Degradation mechanisms

The level of knowledge of the factors that determine the rate of deterioration and damage (herein referred to as degradation), and hence life span, for each component varies considerably and can be very different for different types of the same basic component of the infrastructure. For civil engineering infrastructure, rates of deterioration can be very dependent on both general- and microclimate, so there are considerable variations across the different climates encountered in Europe, whilst track degradation is far more closely related to traffic tonnage and is likely to be similar for similar situations across Europe.

Not only are there current variations in climate across Europe, but future climate change effects are likely to alter the picture. For example, it could be that the limiting degradation mechanisms on certain assets will actually change. These possibilities will be taken into account when developing predictive tools. Climate change factors that could substantially alter asset life predictions include:

- warmer, wetter winters,
- hotter, drier summers,
- increased flooding and rising sea levels,
- more extreme weather events.

The MAINLINE partners have drawn up Table 1 in the DoW which lists the major rail assets described above and ascribes a score to a number of factors relevant to research into deterioration/damage models. Because there are different levels of knowledge between

Western Europe and Eastern Europe and the developing economies, the values quoted in table 1 have been simplified to 2, 5 and 8, with (in terms of either existing knowledge or the chance of a successful research outcome) 2 meaning poor, 5 meaning fair and 8 meaning good.

Component of elderly rail infrastructure	Relevant degradation mechanisms	Current knowledge level	Potential to increase knowledge	Success in 3 years Deterioration models	Availability of validation data
Earthworks					
Sub grade (natural ground)	Bearing capacity failure, lack of shear strength, settlement	8	2	5	5
Cuttings	Soil Erosion, Creep deformation, Stability Rock Erosion, Freeze/thaw	5	8	5	5
Embankments	Erosion, Creep deformation, Stability	5	8	5	8
Bridges					
Masonry	Freeze/thaw, Water percolation, Foundation settlement, Deformation, Fatigue	5	8	2	5
Metallic	Corrosion, Fatigue, Cracking, Dynamic effects	8	8	5	8
Concrete (reinforced, pre-stressed or post tensioned)	Concrete Carbonation, Sulfate attack, Chloride ingress, Alkali silica reaction, Freeze/thaw Reinforcement General (carbonation) corrosion, Pitting (chloride induced) corrosion, Fatigue	8	5	5	8
Tunnels					
Unlined	As for rock cuttings	5	5	2	5
Masonry lining	As for masonry bridges	5	5	2	5
Concrete lining (including sprayed)	As for concrete bridges	8	5	2	5
Metallic lining	As for metallic bridges	8	5	2	5
Other structures					
Retaining/sea walls (masonry, concrete, steel sheet piling)	Stability (plus those for specific materials mentioned above)	5	5	5	5
Drainage / Culverts	Blockages, Crushing due to overloading. For culverts also same as for bridges	8	5	5	5
Track					
Ballasted plain line (rail, sleepers and ballast)	Wear; Also same as for bridges	8	5	2	5
Switches & crossings	Mechanical wear, Dynamic effects	5	8	5	5

Table 1: Degradation models

Estimation of current knowledge and potential for improvement according to DoW. Values express the chance of successful research outcome (2=poor, 5=fair and 8=good).

As the current knowledge of whole life environmental performance is at a far lower level than that for deterioration, and is only just emerging from academia into industry, Table 5.1 has been used to determine the areas in which it is proposed to focus the research within the project.

5.1.3 Recommendations according to the DoW

Based on the interpretations in Table 5.1, the DoW proposed to focus the major part of its effort in addressing embankments, metallic bridges, unlined tunnels and switches and crossings. This effort will be mainly concentrated on providing the data necessary to develop the whole life environmental and economic assessment tool called for in WP5. However, the other asset types shown in the table will not be ignored and the project will ensure that the major project output will be applicable to as many of these other assets as possible. For example, development of deterioration models to account for spatial effects could be applied

to both concrete and metallic bridges, provided that the relevant input and propagation parameters are captured for each distinct case.

In addition, the project will ensure that the other call themes are covered through a rigorous appraisal of the international state of the art for each specific area and the preparation of application guidance specifically aimed at railway administrations. These are discussed in more detail in section 5.2 below.

MAINLINE is continuously in contact with the FP7 project SMART Rail, accepted through the same call. The work in SMART Rail will complement that in MAINLINE and both projects have agreed to share information and minimise duplication of effort. During preparation of this deliverable it has been confirmed by the SMART Rail coordinator that they will be focussing on embankments and bridge scour.

5.2 Benchmarking through questionnaire

Benchmarking of focus areas identified in DoW has been performed based on a questionnaire sent out to infrastructure managers within the MAINLINE project as well as UIC at the end of 2011. Network Rail Infrastructure Ltd (United Kingdom), Deutche Bahn AG (Germany), Mav Magyar Allamvasutak Zartkoruen Mukodo RT (Hungary) and Trafikverket (Sweden) have kindly shared their experience answering the questionnaire. In addition, Technische Universitaet Graz (Austria) has kindly provided input specifically on track assets.

All answers are compiled for each asset type in Appendix A. In Table 5.2 the number of responses for each component is shown.

Asset type	Number of responses
Earthworks	3
Bridges	4
Tunnels	3
Other structures	1
Track	4*

Table 2: Number of responses to questionnaire.

****Answer from track specialists at Technische Universitaet Graz (Austria) has been included as their expertise has not been included in the identification of focus areas in DoW.***

Due to the sparse number of answers this deliverable is limited to benchmarking only.

For each of the 5 asset types the following four questions from DoW are benchmarked if possible:

- Current knowledge level
- Potential to increase knowledge
- Success in 3 years research outcome
- Availability of validation data

In the following sections each of the asset types is treated in a somewhat greater detail.

The questionnaire also provides information about important degradation mechanisms, primary aggressor, inspection methods, time for imminent failure development, trigger points (thresholds) for intervention, costs, assessment codes etc. Answers to these questions are not treated in the following sub sections but the questionnaires provide significant information to the later chapters of this report and throughout the MAINLINE project. Refer to Appendix A for the response to the questionnaire.

5.2.1 Earthworks

The variability in answers related to sub soil (natural ground) seems rather high. In general track geometry is recorded periodically, accessible inspection and monitoring data is sparse and standards/guidelines for conditions assessment such as e.g. Ril 853.8001 and NR/SP/TRK/9030 exist. This substantiate that the current knowledge level is good and potential to increase knowledge within the project period might be poor.

Cuttings and embankments are to some extent exposed to the same deterioration mechanisms (e.g. erosion, creep deformation and loss of stability). The potential to increase knowledge seems to be good for both components but answers to questionnaires reveal that available inspection and monitoring data might be greater for embankments.

As the SMART Rail project will be focusing on embankments, cuttings are chosen for further studies within the MAINLINE project. However, embankments and sub soil may be revisited at a later stage of the project if appropriate.

5.2.2 Bridges

Response to the questionnaire reveals that the current knowledge level is quite high for all bridge types compared to some of the other asset types.

There is a rather high variability in the fraction of each bridge type within the bridge stock of each country in the EU. It is however evident that the majority of old bridges are metallic or masonry. On the other hand, as shown in Figure 2, a vast number of 20-50 year old concrete bridges are in service. It is foreseen that allocation of maintenance resources for concrete bridges is an emerging challenge.

In general, maintenance costs per bridge related to metallic bridges seem to be greater than those of masonry and concrete bridges. Part of the reason could be that metallic bridges are in general older than concrete bridges.

Due to significant ongoing work on degradation models for masonry bridges and the relative limited available work, it is suggested that during the forthcoming three year period the possibility of a step change is poor for masonry bridges; therefore metallic bridges are chosen for more detailed studies within the MAINLINE project.

Masonry and concrete bridges may be revisited at a later stage of the project if appropriate.

5.2.3 Tunnels

Response to the questionnaire reveals that all 3 countries have masonry and concrete lining, 1 of the 3 countries have metallic lining and only 2 of the 3 countries have unlined tunnels. Although the response is limited, it suggests that focus is directed towards lined tunnels.

There is some variability in the response but one infrastructure manager indicates that imminent failure develops more rapidly for concrete linings than for masonry linings. On the other hand another infrastructure manager indicates that degradation of masonry lining is more costly and has the greatest lack of knowledge when compared to concrete lining.

Based on the above statements and due to the fact that masonry and concrete linings experience some of the same deterioration mechanisms focus at this stage of the project should be on both types on lining.

Unlined and metallic linings may be revisited at a later stage of the project if appropriate.

5.2.4 Track

Response regarding plain line (i.e. total track superstructure), sleepers, ballast and switches and crossings indicate that the greatest lack of knowledge is related to switches and crossing. Furthermore, availability of validation data seems good.

On the other hand, it seems that track degradation and performance models need further additions to answer main questions within the MAINLINE project. As track models are basis for switches and crossing models (incl. more components and parameters), we propose to start with track models and then concentrate on switches and crossings.

5.2.5 Other structures

Retaining walls, coastal/river defences and culverts are identified as ‘other structures’.

It is noted that only one infrastructure manager (MAV) has provided a response to the questionnaire related to 'other structures'. Based on this response it seems as if retaining walls and coastal/river defences have the most costly degradation mechanisms. To a large extent the knowledge level is the same for both components. However, it seems that availability of evaluation data is better for retaining walls.

Based on the above statements focus should be directed towards retaining walls.

Coastal/river defences and culverts (masonry and concrete) may be revisited at a later stage if appropriate.

5.2.6 Monitoring and examination techniques

Monitoring and examination techniques/systems pertaining to the selected assets as mentioned in the answers to the questionnaire involve:

- *Embankments*: Visual walk-over inspections, inclinometer readings, displacement meter readings, geodetic measurements.
- *Metallic bridges*: Visual inspections, dye penetrant testing, ultrasonic testing (uncommon), depth gauges, deformation measurements by extensometers, measurements by laser scanning, bond tests, hammer tapping, use of torque spanner, visual inspections supported occasionally by Non-Destructive Testing (NDT)/ partially destructive testing, emerging electromagnetic techniques, acoustic emission monitoring.
- *Concrete and masonry lined tunnels*: Visual and tactile examinations, Periodic inspections as per relevant manuals.
- *Switches and crossings*: regular inspections in relation to loading, periodic visual inspections, recording car, Ultrasonic NDT, Switch blade deflector.
- *Retaining walls*: Visual inspections, geodetic surveys.

The above list made from response to a questionnaire is not exhaustive.

WP4 of the MAINLINE project will examine these techniques in detail to investigate how these and other monitoring and examination systems can provide suitable inputs to degradation models being considered in the project. WP4 will build on work done on two Framework projects *Sustainable Bridges* and *Innotrack*. Other sources of state of the art

knowledge include: reports generated by the UK's Rail Research & Safety Board (RSSB) - for example, reports T844 [1] and T853 [2] that examine remote condition monitoring IT system architecture across the rail industry to determine if they are being utilised optimally; and a report on the French national research project – MIKTI [3].

5.3 Recommendation of assets to be considered

Based on benchmarking of the DoW focus areas the following asset types are identified as the key focus areas:

- Cuttings,
- Metallic bridges,
- Tunnels with concrete and masonry linings
- Plain line (total track superstructure) and switches and crossings
- Retaining walls

The above asset types and their degradation and performance characteristics will be specified in the following chapters. Other asset types and their degradation and performance characteristics will be briefly specified in the following chapters with due reference to state-of-the-art.

5.4 References

[1] T844 (2009), Mapping current remote condition monitoring activities to the system reliability framework, RSSB
http://www.rssb.co.uk/SiteCollectionDocuments/pdf/reports/Research/T844_rb_final.pdf

[2] T853 (2010),Remote condition monitoring IT survey, RSSB
http://www.rssb.co.uk/RESEARCH/Lists/DispForm_Custom.aspx?ID=907

[3] MIKTI 2010, Steel-concrete composite bridges, National Project, ENPC press, Mama-la-Vallée France

6. Earthworks

6.1 Asset types

Railway earthwork structures are built in order to achieve vertical alignment of the track. There are typically four types of asset within the earthworks element class, namely:

- Natural ground
- Embankments
- Soil cuttings
- Rock cuttings

In locations where the natural land lies flat and at the required height, the track and foundations can be constructed directly on the natural ground. Embankments are built where it is necessary to raise the level of the track above the level of the natural ground. This is achieved by placing materials, most commonly locally excavated soil or rock, on top of the natural ground. Cuttings are required where the track needs to pass through high ground at a lower level. Cuttings consist of excavations in the natural ground, which may be either soil or rock, with track laid at the foot of the side slopes. Where track corridor runs along sidelong ground it may be supported by an embankment on one side and a cutting on the other.

Prevalence

Earthworks form a significant proportion of many railway infrastructure networks. In the UK for example, there are approximately 8,762km of embankments, 5,763km of soil cuttings and 495km of rock cuttings [7]. This gives a total of 15,020km of earthworks (excluding natural ground) across the total track length of 17,600km, representing 85% of the network. Earthworks will be most prevalent in regions with closely spaced land contours.

Composition materials

Earthwork construction materials are variable in nature, and largely dependent on regional geology. Cuttings are man-made slopes through natural ground, and therefore consist of the naturally occurring strata at that location. Modern embankments may be constructed from specially selected fill materials, however they are most commonly constructed from locally sourced materials and their composition is therefore also dependant on regional geology. The European Environment Agency published a map showing the main types of soil found in Europe (See Appendix B). This shows that there are a wide range of soils that feature across the different European regions.

As is true for any other engineering structure, the engineering properties and deterioration mechanisms are different depending on the construction material used.

Age

In the UK the majority of earthworks were constructed between 100 and 150 years ago, however the age of earthworks across Europe will vary depending on the age of the network. Figures published by the European Environment Agency show that the total length of traditional railway infrastructure has actually decreased slowly over recent years, suggesting that newly built railway earthworks are uncommon (See Appendix B Figure B-3). However the figures also show that a large number of new high speed rail networks have been constructed over the past two decades; and many new earthworks will have been constructed as part of these new networks. The newly constructed earthworks will have been designed to resist modern loading conditions and may have been built using new construction techniques and materials.

6.2 Damage and deterioration mechanisms

The four asset types may deteriorate in different ways. Furthermore, a particular asset type may deteriorate in different ways depending on its composition. The following sections will outline the main damage and deterioration mechanisms for each asset type, for a range of soil and rock types, identifying their temporal and spatial characteristics as well as sensitivities to load evolution and climate change where applicable.

6.2.1 Natural ground

Track founded on natural ground can deteriorate as a result of deformation and weakening of the underlying foundation material. The main deterioration mechanisms for natural ground track formations can be summarised as follows:

- *Reduced bearing capacity* – a result of reduced shear strength in cohesive soils caused by excessive pore water caused by groundwater. This kind of failure will be sensitive to climate change effects, as increased rainfall and rising sea water levels may cause excessive groundwater.
- *Settlement* – can occur where there are soft sub layers of soil for example peat and clay. There are several possible causes of subgrade settlement, as outlined by Selig and Waters [6]:
  - Consolidation from weight of added earth or from ground water lowering (or conversely swell from removal of earth or ground water rise)
  - Shrinkage and swelling from changes in moisture
  - Progressive deformation from cyclic train loading inducing compaction, consolidation and shear deformation. The DB questionnaire response reports that settlement deterioration develops with increasing load and traffic speed, so this mechanism is likely to be sensitive to load evolution.
- *Frozen subgrade* – cold temperatures can cause the subgrade material to freeze resulting in a change in strength and stiffness. Freeze heaving and thaw softening effects can also cause foundation instability. Increased frequency of extreme weather events could see an increase in freeze thaw deterioration.
- *Mining subsidence* – Foundation deformation can result from previous mining activities in the bedrock, even if the overlying strata are sound. The collapse of old mine shafts under or adjacent to embankments may cause local settlement which can be both sudden and catastrophic and require remediation work.

6.2.2 Embankments

Embankment failures can be broadly classified into two categories; shallow failures and deep seated failures, as illustrated in Figure 3 below. Shallow failures do not exceed 2m in depth and are commonly translational in nature. The rupture surface in deep failures exceeds 2m in depth and can take a variety of shapes depending on ground conditions. Whilst shallow failures are contained entirely within the side slopes of the embankment, deep failures can extend beyond the toe, affecting property and operations outside of the embankment land take.

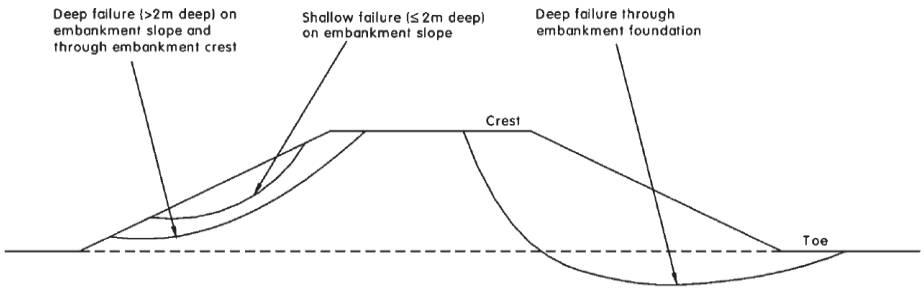

Figure 3: Examples of rupture surfaces for shallow and deep embankment failures [1]

All types of slope failure can be analysed in terms of the principles of soil mechanics. The most common model used for slope stability calculations is the classic limit equilibrium model. Figure 4 below contains an extract from the CIRIA Report C550 [1] which explains the basic soil mechanics principles that govern slope stability.

In the classic limit equilibrium model, which is used for most calculations of slope stability, the strength of the soil, and hence its resistance to failure, is given in terms of effective stress by:

$$\tau = c' + (\sigma_n - u) \tan \varnothing' \quad \text{(British Standards Institution, 1981)}$$

Where τ = the resistance to shear along an actual or potential rupture surface

c' = the cohesion of the soil in terms of effective stress

σ_n = the total normal stress on the rupture surface under consideration

u = the pore water pressure

$\varnothing'$ = the angle of shearing resistance with respect to effective stress

c' and $\varnothing'$ are referred to as the materials strength parameters.

The value of τ thus depends on the value of u. If u increases, the $(\sigma_n - u) \tan \varnothing'$ term will decrease, which may lead to failure of the slope in soils with low values of c' and $\varnothing'$. In soils with a high clay content, $\varnothing'$ may be lower than the angle of the slope and the influence of c' is large. For low-clay-content soils and granular materials, c' will be low or zero and the strength will depend principally on $\varnothing'$, especially if the material is free-draining.

Most over-consolidated clays, when freshly excavated, have negative pore water pressures, ie suctions, or low positive values. Embankments constructed with these materials will be stable in the short term (ten years) at slope angles of up to 1 (vertical) to 2 (horizontal) (1:2 or 26 degrees) if the material is well compacted, and 1:2.5 (22 degrees) to 1:3 (18.4 degrees) if loosely tipped. Hence, at the construction stage the embankment appears stable, although this stability is sometimes marginal and failures have occurred.

With time, the negative pore pressures decrease, or the positive pore water pressures increase as water percolates through the fill. This is largely restricted to the outer 1–1.5 m of the slope in well-compacted highway embankments and occurs deeper within loosely tipped railway and canal embankments. Softening of the clay occurs due to the ingress of water, and shrink-swell movements can occur in response to seasonal changes in moisture content and vegetation. These lead to progressive movement of clays and detrimental changes in the values of c' and $\varnothing'$, as the effects of over-consolidation on the clay are removed. These can lead to movement on a rupture surface. Once movement is initiated, the shear strength along the rupture surface will rapidly decrease towards a lower residual value.

Many old railway and canal embankments contain rupture surfaces and zones of failed material. The associated movements have been exacerbated by the addition of ballast or other material to the top of the embankment. They are in a metastable condition, and further movements can be initiated by changes in pore water pressure. A particularly critical time is when a period of intense rain follows a long dry spell. Monitoring of railway embankments has indicated that there may be a very rapid increase in pore water pressures after the wet spell begins. Ultimate limit state deep slope failures may be initiated by this rapid change. It is apparent, therefore, that pore water pressure has a critical role to play in slope stability. It is the most variable parameter and the one whose effect is most rapid.

For further information on the loss in performance of infrastructure embankments see Vaughan (1994) and Crabb and Atkinson (1991).

Figure 4: Soil mechanics principles for slope stability (extracted from [1])

There are several deterioration mechanisms which can contribute to poor slope stability and loss of embankment performance. The CIRIA Report C550 outlines the following principal causes of deterioration;

- *The presence of water* – The performance of an earthwork embankment is often related to rainfall, either in the form of short-lived storm events which can generate rapid failures, or long lasting events which modify ground water profiles. Increased pore water pressure can result in reduced shear strength and subsequent slope instability. The cycle of swelling during wet periods and shrinkage during dry spells can also cause deformations. Future climatic events might be different from those that earthworks

embankments have typically experienced to date, so the performance of embankments may change in the future.

- *Erosion* – Migration of rivers and streams can result in scour and undermining of embankments some time after their construction. Such deterioration may occur gradually over time, or more quickly as a result of a major flooding event. Increased flooding is predicted as a result of climate change which could see the rate and frequency of this type of deterioration increase (see Appendix B which shows predicted flooding risk across Europe over a 100 year return period). Where railway embankments have been constructed in waterfront areas coastal or river defences will have been constructed to protect from erosion deterioration (see Section 10 for more details on coastal and river defences)
- *Method of construction* – Failure commonly occurs at the interface between the natural ground and embankment fill. Railway embankments were not generally benched into the existing ground, and in many cases were constructed on top of the original topsoil. In these cases the interface with the top soil and embankment fill forms a potential rupture surface, along which a progressive failure could develop.
- *Foundation inadequacy* – The failure of an embankment could result from failure of the sub-grade material on which it is constructed. The possible foundation failure mechanisms are the same as those outlined for track founded on natural ground in section 6.2.1 above.
- *External factors* such as vandalism, erosion or burrowing animals can change the slope geometry or loading thus undermining the stability of an earthwork and causing deterioration. Another potential cause of major instability is excavation by a third party at the toe of an embankment, for example in a private garden which backs onto an embankment. Even if the excavation does not cause failure, movement is likely to occur which will lead to settlement of the crest and subsequent weakening of the embankment.
- *Surcharging* – Increases in loading from vehicles using low embankments (typically less than 2m high) can cause compression of fill or foundation materials resulting in deformations. Load evolution will see the introduction of high-speed trains on railway embankments which may produce dynamic effects that old embankments were not designed to resist.
- *Failure of supporting structures* – failure of structures adjacent or integral to the embankment could cause local instability. Examples might include the collapse of an underbridge or culvert or the failure of a remedial works such as retaining walls and soil nails.
- *Vegetation* – trees, shrubs and grasses cause seasonal changes in soil moisture content which augmenting the magnitude of the shrink swell cycle. In the case of newly planted vegetation, poor compaction around can also facilitate water ingress. It is important to note however that established vegetation can actually improve slope stability as the roots take up ground water during the growing season, thus decreasing pore water pressure and increasing soil strength. In contrast, the presence of mature trees beyond the site boundary can have a negative effect on embankment performance as they can cause shrinking and swelling at the toe. In summary, there are no general rules governing the effect of vegetation, and its presence may have a positive or negative effect. For this reason careful vegetation management is an important maintenance tool.

In the questionnaire responses, all three infrastructure owners reported different temporal characteristics with regard to embankment deterioration. In fact the time taken for a deterioration mechanism to develop varies on a case by case basis. In some cases the short-term condition just after construction is the most critical, and stability actually increases with time as the foundation materials consolidate and pore pressures are dissipated.

Conversely if the embankment fill is a heavily over-consolidated clay which may have very low or even negative pore water pressures at the time of construction, then stability can decrease with time as water infiltrates the embankment and pore water pressures develop. This sort of failure is usually progressive and can happen over relatively long periods of time, however the final failure is often triggered by periods of sustained heavy rainfall and can happen very suddenly.

6.2.3 Soil cuttings

As with embankments, the stability of soil cuttings is governed by the principles of soil mechanics (see Figure 4). Both embankments and soil cuttings are soil slopes, so their deterioration mechanisms are largely the same.

Soil cutting failures can also be broadly categorised as either shallow or deep failures depending on the location of the rupture plane (see Figure 5 below):

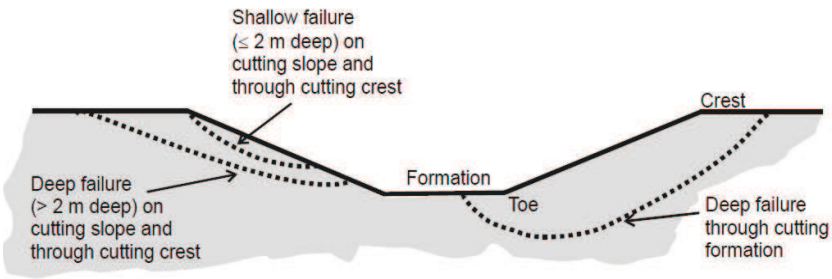

Figure 5: Examples of rupture surfaces for shallow and deep cutting failures [2]

These rupture surfaces are comparable to those observed in embankment failures. The major difference between the two asset types is the location of traffic loading (at the toe in the case of a cutting and at the crest in the case of an embankment) and the likely nature of the fill (embankments are man-made and may be composed of heterogeneous fill or even engineered materials whereas cuttings are composed of natural soil strata).

The CIRIA Report C591 describes the main mechanisms for soil cutting deterioration as the following:

- *Presence of water* – as for embankments, the moisture content of a soil cutting is a major factor in governing slope stability (see section 6.2.2 above).
- *Weathering* – The formation of a cutting exposes soils to weathering that has a detrimental effect on their engineering properties. For example the exposure of clay to water can lead to swelling and a reduction in shear strength.
- *Long term creep* – In residual high plasticity clays and organic soils slow downhill movements of the upper layers may occur. Loose unbound deposits such as scree are also prone to ravelling.
- *Excavations* – As with embankments, excavations on or adjacent to soil cuttings can alter slope geometry and cause instability (see section 6.2.2)
- *Failure of supporting structures and services* – The metallic components of soil reinforcing systems such as soil nails can be degraded through corrosion causing instability. Cuttings may also contain third party services for example water mains that could fail causing catastrophic failure.

- *Erosion* – As with embankments the edges of open-sided cuttings adjacent to watercourses can be degraded by scour (see section 6.2.2)
- *Mining subsidence* – Ground movements associated with past mining activity can affect the stability of the side slopes and gradient along he level of the base of a cutting (see section 6.2.1 for more details on mining subsidence)
- *Landslides* – Some UK railway infrastructure was constructed in areas where landslides had previously occurred. The construction of a cutting or changes to an existing cutting in such an area can re-activate ground movements long existing slip surfaces. Such movements could occur quickly in response to a large change in stability or could occur progressively for example as a result of seasonal variation in pore water pressure.
- *Vegetation* – as with embankments, the presence of vegetation can affect the performance of cutting slopes (see section 6.2.2)

6.2.4 Rock cuttings

Rock cuttings have very different failure modes to soil cuttings so analysis is very different. Whereas soil strength and stiffness are the governing factors for soil slope stability, rock slope stability is largely dependent on discontinuities within the rock mass, their orientation, persistence and strength parameters. The key principles of rock mechanics are outlined in the extract from CIRIA C591 [2] in Figure 6 below:

Failure of a rock cutting can occur by a number of modes, therefore there is no generic method of stability analysis that applies to all cases. In many cases the mode of failure of a rock slope is dominantly controlled by the presence of discontinuities and is dependent on their orientation, persistence and shear strength. However, care must be taken with interbedded soils and rocks where the soil will tend to dominate stability. The shear strength of a discontinuity of infinite persistence, in the absence of infill materials and significant large scale roughness is determined by:

$$\tau = c + \sigma_n \tan \phi$$

where

τ = the shear resistance of the discontinuity

c = the apparent cohesion of the discontinuity

σ_n = the total normal stress acting on the discontinuity

ϕ = the angle of shearing resistance

Where the discontinuity has significance large scale roughness or waviness, which can be assessed in the field, a further component of strength is added:

$$\tau = c + \sigma_n \tan (\phi + i)$$

where

i = angle of the roughness or waviness relative to the angle of the discontinuity

The effect of water pressure in a rock slope on the strength of discontinuities is to reduce the normal stress acting on the discontinuity:

$$\tau = c + (\sigma_n - u) \tan \phi$$

where

u = the uplift water pressure acting on the discontinuity

Unlike in soil mechanics, the angle of shearing resistance and the apparent cohesion are the same whether water is present or not, these parameters are unaffected by the presence of water in the slope. In general, the effect of water pressure on the stability of rock slopes is of considerably less importance than in soil slopes, as such slopes are generally free draining through the discontinuities present.

Figure 6: Rock mechanics principles for slope stability (extracted from [2])

The CIRIA Report C591 outlines the following factors as being the principal influences on loss of performance in rock cuttings;

- *Weathering* – weak sedimentary rocks can weather easily and quickly, resulting in considerable material loss at the cutting face. Grainfall may accumulate at the base of slopes as a result. Rocks containing calcium carbonate cements, such as limestone and some sandstones have distinctive weathering patterns which cause degradation.
- *Presence of discontinuities* – rock masses contain discontinuities for example bedding planes, joints, faults, fractures and shear zones. The number, orientation, persistence and shear strength of rock cuttings all effect slope stability. Many older cuttings may be prone to failure as a result of discontinuities as the field of rock mechanics was less well understood at the time they were designed

- *Construction method* - Most railway cuttings were formed by bulk-blasting with gunpowder, and such blasting techniques can cause significant fracturing of the rock mass which can lead to rock fall and ravelling problems at a later stage.
- *Climatic influences* – In regions that experience large diurnal temperature changes freeze thaw weathering can cause considerable degradation of the rock mass
- *Vegetation* – When vegetation grows within rock discontinuities root wedging can occur, causing local but significant degradation of the slope. Large trees can also become unstable and topple, causing rock falls.
- *Failure of slope support systems* – Where a rock slope has been remediated, for example with rock bolts, the failure of those structures can reactivate slope instability. In addition the failure of remedial measures such as rock catch nets can result in a limit state being reached.

The above factors can result in planar, wedge or toppling failure as depicted in Figure 7 below.

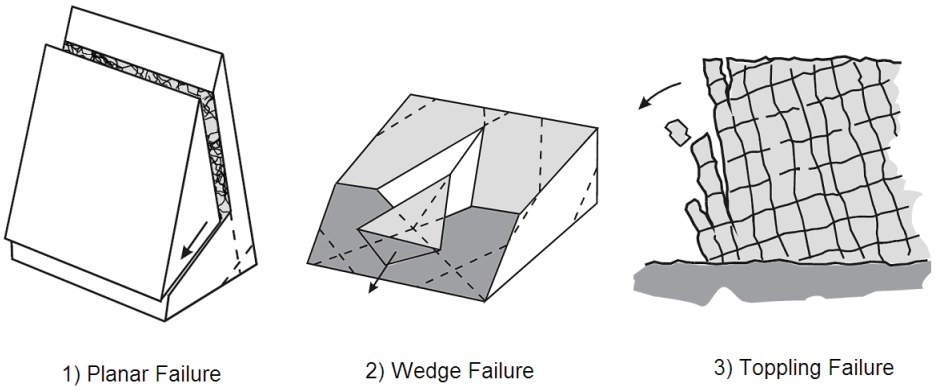

Figure 7: Rock slope failure mechanisms (diagrams extracted from [2])

6.3 Associated performance and limit states

Current design codes require that both ultimate and serviceability limit states are considered. An ultimate limit state can be defined as a state of collapse, instability or form of failure that may endanger people or property or cause major economic loss. In the case of earthworks, such movement would affect adjacent railway or third party infrastructure for example track, services or highways. The serviceability limit state is a state of deformation that affects the use of the earthwork, impairs its durability or requires excessive maintenance. If a serviceability issue is not addressed it might eventually deteriorate into an ultimate limit state. It is difficult to define serviceability limit states for earthworks in terms of a limiting deflection, as is done for bridges and retaining walls. This can make the division between ultimate and serviceability limit states unclear.

It is important to establish details of the geology when assessing the performance limit states of an earthwork. With cuttings in particular, the details of the strata and their properties can have a significant effect on the behaviour. With railway embankments, the fill is often heterogeneous and so defining its properties can be more difficult.

The consequence of a given slope failure varies depending on the type of earthwork. For example a land slip on an embankment could undermine the track, causing misalignment and affecting ride quality (serviceability limit state) whereas the same slip on a cutting slope

could result in debris falling onto the track causing an obstruction (ultimate limit state). The consequences of a given failure mechanism are also linked to the geometry of the earthwork in question, for example a large soil slip on a shallow cutting may have little effect on traffic where the distance between the toe of the slope and the track is large, however even a small rock fall adjacent to a narrow track corridor could have catastrophic consequences.

The following sections will outline the main limit states for each of the four types of earthwork asset.

6.3.1 Natural ground

Serviceability Limit States

- *Deformation* – if the foundation in compressible, the dynamic traffic loading can cause excessive deformations leading to distortion of the line and level of the track and consequently poor ride quality. This can require a speed restriction to be put in place.
- *Settlement* - Settlement is usually non uniform along and across the track which can result in track misalignment. This can also affect ride quality and require speed restrictions to be imposed.

***Ultimate* Limit States**

- *Bearing capacity failure* - failure of the track foundation can cause collapse
- *Deep failure* – a rupture surface may pass through the natural ground if it is composed of a weak material. Strength reduction can propagate along these surfaces causing a deep seated failure beneath the track.

6.3.2 *Soil* embankment

Serviceability Limit States

Many railway embankments suffer serviceability limit state failure, where loss of performance generally occurs subtly and over a long period of time. Serviceability failures can result from several mechanisms and have a major affect on railway operations. The CIRIA Report C550 on Infrastructure Embankments describes the following serviceability limit states:

- *Internal deformation* – High plasticity clays, particularly those that are loosely tipped can undergo significant shrinking and swelling in response to seasonal variations in moisture. This can cause up to 50mm of movement which can cause track misalignment and could eventually trigger ultimate slope failure
- *Ponding of water beneath rails* – This can occur when water is trapped in ballast founded upon relatively impermeable clay. This situation can worsen dynamic loading from trains which may lead to reduced strength and settlement of the fill material.
- *Internal fire* – railway embankments that contain combustible materials (for example colliery spoil, incompletely burnt ash or wood) can be vulnerable to internal fire as a result of the oxidation process of pyrite and organic matter. The increased temperature can cause damage to services and the loss of vegetation on the embankment side slopes.
- *Ravelling* - railway embankments often have steep granular shoulders (usually ballast) which are loosely tipped and under loading from both train vibrations and settlement or erosion of the lower slope. As a result the granular material at the crest can become unsettled causing individual particles to move downslope, undermining cables and services near the crest.

- *Differential Settlement* – At bridge approaches there is often a difference in settlement between the embankment fill and the more rigid bridge structure. This can result in a change in rail level and stiffness, both of which affect ride quality.

Despite the large amount of work that has gone in to understanding this complex field, there are still gaps in the geotechnical understanding of embankment serviceability limit states.

Ultimate Limit States

Ultimate limit state embankment failures occur less frequently than serviceability failures, but the consequences are much more severe, often resulting in traffic disruption. The majority of ultimate limit state failures occur in embankments composed mainly of over-consolidated clays and rarely occur in granular embankments, unless they are founded on weaker materials or subject to excessive pore water pressures.

- *Deep Rotational failure* – in clay embankments, settlement can result in dishing in the centre of the embankment. The causes ponding on the top surface of the clay which can further increase pore water pressure and cause rotational failure
- *Progressive failure* – Most clay railway embankments have a residual internal rupture surface, remaining from failures that occurred during or just after construction. In high plasticity clays softening can occur at the toe causing a reduction in strength which propagates back through the embankment along the rupture surface eventually reaching the crest and causing ultimate failure.
- *Cess heave* – In lower plasticity clays the soil at the crest can become overstressed as a result of water infiltration (reducing strength) and traffic loading. This overstressed soil beneath the track is squeezed sideways and upwards causing a bearing capacity failure
- *Erosion* – during heavy rainfall the increased flow of water can lead to internal erosion, particularly in low-plasticity and granular materials. This can result in ultimate slope failure.
- *Landslide* – embankments on side long ground, often in areas of old landslips can fail by the reactivation of an existing in situ landslide.
- *Mudslip (washout failure)* - mudslips can be caused by flooding, either due to heavy rainfall events or by burst watermain .
- *Culvert (or other structure) collapse* – collapse of culverts or other structures located within or adjacent to an embankment can lead to a local lack of support. This can either directly cause slope failure, or can lead to flooding and softening which induces a failure at a later stage.

6.3.3 Soil cuttings

Serviceability Limit States

Serviceability Limit States in cuttings are most commonly due to ground movements, but can also cover aesthetic and environmental issues. The CIRIA Report C591 gives the following examples of possible serviceability limit states for infrastructure soil cuttings:

- *Downward creep* – slow downward movement of the upper layers of soil causing disruption to drains, fences and vegetation
- *Poor aesthetic appearance* - which could be due to litter, graffiti, damaged fences or excessive vegetation cover
- *Overgrown vegetation* - cutting vegetation imposing on the transport corridor, or causing excessive leaf drop onto track

- *Livestock and wild animals* - straying onto the earthwork and too close to the track potentially causing disruption to services. This could occur if fencing is insufficient or damaged

It is also important to note that a cutting should not be considered in isolation and that slope failure can be triggered be activities outside of the cutting land take. Similarly a cutting failure can have an effect on infrastructure and property outside of the land take.

Ultimate Limit States

The Ultimate Limit States in soil cuttings are generally generated by either shallow or deep seated ground movements during operational loading. However miscellaneous environmental conditions can also be the cause. The CIRIA Report C591 gives the following examples of ultimate limit states for infrastructure soil cuttings:

- *Deep seated failure* - Movement of a substantial volume of soil along a deep-seated slip plane that causes disruption to traffic movement along the transport corridor
- *Crest failure* - Ground movements that compromise the stability of structures or services on or adjacent to the crest of a slope
- *Flooding* - Flooding of the track due to slope drainage failure or washout failure
- *Track Obstruction* – Any object falling onto the track surface has the potential to cause a major accident and severe damage to infrastructure. This could be as a result of trees falling onto the track due to weakness of soil beneath tree roots, or in an extreme case a vehicle leaving a carriageway at the top of a cutting and falling onto a transport corridor below.

6.3.4 Rock cuttings

Serviceability Limit States

The CIRIA Report C591 gives the following typical serviceability limit states for infrastructure rock cuttings:

- Minor rockfalls
- Ravelling and spalling
- Erosion and grainfall
- Frost shattering
- Vegetation damage such as root jacking

It is often the failure of the rock material rather than the rock mass that causes serviceability failures. These failures do not cause major disruption to the railway and can usually be controlled by small scale remedial works and appropriate maintenance regimes.

Ultimate Limit States

The CIRIA Report C591 gives the following examples of ultimate limit states for infrastructure rock cuttings:

- *Rock falls* – pieces of rock may become dislodged from steep slopes, for example as a result of frost shattering, and fall onto the track or cess. This can impose immediate danger to railway users, cause a disruption to operations and damage infrastructure.
- *Wedge, Planar or Toppling failure* – as described in 6.2 above, a block of rock moves downslope, potentially falling on track. These failure modes all result from the orientation of discontinuities in the rock in relation to the exposed rock face. They can

result in large pieces on rock falling onto the track or cess with immediate consequences to operations, safety and infrastructure.

- *Rock Mass failure* – these failures can occur where there are closely spaced discontinuities and are not related to the orientation of the slope cut. In these instances the slope failure the slope can be closely likened to a soil slope failure (see ultimate limit states for soil cuttings above)
- *Top soil failure* – failures of mixed rock and soil slopes often take the form of soil materials failing over the top of the underlying rock. This can result in the limit states observed for soil cuttings.

Rock slopes generally display little evidence of the early development of failure, but when the ultimate limit state is reached it is normally immediately visible to infrastructure users or owners.

6.4 Time dependence of performance states

The performance state of a given earthwork over a defined period of time will depend upon the deterioration rate of the asset in question. The questionnaire responses from infrastructure owners NR, DB and MAV each report different trends with regard to the time dependency of failures observed for the different earthwork types. This is unsurprising because, as discussed in section 6.2, the rate of deterioration of a particular type of earthwork is highly dependent upon geometry, geology and environment as well as other external factors. The large number of variables involved means that the rate of change in performance can vary on a case by case basis. This poses a challenge when attempting to predict how and when an earthwork will reach a performance limit state.

6.4.1 MOUCHEL findings from the CeCost CP5 Project

The above mentioned challenge was also encountered by Mouchel when delivering the CeCost CP5 asset management project for Network Rail. A team of engineers was required to carry out a 100 year whole life costing exercise for a sample of 152 earthworks assets including embankments, soil cuttings and rock cuttings. In order to complete this task it was necessary to quantify a standard rate of deterioration for each asset type in order to predict when repairs would be required over the next 100 years, and how extensive they would be. Because of the complex and variable failure mechanisms involved it was not possible to use a theoretical model (based on soil or rock mechanics) to determine the rate of deterioration for each asset type. It was also appreciated that major failures often occur as a result of extreme or accidental external events, and are by their nature unpredictable. Network Rail repeat earthwork examination data records were used together with engineering judgement to establish a reasonable estimate of deterioration rate.

Network Rail operates and maintains an online database containing examination records for its UK wide network of earthworks. This covers approximately 8762km of embankments, 5763km of soil cuttings and 495km of rock cuttings [7]. During examination each 100m of earthwork is given a score based on the Soil Slope Hazard Index (SSHI) algorithm or Rock Slope Hazard Index (RSHI) algorithm. The SSHI and RSHI scoring systems take into account several factors related to the potential deterioration mechanisms discussed in Section 6.2, including slope geometry, geology and drainage as well as evidence of past failure.

Raw data relating to these factors is collected during site inspection and subsequent desk study, and is input into the algorithm in order to give the earthwork 3 numerical ratings:

1) Actual Failure factor – built up from evidence of past, ongoing or imminent failures for example observed slips, track misalignment, cracks, remediations
2) Potential Failure Factor – taking account of features that could increase the probability of future failures for example geology, slope angle, drainage presence/functionality, previous mining activity, observed animal activity, proximity to watercourses
3) Earthwork Factor – takes account the scale of the earthwork of a given length, based on its height

These three factors are then combined to classify its condition as serviceable, marginal, poor using the failure flow diagram shown below.

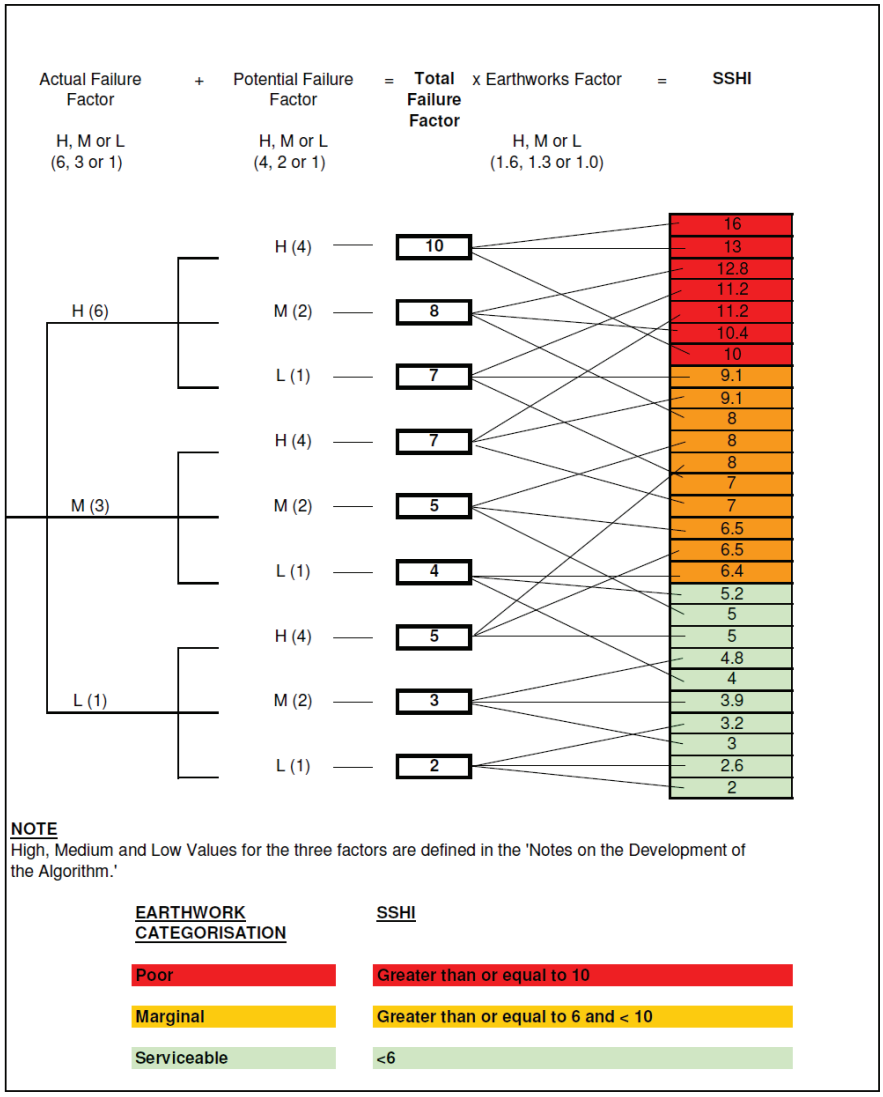

Figure 8: SSHI Failure Flow Diagram [7]

Under Network Rail's policy every 100m length of earthwork is examined every 1, 5 or 10 years depending on the current condition of the slope. MOUCHEL was therefore able to use the repeat SSHI data for embankments and soil cuttings to analyse how many assets deteriorated to a worse condition over a given number of years. This data was then converted into a percentage of the global data set, and an estimated deterioration rate was calculated, as shown in Figure 9 below:

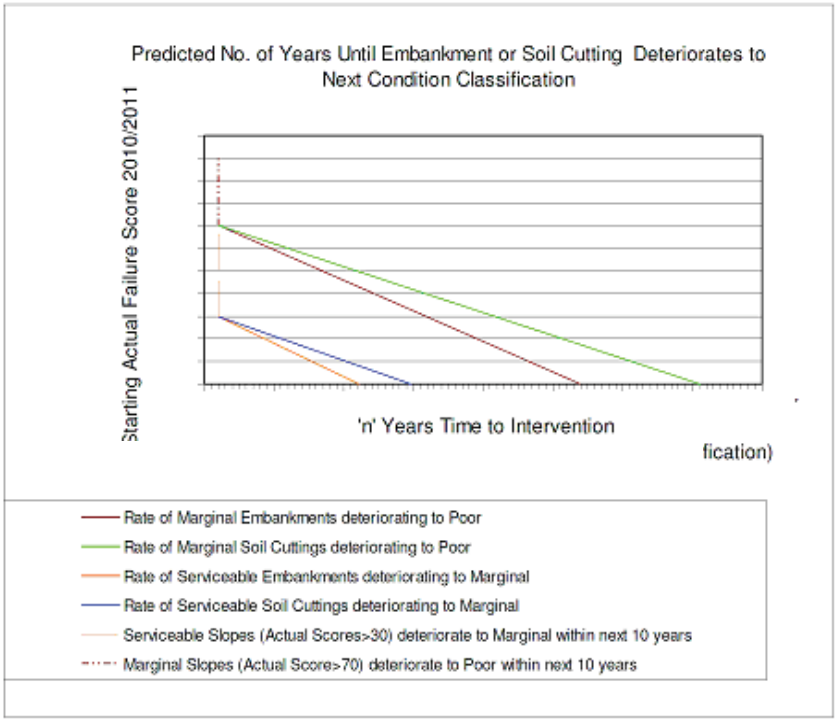

Figure 9: Decaled slope deterioration graph for embankments and soil cuttings (extracted from CeCost CP5 Design Basis Statement Earthworks [7])

The graph above was used to estimate the length of time it would take for an embankment or soil cutting of a given current condition to deteriorate to the next condition classification.

6.4.2 Areas for further research

Rock slope deterioration

The complexity of the RSHI algorithm and the lack of repeat data for rock cuttings meant that it was not possible to estimate a deterioration rate as part of the CeCost CP5 project. If repeat condition assessment data becomes available for a large enough sample of rock cuttings it may be possible to conduct similar statistical analysis of deterioration rates.

Differentiated deterioration rates depending on slope characteristics

The deterioration model developed for soil slopes could be further refined when more repeat examination data becomes available. It may be possible to differentiate the rate of deterioration for particular slopes with similar characteristics, for example composition and geometry. This would allow an appropriate model to be adopted in different regions where particular slope characteristics are more common than others.

Influence of age

The UK earthworks analysed as part of CeCost CP5 had already matured to between 100 and 150 years old and it was therefore assumed that they had reached a constant rate of deterioration. It was not possible to validate the linear constancy of this rate with the data that was available at the time, however validation would be possible if a large number of repeat data sets, of reliable quality became available.

Effect of climate change

The effects of climate change could also be incorporated into the model. Increased rainfall and risk of flooding in the future could alter deterioration rates and increase the likelihood of failure.

6.5 Summary

Earthworks form a large proportion of railway networks across Europe. There are four types of earthwork asset, namely natural ground, embankments, soil cuttings and rocks cuttings. There are also numerous possible soil and rock types from which each type might be constructed. As a result there are a wide range of possible damage and deterioration mechanisms that affect earthworks structures.

Embankments and soil cuttings both consist of soil slopes and therefore deteriorate in much the same way. The major factor affecting soil slope stability is the presence of water, which can trigger a range of deterioration mechanisms. As a result, future deterioration rates for earthworks constructed in cohesive materials such as clay will be sensitive to the effects of climate change, which is predicted to cause increased rainfall and flooding. Rock cutting deterioration is less affected by the presence of water. Instead stability is largely governed by the presence and orientation of naturally occurring discontinuities within the rock mass. Track founded on natural ground can deteriorate as a result of weakening and deformation of the sub-layers of soil. The deterioration mechanisms that affect natural ground can also effect embankments and cuttings by causing deep seated failures in the underlying strata.

Serviceability and ultimate limit states are less well defined for earthworks than they are for other civil engineering structures. However, two CIRIA guides have been published which outline a range of possible limit states for earthworks and cuttings.

Another research project called SMART Rail is currently in progress, which is focussing on embankment asset types. Hence the MAINLINE project will focus on cuttings only from this point onwards, and link into SMART Rail for embankments.

6.6 References

[1] Perry et al, 2001 - CIRIA C550 Infrastructure embankments – condition appraisal and remedial treatment

[2] Perry et al, 2003 - CIRIA C591 Infrastructure cuttings – condition appraisal and remedial treatment

[3] Rail.co, 4 Aug 2011, Rail services back to normal after South Croydon mudslide, article accessed at:http://www.rail.co/2011/08/04/rail-services-back-to-normal-after-south-croydon-mudslide/

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7. Bridges

This chapter deals with degradation and performance specification for bridges.

Response to the questionnaire reveals that the current knowledge level is quite high for all bridge types compared to some of the other components.

There is a rather high variability in the ratio of each bridge type within the bridge stock of each country in the EU. It is however evident that the majority of old bridges are metallic or masonry. On the other hand, as shown in Figure 2, a vast number of 20-50 year old concrete bridges are in service. It is foreseen that allocation of maintenance resources for concrete bridges is an emerging challenge.

In general maintenance costs per bridge related to metallic bridges seems to be greater than those of masonry and concrete bridges. Part of the reason could be that metallic bridges in general are older than concrete bridges.

Due to significant ongoing work on degradation models for masonry bridges and the relative limited available work, it is suggested that during the forthcoming three year period the possibility of a step change is poor for masonry bridges; therefore metallic bridges are chosen for more detailed studies within the MAINLINE project. Quite extensive guidance is also provided for concrete bridges where allocation of maintenance resources is an emerging challenge.

Masonry bridges are addressed to a lesser detail in this chapter.

Focus in this deliverable is directed towards existing bridges but in the coming deliverables attention is also given to new bridge types used for replacement as they also have to be included in LCC and LCA analyses.

It is noted that only rail carrying bridges is part of the MAINLINE scope.

7.1 Metallic asset types

According to section 5.2.2, metal bridges are recommended for further consideration.

From a large survey of European metallic bridges, it was found that 3% are cast iron, 25% wrought iron and 53% steel. Only 11% have spans longer than 40m. In addition, the survey, undertaken as part of the Sustainable Bridges project, identified country specific characteristics through a questionnaire. In the following, the main conclusions from this study are highlighted.

For the vast majority of short span metallic bridges (ranging between 4m to 10m), two types of cross-sections appears to be the most common in the European railway network. One is the “semi-through” or “half-through” cross-section, see Figure 10. It is built up by main plate girders on each side of the track (usually with additional girders between multiple tracks), cross-girders between these and rail bearers under each rail. The other main type has the main plate girders placed directly below the tracks, see Figure 11. Older bridges have usually the sleepers connected directly to the load-carrying members, but there are also ballasted bridges for both bridge types, with steel or timber decks supporting the ballast. Depending on the type and depth of ballast, the load transfer from the track to the main structural system needs to be determined. For very short spans (less than 4m), several simple bridge types exist, typically consisting of several rolled sections supporting the sleepers, or forming a deck for a ballasted track. For younger bridges, corrugated steel pipes or arches are common for such very short spans.

For medium span bridges (10m to 40m), the same two types of cross-sections as for the shorter span bridges dominate. However, as the span gets towards the upper limit, truss bridges are also increasingly common.

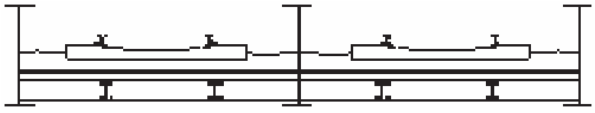

Figure 10: Half-through plate girder bridge, with cross girder, floor plate, rail bearers and ballast.

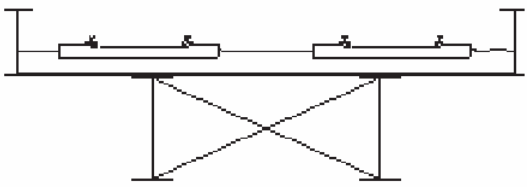

Figure 11: Metallic girder and metallic deck with ballast.

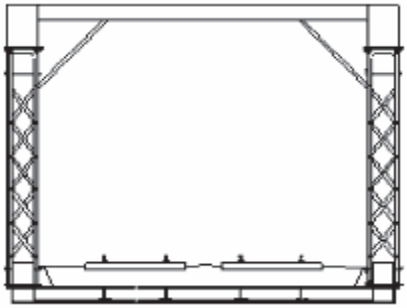

Figure 12: Through truss bridge, with cross girder, floor plate, railbearers and ballast.

As the span increases further (to over 40m) truss bridges become more dominant, though lattice type bridges also occur. Through truss sections, see Figure 12, and half-through truss sections are common, as well as deck-truss bridges with the track on top of the truss. Steel box arch and truss arch bridges are also common across large spans, however cable-stayed and suspension bridges are very unusual for railways.

Up until the 1950s, metallic bridge girders and truss members were formed by riveted sections. From this time onwards, rivets were successively phased out, with welding becoming the preferred method for joining. Since the majority of existing metallic bridges are from the pre-1950s period, particular attention will be directed towards damage/deterioration phenomena and performance modelling relevant to riveted bridges. However, the main phenomena are expected to remain, in principle, the same for younger existing steel bridges as well as new steel bridges.

As stated, younger existing steel bridges are also present in the European bridge stock. Damage and deterioration mechanisms such as corrosion are to a large extent similar to those relevant for older bridges. Their distribution as function of time may, however, is different.

Limit state assessment of these younger steel bridges may be performed based on standards for design of steel structures like Eurocode 3. Guidance on assessment of steel structures incl. reasons to relax requirements from design standards and possible enhancement of structural models, please refer e.g. [1] and [5].

7.2 Metallic properties

Any safety assessment for an old bridge requires the knowledge of the structural resistance and therefore information with regard to the material properties.

The relevant information when determining characteristics of old metal bridge materials is:

- Chemical analysis of the material, which is useful for the material identification. For the old metals that are considered here, no limit requirements had to be fulfilled. The relevant chemical components with effect on the material properties are:
  - C Carbon, increasing hardness, strength and hardenability, but decreasing toughness and weldability,
  - Si Silicon, similar effects as carbon, leading to brittle behaviour in high amounts,
  - Mn Manganese, positive effect on both strength and toughness properties,
  - P Phosphorus, increasing hardness and brittleness,
  - S Sulphur, negative effect on strength and toughness properties,
  - N Nitrogen, unwanted element, reducing toughness and weldability;
- Mechanical properties determined by tensile tests, with the relevant parameters:
  - f_{yT} Yield strength/elastic limit in tension. Modern codes are based on the upper yield strength (R_{eH}).
  - f_{yC} Yield strength/elastic limit in compression. Modern steels have the same value in tension and compression but this is not the case for older metals such as cast iron.
  - f_u Ultimate tensile strength,
  - ε_u Ultimate elongation;
- Fatigue properties, given e.g. by fatigue resistance curves/S-N-curves;
- Fracture properties are required for an assessment of the remaining service life, once damage such as cracks have been detected. These are fracture toughness values and threshold values for crack growth, as well as the test temperature of the Charpy energy test, $K_{mat}(K_{IC})$, $J_{mat}(J_C)$, ΔK_{th}, $T_{27\,J}$ measured on full thickness, see [27] - [30].

In Table 3 an overview of definitions and characteristics of bridge metals is provided.

Grey cast iron	Iron alloy containing 2.5 - 4% by weight of carbon and other elements such as silicon, phosphorous, manganese and sulphur. After casting, grey cast iron is cooled slowly, allowing the graphite to form discrete flakes, which act as stress raisers. Cast iron is weak in tension, brittle, fatigue-sensitive and exhibits a significant size effect. In addition to variation in the internal microstructure, grey cast iron is characterised by macrostructure defects produced during casting.
Ductile cast iron	Ductile cast iron contains spherical graphite nodules. Under tensile load these spherical inclusions do not act as stress raisers to the same extent as the graphite flakes in grey cast iron. Consequently, spheroidal cast iron is ductile in both tension and compression.
Wrought iron	Wrought iron was formed by reheating cast iron to a high temperature and stirring to remove carbon and other impurities. The cooled iron was then reworked producing a matrix of pure iron and elongated slag stringers. Repeated reworking gave finer

	slag stringers resulting in improved strength and ductility. Wrought iron is tough and malleable and has a high anisotropy. The fatigue behaviour depends to some extent upon the amount of cold working carried out during manufacture. Cracks typically propagate from rivet holes or similar large stress concentrations.
Historic steel	Steels produced before 1922 can be of poor quality and may be laminated or include other defects. It is difficult to predict the properties of historic steel unless records have been kept and sample testing is likely to be required. In the absence of other information some countries have national recommendations e.g. UK Highways Agency and Rail Net Denmark.
Modern steel	Modern carbon steel is characterised by linear-elastic behaviour up to the onset of yield, followed by considerable ductile plastic strain up to ultimate rupture, generally accompanied by strain hardening. Design guidance is well documented in Eurocode 3.
Stainless Steel	Stainless steels are corrosion and heat resistant steels containing a minimum of 10.5% chromium. Supplementary rules for the design of stainless steel structures are provided in Eurocode 3. Stainless steel is a relatively new structural material and will therefore not be considered further in this report.

Table 3: Definitions and characteristics of bridge metallic materials [26]

Table 4 below outlines the typical properties of cast iron, wrought iron and steel as outlined in CIRIA C595 [26].

		Cast iron		Wrought iron	Steel	
		Historic	Modern (BSI, 1997a)		Historic (pre-1950)	Modern (post-1950)
Modulus (GPa)	tensile	66–94*	100–145	154–220	200–205	200–210
	compressive	84–91*				
Strength (MPa)	tensile	65–280	150–400	278–593**	286–494	275–355
	compressive	587–772	600–1200	247–309		
Elastic limit (MPa)		See Figure 2.5		154–408	278–309	275–355
Ultimate strain (%)		Figure 2.5	0.5–0.75 (tension)	7–21	18–20	18–25
Poisson's ratio		0.25	0.26	0.25	0.26–0.34	0.30
Density (kg/m^3)		7050–7300		7700	7840	
Coefficient of thermal expansion ($10^{-6}/^0$C)		10–11		12	12	

* Secant modulus, due to non-linear σ-ε curve.

** UTS is parallel to the grain. UTS perpendicular to the grain is about two-thirds to three-quarters of this value

All values in this table are representative. Reference should be made to Bussell (1997) and Highways Agency (2001) for further information, together with any test data.

Table 4: Typical properties of metallic bridge materials [26]

Material properties can be impaired by manufacturing flaws introduced either during original manufacture, during fabrication and erection, or during subsequent repairs. Examples are:

- Porosity, shrinkage cracking and machining notches in cast iron members.
- Embrittlement of castings when repair welded.
- Cracks induced in wrought iron or steel members and during cold forming.
- Loss of ductility in wrought iron and steel members, due to slag and inclusions respectively, when strengthened by welding, particularly where through-thickness tensile stresses are introduced.
- Hydrogen embrittlement of higher strength steels (eg Grade 10.9 bolts, especially when electroplated).

7.3 Damage and deterioration mechanisms for metallic bridges

7.3.1 General

A recent paper [9] reviewed the causes of failure and collapse in metallic bridges, through analysis of past events. In total, 164 cases were examined, with 34% corresponding to railway bridges. Figure 13(a) shows the underlying failure modes in cases where total collapse of the bridge was experienced, whereas Figure 13(b) depicts the same for the case of damage or failure short of total collapse.

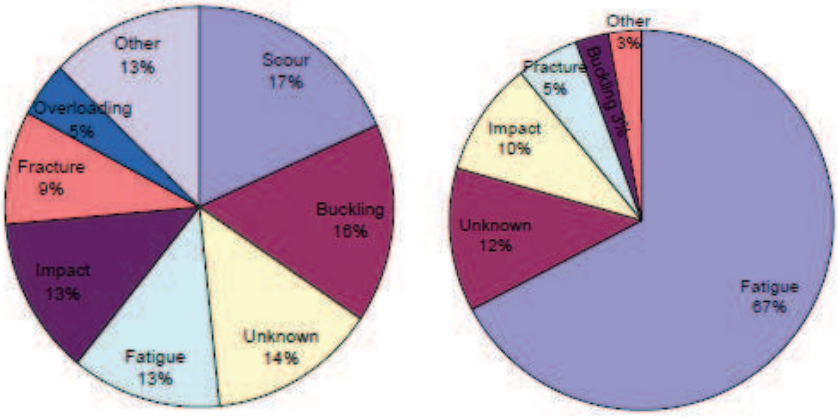

Figure 13: Failure modes for (a) collapsed and (b) non-collapsed metallic bridges

It is clear that in both charts, fatigue/fracture plays a major role in undermining bridge resistance, and is by far the dominant factor in causing damage or failure short of total collapse. Hence, fatigue will be a focus area in the following sections.

It is also worth noting that scour is a very important contributor to bridge collapse.

Buckling is the other factor that plays a role, albeit at a smaller scale compared to fatigue.

However, what is not possible to decipher from the above is the extent to which these failures/collapses occurred on bridges that were in a good condition or had suffered some form of deterioration which had eroded part of the safety margin that may have been available at the outset.

Furthermore, it should be noted that the distribution of failure modes shown in Figure 13 (related to collapse or failure short of collapse) may not correspond to the actual distribution of money spend on effects of degradation mechanisms in metallic bridges now or in the future.

Putting fatigue aside, deterioration in metallic bridges occurs as a result of environmental factors (temperature, precipitation, humidity, wind) reacting with the bridge materials, thus leading to a reduction in geometric or material properties. Corrosion is the most obvious

manifestation of the influence of environmental factors on metallic bridges. Hence, corrosion will also be a focus area in the following sections.

7.3.2 Fatigue

Several authors have provided significant information to this field of expertise, particularly in relation to bridges [1], [2], [4], [5], [6], [7] and [8].

To be able to make accurate assessments of existing bridges, it is important to know the behaviour of bridges exposed to fatigue, and how the old materials behave due to cyclic exposure. The technique of riveting is no longer used in bridges due to more developed methods of assembling plates as welding. Due to this fact there is often missing information in codes how to deal with and assess riveted structures.

In old truss bridges with rails on wooden sleepers which lie on rail bearers and cross girders (open decks), fatigue problems will probably start in these elements, before having problems in the main girders. The reason is that the short elements have to endure a greater number of stress cycles than the elements with greater spans. For the small elements each axle load or boogie represents a cycle of stress range but for the main girder it is only each train which gives a cycle of stress range.

The hole drilling, the assembling techniques and the clamping force constitute essential aspects of the construction process which can affect the strength and the lifetime of old riveted structures. The assembling technique of riveting was to drive a hot rivet through the parts that were to be connected. The rivet was then formed by hammering the shank to form another head. When the rivet cooled the material contracted which created a compressive force on the assembled parts, called clamping force. The magnitude of the force differed significantly between rivets depending on the persons conducting the riveting.

Methods of producing rivet holes in old bridge structures were drilling, punching, sub drilling and reaming, and punching and reaming. The surface conditions of rivet holes are believed to be an influencing factor on the fatigue life of riveted structures. Opinions concerning the method best suited for producing rivet holes are not unanimous. The degree of clamping by the rivet and the quality of the hole surface (which can vary with punch wear) can influence the fatigue life of the plating. These factors need to be investigated (see section 7.4.2 below). They also present problems of inspection by normal NDT methods (see WP4).

Other aspects which can pose problems of assessment are analysis of complete riveted joints particularly where prying forces exert tension on rivets (e.g. cross member to main girder connections made with cleat angles).

The processes leading to a fatigue failure are often explained in three stages, each stage with its own characteristics.

1. Crack initiation: a crack is forming in the metal microstructure. Cracks will initiate through plastic deformations due to tension on grains in the steel structure. This occurs when the stresses in a crystal reach its yield point and the crystal begins to deform plastically. Plastic deformations, in the crystals often have its origin at a notch or at a stress raiser such as dislocations, blisters, and inclusions of impurities, etc.

2. Crack propagation: the crack is growing in the material. The crack propagation occurs due to continued cyclic loading, making cracks form into one or more main cracks. A plastic zone forms in front of the crack with the size of a few grains. Growth of cracks depends on the internal structure of the material, the size of grains (governing both the crack direction and the ability to endure fatigue in the first stage), but at the end, the crack direction becomes normal to the far field tensile axis.

3. Rapid crack growth: the component or the structure is failing rapidly. This late stage in the fatigue process is leading to failure when the remaining cross-section can no longer stand the loads. The two main failure modes are brittle and ductile. Brittle

failure leads to a rapid collapse. Ductile failures are characterised by a plastic deformation of the remaining cross section.

The number of cycles for the different stages in the fatigue process can vary significantly from hundred to millions of cycles depending on stress range, stress initiation factors, material properties, etc.

The majority of the old bridges still remaining today are made of steel; however there are many made from wrought iron still in service. The knowledge concerning wrought iron bridges is not as extensive as for steel bridges. The investigations in [1] indicates that there is no obvious difference in the fatigue endurance between steel and wrought iron structures concerning fatigue, though additional evidence in this respect would be welcome.

In addition to fatigue related failure of riveted bridges, weld-related cracking needs to be addressed for bridges build between the 1950s and now. Weld-related cracking is a result of welding discontinuities, residual stresses, and decreased strength and toughness in the weld material and heat-affected zone (HAZ). Design and fabrication methods also affect weld integrity. Stress concentrations from notches, residual stresses, and change in microstructures resulting in reduced toughness can also be caused by flame cutting.

Furthermore, bolts and bolted connections are also of concern in relation to fatigue. The presence of geometrical discontinuities (holes, change of section) causes stress concentrations which increase the stresses locally and influence resistance to fatigue. Stress concentrations occur in bolts at the thread roots, thread run-out and at the radius under the head. Fatigue failures in bolts in fluctuating tension commonly occur at this last location or in the first thread under the nut. The design of the joint is very important as the fatigue strength finally depends on the real path of the load through the connection, and the fluctuation in stresses of the fatigue sensitive regions. Differentiation between the tension and shear load cases together and the effect of using preloaded blots is not detailed in this deliverable.

7.3.3 Corrosion

Corrosion is a problem for metal structures (Figure 14). Unless treated with some kind of protection, the resistance of structural details will decrease due to corrosion due to (a) reduction in area (b) degradation of mechanical performance (c) build up of corrosion products at connection details and (d) a notching effect that creates stress concentrations, i.e. corrosion is a matter of strength, stability, serviceability and durability.

Corrosion is deterioration of the material due to reaction with its environment. All corrosion processes include electrochemical reactions. Galvanic corrosion, pitting corrosion, crevice corrosion, and general atmospheric corrosion are purely electrochemical. Stress corrosion, however, result from the combined action of chemical plus mechanical factors.

The type and amount of corrosion depend on many factors that include design details, material properties, operation and maintenance, environment, and coating system. In general, the primary factors are the local environment and the protective coating system.

Concerning old metal bridges some degree of corrosion will always be present due to the assembling technique with layered parts making corrosion protection hard to perform and maintain.

Figure 14: Corrosion on riveted bridge elements

It is difficult to compare the amount and the severity of corrosion. In general, it is important to distinguish between global and localised corrosion.

Global corrosion can be found in exposed elements, e.g. flanges and webs of plate girders, and is characterised by a general thinning of the plates. This reduction in area can have an effect in buckling resistance, since the plated elements would have become more slender and hence more prone to buckling. It may also have an indirect effect on fatigue resistance, since thinner cross-sectional areas lead to higher stresses, hence smaller fatigue lives.

A further consideration is separation of stacks of plating due to expansion of corrosion products between them. In cases where rivet spacing is large, or where rivet heads have corroded excessively, rust expansion can continue indefinitely resulting in reduction of buckling strength and/or loss of rivet heads. The pressure exerted by expanding rust is not known definitely, but a value of 30N/mm^2 has been suggested.

Localised corrosion (crevice, pitting, galvanic and possible stray current corrosion[4]) occurs in a number of relatively confined locations, such as corners, joints, areas adjacent to holes etc. This type of corrosion can lead to localised failure in the first instance, though this may be lead to more extensive failure or even collapse if load re-distribution cannot be accommodated.

The fatigue life will not be influenced in the same way if the corrosion damage is located at the compressed flange rather at the tension flange. Especially corrosion near rivets increases the local stress levels which lead to lower fatigue endurance. The rough surfaces due to corrosion acts as a stress raiser which can cause the growth of cracks, the amount of corrosion that can be allowed before it becomes a larger stress raiser than the rivet holes can however not be established.

It should be kept in mind that the ductile parts of old steel are located at the surface of plates and angles. A corroded structure will have a reduced cross section consisting of more brittle material which increases the risk of a brittle fracture especially in low working temperatures.

The tolerable deterioration of rivet heads is also a main topic for condition assessment of old riveted steel railway bridges. Possible results of rivet head corrosion are:

- Loss of pre-stress
- Constitutional change of riveted connection

[4] Please refer section 7.5.3.

- Loss of position permanence
- Gaping of plies followed by stress corrosion cracking

This has not been investigated any further at present.

Summary

Table 5 is a summary of temporal and spatial characteristics together with sensitivity towards load evolution and effects of climate change for fatigue of riveted metallic bridges as presented above.

Riveted metallic bridges	
Temporal characteristics	In general, changes happen within months or even years depending on crack growth stage of the structure in question. The effect of corrosion and notches can lower the fatigue endurance.
Spatial characteristics	Fatigue attacks where stress raisers are present (small scale) but typically bridges have many fatigue-prone details. Corrosion can be of a global or localised nature.
Sensitivity towards load evolution	Fatigue is sensitive both to live load intensity as well as frequency.
Sensitivity towards effects of climate change	Corrosion increases with increasing temperature and humidity (temporal effect) and the effect of corrosion and notches lowers the fatigue endurance by several detail categories. Furthermore, corrosion spreads more rapidly with increasing temperature and humidity (spatial effect), i.e. more stress raisers could emerge.

Table 5: Summary for riveted metallic bridges

7.4 Associated performance and limit states for metallic bridges

7.4.1 General

This section looks at performance requirements in terms of structural strength/serviceability/durability. Both normal and accidental loads need to be considered.

Performance assessment of existing metallic bridges must include an overall conventional safety evaluation for all the joints and all the structural components against actual operating conditions. The evaluation has the purpose to identify and predict risks in terms of stability, strength and fatigue, and to localize the hot spots for which failure due to damage and undetected cracks could lead to bridge collapse.

As for any structure, the study of the operating conditions is essential. They concern several aspects of the life of the structure: changes in number of tracks, evolution of loads, reported failures, exceptional events (damage due to external sources)… A large number of metallic bridges have been subjected to repairing and reinforcing processes due to a variety of damage or due to modifications in their operating conditions. For those bridges, safety and serviceability questions arise versus modern loads; the question of assessing their remaining service life has to be asked.

By nature, fatigue is a matter of structural strength/capacity. However, according to the Eurocode it is not directly affiliated to the ultimate limit state (e.g. separate fatigue actions and differentiated target reliability). Fatigue in metallic bridges is only a strength/capacity issue if designed correctly. For assessment this may not necessarily be the case as e.g. rivets failing in fatigue may cause slippage/greater deflection and a serviceability issue occurs (poor ride quality or perhaps derailment risk).

Accidental limit states are single point in time load cases that differ from many load cycles in the fatigue load case. However, fatigue crack growth may reduce the capacity also in the accidental limit state.

Durability is affected by fatigue crack growth. The concept of durability is essential the background of calculating the remaining lifetime based on damage accumulation or crack growth based on fracture mechanics, i.e. in this presentation it is considered that durability is covered by structural strength/capacity limit state.

7.4.2 Structural strength/capacity

For fatigue assessment, the procedure would require the knowledge of the fatigue cycles applied to the different details. This information is generally not readily available.

Some studies tend to prove that the eventual fatigue damage accumulated for old structures until the end of WWII or even later is negligible in comparison to the cycles applied during the past 50 years (due to change of loading). This is consistent with the finding that the cut off limit for stress range is 40 MPa, please refer to [1].

A simplified procedure in Germany exists for the remaining service life assessment based on fatigue damage accumulation using S-N-lines. The verification is implemented in the DB-Guideline 805 [3] and elaborated in [1].

Most design rules for steel structures, for instance those in Eurocode 3, are applicable also to riveted structures. However, some information is missing on how to deal with the special case that elements are intermittently connected in contrast to welded structures that are connected continuously. One such issue is how to define the cross section class of a riveted member. One question that is not covered in a reasonable way is the distance between rivets in the direction of stress. Another is to quantify the positive effect of restraint to local buckling provided by the connecting angles.

The traditional method for assessing the resistance of steel bridges is based on elastic analysis. In case the resistance in the ultimate limit state is insufficient it is possible that allowing for plastic deformations gives a more favourable answer. Such a situation may occur if one wants to allow one or a few exceptionally heavy trains that do not contribute to the fatigue.

With regard to the effect of corrosion, performance requirements pertaining to strength that need to be considered include tension capacity, global and local buckling capacities, and bearing capacity. The effect of corrosion is typically taken into account through reduced cross-sections, though additional influences may arise as a result on highly localised corrosion in fatigue sensitive areas that may require consideration of the combined effect of corrosion and fatigue.

7.5 Concrete bridges

7.5.1 Asset types

For shorter span concrete bridges (spans up to 10 m), non pre-stressed simply supported beam or slab bridges are common as well as frame bridges. The most common cross-sections are slab and trough cross-sections. Also, solid, wide, “slab-like” beam cross-sections are common for the larger spans. In UK, the most common concrete bridge type consists of solid, rectangular reinforced or pre-stressed prefabricated beams, placed close to each other and connected to form a flat deck. From a large survey performed by the Sustainable Bridge project among 16 European infrastructure managers some 62% of all railway bridges have spans less than 10 meters [10].

For medium span bridges (10 to 40 m), pre-stressed, often continuous, girder bridges are the most common, but continuous slab and frame bridges are also present. Slab and “slab-like” beam cross-sections are still common, as well as the trough cross-sections. For increasing spans, T and box cross-sections become more common. From [10] some 34% of all railway bridges have span between 10 and 40 meters.

The larger span bridges (over 40 m) represent a very small fraction of the total concrete railway bridge stock (approx. 4% according to [10]). However, each bridge represents a large investment. Here, pre-stressed T- and box-girder bridges are common, but also arch bridges are present.

7.5.2 Material properties

The materials relevant for the degradation and performance specification of concrete bridges are:

- Concrete
- Reinforcement
- Pre-stressing steel

In addition to this, the interaction between the reinforcement or the pre-stressing steel and surrounding concrete is of importance. For a detailed treatment of material properties related to concrete bridges please refer to [1].

7.5.3 Damage and deterioration mechanisms

According to infrastructure managers responses to the questionnaire (see Appendix A), MAINLINE - Description of Work and Sustainable Bridges survey [10] the dominating degradation mechanisms in concrete bridges are:

- Corrosion of pre-stressing tendons
- Reinforcement corrosion (general from carbonation and pitting from chloride ingress) which leads to cracking and spalling of concrete cover
- Sulphate attack
- Alkali Silica Reaction (ASR)
- Freeze/thaw
- Fatigue of reinforcement
- Stray current corrosion
- Impact

Each degradation mechanism is treated below.

Corrosion of pre-stressing tendons

In pre-stressed reinforcement, uniform, localised as well as stress corrosion may occur. However, the corrosion is most often restricted to limited parts of the tendon. Since pre-stressing tendons normally have a thicker concrete cover than the non pre-stressed reinforcement, corrosion is normally not governed by carbonation or chloride penetration through the concrete cover. Instead, corrosion occur primary through failure of the tendon corrosion protection system and secondary through defect drainage or other cast-in systems, or chloride penetrations through construction or element joints and through major cracks, etc. Failure of the tendon corrosion protection system may typically consist of grout voids in the tendon ducts, open or partly open grouting in- and outlets, leaking and damaged metal ducts and poor concrete covering the anchorage devices.

In [12] inspection, testing and assessment of post-tensioned tendons are addressed. For assessment of the bridge resistance it is important to quantify the extent of the corrosion, i.e. to determine the remaining cross-sectional area of the tendon and the remaining pre-stressing force.

Reinforcement corrosion

The effect of corrosion on concrete structures is described in e.g. [12], [13], [25] and [31]. The following representation follows [1].

There are mainly two types of environmental actions leading to corrosion of reinforcement embedded in concrete bridges, carbonation of the concrete and chloride penetration. Substantial research has resulted in well verified models describing how the carbonation and chloride penetration advances through an un-cracked concrete cover, see e.g. [14]. However, existing models describing the subsequent corrosion development is more rudimentary, and the influence of cover cracking and other deficiencies of the concrete protecting the reinforcement are still not well known.

Reinforcement corrosion can be subdivided into two main effects:

- The reinforcement cross-section is reduced, leading to a reduced capacity. Furthermore, the reinforcement can become more brittle in case of localised corrosion.
- The rust occupies a larger volume than the steel it was formed from. This leads to splitting stresses in the concrete, which eventually may lead to spalling of the

concrete cover. In addition to risk for personal or property damage when pieces of concrete falls off, and a bad appearance of the bridge, this influences the structural response in several ways:

- The bond between the reinforcement and the concrete is influenced. When the bond is deteriorated the deflections of the bridge will increase. A more serious matter is that the load carrying capacity can decrease if the corrosion occurs at splices or in anchorage zones.
- If the concrete cover is spalled off in a compression zone, the inner lever arm will be reduced and, consequently, the moment capacity will decrease. Furthermore, if there is compression reinforcement this may buckle, leading to further capacity reduction.

When evaluating the consequences of reinforcement corrosion, it is important to be aware of the different kind of corrosion that may occur. There are three main type of reinforcement corrosion that occurs in concrete bridges:

- *Uniform corrosion* consists of a uniform surface attack of the steel with no discrete anodic and cathodic sites on the reinforcement e.g. from carbonation. It leads to an approximately equally distributed layer of corrosion around and along the reinforcement. When uniform corrosion occurs on reinforcement cast in concrete, the volume increase causes splitting stresses in the concrete, influences the bond and will eventually lead to cover cracks and spalling. Prior to formation of concrete cracks, the reduction of reinforcement area is negligible.
- *Localised or pitting corrosion* occurs due inhomogeneous corrosive environment or inhomogeneous reinforcement surface conditions. Here, a localised (small) anodic area forms an electrochemical cell with a large cathodic area, leading to locally high corrosion rates. If there are high concentrations of ions, such as chlorides, e.g. where there is a concrete crack, a pitting mechanism may occur. This can lead to a substantial reduction of reinforcement area locally, without any splitting cracks. This may eventually lead to rupture of the reinforcement, something that can be hard to predict through inspection.
- *Stress corrosion* may occur in pre-stressed reinforcement.

Cracking and spalling of concrete cover

For reasons of function, durability and appearance cracking is a major concern in concrete construction. The increasing use of more cement and lower water to binder ratios for higher concrete qualities has rendered this problem more acute in recent years. For this reason more severe crack prevention requirements have been specified by proprietors and normative bodies in the last decades, especially for civil engineering structures in severe environmental conditions.

In essence, cracks are either structural or non-structural in character. Non-structural cracking occurs before or after the material has hardened: with the former, cracking may be due to drying shrinkage; with the latter to corrosion of the reinforcement, freeze-thaw effects, temperature variations and Alkali-Silica reactions. Structural cracking can occur through overstressing of the material or through ground movements.

Cracking in concrete can be classified according to [11].

Methods on how to evaluate cracking in concrete due to hydration, temperature, restrains and settlements are given in e.g. [15].

According to Eurocode EN-1992-1-1 (2008) recommended crack widths can be stated with reference mainly to the corrosion risk for the reinforcement.

Spalling occurs as a localised depression on the surface of a bridge. It can be caused by corrosion and by frictional forces generated by thermal movements. Where unchecked, spalling will often lead to the exposure of the reinforcement in concrete structures.

Sulphate attack

Sulphate attack can be either external or internal. External attack is due to penetration of sulphates in solution, e.g. in groundwater, into the concrete from outside. Internal attack is due to soluble source being incorporated into the concrete at the time of mixing, e.g. gypsum in the aggregate.

Behind the reaction front of external sulphate attack, the composition and microstructure of the concrete will have changed. These changes may vary in type or severity but commonly include:

- Extensive cracking
- Expansion
- Loss of bond between the cement paste and aggregate
- Alteration of phase composition, with mono-sulphate phase converting to delayed ettringite and, later stages, gypsum formation. The necessary additional calcium is provided by the calcium hydroxide and calcium silicate hydrate in the cement paste

The effect of these changes is an overall loss of concrete strength.

Alkali-Silica reaction (ASR)

ASR is a complex chemical reaction that takes place between the alkali hydroxides in the hydrated concrete and certain siliceous aggregates. The reaction product imbibes water, and in some situations, expansion stresses can be developed in the concrete. These stresses can be sufficient to crack the concrete. The reaction is slow and it can take many years for visual damage to develop.

Damage due to ASR may in some cases develop extremely fast. If the concrete contains sufficient siliceous aggregates (often in the sand) it may only take 10 years from failure of waterproofing until the majority of the bridge deck deteriorates due to cracks forming horizontally. This even happens for railway bridges where the presence of alkali (from de icing salts) is not present, i.e. water is sufficient to drive the development of ASR.

The development of this cracking is very variable and depends on presence of reinforcement or other restraint. Both the strength and stiffness of isolated and unreinforced concrete samples are reduced by ASR. However, tests on structural elements and actual structures have indicated no significant adverse effect of ASR deterioration, due to restraint by reinforcement or undamaged concrete. E.g. BA 52/94 gives guidance on the assessment of concrete structures affected by ASR [24].

Frost damage

Freezing damage in concrete is caused by the volume expansion of freezing water in the concrete pore system. If the expansion cannot be accommodated in the pore system, but is restrained by the surrounding concrete, it induces tensile stresses in the concrete. The tensile stresses cause cracks, which affect the strength, stiffness, and fracture energy of the concrete as well as the bond strength between the reinforcing bar and surrounding concrete in damaged regions; see [16] and [17].

Two types of freezing damage can be distinguished, [18]:

1. Internal freezing damage caused by freezing of moisture inside the concrete. This may cause cracking and substantial reduction of strength and stiffness.
2. Surface scaling, which is usually caused by freezing of salt water in contact with the concrete surface. This damage usually results in spalling of the concrete surface, while the remaining concrete is mainly unaffected.

The effect of internal freezing damage can be modelled by changing the material properties for the concrete, see e.g. [19]. The effect of surface scaling can be modelled as changes in the geometry, i.e. reduction in dimensions, see e.g. [19].

Fatigue of reinforcement

The fatigue behaviour of steel reinforcement is similar to fatigue of elements in steel construction. The fatigue relevant parameters are the stress range, the number of stress cycles and discontinuities both in the cross section and the layout of the steel reinforcement, resulting in stress concentration at possible fatigue damage locations.

Fatigue life of steel reinforcement can be divided into a crack initiation phase, a steady crack propagation phase and brittle fracture of the remaining section.

The fatigue behaviour of the reinforcement can be represented by means of the S-N-diagram (Wöhler line) in a double-logarithmic representation. The nominal fatigue strength is commonly defined by the stress range amplitude at 2 million cycles. This value is called *fatigue category* and refers to a given S-N-diagram.

A literature review regarding the fatigue strength of various types of steel reinforcement has been performed in [1]. Furthermore, [1] includes new methods and guidance on assessment procedures. BA 38/93 'Assessment of the fatigue life of corroded or damaged reinforcement bars' [23] also gives guidance on assessment of fatigue life.

Stray current corrosion

There is a risk of corrosion of embedded or exposed steel caused by stray currents from an electrically powered railway passing a bridge.

The traction power is distributed to the trains via an overhead catenary wire system and collected by the train engines by pantographs.

In DC systems, the rails in the railway track are insulated from the ground in order to reduce stray currents, and the traction current will ideally return back to traction substations via the electrically continuous rail system. However, because of unavoidable imperfections of the rail insulation, parts of the return current will find additional return paths, for example through the soil and through the bridge structure.

In AC traction systems the rails are usually not insulated from the ground for safety reasons. Owing to the bi-polar nature of the current the amount of corrosion caused by AC stray currents is much less than corrosion caused DC stray currents of the same magnitude.

Provisions against the effects of stray currents caused by DC traction systems, please refer EN 50122-2.

The risk of corrosion from stray currents may be assessed by

- identifying the possible stray current paths
- estimating the magnitude of the stray currents

The assessment of the magnitude of stray currents heavily depends on the quality of the input data. Among the most essential data are:

- Traction system voltage
- Distance to the traction power substations
- Type of locomotives
- Frequency of train traffic
- Specific resistance of the soil

Impact

Impact is not treated in this deliverable.

Summary

Table 6 is a summary of temporal and spatial characteristics together with sensitivity towards load evolution and effects of climate change for concrete bridges as presented above.

Concrete bridges	
Temporal characteristics	In general (except impact), changes happen within years or decades.
Spatial characteristics	Corrosion, cracking, spalling, alkali silica reaction and impact can be of a global or localised nature. Fatigue attacks where stress raisers are present (small scale). Sulphate attack is generally of local nature. Freeze/thaw is generally of global nature.
Sensitivity towards load evolution	In general the degradation rate is influenced by live load intensity because it may ease penetration into the bridge due to structural cracking. Fatigue is sensitive both towards live load intensity as well as frequency.
Sensitivity towards effects of climate change	Corrosion increases with increasing temperature and humidity (both are needed) and the effect of corrosion and notches lowers the fatigue endurance. Furthermore, corrosion spreads more rapidly with increasing temperature and humidity (spatial effect).

Table 6: Summary for concrete bridges

7.5.4 Associated performance and limit States

General

This section looks at performance requirements in terms of structural strength/serviceability/durability. Both normal and accidental loads need to be considered.

Performance assessment of existing concrete bridges must include an overall safety evaluation for all critical cross sections against actual operating conditions. The evaluation has the purpose to identify and predict risks in terms of stability, strength and fatigue, and to localize damage that could lead to bridge collapse. Furthermore, for railway bridges comfort and deformation criteria's are also important issues related to risk of derailment etc. For a detailed treatment on methods of dealing with deterioration in assessment of concrete structures please refer e.g. [21] and [22]. For a state of the art review and proposal of new highly advanced methods on how to assess the maximum realistic capacity of concrete railway bridges please refer to [1].

As for any structure, the study of the operating conditions is essential. They concern several aspects of the life of the structure: changes in number of tracks, evolution of loads, reported failures, exceptional events (damage due to external sources)… Safety and serviceability questions arise versus modern loads; the question of assessing their remaining service life has to be asked.

For a comprehensive treatment of Service Life Design of concrete structures please refer to [20].

The concept of durability is essential the background of calculating the remaining lifetime, see e.g. [14].

In order to assess the residual life of deteriorated concrete structures, several aspects of the structural assessment of a deteriorating structure need to be addressed:

1. The need to establish the level of present performance by establishing the type, extent and cause of the damage
2. The establishment of the deterioration rate
3. The prediction of the loss of the structural capacity
4. The identification of the minimum acceptance level of performance
5. The urgency of intervention

7.6 Masonry bridges

7.6.1 Asset types

There are probably close to a million masonry arch bridges worldwide [36]. All are ageing and most are carrying loads well in excess of those envisaged by their builders. A recent survey [32] revealed that approximately 40% of the European Railway bridge stock comprises masonry arch bridges and that over 60% of these are over 100 years old.

Figure 15 shows a typical example of a masonry arch bridge with backfilled spandrels. Masonry arch bridges have one feature in common – an arch which acts as the primary structural element carrying the imposed load down to the foundations. The arch may take various shapes including; semi-circular, parabolic, segmental, elliptical, gothic pointed and may comprise different materials – see section 7.6.2 below. The backfill over the arch may be contained by spandrel walls which extend beyond the abutments to provide wing walls. Sometimes internal spandrel walls may also exist, especially for spans larger than about 12m. It is not easy to ascertain the internal construction simply by looking at the external faces, and it is important to develop a good knowledge of the particular bridge under consideration in order to understand its load bearing behaviour, its propensity to different defects and deterioration mechanisms, and its potential failure modes.

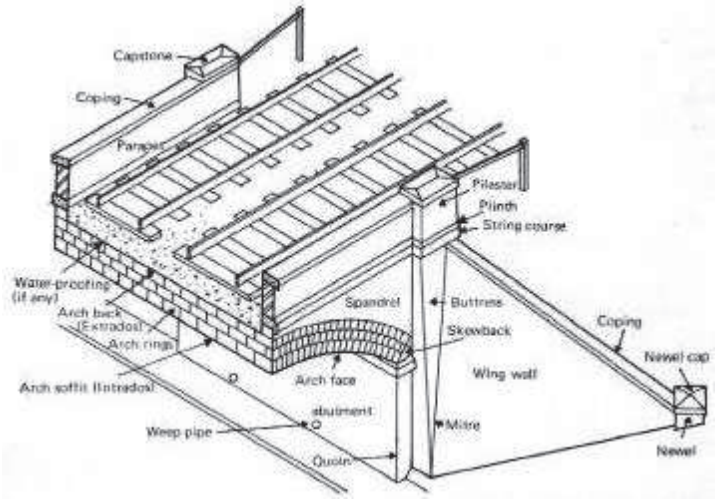

Figure 15: Typical example of masonry arch bridge [1]

It is also important to determine the type of foundation used and to understand whether it has been taken down to rock or any other type of fixed foundation. Where this has not been

possible, the type and form of piling employed is important in understanding how the bridge reacts to its loading.

Masonry arch bridges can be single or multi-span; they can be square, skew or curved in plan. In the case of multi-span bridges, the individual spans can vary considerably as can the height and proportion of the piers.

Railway bridges in Europe almost invariably contain multiple arch rings. These rings are either simply connected by uninterrupted mortar joints or crossed by headers which provide an interlocking effect and greatly increases the shear strength of the connection between rings (see Figure 16). The type of arch construction shows certain regional consistencies around Europe, for example arch bridges in Britain are generally built without headers, while in Southern Europe arches with interlocking headers are more common.

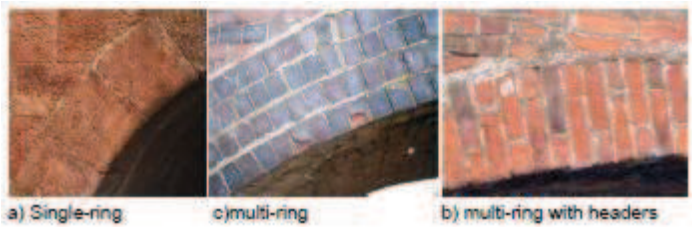

Figure 16: Different degrees of shear connection in masonry arches [1]

7.6.2 Material properties

The materials used in masonry construction include a variety of bricks and stone units, typically separated by beds and vertical joints comprising some type of mortar [1].

Mortars

Depending on the type of masonry, the volume of mortar per unit volume varies significantly, from 0% in the case of some ancient arches which were constructed from perfectly fitted dressed stone to over 20% in the case of some random rubble bridges. The mortar, historically, was a lime or lime/cement mortar, hence being "plastic" and forgiving. The proportion of mortar, its type and characteristics are an important influence on the performance and behaviour of a masonry bridge's structural elements, particularly the arch barrel. Modern Portland-type cements have dominated the 20th century but such cements were not used commonly until the second half of the 19th century.

Bricks

Research conducted by British Rail in the UK has revealed the considerable difference in strength and modulus of various types of bricks (whose colour - red, yellow, blue- is influenced by the raw clay material as well as the addition of minerals and pigments) used in the construction of railway bridges between 1840 and 1910. For example, the elastic modulus varies between 5 and 15 kN/mm^2, and the corresponding characteristic strength between 16 and 70N/mm^2.

Stone

Sandstone and limestone are the most commonly used types of stone in masonry arch bridges.

Soil and backfill

Backfill is almost certain to be local material that was convenient to the site and may vary considerably in nature both vertically and laterally. Historically there may have been poor

specification of the quality and consistency of the fill (materials ranging from ash, clay, sand, gravel, peat and concrete have been found in arch backfill) or its method of placement and compaction. Additionally recent works may have removed the surface layer of material and replaced it with an engineered fill, or incorporated reinforcement such as concrete slab or steel plate elements. Parts of the fill may also have been lost through washout. A first stage is to determine this potential variability before making a judgement as to the likely consequences on bridge capacity.

7.6.3 Damage and deterioration mechanisms

Masonry bridges are generally durable and resilient to environmental aggressors. However, with time, environmentally driven deterioration (e.g. due to water ingress, temperature variations, exposure to high winds and combinations of the above) can take place, particularly if movements have occurred within the structure, which may promote the formation of cracks and settlements. These movements could be the result of long-term soil changes leading to soil-structure interaction, hydraulic loads on pier and foundations (and occasionally on the arch itself) but can also be attributed to live loading over the bridge.

The mortar is often the weakest constituent in the materials making up the masonry arch bridge (as it should be since this is the most readily repairable part), hence defects are often seen first along the mortar lines, though this cannot be generalised. In the Sustainable Bridges project [33], an attempt was made to classify the main types of defects observed on masonry bridges. Figure 17 presents the proposed classification. Most of these defect types have an influence on both the condition and the load carrying capacity of the bridge – the exception is contamination, which is only deemed to affect condition. It should be noted that this represents just one possible classification scheme for defects, others, possibly more in line with schemes applied to other structures, could equally be developed, see eg. [34] and [35].

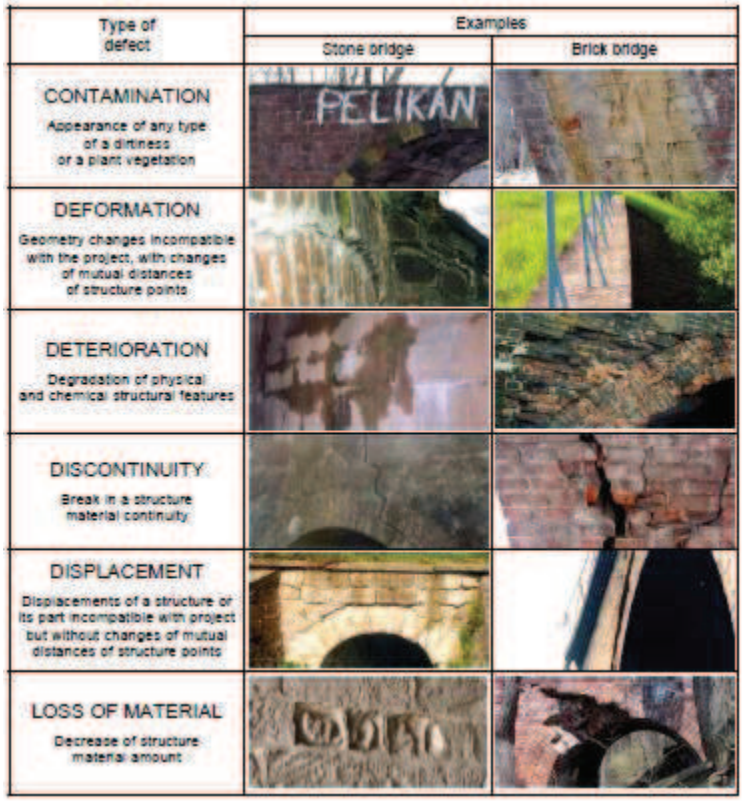

Figure 17: Classification of main types of defects on masonry arch bridges

Fatigue can also occur in masonry bridges, as typically attributed in other materials and forms of construction to repeated heavy load cycling. Although there are still some doubts as to the exact nature and manifestation of fatigue damage in masonry arch bridges, the experience of maintenance engineers with regard to very heavy traffic loads experienced in the last 30 to 40 years suggests that there is damaging potential akin to a fatigue process [1]. There is limited evidence on the extent to which the failure mechanism and corresponding load is affected by such cycling, and this remains an area of active research in the masonry arch community.

7.6.4 Associated performance and limit states

The properties of masonry are influenced by a large number of factors, such as material properties of the units and mortar, arrangement of bed and head joints, anisotropy of unit, dimension of units, joint width, quality of workmanship, degree of curing, environment and age. Unfortunately some of the most important test data required in masonry analyses are known to be very variable and sensitive to the test methods employed and hence may not represent the performance of the masonry in the actual structure.

Masonry is a brittle material with very low tensile strength that can sustain substantial plastic deformation in compression. As a result of their inability to resist bending forces, masonry structures under loading will deform and crack unless they can resist those loads through a path of compressive internal forces. As a consequence of this, cracking is quite common in masonry structures and should not be automatically associated with structural distress. Moreover, the durability of masonry is not as severely affected by cracking as, say, reinforced concrete, and in many cases the plasticity of most historical lime mortars will allow those cracks to be gradually sealed by autogenous healing.

As highlighted in the Sustainable Bridge project outcomes [1], the application of Limit State Design Philosophy for masonry bridges is at variance with their application to other materials. This is because there is usually no structural metal in the bridge to corrode and hence limits on crack widths etc have little meaning. The guidelines developed in that project, therefore offer the following definitions:

The ultimate limit state (ULS) for masonry arch bridges can be defined as – the condition at which a collapse mechanism forms in the structure or its supports.

The permissible limit state (PLS) for masonry arch bridges can be defined as – the condition at which there is a loss of structural integrity which will measurably affect the ability of the bridge to carry its working loads for the expected life of the bridge.

In determining the ULS (collapse load) the bridge owner is assured of the ultimate capacity but little else. Based upon UK field tests in the 1980's two modes of failure were reported – the formation of a mechanism comprising hinges (4 in number for a single span bridge) or a 'snap-through' failure. These modes of failure were, to some extent, pre-determined by the nature of the loading system (a full-wide 'knife edge load' applied at about the quarter span). The loading was applied monotonically through to failure.

At this point that it is very important to appreciate that although there is some general agreement as to the definition of the ULS as the condition at which a collapse mechanism forms in the structure or its supports, no such agreement exists for other limit states. In addition, it is worth noting that a Durability Limit State and a Fatigue Limit State are also considered, though both are more problematic to quantify when compared to metallic or concrete structures.

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8. Tunnels

8.1 Asset types

A range of railway tunnels exist across Europe of varying type, age and construction method. The principal types of railway tunnel are outlined below, categorised by their primary support systems;

- Unlined tunnels
- Masonry lined tunnels (brick or stone block-work)
- Metal lined tunnels (steel or cast iron)
- Concrete lined tunnels (pre-cast segmental, in-situ or sprayed)

Construction method

The method used to construct a tunnel is often a key factor in determining its possible modes of deterioration and failure. The construction technique adopted will be dependent on the available technologies at the time of construction, local geology and groundwater, required depth and length of the transport corridor and a range of economic factors. The two techniques that were used to build most railway tunnels in the UK are cut and cover and boring [1].

- *Cut and cover* – The tunnel structure is built in a large open excavation and covered by fill. The surface is then returned to use.
- *Boring* – The tunnel is constructed by sinking shafts or driving adits from the ground surface at various points along the line of the tunnel and excavating out laterally from the base. The construction shafts are either left open for ventilation, or sealed and backfilled.

Typical cross sections of cut and cover and bored tunnels are shown in Figures 18 and 19 respectively.

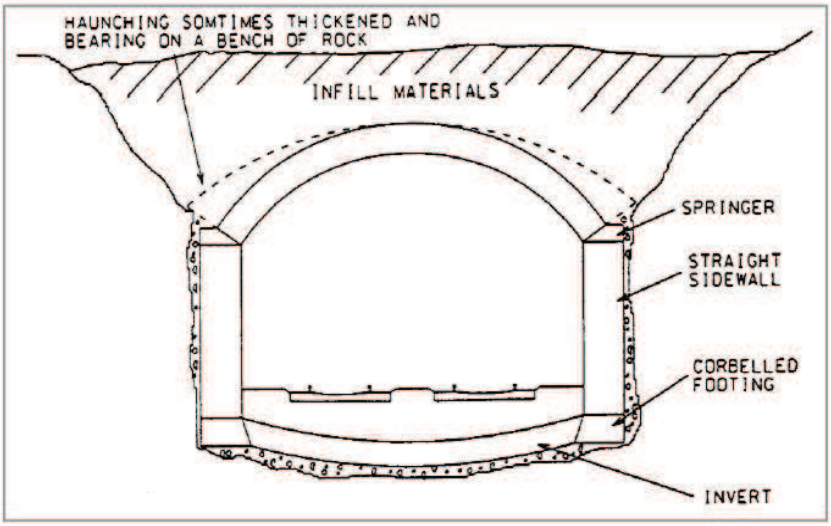

Figure 18: Typical section through a cut and cover railway tunnel [1]

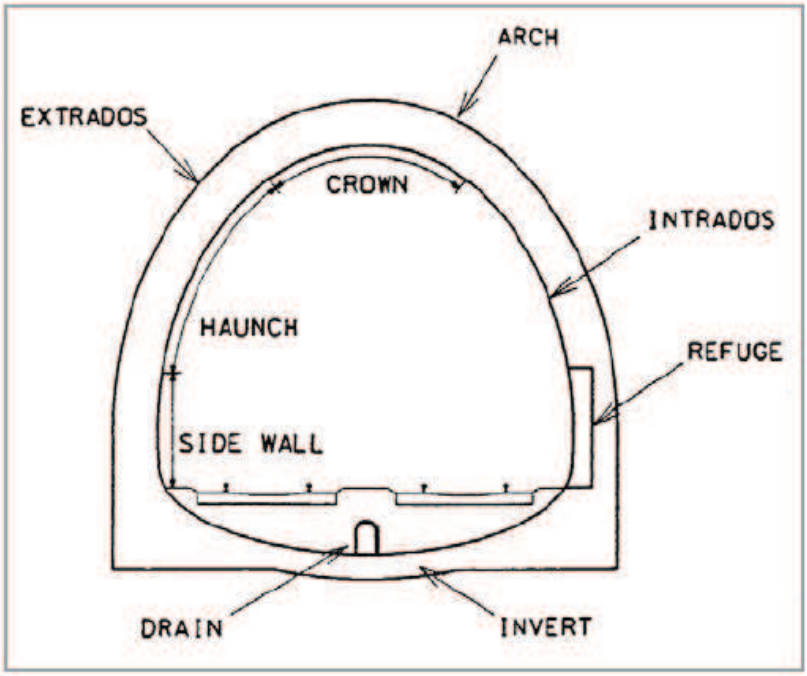

Figure 19: Typical cross-section of a bored railway tunnel [1]

Materials

In the case of unlined tunnels, the in situ geology provides the primary support system for the structure. In such cases the excavation route and profile will be chosen so as to optimise the strength and stability provided by the existing geological structure.

In the case of lined tunnels, over time the lining material used in tunnel construction has progressed from timber, through stone, brick and cast iron to modern steel, pre-cast concrete, reinforced concrete and sprayed concrete.

Tunnel types found across Europe

The mix of tunnel types and total length of tunnels varies between different European railway networks. For example the number of tunnels at Hungarian railway lines is very small (19). The types of tunnels are reinforced concrete, concrete lining on formerly masonry structures, masonry (only stonework) lining and unlined tunnels. Deutsche Bahn typically have only masonry and concrete lined tunnels. Network Rail have a wide range of tunnel types including unlined, masonry lined, concrete lined and metallic lined, however masonry (brick and stone lined) tunnels are the most common.

Selection of assets for further study in this report

Tunnel assets included in this section are bored tunnels and tunnels constructed by the cut and cover method where the depth of overburden cover is such that the effects of actions imposed on the lining originating from the surface land are negligible. Cut and cover structures where the depth of over burden is such that actions imposed on the lining originating from the surface land is influential on the capability of the tunnel lining will be dealt with as bridges.

Only lined tunnels and lining materials are included in this section; unlined tunnels are not included. This is because there is currently an internationally recognised understanding regarding the structural behaviour of rock masses around unsupported (unlined) tunnels using either existing standard geotechnical systems or bespoke management systems developed specifically for unlined tunnels. The degradation of unlined tunnels is readily

detectable allowing for timely and effective intervention. On the other hand, the understanding of how the degradation in durability of brick and masonry linings on ground – support interaction in older tunnels and its effects their ongoing capability is less readily understood; and therefore it is this area that the project feels it should concentrate its efforts on.

8.2 Damage and deterioration mechanisms

In lined tunnels, the lining component forms the primary support system for the structure. Therefore the condition of this type of tunnel is governed by the performance of the lining structure and its construction materials. Structural deterioration of the tunnel lining can lead to failure, causing injury or loss of life as well as economic loss related to service disruption and costly remedial or replacement works. In addition, material deterioration due to external or internal tunnel conditions can lead to changes in the material properties. This can cause a range of issues from minor defects such as staining to a reduction in the strength of the lining. If left untreated, material deterioration mechanisms can lead to structural failure over time. Sections 8.2.1 and 8.2.2 will outline the principal causes of structural and material deterioration respectively.

Fire damage is a unique mechanism for structural and material degradation in tunnels, so this subject is treated separately in Section 8.3.2. The performance of other tunnel components such as shafts and cross passages can also affect the overall condition of the asset, and this is discussed in Section 8.4.

8.2.1 Structural deterioration

The CIRIA guide C671 on the inspection, assessment and maintenance of tunnels [1] provides detailed information on the main deterioration mechanisms for lined tunnels. The key structural deterioration mechanisms affecting lined tunnels are outlined below.

Geotechnically driven faults

- *Groundwater issues* – Progressive washout failure due to water ingress. Poor drainage leading to softening. Groundwater can cause certain soils such as clays and marls to swell. This causes an increase in ground loading on which may exceed the resistance of the structure. Groundwater may also contain aggressive chemicals which can attack the tunnel lining material (see Section 8.2.2 for more details on material degradation). The flow of water through porous soils can also cause erosion thereby undermining the tunnel lining. In areas where there is a gap between the lining and the ground, ground movements may cause debris to accumulate thus increasing loading, and rock movements may induce point loads on the lining structure.
- *Ground movements and rock movements* – The majority of movement occurs shortly after construction as the ground responds to stress redistribution. However movement can also continue over a number of years and may cause degradation of ageing infrastructure tunnels. Long term stress changes can result in lining distortions which have the potential to reduce tunnel clearance to the point that the passage of trains is compromised. In extreme cases lining distortion can lead to localised collapse of the tunnel structure.
- *Differential loading and settlement* – Excess water beneath the sidewall footings can result in softening in this area causing settlement. Both differential loading and settlement can induce tensile bending, shear and tension within the lining structure. Settlement commonly causes tension cracks to appear in the horizontal plan. Shear cracks can occur as a result either differential loading or settlement and generally appear as a lip or

step in the lining. Typical forms of lining deformation observed in brick-lined tunnels are shown in Figure 20 below:

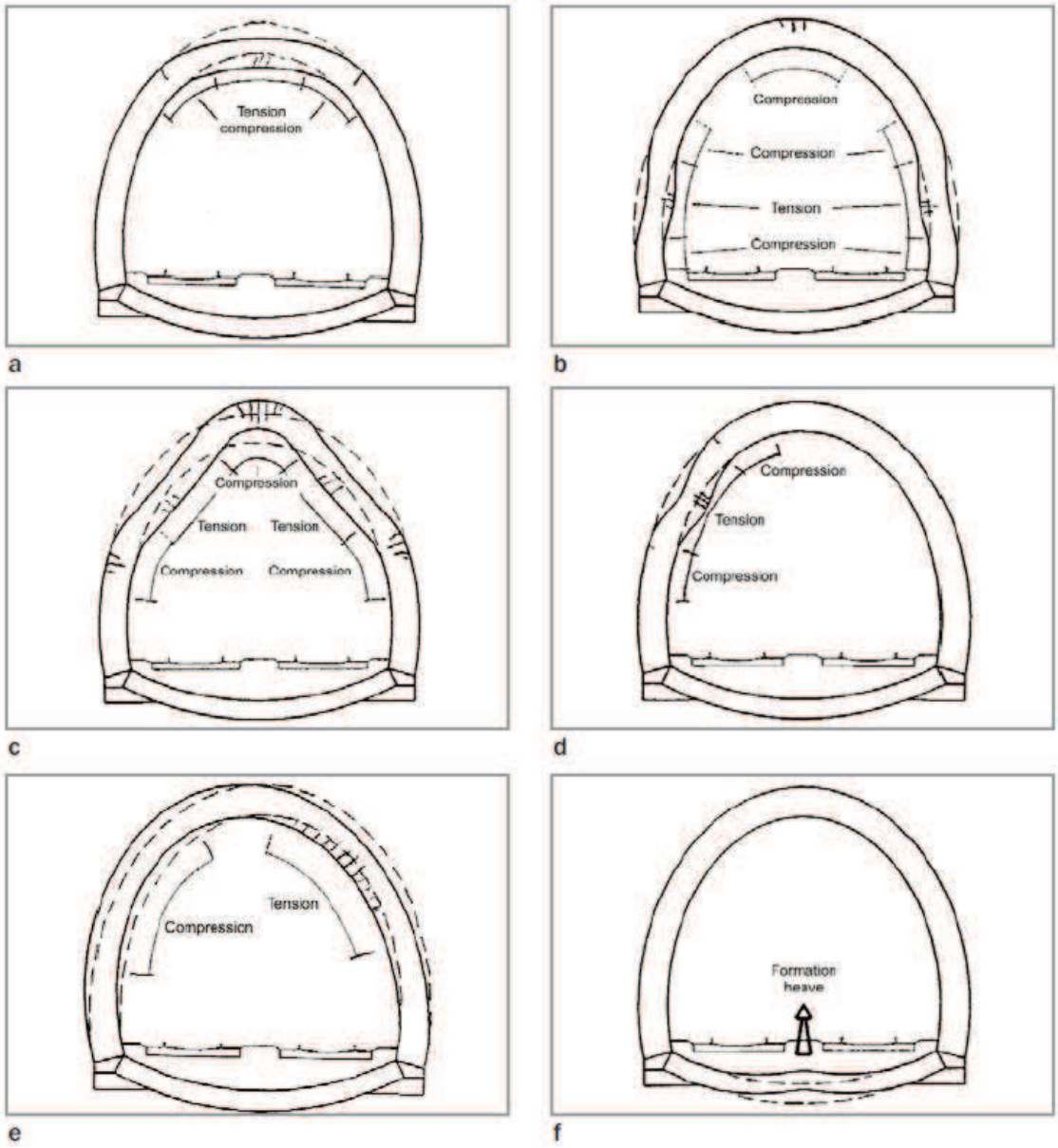

Figure 20: Typical forms of lining deformation in brick-lined tunnels [1]

- *Soil expansion* – Some clays and soft rocks such as shale and mud rocks are susceptible to expansion when exposed to air or water, or when their confining pressure is reduced. Such expansion can cause an increase in pressure exerted on the lining extrados. Soil expansion in the underlying strata can result in rising formation and subsequent failure of the invert.
- *Mining subsidence* –If a tunnel has been constructed in an area with a history of mining deep seated deformations and movements can occur resulting in subsidence (see Section 6 for more details on mining subsidence). In the case of tunnels, this can cause the tunnel arch to sink in relation to the formation level, leading to reduced clearance.
- *Interaction with other tunnels or cross passages* – extra load from new construction of adjacent tunnels or other structures.

Repair and enhancement activities

There are very few cases where it would be practical, economically justifiable or socially acceptable to substantially replace ageing tunnel assets [1]. Therefore a tunnelling asset may be considered to have an infinite design life, necessitating a rigorous and proactive approach to maintenance. In many cases repair and enhancement works carried out on an existing tunnel structure will alter the fabric and behaviour of the lining. Under these circumstances assessments should be carried out to determine the potential long term and short term effects of carrying out the work. Badly specified tunnel repair and modification work using unsympathetic techniques or incompatible materials can contribute towards structural instability in the following ways;

- *Invert modifications* – Modification made to the invert may reduce its capacity to resist heave or swelling of the tunnel floor. It is important that temporary works are designed, implemented and monitored for invert modifications/repairs to mitigate the risk of instability of the lining during the temporary state. Instability of the lining while invert modifications are taking place can have catastrophic consequences if not controlled.
- *Drainage work* – remedial solutions that involve the introduction of more efficient drainage or drying out the lining will result in changes in water flow through the surrounding soil. This can induce surface settlements and has the potential to cause structural problems at the surface.
- *Segmental lining repairs* – Individual lining components may be replaced or repaired as part of maintenance work. If replacement lining components are handled careless or poorly installed they may cause future deterioration by allowing water ingress through joints or cracks.
- *Patch repairs* – patch repairs in masonry linings can create in a hard spot in the structure if replacement materials are significantly stronger or stiffer than the existing materials that surround it. This can change local stress distributions, potentially causing deformation and subsequent failure. Local patch repairs in deteriorating concrete may cause deterioration to accelerate in the adjacent areas through the incipient anode effect.
- *Inadequate temporary formwork during repair* – Some tunnel repairs, for example renewal of brickwork over the crown of the tunnel may necessitate the use of temporary formwork. Improper design or premature removal of formwork may cause structural instability that can lead to damage or ultimate failure.
- *Repointing* – before carrying out repointing on masonry linings it is necessary to clean out the joints using either mechanical tools or high pressure water jetting. The use of inappropriate cleaning methods can cause damage to the surrounding masonry units.
- *Application of protective coatings* – Where protective coatings are applied to existing linings incorrect specification and application of the coating can result in damage to the lining material, particularly in the case of old stone and brick. Before application of protective coatings it is necessary to clean of existing surface and this can also cause damage. There is a further risk that if the substrate material is not dry at the time of application, the waterproof coating can actually entrap water and thus worsen deterioration. Similarly, the coating may block drainage paths or water outlets causing a build up of water behind the lining.
- *Routine cleaning* – tunnel cleaning is often carried out as part of a routine maintenance cycle. The use of inappropriate cleaning methods can cause damage to the tunnel lining, especially if they are used at regular intervals.
- *Bolting into cast iron* – It may be necessary to drill into cast iron lining segments when carrying out plate repairs or installing new operational equipment inside the tunnel. In

these cases there is a risk of shattering the brittle metal if inappropriate drilling equipment is used.

- *Welding cast iron* – Welding may have been carried out on a cast iron lining as part of a previous repair. If the temperature was not strictly controlled during the welding process it is possible that in situ stresses may have been created within the lining which can result in distortion, hardening and extra cracking.
- *Bimetallic corrosion* – If a different metal is used in the repair of an existing metallic lining there is a risk that bimetallic corrosion can occur. This occurs when metals of significantly different nobility come into contact and form a galvanic couple, and is a particular problem in wet environments. This sort of corrosion usually occurs in localised areas, especially around fixings.

Operational loading

- *Cyclic loading and fatigue effects* – loading from the repeated passage of trains principally affects the invert component of tunnel structures. There is little known about the effects of cyclic loading on masonry lined tunnels however recent research indicates that is could be of structural significance. In addition, MAV report that crack formation in their tunnels is highly sensitive to load evolution.

Water ingress

- *Ice formations* – In extreme weather conditions the wetting of the lining surface can lead to formation of icicles. Such damage generally attacks over a small area where the water ingress is extensive. Climate change may affect the intensity and frequency of ice formation and damage. Ice formations can impair clearance or fall onto track/train.
- *Freeze thaw actions* - Freeze and thaw cycles can cause damage to the wet surfaces of concrete and masonry linings. MAV observe that this problem occurs mainly in short tunnels in winter time.
- *Ring separation* – water ingress through masonry linings can cause washout out of water between courses, thus causing ring separation or voiding within the structure. This can result in the loosening and fall out of bricks in localised areas or in more extreme cases cause the complete collapse of one or more courses.

Vegetation

Vegetation is commonly found in the areas around tunnel entrances, shafts and in areas of shallow cover. Vegetation causes the biggest problem in masonry lined tunnels, but can also cause degradation in segmental linings.

- *Risks from vegetation growth* – root growth can cause local damage to structural elements, particularly in masonry lined tunnels. Settlement or heave of tunnels can also occur as a result from the seasonal changes of mature deciduous trees. Vegetation growth can also prevent or impair proper examination of the structure
- *Drainage impairment* – vegetation can accumulate in drainage channels causing blockage. This can cause a range of issues related to groundwater and water ingress as discussed above. The presence of plant roots can also inhibit the drying out process of wet masonry

8.2.2 Material deterioration

Tunnel linings may be of masonry, concrete or metal construction. Each of these lining materials has unique characteristics and properties and can therefore deteriorate in different ways. The principal material deterioration mechanisms for masonry, concrete and metal tunnel linings are described below.

Masonry linings

The deterioration of stone, brick and mortar is covered in detail in Section 7.6 on masonry bridges. The majority of masonry deterioration mechanisms are related to the presence of water and chemical contaminants. A key mechanism that is particularly relevant to tunnel structures is sulphate attack as sulphates are often present in the groundwater, soil and rock that surrounds tunnel linings.

MAV have observed that deterioration of their stone linings occurs over large areas and the intensity depends on the type of stone or brick and presence of aggressive agents in the internal and external tunnel environment. It has gradual changing characteristics and develops slowly over time.

If left untreated masonry deterioration can result in structural instability and failure, for example the fall of bricks or stones onto the track or passing trains.

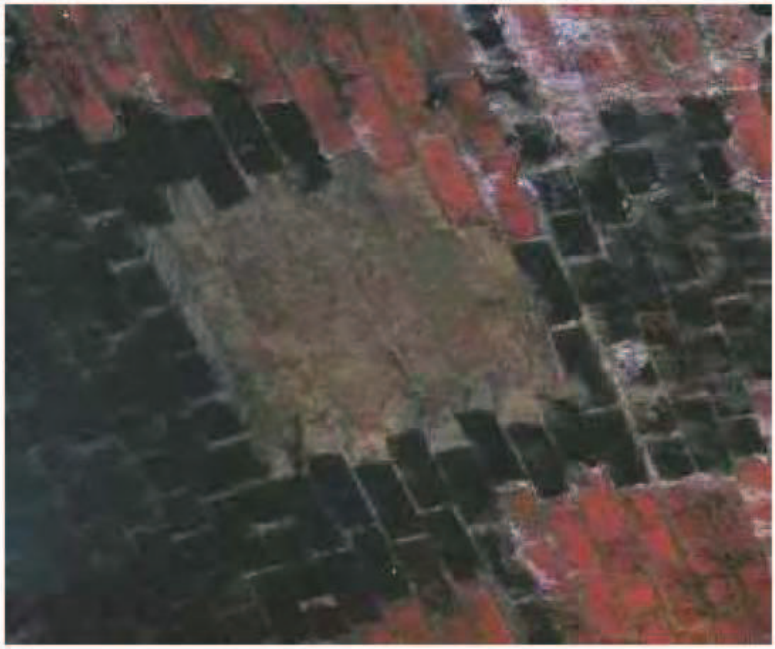

Figure 21: Displaced brickwork in the crown of a masonry lined tunnel

Concrete linings

The deterioration of concrete is covered in detail in Section 7.5.3 on concrete bridges. Problems may include cracking and spalling, reinforcement corrosion, sulphate and acid attack and freeze thaw damage.

As is the case for masonry linings, a particularly relevant mechanism for concrete tunnel structures is sulphate attack as sulphates are often present in the surrounding ground and groundwater. In the case of segmental pre-cast concrete lined tunnels, cracking and spalling may occur as a result of changed loading conditions or construction damage. Segmental concrete linings commonly have gaskets between the segments which form part of the waterproofing system. Degradation of the gasket can lead to leakage causing corrosion of

the lining. Freeze-thaw damage is typically only seen in areas exposed to cyclic fluctuations in temperature for example near portals and shafts. Freeze-thaw damage is unlikely to be a problem in deep tunnels as they are well insulated by the ground.

Concrete deterioration may lead to a loss of strength, for example by a reduction in the effective section thickness or loss of reinforcement, and thus has the potential to cause structural failure.

Figure 22: Example of acid attack in a concrete lined London Underground tunnel [1]

Metal linings

Deterioration of metallic materials is covered in detail in section 7.3 on metallic bridges. Potential problems include corrosion, rusting, distortion, movement of segments and cracking.

Corrosion of metal tunnel linings is governed by the agressivity of the surrounding soil, with peat and clay being two of the most corrosive types. The rate of corrosion is controlled by soil porosity, drainage and groundwater constituents. Cracking of metal tunnel linings may occur as a result of impact damage, changed ground or operational loading or as the result of maintenance work, for example weld repairs.

Metal deterioration can result in section loss and lead to structural failure as a result of the reduced strength capacity.

Figure 23**: **Example of corroded cast iron lining in a London Underground tunnel [1]

8.2.3 Fire damage

Fires in tunnels are a particularly serious threat as they can reach higher temperatures than fires occurring above ground. The insulated underground environment also means that high temperatures can be sustained over a much longer time period. Another important characteristic of tunnel fires is that they are often caused by vehicles, and the rupture or explosion of fuel tanks can cause the growth, extent and severity of fire to increase rapidly.

Fires can place exceptional demands on tunnel structures, causing movement of the tunnel lining as well as changes to its stiffness, effective section and interaction with the ground. A detailed description of the effects of fire on concrete, masonry and metallic lined tunnels can be found in CIRIA C671 [1].

Fire damage develops very rapidly once instigated; however such events are accidental in nature so it is impossible to predict when fire damage might be triggered during the life of a tunnel asset. Whilst the likelihood of tunnel fires may by reduced through the implementation of improved health and safety measures, modern tunnel fires continue to occur. It may be possible to use probabilistic methods to determine potential costs of reactive work to remediate fire damage across a stock of tunnels.

8.2.4 Performance of other tunnel elements

There are a number of major elements other than the lining that may exist within a tunnel asset. The deterioration of the tunnel lining may be linked to the performance of other tunnel elements as described below.

Shafts, shaft eyes and chimneys

Shafts can cause problems for both the tunnel and the land above it. In certain circumstances a shaft may collapse, causing severe damage to the tunnel below. Other less severe but significant problems might be the ingress of water or circulation of cold air through open shafts. These effects can cause degradation such as corrosion and freeze thaw damage to occur. Where friction is lost between the shaft lining and surrounding ground the additional load is taken by the tunnel structure below. This can result in overloading problems for the tunnel lining.

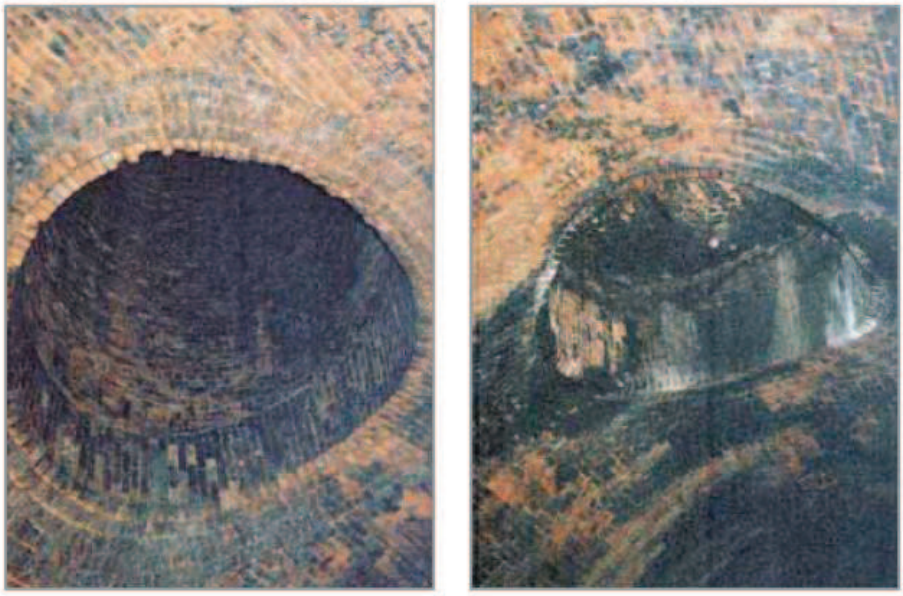

Figure 24: Examples of open and closed shafts in a brick-lined tunnel [1]

Cross-passages, adits and headings

Cracks may occur in cross passages or adits as a result of overloading or inadequate strength of the cross passage structure. This can allow water ingress and gradual deterioration.

8.3 Associated performance and limit states

Tunnels are thought to be irreplaceable and therefore intervention is of a pre-emptive nature with early maintenance being the driver for continued serviceability.

Assessment is often of a qualitative nature rather than quantitative. One example is historic masonry tunnels where it is difficult to carry out numerical modelling for limit state conditions because of many changes in material condition and durability through the length of the tunnel.

The quantitative assessment is carried out as a detailed examination and the parameters that govern limit state conditions are required to be assessed during the examination. The DARTS project (Durable And Reliable Tunnel Structures) is one project where serviceability and ultimate limit states were investigated in relation to deterioration of concrete tunnels. The serviceability limit state relates to depassivation of the reinforcement, cracking and spalling (only economic consequences) and ultimate limit state relates to spaling and reduction of cross section capacity and bond.

8.4 Analysis of historic degradation trends

The performance state of a tunnel asset over its life will depend upon the deterioration rate of the asset in question. Analysis of repeat condition data obtained during inspections may help to develop models for the degradation trends of various tunnel types.

8.4.1 Network Rail Tunnel Condition Marking Index (TCMI)

Network Rail has designed a system called the Tunnel Condition Marking Index (TCMI) which they use to measure and demonstrate the change in condition of their tunnel stock over time [3]. The current system can be applied to brick, stone and concrete lined tunnels (excluding segmental concrete) as well as a limited number of metal components such as cast iron shaft eyes and Armco linings. There are plans to incorporate other tunnel construction types such as segmental concrete and segmental cast iron during subsequent phases of development of the TCMI system.

The TCMI system provides a quantitative condition mark for each tunnel after it has undergone detailed examination. The condition mark grades the structure on a scale of 0 to 100 where 100 indicates a tunnel in ‘perfect’ condition and is based on non-judgemental recording of defects by the examiner. TCMI scores are generated at the minor element, major element or tunnel bore levels. The condition of each element is recorded as an alphanumeric severity/extent code on the defect record sheets during inspection. These codes capture information on the type, area or length and number of defects observed and are fed into various algorithms to produce a TCMI score. Construction defects are noted but not scored. The rationale behind this is that they are in-built defects and not a result of deterioration of the lining over time. Repaired defects may still be recorded, for example if a bulge has been restrained by anchor bolts but the bulge is still present. In such instances the repair work will not improve the TCMI score, but it should arrest the decline in score over time, provided all other conditions remain the same. For detailed information on the TCMI algorithm and inputs please refer to Network Rail standard NR/L3/CIV/0006/4C [4]

The TCMI scores for each element are banded into red, amber and green categories as shown in the example output report in Figure 25. Reporting in this way not only provides an overview of the condition of the whole tunnel but also enables easy location of sections that are in a poor condition.

SUMMARY TCMI SCORES	
BORE	96
PORTAL 1	99
PORTAL 2	89

Lowest Score & Location	
52	TS2 ; USC
68	TS2 ; DSC
74	TS1 ; USC

Key:	>	<=
	0	40
	40	80
	80	100

Major element scores in bold
Minor element scores in normal font

Shaft Eye Scores

	EO1
	100
EYU1	100
EYR	100

Side Voussoirs

	SV1
	100
FRV	100

Adits/Cross Passages

	AD1
	100
LSW	100
CRN	100
HSW	100

Tunnel Bore Minor Element Scores :-

	TS1	TS2	TS3	TS4	TS5	TS6	TS7	TS8
	85	**74**	**100**	**100**	**100**	**100**	**100**	**100**
USW	88	88	100	100	100	100	100	100
USH	90	86	100	100	100	100	100	100
USC	74	52	100	100	100	100	100	100
DSC	96	68	100	100	100	100	100	100
DSH	81	88	100	100	100	100	100	100
DSW	85	86	100	100	100	100	100	100

Figure 25: Example TCMI output report [3]

By examining repeat TCMI data it would be possible to analyse degradation trends for different tunnel types over time. By translating specific historic tunnel examinations into TCMI defect extent and severity codes and evaluating the retrospectively applied TCMI scores from each of these historic reports, it will be possible to obtain a tunnel specific degradation curve for a given tunnel. By further evaluation, historic interventions could also be applied to the curve to evaluate the effectiveness of maintenance / repair schemes in terms of attaining optimum whole life cost.

8.5 References

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9. Track

Railway track faces different types of traffic, either mixed or pure passenger traffic respectively freight traffic. However, in Europe mixed traffic is dominating with exception of high speed lines. MAINLINE is focusing on prolongation of service life of existing track and therefore on conventional ballasted mixed traffic lines.

Not part of this research is the influence of different rolling stock and the interactions between wheel and rail, which can be described by the wheel to rail forces and the equivalent conicity.

9.1 Asset types

MAINLINE is aiming for optimisation of existing infrastructure assets, in term of track ballasted superstructure together with open track and switches and crossing.

Track itself must be understood as a civil engineering structure consisting of the foundation (substructure: subsoil, load distribution layer, water drainage system) and the superstructure (rails, fasteners, rail pads, sleepers, and ballast).

Figure 26: Track structure (Piereder, ÖBB)

In Europe all main track components differ not only between the countries but also within each country. Rails normally differ in profile (rails are still produced from 18.3 kg/metre to 71.4 kg/metre) and steel grade (R200, R260, R320, R350, R350HT, R370CrHT, R400HT). A large number of old fastening systems can be found, while there are just a few modern systems available, but all differ due to rail foot width and the sleeper type. The sleeper geometries differ in a wide range from country to country. Generally wooden sleepers and concrete sleepers are in use, while steel sleepers especially for main lines are less common. For main lines and high speed lines concrete sleepers are more and more equipped with under sleeper pads. Artificial wood sleepers can be found for very specific circumstances. Especially the ballast quality differs in a wide range. Volcanic rock with high resistance against abrasion is in use as well as all types of granite and weak limestone or even slag.

In case of turnouts these variations are topped by different turnout geometries and technologies for the drives and signalling devices, not to speak about heating.

However, the behaviour of the total system is relevant for its performance and this performance again is influenced by the behaviour of all components and the interaction

between these components. The performance, respectively the deterioration is influenced by the traffic volume, of course, but also by the alignment to a large scale. Sharp curved tracks are subject to different deterioration and damage mechanisms compared to straight track sections.

Generally, the above statements are true for turnouts as well. The geometry of switches influences the behaviour (straight versus curved turnouts) as well as operational use (percentage of diverging trains). The specification of components must also cover the tongue rail and the frog type.

Thus investigations must cover the behaviour of the entire track, respectively turnouts.

9.2 Damage and deterioration mechanisms

In general track status can be described by two factors, the geometrical state and the structural state of all track components. However, these two factors influence one another, as a poor geometry leads to high forces degrading the structural state of the components and vice versa.

It is the purpose of maintenance to keep track within its geometrical limits (track gauge, longitudinal level, alignment (straight section, curve, transition curve), cross level (super elevation), and twist) and simultaneously to ensure long service life in minimising the forces within track.

To make it even more complicated, deterioration depends on weather conditions, humidity and other external conditions. So the seasons even influence the risk for different types of failures (buckling, rail breaking).

However, track and turnout deterioration is not just defined by the components behaviour, but by the interactions of all components and thus the behaviour of the civil engineering structure named 'track' respectively 'turnout'.

Track Geometry Behaviour over time (Ballast)

Track geometry data and track status data (age of components, executed maintenance actions) are monitored in order to find the optimal points in time for maintenance actions (tamping, ballast cleaning, additional single failure tamping) or re-investment.

The degradation rate b of the exponential approach ($Q_{time} = Q_{initial} \times e^{b}$) is thereby itself a function of

- Traffic volume
- Axle load
- Track design (radius)
- Superstructure (rail profile, sleeper type, ballast quality)
- Subsoil quality

The subsoil quality (subsoil itself, load distribution layer, drainage system) influences the overall behaviour, respectively the ballast. Under extreme boundary conditions the overall behaviour can change from an exponential function to a logarithmic function during the first years of service.

Ballast deterioration means

- Cracking of ballast stones
- Abrasion
- Fouling

- Contamination
- Appearance of mud spots

Track geometry is monitored by track measurement cars. There are two different principles the measurements are based on: cord or inertial measure systems. Data from the two different systems cannot be compared directly, i.e. a transfer function is needed.

Single failure reports also indicate the overall behaviour of a certain line-segment.

Rails

Continuous welded rails and jointed rails make the main difference in terms of rail degradation. Within continuous welded rails differences in temperature lead to stresses in the rails and thus provoke the problem of buckling. Consequently main focus must be addressed to high side resistance of track and thus sufficient and clean ballast bed at the sleepers faces. This problem is reduced on jointed rails, as temperature-stresses can be reduced by expansion of the rails. However, the joints themselves have high maintenance demands. This is an example demonstrating the interaction of the different track components is important and analysing the components separately is not recommended.

There are different impacts on rails caused by different mechanisms consequently leading to different damages and therefore maintenance actions. These are listed below.

<u>Wear (Side-Wear)</u>

This is actually important when it comes to narrow curved tracks. The degradation rate is influenced by

- Traffic volume
- Axle Forces (steering or non-steering axles)
- Radius
- Rail Steel Grade

Rail wear can be monitored with optical devices by track measurement cars.

<u>Rolling Contact Fatigue (RCF)</u>

For tracks with wide curves, RCF is much more relevant than side-wear. The degradation rate (crack depth growth rate) is influenced by

- Traffic volume
- Axle Forces (slip, esp. Locos)
- Radius
- Rail Steel Grade

RCF failures can be identified with eddy current measurements.

<u>Fatigue</u>

The multiple changes of compression and tensile stresses may cause an exceeding of the fatigue resistance. This process is triggered by

- Traffic volume
- Axle Load Distribution
- Rail Profile
- (Temperature)

- (Track Geometry)

Indicators of an exceeded rail fatigue resistance and rail breakages may be identified using ultra sonic measurements.

Corrugation

This process is triggered by

- Traffic volume
- Radius
- Total elasticity of track
- Rail Steel Grade

Corrugation waves are measured by the recording car.

While side-wear, corrugation, and RCF can be handled by rail grinding, an exceeded rail fatigue resistance leads necessarily to the exchange of rails.

Rail Pads

Again, this is mostly a phenomenon of curved track. Worn out rail pads have to be changed, in order to reach appropriate service lives of concrete sleepers. The wear of rail pads is a function of

- Traffic volume
- Radius
- Horizontal Rail Forces
- Rail Pad Stiffness

Rail pad wear can be identified by monitoring the rail inclination.

Fasteners

Due to overloading and/or geometric defaults, fasteners can simply break or loose their clamping force.

Sleepers

Degradation of wooden sleeper is very often the missing frictional connection or the process of rotting. This is mostly influenced by the sleeper age and the drainage situation. The deterioration process is influenced by

- Traffic volume
- Radius
- Axle Load
- Humidity

The behaviour of wooden sleepers can be monitored by measuring the gauge and the rail foot distance.

Concrete sleepers show very little deterioration as long as ballast bed shows a sufficient thickness, respectively the elasticity of ballast bed is working properly. Deterioration is materialised as cracks.

Concrete sleepers can under some circumstances deteriorate due to delayed ettringite formation (DEF). The phenomenon leads to an internal expansion and, gradually, to cracks. In combination with moisture and/or frost erosion the deterioration may increase after a few years and become so severe that the sleepers have to be replaced by new ones [1].

The ettringite is normally formed under the hardening process of concrete and is transformed to monosulphate with less water content after a few hours. However, if heat curing (steam-curing) and/or cement with high sulphate contents are used, ettringite may form several years after the hardening causing delayed expansion and cracking. The phenomenon has, time after another, occurred in railway sleepers in many countries [2]. However, recommendations regarding maximum cement content and curing temperature is easily forgotten in the strive to increasing production efficiency.

Corrosion of duo-block sleepers, i.e. twin concrete pads with embedded rail support and fastening connected with a metal tie bar is also of importance.

Subsoil / Load Distribution Layer

Even if the subsoil condition is the most critical input related to track behaviour, description is still difficult. The behaviour is influenced by

- Ballast Quality
- Size Distribution of Ballast Stones
- Pollution of Ballast and Load Distribution Layer
- Drainage Condition
- Load Capability of Subsoil

Georadar analyses can give indications of the subsoil quality.

The degradation model shows maintenance necessities due to different wear or damage phenomena. Considering maintenance necessities for the asset (track) as a whole, lead to an adjusted track strategy. At a certain point of time maintaining a single component leads to higher costs than renewing the total system. The economical effect of exchanging rails only when the sleepers show a remaining service life of some years can be calculated.

Switches and crossings

For switches and crossings all mechanisms described above are valid. Additionally, the following aspects have to be considered.

Frog

Appearing cracks at the frog surface and wear of the front end of the frog. The process depends on

- Traffic volume
- Type of component (material, fixed or diamond)

Frog deterioration can be fought by surface welding.

Tongue Rail

The deterioration process is similar to general rail wear. However, tongue rails face higher wear due to reduced support. Moreover, the profile is reduced as well.

Wear is triggered by

- Traffic volume
- Number of diverging trains
- Rigidity of bogies
- Rail Steel Grade

Check rails face minor wear as long as the geometry of the turnout is properly maintained.

Drivers and Control Devices

Switches and Crossings (S&C) are important parts of the rail system with high maintenance costs. One main contribution to the high costs is repair and exchange of S&C that does not function due to degradation of drivers and control devises, see [3] and [4].

9.3 Associated performance and limit states

Studying track behaviour is based on time rows of recording car data and status date (traffic volume, type and age of track and turnouts and maintenance actions executed). Thus accidental load cases are not taken into account. However, overloading of railway vehicles to a certain amount is within the operational load collectives.

The loading of most of the main lines is increasing with 0.5 to 2 per cent annual resulting in a traffic increase within the service life of track of 15 per cent to 80 per cent. These figures are based on constant axle loads, i.e. the increase is caused by a higher train frequency. However, as track behaviour is analysed using measuring data in time rows, the effects of the load increase are covered.

The resistance of ballast is decreasing over time, unless new ballast is added by implementing ballast cleaning. Wooden sleepers start to rot and thus deliver reduced loading capabilities. Rail pads also deteriorate. Rails show very little change of the behaviour. However, in case of poor water drainage, the whole system deteriorates quickly, as it does if the sub-layers cannot bear the loads.

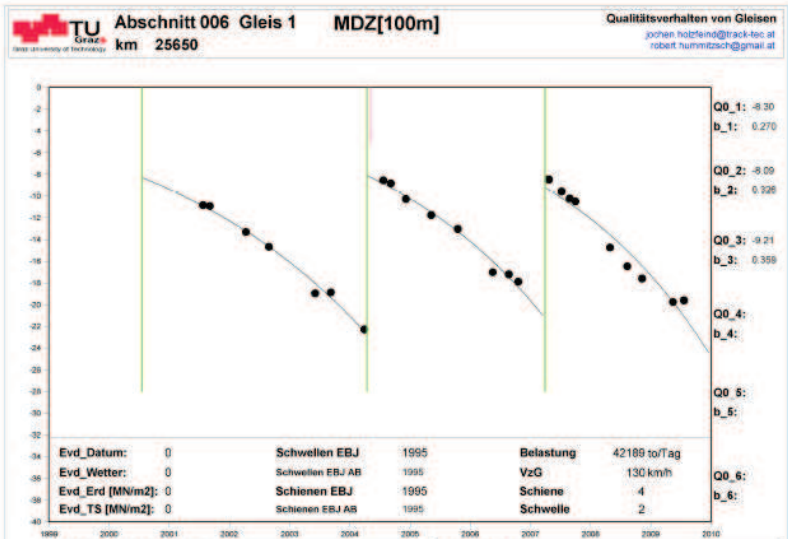

Figure 27: Track quality behaviour over time including maintenance actions

(TUGraz)

Furthermore deterioration of the components interacts. If rail pads are worn out and not changed in time concrete sleepers face irreversible damage within short time.

Track geometrical limit states

EN 13848-5 standard [5] contains limit values (limit value ranges) for AL, IL, IAL size limit categories. Besides railway administrations have also constructional, maintenance and closing (terminating the operation) limits.

Limit states for the main track components

Rail

- Wear limit values
- RCF limit values (This is not examined by all railway administrations)
- Ultrasonic faults intervention arrangements
- Limit values for neutral temperature zone. There is not a well suited technology like for the inner rail faults or rail wear. Only manual measuring instrument is available with a lot of uncertainty, and it is impossible to survey the whole network.
- There are different rules for the allowed deviations of the actual temperatures from the neutral temperature zone at different railway companies. The neutral temperature zone also changes by countries orientated to the climate of the country.

Fasteners

For tightening there are momentums, for other fastener states there are no general limit values available.

Sleepers

There are no general limit values available.

Ballast

Often the contamination is a limit value given in % of polluted ballast. However, the value is seldom measured.

Turnouts

As turnouts are safety relevant, internal regulations for the geometry and limit values for the components are in use.

As the limit states are defined by the speed of the trains and the axle load or metre load poor quality levels can be accepted by reducing train speed and/or axle or metre load. In case of track and turnouts the reduction of train speed is commonly used (permanent slow orders), especially as they do not imply relevant costs for the infrastructure owner. However, speed restrictions lead to additional operations costs for the railway operating companies and thus – from a system point of view – should be avoided in any case in main lines.

9.3.1 Durability

Durability is a major question for ballast. Even maintaining the alignment shows an impact on ballast wear. Thus service life can be limited by a certain number of tamping actions necessary due to the traffic passed.

Standard rails 60E1 have no limitation in straight sections as longs as preventive head check grinding is executed. Service life of light rails (up to profiles of 54 kg per metre) is limited to 300 – 450 mio. gross tons passed. These values depend on the axle load distribution, the temperature ranges, and strongly to the track failures. In sharp curves the outer rail faces strong side wear and the inner rail suffers corrugation. Both effects can be reduced by using higher steel grades as offered by head hardened rails and by lubrication.

Assuming sufficient water draining and an elastic ballast bed wooden sleepers can reach some 30 years and more of service life, concrete sleepers even up to 50 years. High horizontal forces in curves can limit these service lives due to problems with fasteners and rail support.

9.3.2 Serviceability

If rails are worn out it is technically possible and economically advisable to change them. If ballast is polluted and/or worn out it might be acceptable to clean it, if the other components of the track are still showing high residual service lives. If this is not the case or sleepers are worn out the entire track structure should be replaced in order to provide a high initial quality and thus long service life. Residual service lives of components can be used when reusing these components on sidings or in branch lines.

9.4 References

[1] Thun, Håkan, Utsi, Sofia and Elfgren, Lennart (2008): Load carrying capacity of cracked concrete railway sleepers. *Structural Concrete*, Journal of fib, Vol. 9, No 3, pp 153-161. ISSN 1464-4177.

[2] Scrivener K. and Skalny J. P. (2005): Conclusions of the International RILEM TC 186-ISA Workshop on Internal Sulphate Attack and Delayed Ettringite Formation. 4-6 September 2002, Villars, Switzerland. Rilem, *Materials and Structures*, Vol. 38, No. 280, pp. 659–663.

[3] Nissen, Arne (2009): Development of Life Cycle Cost Model and Analyses for Railway Switches and Crossings. Ph D Thesis, Luleå University of Technology, Luleå 2009, 132 pp, ISBN 978-91-7439-026-1.

[4] Ekberg, Anders and Paulsson, Björn, Editors (2010): Innotrack – Innovative Track Systems – Concluding Technical Report. Paris: UIC, 288 pp. ISBN 978-2-7461-1850-8.

[5] EN 13848-5 Track geometry quality evaluation, 2008+A1 2010

Furthermore, important references are listed below:

CR INS TSI C(2011) 2741 final 26.04.2011.

EN 15302, (2010), Method for calculation of equivalent conicity

UIC leaflets:

519 Method for calculation of equivalent conicity 2004.12.01

712 Catalogue of Rail defects 2002.

715-2 Recommendations for management of rails 2003. August

716 Maximum permissible wear profiles for switches 2004. May

720 Laying and maintenance of CWR track 2005. March

721 Recommendations for the use of hard quality rails 2005. March

725 Treatment of rail defects 2007. June

Innotrack documents:

D4.1.2 Rail degradation algorithms

D4.1.6 Detailed analysis of rail infrastructure problems

D6.4.1 LCC and RAMS calculation

TecRec Rail re-profiling 2011. March

UIC (2010), Monitoring track condition to improve asset management, ISBN 978-2-7461-1871-3

UIC Maintenance of switches and crossings

A guideline to best practice of squat treatment (Zili Li) 2011. October

Veit, Peter: Life Cycle Costing in Practice in Europe. - in: Guidelines to Best Practices for Heavy Haul Railway Operations. (2009), p. 2-22 - 2-34

Veit, Peter: Instandhaltung und Anlagenmanagement. - in: Handbuch Eisenbahninfrastruktur. (2006), p. 873 - 928

10. Other structures

10.1 Asset types

There are a number of other civil engineering structures that exist across European railway networks but do not fall within the main categories covered in Sections 6 to 9 of this report. These miscellaneous asset types are listed below:

- Retaining Walls – this includes masonry or concrete gravity walls, gabions, dry stone walls, cantilever walls including sheet piles, Catch walls and fences and facing walls. Network Rail also class reinforced soil slopes at an angle greater than 70 degrees as retaining walls, however they will not be considered in this section.

- Coastal and River Defences –these may be concrete, blockwork, brickwork or masonry gravity walls with exposed faces fronting on to the sea or a river.

- Drainage - this section will only discuss culverts. Earthwork drainage and track drainage are covered in sections 6 and 9 of this report respectively.

10.2 Damage and deterioration mechanisms

Whilst these asset types cannot themselves be defined as earthworks, bridges, tunnels or track, their deterioration mechanisms are often closely linked to one or more of these structures. For example, factors affecting retaining wall deterioration will be closely linked to earthwork deterioration, and a culvert may deteriorate in much the same way as a bridge structure. The failure of one of these asset types may also be closely linked to the failure of a major asset, for example a retaining wall failure may cause a catastrophic earthwork failure. The following sections will outline the deterioration mechanisms that are specific to each of these asset types and highlight any similarities and links with the main asset categories discussed in previous chapters.

10.2.1 Retaining Walls

The performance of a retaining wall is governed by the interaction between the ground and the structure itself. Therefore deterioration may arise as result of changes in the ground conditions as well as deterioration of the structure itself.

- *Differential settlement* – as discussed in earthworks section
- *Soil parameters* – As outlined in section 6 on earthwork structures, soil behaviour can be affected by a number of factors which change with time and environmental conditions. These can cause deterioration to a retaining wall by altering the loading from the retained earth.
- *Groundwater* – pore water pressure can build up behind the retaining wall leading to overturning
- *Vegetation* – The growth of vegetation in and around a retaining wall can cause significant damage. The growth of bushes and saplings near the wall can cause root damage and can effect the water content of the adjacent soils and causing geotechnical instability

- *Material and structural degradation* – Depending on the construction material, the retaining wall structure itself may deteriorate via concrete chemical attack, steel corrosion, fatigue deterioration etc. Refer to Section 7 on bridge structures for more information on these deterioration mechanisms.

10.2.2 Coastal and River Defences

The performance of a coastal or river defence may be affected by a variety of circumstances related to loading, environment and material durability. The deterioration mechanisms can be broadly categorized as follows:

- *Exceedence of original design loads* - many of the coastal and river defences found on the railway network in the UK are more than 100 years old and were not designed to resist the loading and vibration from modern traffic running above the retained earth. Many bridge abutments form coastal defences and are particularly susceptible to deterioration as a result of increased traffic loading (see chapter 7 on bridges). As traffic loading continues to increase in frequency and intensity the rate of deterioration of these structures may increase. Overloading can also occur if heavy materials or plant is placed on the retained surface.
- *Accidental Impact*– accidental impact from a boat or ship can cause serious damage to a waterfront wall, usually in the form of loosening of the individual components of the wall and cracking. Impact from flood debris may also cause significant damage to a river defence.
- *Man-made changes to the wall and environment* – excavation at the toe of a waterfront wall (as a result of over-dredging or adjacent underwater blasting), adjacent construction work causing damage to wall anchoring, increased loading, removal of backfill or impaired drainage
- *Mining subsidence* – As discussed in Section 6, subsidence can occur as a result of previous mining work below the structure. In the case of waterfront walls this can cause cracking and tilting of the wall.
- *Wave and current scour* – this can cause erosion of the ground level in front of the wall and is a common cause of deterioration. The predicted increase in flooding may cause unprecedented scour deterioration to old river defences
- *Rising water levels* – global sea levels are predicted to rise and increased rainfall could see an increase in river levels. Increased water levels will increase the static pressure acting on a water front defence and could cause future damage or failures. Raised sea levels could also increase the height of waves reaching the wall, worsening the effects of wave erosion
- *Vegetation* – vegetation can affect waterfront walls in the same way as retaining wall (see section 10.2.1 above). In particular, vegetation damage can leave a coastal or river defence vulnerable to water penetration and frost action
- *Structural deterioration* – the general deterioration of the structure can take a variety forms, depending on the construction material. These are detailed in the discussion on bridges in chapter 7 of this report, but most commonly might include deterioration of brick mortar, deterioration of concrete, loss of stone, and cracking.
- *Geotechnical failure* – As outlined in section 6 on earthwork structures, soil behaviour can be affected by a number of factors which change with time and environmental conditions. These can cause deterioration to a waterfront defence by altering the loading from the retained earth.

10.2.3 Culverts

Examples of culvert failure mechanisms are outlined below. These failures may begin as serviceability issues, affecting the drainage efficiency but may result in catastrophic failure affecting adjacent infrastructure.

- *Blockage* – culverts may become blocked by vegetation or silt. This is particularly problematic where the flow is insufficient to keep the drain clear. This will result in impaired drainage performance which can in turn lead to subsequent earthwork failure or track flooding. The consequences of such blockage are particularly severe if they coincide with a period of persistent and/or heavy rain.
- *Water-borne debris* – this can divert water flows, affecting drainage performance and can generate scour effects on as the culvert structure and adjacent structures such as bridge foundations.
- *Erosion of surrounding surfaces* – Rainwater and ground water may cause the materials supporting a drainage structure to be eroded away. This can result in impaired performance, instability or deformation of the drainage structure.
- *Structural and material deterioration mechanisms* – Culverts are analogous to bridges in terms of their construction and structural behaviour and may be affected by the deterioration mechanisms described in chapter 7.

10.3 Associated performance and limit states

Information relating to the performance and condition of these asset types should be gathered periodically in order to ascertain maintenance requirements. This information can be assessed against performance criteria or limit states assigned by the asset owner. The specific performance requirements for the asset types covered in this section are discussed below.

Retaining walls and coastal/river defences

Retaining walls and coastal and river defences are both types of retaining structures. They therefore have largely the same performance requirements. Ultimate limit states for retaining structures include instability and failure by rupture. These can be caused by slip failure, bearing capacity failures and translational or rotational ground movement. The British Standard for Retaining Structures [4] outlines the typical ultimate limit states for retaining walls, as shown in Figures 27, 28 and 29 below.

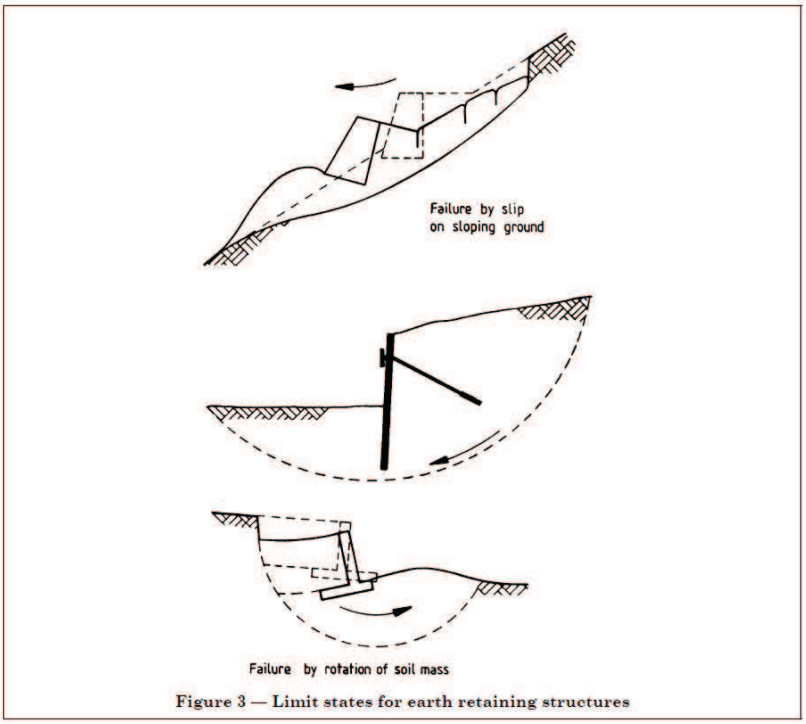

Figure 28: Retaining wall limit state: extract 1 from BS 8002:1994 [4]

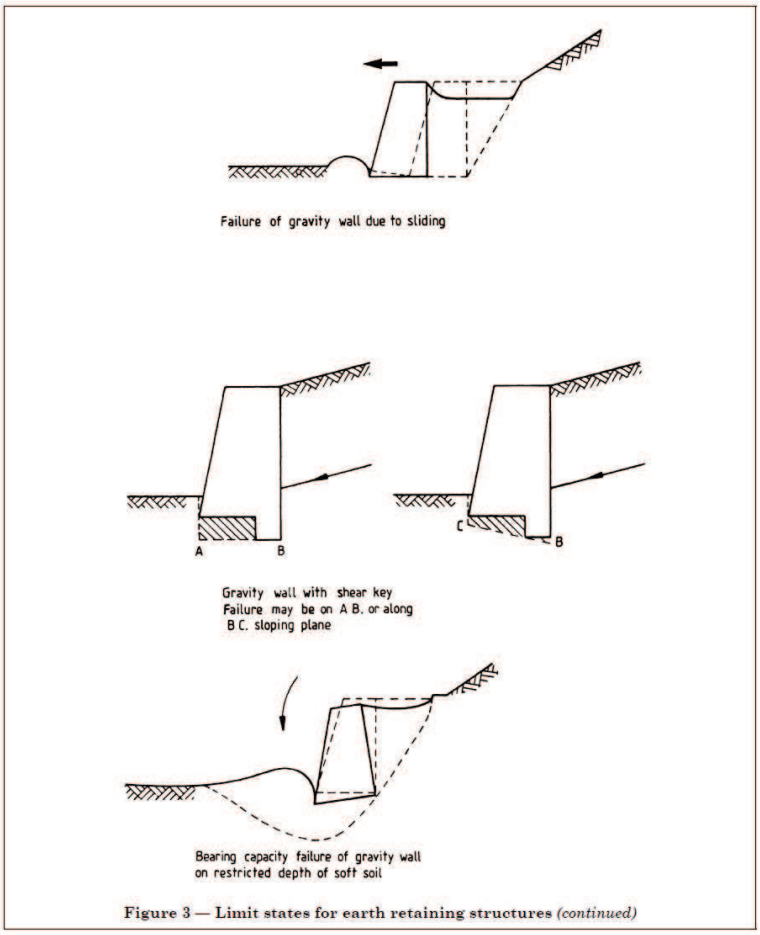

Figure 29: Retaining wall limit state: extract 2 from BS 8002:1994 [4]

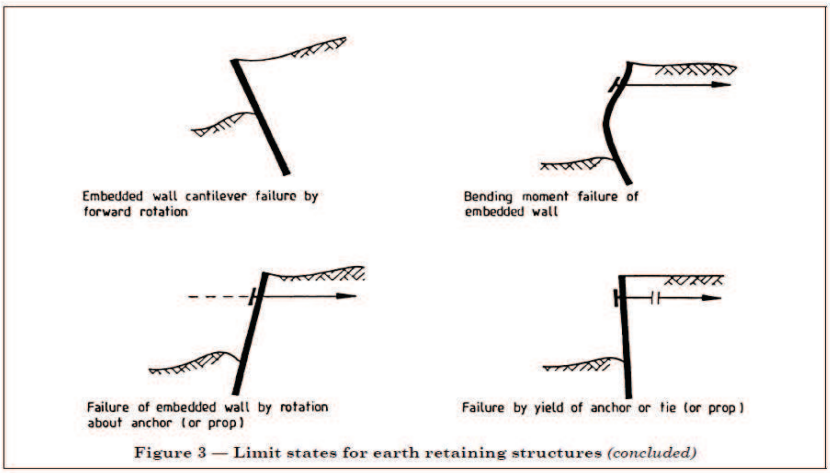

Figure 30: Retaining wall limit state: extract 3 from BS 8002:1994 [4]

For most earth retaining structures however, the serviceability limit state of displacement is the governing factor for assessing performance rather than the ultimate limit state of overall stability. This is because the deformation of an earth retaining structure causes soil deformations which are large in comparison with the normally acceptable strains in service, and hence has a direct effect upon the loading on the structure [4].

In the case of coastal and river defences, a key function of the structure is to prevent flooding of the railway infrastructure by tidal or non tidal waters. For this reason these structures should also be assessed for their ability to resist extreme loading caused by flooding storms or high tides, as well as normal loading conditions. If a structure is deemed to be susceptible to damage or reduced capacity as an event of such extreme loading it may be necessary to prepare a flood warning plan [5] and/or undertake maintenance or renewal work.

Culverts

The CIRIA guide C689 [2] identifies the following performance requirements for infrastructure culverts:

- *Structural capacity* - The form of structural assessment will depend on several criteria including the culvert span, depth, form of construction and degree of live loading. For more details on the associated structural performance and limit states refer to section 7 on bridges.
- *Hydraulic performance* – Particular to culverts is the requirement for a hydrological assessment. This is used to estimate the flow conditions and to understand sediment and debris loads that may impact upon hydraulic performance. Assessment of hydraulic performance may need to consider a range of flow conditions, particularly low-flows as these are important for understanding ecological impacts.
- *Legislative requirements*, for example environmental performance, riparian duties or health and safety legislation. These will vary from country to country, but may be a significant factor in determining when maintenance or remedial work is required.

Over time the performance requirements for a culvert may change as a result of changes in loading, (both hydraulic and structural), changed legislative requirements or changes to the wider environment such as nearby developments or changes to the groundwater regime.

10.4 Summary

The structures described in this section are closely linked to the other asset types previously discussed. Culverts are essentially bridge structures, and therefore exhibit largely the same deterioration mechanisms. Retaining walls and coastal/river defences, when constructed in concrete or masonry, can also be compared to bridge abutments.

The key differences in the deterioration of these 'other' structures arise from the different environmental conditions that they are subjected to. In the case of culverts and waterfront defences the presence of flowing water is a major cause of deterioration. For this reason these structures may be particularly sensitive to the predicted rising sea levels and increased flooding caused by climate change. A further key cause of deterioration applicable to all three of the 'other' structures is the interface with soil, which can trigger deterioration through chemical attack. Changes in soil parameters, in particular pore water pressure can also cause instability and failure.

These structures often form an integral part of a larger structure, for example a culvert running through an embankment, or a coastal defence that serves as a bridge abutment. For this reason the deterioration of one of these structures may have both first order effects which affect the serviceability and capacity of the structure itself, and second order effects which affect an adjacent structure. The second order effects may be disproportionately large, for example a culvert serviceability issue caused by silt blockage could trigger a major failure of an embankment due to excessive groundwater.

10.5 References

[1] R.N.Bray and P.F.B Tatham, 1992 - CIRIA B 13 Old Waterfront Walls

[2] M Balkham et al, 2010 - CIRIA C689 Culvert Design and Operation Guide

[3] Network Rail, 2010 - NR-L3-CIV-005 Railway Drainage Systems

[4] British Standards Institute, 2001 - ENV 1997 Part 1 Geotechnical design 1995 BS8002 Retaining Walls

[5] Network Rail, 2004 – NR-SP-CIV-089 Management of Existing Coastal, Esturine and River Defences

11. Conclusion

This deliverable specifies relevant degradation and performance states for selected assets to be given special focus in the MAINLINE project and Work Package 2 in particular which aims at the enhancement of degradation and performance models to provide a step change in safe, cost effective management and/or life extension of these assets.

Special focus areas are addressed in the Description of Work (DoW) and benchmarked by response of infrastructure managers to a questionnaire that has been devised specifically for the purposes of this work package.

The following asset types are identified as focus areas:

- Cuttings,
- Metallic bridges,
- Tunnels with concrete and masonry linings,
- Plain line (total track superstructure) and switches and crossings,
- Retaining walls.

For the above mentioned assets there is a high probability for knowledge increase within a 3 year period and useful validation data is available.

For all asset types within the scope and for the above mentioned in particular degradation and associated performance characteristics and limit states are addressed. Furthermore, their temporal and spatial characteristics as well as sensitivity to load evolution and effects of climate change are addressed.

12. APPENDIX A – Response to questionnaire

See separate pdf-file (to be printed in A3-format).

Appendix A - Response to questionnaire

A.1 Earthworks

A.2 Bridges

A.3 Tunnels

A.4 Track

A.5 Other structures

Appendix A.1 - Earthworks

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	EARTHWORKS (where possible please give indication of the number/quantity of assets in each category)		
	Sub-grade (Natural Ground)	**Cuttings**	**Embankments**
Q1. Describe experienced degradation mechanism(s)	track level defects due to sub soil deformation	land slides and slopes into track	land slides, track level defects
Q2. What is the primary agressor for this degradation?	bad sub soil (soft layers: peat, clay etc.)	insufficient slope stability, insufficient drainage, slope at rock sides	insufficient capability, insufficient drainage
Q3. How is this degradation monitored or inspected?	track recording device		
Q4. How fast does an imminent failure develop over time?	due to sub soil and/or increasement of load and speed	depending on individual cases	depending on individual cases
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)	see Ril 820.2001, if sub soil soil improvemnet necessary due to geotechnical survey and assessment	track level see Ril 820.2001	depending on individual cases
Q6. Are interventions related to a condition or a safety assessment?	see Q5	due to condition assessment	due to condition assessment
Q7. Rate degradation in terms of costs (Operations & Maintenance and Renewal) (1-10, 1 being the most costly)	maintenance : frequent tamping activities, renewal/strenghtening: PSS _Einbau (ca. 500€/m track), retrofitting with piles and colums (>> 1000 €/m)	maintenance: clearence of side path, jetting of drainage, renewal drainage (Abfanggraben),slope reanchorage and eatrth concrete etc., cost due to source and mechansim	maintenance: tamping of track, jetting of drainage, renewal drainage (Abfanggraben),slope reanchorage and earth concrete, retrofitting (piles and columns) etc., cost due to source and mechansim
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	none	none	none
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	Ril 853.8001 - safety risks and defect category 1-4 and assessment categories 1-4	Ril 853.8001 - safety risks and defect category 1-4 and assessment categories 1-4	Ril 853.8001 - safety risks and defect category 1-4 and assessment categories 1-4
Q10. Do you have accessible inspection or monitoring data for this mechanism?	no	no, but in same cases (setting pins, inclinometer, extensometer)	no, but in same cases (setting pins, inclinometer, extensometer)
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	see Ril 820.2001 (longitudanal level, altitude, twist, rise, gauge track)	condition assessment: change od slope, indication of slopes: cracks and splittings, influx, vegetation, side path cracks, indication of ballast flow, slant of overhead line mast etc.	condition assessment: change od slope, indication of slopes: cracks and splittings, influx, vegetation, side path cracks, indication of ballast flow, slant of overhead line mast etc.
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	see Ril 820.2001, periodic depending on line category	see Ril 836.8001: inspection in 1 or 3 years steps, inspection of construction type category 3 every 6 years	see Ril 836.8001: inspection in 1 or 3 years steps, inspection of construction type category 3 every 6 years

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	EARTHWORKS		
	Sub-grade (Natural Ground)	**Cuttings**	**Embankments**
Q1. Describe experienced degradation mechanism(s)	Detereioration is exponential Types of deterioration süllyedés soil failure moistering	Detereioration is exponential slope scaling slope sliding slope landslide wash out	Detereioration is exponential slope scaling slope sliding slope landslide wash out loss of stability
Q2. What is the primary agressor for this degradation?	water faulty planning	water faulty planning	water faulty planning
Q3. How is this degradation monitored or inspected?	sinking with measuring tube geodesy visual inspection evaluation of measuring graphs of superstructure	slope dtereiorations visually inclinometer displacement meters geodesy	slope dtereiorations visually inclinometer displacement meters geodesy
Q4. How fast does an imminent failure develop over time?	exponential	exponential	exponential
Q5. Rate degradation in terms of costs (O&M and renewal) (1-10, 1 being the most costly)	sinking 3 soil failure 1 moistering 5	slope scaling 4 slope sliding 3 slope landslide 4 wash out 6	slope scaling 4 slope sliding 3 slope landslide 4 wash out 6 loss of stability 1
Q6. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	4	4	4
Q7. What documents are used to assess this degradation mechanism? (guidelines, codes, internal documents)	D 11 sub-structure instructuion book evaluation of measuring graphs of superstructure	D 11 sub-structure instructuion book	D 11 sub-structure instructuion book
Q8. Do you have accessible inspection or monitoring data for this mechanism?	Yes	no	yes
Q9. What key paramter(s) is/are recorded through monitoring or inspection?	sinking changing in ground water level displacements continuity of surfaces geometry of surfaces	sinking changing in ground water level displacements continuity of surfaces geometry of surfaces	sinking changing in ground water level displacements continuity of surfaces geometry of surfaces
Q10. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	periodic	periodic	periodic

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	EARTHWORKS (where possible please give indication of the number/quantity of assets in each category)		
	Sub-grade (Natural Ground)	**Cuttings**	**Embankments**
	Bearing capacity failure under track/ a requirement for excessive track maintenace.		
Q1. Describe experienced degradation mechanism(s)	Boulder fall from Glacial Till onto track.	Physical weathering (freeze thaw and effects of scour and flooding)	Physical weathering, animal burrowing, flood and scour.
Q2. What is the primary agressor for this degradation?	Water leading to softening and reduction of shear strength of cohesive soils. Physical weathering.	Rainfall and ground-water (and sometimes river flooding). Freezing winter temperatures cause rock-fall.	Rainfall, flood and scour. Also train traffic and track maintenance.
Q3. How is this degradation monitored or inspected?	Track inspection. Problem sites are monitored.	Examination by walk-over inspection using Soil Slope Hazard Index. This is a repeatable and reproducible examination system. Output is SSHI score and condition classification. Repeat examinations allows degradation rates to be determined.	Examination by walk-over inspection using Soil Slope Hazard Index. This is a repeatable and reproducible examination system. Output is SSHI score and condition classification. Repeat examinations allows degradation rates to be determined.
Q4. How fast does an imminent failure develop over time?	Rapidly.	Water related failures are very rapid. Rock fall is rapid Rotational failures can be slow - monitoring often used to mitigate risk.	Slowly develop over time - monitoring often used to mitigate risk.
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)	Reactive to excessive track maintenance.	Soil cuttings - Soil Slope Hazard Index > 10 leads to an Evaluation to determine Ground Investigation and monitoring requirements followed by repair where necessary. Rock cuttings - Rock Slope Hazard Index > 100 leads to an Evaluation followed by repair where necessary.	Either ultimate failure (slip) requires reactive repair. Serviceability failure causing track settlement is also a driver for repair.
Q6. Are interventions related to a condition or a safety assessment?	Intervention is done in relation to slope failiure.	Soil Slope Hazard Index > 10 leads to an Evaluation to determine Ground Investigation and monitoring requirements followed by repair where necessary. Rock cuttings - Rock Slope Hazard Index > 100 leads to an Evaluation followed by repair where necessary.	Interventions are usually due to track settlement or due to monitoring results
Q7. Rate degradation in terms of costs (Operations & Maintenance and Renewal) (1-10, 1 being the most costly)	1. Increased track maintenance costs	1.Flood and scour related cutting failure repairs 2. Rock-fall (scaling and netting) 3. Shallow failure of slopes due to weathering of soil cuttings	1. Flood and scour damage repairs 2. Shallow failure of embankment side slopes repairs 3. Animal burrowing repair
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	Limited knowledge.	Examinations using Soil Slope Hazard Index (SSHI) have been carried out since 2005 and produces a numeric score. Cutting condition is classified into poor, marginal or serviceable condition based on SSHI numeric score. Repeat examination information is available for some cuttings and embankments. Degradation rates are available based on this limited repeat examination data.	Examinations using Soil Slope Hazard Index (SSHI) have been carried out since 2005 and produce a numeric score. Embankments are classified into poor, marginal or serviceable condition based on SSHI score. Repeat examination information is available for some cuttings and embankments. Degradation rates are available based on this limited repeat examination data.
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	NR/SP/TRK/9030 Formation Treatments	Network Rail specificiation NR/L3/CIV/065 Examination of Earthworks	Network Rail specificiation NR/L3/CIV/065 Examination of Earthworks
Q10. Do you have accessible inspection or monitoring data for this mechanism?	No.	Yes - examination data is available through a web-based database.	Yes - examination data is available through a web-based database.
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	Ground Penetrating Radar are available for some sites. Track geometry recorded from Track Recording Vehicle.	Rate and depth of slope movement are monitored. Examination reports give data on tension cracks.	Rate and depth of slope movement are monitored.
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	Track geometry is recorded periodically.	Usually monitoring is periodic (relevant time interval depends on rate of movement of slip) Between 5 and 10 cutting sites have remote monitoring and these are continously monitored.	Usually monitoring is periodic (relevant time interval depends on rate of movement of slip)

Appendix A.2 - Bridges

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	BRIDGES (where possible please give indication of the number/quantity of assets in each category)		
	Masonry	**Metallic**	**Concrete (reinforced, perstressed and post-tensioned)**
Q1. Describe experienced degradation mechanism(s)			
Q2. What is the primary agressor for this degradation?	incoming water		
Q3. How is this degradation monitored or inspected?	regular inspection		
Q4. How fast does an imminent failure develop over time?	depending of indicidual characteristics		
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)			
Q6. Are interventions related to a condition or a safety assessment?	related to condition assessment	related to condition assessment	related to condition assessment
Q7. Rate degradation in terms of costs(Operations & Maintenance and Renewal) (1-10, 1 being the most costly)			
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)			
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	Ril 804.8001, bridges will be out in categories of condition	Ril 804.8001, bridges will be out in categories of condition	Ril 804.8001, bridges will be out in categories of condition
Q10. Do you have accessible inspection or monitoring data for this mechanism?	no, but in some cases information might be available	no, but in some cases information might be available	no, but in some cases information might be available
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	crack development	crack development, surface condition, lack of painting	surface condition, reinforcement covergae, crack development
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	mainly periodic, inspection routines every 6 year; only in cases of doubt the interval is once a year or even shorter	mainly periodic, inspection routines every 6 year; only in cases of doubt the interval is once a year or even shorter	mainly periodic, inspection routines every 6 year; only in cases of doubt the interval is once a year or even shorter

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	BRIDGES		
	Masonry	**Metallic**	**Concrete** **(reinforced, perstressed and post-tensioned)**
Q1. Describe experienced degradation mechanism(s)	1. weathering of stones 2. freeze-thaw damage of units 3. Mortar loss 4. Crack under tracks 5. Cracks under spandrels 6. Loss of bricks, stones 7. Bulging of spandrels	1. Defects of welds (e.g. cracks) 2. Corrosion and reduction in cross section 3. Cracks around rivet holes 4. Cracks in steel plates 5. Deformation of bearings 6. Deformation of steel elements 7. Loss of surface coating 8. Slackening of connections (bolted, riveted)	1. Loss of concrete cover 2. Corrosion of reinforcement 3. Cracking of concrete 4. Water penetration 5. Freeze-thaw damage, loss of strength 6. Carbonation of concrete
Q2. What is the primary agressor for this degradation?	1. Air pollution, humidity 2. Temperature, wettness 3. Wind, wettness 4. Traffic loading 5. Traffic loading, difference in stiffness 6. Freeze-thaw cycles, traffic loading 7. Increased lateral earth pressure	1. Traffic loading (large number of repeat) 2. Environmental effects, insufficient cover 3. Traffic loading (large number of repeat) 4. Traffic loading (large number of repeat, overloading) 5. Traffic loading (large number of repeat, overloading) 6. Overloading 7. Environmental effects 8. Traffic loading (large number of repeat)	1. Corrosion of steel, wetness 2. Wetness, poor quality of concrete, insufficient cover, chloride 3. Overloading, movements 4. Damaged insulation 5. Temperature, wettness 6. Air pollution
Q3. How is this degradation monitored or inspected?	1. Usually visually, in some cases samples are taken and investigated in laboratory 2. Usually visually, in some cases by videoendoscopy 3. Visually 4. Visually 5. Visually 6. Hammer tapping 7. Visually	1. Visually, dye penetration test, rarely ultrasonic test 2. Visually, depth gauges 3. Visually, dye penetration test, rarely ultrasonic test 4. Visually, dye penetration test, rarely ultrasonic test 5. Visually, deformation measurements by extensometers 6. Visually, deformation measurements by extensometers, measurements by laser scanning 7. Visually, cover depth measurement, bond test 8. Hammer tapping, torque spanner	1. Usually visually, hammer tapping, covermeter 2. Usually visually, rarely by chemical or NDT tests 3. Visually or by glass plates 4. Visually 5. Visually 6. Chemical test in situ or in lab
Q4. How fast does an imminent failure develop over time?	1. Development requires decades depending on stone quality 2. Development requires years or decades depending on parameters such as stone quality, presence of wettness, temperature 3. Development requires years or decades depending on parameters such as mortar quality, presence of wettness, temperature, wind intensity 4. Development requires years or decades depending on parameters such as height and quality of fill, dimensions of the arch, dynamic effects 5. No data 6. Development requires years or decades depending on parameters such as height and quality of fill, dimensions of the arch, dynamic effects 7. Development requires years or decades depending on parameters such as height of spandrel and fill, lateral earth pressure	1. Development requires months or years 2. Development requires years or decades depending on explosure conditions 3. Development requires months or years 4. Development requires months or years 5. Development requires years or decades 6. Development requires years or decades 7. Development requires years or decades 8. Development requires years or decades	1. Development requires years or decades depending on concrete quality and wetness 2. Development requires years or decades depending on parameters such as concrete quality, wetness, concrete cover 3. Development requires years or decades depending on structural dimensions, loading, condition of foundations 4. Development requires years or decades depending on quality and type of insulation, quality of concrete 5.Development requires years or decades depending on quality and type of insulation, quality of concrete 6. Development requires decades
Q5. Rate degradation in terms of costs (O&M and renewal) (1-10, 1 being the most costly)	1. 6 2. 4 3. 6 4. 5 5. 6 6. 5 7. 3	1. 7 2. 4 3. 4 4. 3 5. 4 6. 3 7. 4 8. 8	1. 6 2. 4 3. 5 4. 3 5. 4 6. 6
Q6. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	1. 2 2. 4 3. 3 4. 7 5. 6 6. 8 7. 6	1. 7 2. 7 3. 3 4. 4 5. 6 6. 7 7. 9 8. 8	1. 8 2. 8 3. 7 4. 7 5. 6 6. 8
Q7. What documents are used to assess this degradation mechanism? (guidelines, codes, internal documents)	Bridge inspectors guide (under preparation)	Bridge inspectors guide (under preparation)	Bridge inspectors guide (under preparation)
Q8. Do you have accessible inspection or monitoring data for this mechanism?	Only inspection files at individual bridges Monitoring was not carried out	There are mainly individual measurements. Monitoring data are available only in unique cases.	Only inspection files at individual bridges Monitoring was only rarely carried out
Q9. What key paramter(s) is/are recorded through monitoring or inspection?	1. depth of degradation 2. visual view 3. depth of loss 4. crack length, width, location, orientation 5. crack length, width, location, orientation 6. depth and area of loss 7. angle of bulging	1. length and depth of crack 2. visual view, corrosion depth 3. length, orientation and (depth of crack) 4. length, orientation and (depth of crack) 5. movements, incination 6. rate of deformation, shape 7. depth of coating 8. qualitatively by hammer, torque moment	1. Area with lost cover, visible signs of the initiation of loss of cover 2. Loss of effective cross-section area of reinforcement, quality of ribs on reinforcement 3. Crack width, crack opening under traffic loading, temperature 4. Area of wetness, clearness of drainage 5. Concrete strength 6. Depth of carbonation
Q10. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	Only periodic. Experts's inspection is carried out in every 5 years. Artisan's inspection is carried outevery year.	Only periodic. Experts's inspection is carried out in every 5 years. Artisan's inspection is carried out every year. Deflection monitoring is rarely applied for a limited time period.	Usually periodic. Experts's inspection is carried out in every 5 years. Artisan's inspection is carried outevery year. Crack monitoring is rarely applied for a limited time period.

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	BRIDGES (where possible please give indication of the number/quantity of assets in each category)		
	Masonry	**Metallic**	**Concrete**
	Approx 20,000 bridges	Approx 16,000 bridges	**(reinforced, perstressed and post-tensioned)** Approx 4,000 bridges
Q1. Describe experienced degradation mechanism(s)	a) Mortar degradation	a) Corrosion	a) Degradation of concrete
	b) Spalling	b) Fatigue cracking	b) Rebar corrosion
	c) Barrel cracking		c) Corrosion of post tensioning tendons/bars
	d) Spandrel separation		d) Pre stressed/post tensioned anchorage slippage
Q2. What is the primary agressor for this degradation?	a) Water ingress	a) Weather	a) ASR/Sulfate attack/thaumasite
	b) Freeze/thaw and/or overloading	b) Poor design detailing, loading	b) Loss of passivation due to carbonation Chloride ingress
	c) Overloading, settlement		c) Poor grouting of ducts
	d) Surcharging		d) Poor design or construction
Q3. How is this degradation monitored or inspected?	All cases - visual inspection	a) Visual inspection supported occasionally by non destructive or partially destructive testing	a) Visual inpsection supported by petrographic analysis
		b) Visual inspection (not satisfactory). Dye penetrant (probably only suitable for welded structures) Electro magnetic techniques (still largely experimental) Acoustic emission monitoring	b) Half cell potential etc to give indication of likely areas of corrosion. Scisite electro magnetic probe to detect corrosion Prototype fibre optic corrosion detector Fibre optic PH and chloride sensors
			c) Radar/sonic tomography (not fully successful) (See BA86) Intrusive visual inspection using fibre optics
			d) Not known
Q4. How fast does an imminent failure develop over time?	All cases - Decades	a) Decades	All cases - Decades
		b) Decades to initially develop but progress to failure then relatively quick	
Q5. What are the current trigger points (thresholds) for intervention?	a) Joints wasted by 12mm	a) Loss of structural capability	Cracks visually unacceptable
(e.g. visual condition worse than X, crack size larger than Y)	b) Loss of over 25mm of brickwork	b) Cracking observed/Increasing deflections	b) Spalling of covercrete
	c) Cracking increasing in width or length over time		c) Result of inspection/Excessive deflections
	d) Cracking or oversailing exceeding 12mm		d) Increasing deflections
Q6. Are interventions related to a condition or a safety assessment?	Condition	a) Either	a) Condition
		b) Safety	b) Condition
			c) Safety
			d) Safety
Q7. Rate degradation in terms of costs(Operations & Maintenance and Renewal)	a) 10	a) 4	a) 3 to 6
(1-10, 1 being the most costly)	b) 4	b) 1	b) 1 to 5
	c) 1 to 3		c) 1 to 4
	d) 1 to 6		d) 1 to 3
Q8. Is there a lack of knowledge with respect to this degradation mechanism?	a) 8	a) 4	a) 6
(1-10, 1 being the degradation mechanism with the greatest lack)	b) 7	b) 3	b) 6
	c) 5		c) 4
	d) 3		d) ?
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	CIRIA report C656 NR Guide on arch defects	CIRIA report C664 NR Guide to Hidden Details	CBDG Technical Guide 2 Concrete Society publications
Q10. Do you have accessible inspection or monitoring data for this mechanism?	Inspection reports - Yes Monitoring reports - No	Inspection reports - Yes Monitoring reports - No	Inspection reports - Yes Monitoring reports - No
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	Inspection - Visual condition Monitoring - Not used	Inspection - Visual condition Monitoring - Not used	Inspection - Visual condition Monitoring - Not used
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	Periodic (no set intervals, work undertaken as deemed necessary by asset steward)	Periodic (no set intervals, work undertaken as deemed necessary by asset steward)	Periodic (no set intervals, work undertaken as deemed necessary by asset steward)

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	BRIDGES (where possible please give indication of the number/quantity of assets in each category)		
	Masonry **Not applicable, very few assessts within Trafikverket**	**Metallic**	**Concrete** **(reinforced, perstressed and post-tensioned)**
Q1. Describe experienced degradation mechanism(s)		Corrosion, fatigue and collision with traffic	Fatigue, freeze-thaw and corrosion of rebars
Q2. What is the primary agressor for this degradation?		Environmental loads and traffic loads	Traffic loads, climate, and environmental loads.
Q3. How is this degradation monitored or inspected?		Visual inspections and theoretical estimations	Theoretical estimations and visual inspections
Q4. How fast does an imminent failure develop over time?		Corrosion is slow (years) fatigue failures develop over long time (years), however failures are brittle (parts of second)	fatigue failures develop over long time (years), however failures are brittle (parts of second) Corrosion is slow (years), freeze-thay (years)
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)		corrosion is typically not allowed to give significant area loss Remaining fatigue capacity must be enough until critical parts can be exchanged	Fatigue , same as for metallic. Concrete structures must meet safety demands. Often are intervention undertaken early to avoid costly or othervise difficult operations later.
Q6. Are interventions related to a condition or a safety assessment?		Both	Both
Q7. Rate degradation in terms of costs(Operations & Maintenance and Renewal) (1-10, 1 being the most costly)		Maintenance is about 15 % och building costs	Maintenance is about 10 % och building costs
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)		Fatigue models seem to be conservative	Fatigue models seem to be conservative
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)		Trafikverket's handbook for bridge inspection is the basis. In addition, the outcome from Sustainable bridges is also used.	Trafikverket's handbook for bridge inspection is the basis. In addition, the outcome from Sustainable bridges is also used.
Q10. Do you have accessible inspection or monitoring data for this mechanism?		Background data exist, however, not sceintfically compiled	Background data exist, however, not sceintfically compiled
Q11. What key paramter(s) is/are recorded through monitoring or inspection?		Extent of initiated corrosion	Cracks and area loss
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).		Periodic inspection is the most common (99%), maximum intervall is 6 years. The inspector gives suggestion for shorter intervalls. Examples of periodic inspection twice a year exist	Same as for metallic. However, shortened intervalls for concrete bridges are not as common as for steel.

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Appendix A.3 - Tunnels

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	TUNNELS(where possible please give indication of the number/quantity of assets in each category)			
	Unlined **none**	**Masonry lining**	**Concrete Lining**	**Metallic lining** **none**
Q1. Describe experienced degradation mechanism(s)		alteration, weathering, cracks, open joints, influx	developments of cracks, bare reinforcement, influx	
Q2. What is the primary agressor for this degradation?		ageing process	bad construction quality	
Q3. How is this degradation monitored or inspected?		periodic inspection	periodic inspection	
Q4. How fast does an imminent failure develop over time?		slow	very quickly	
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)		structural safety, transport safety and operational safety	structural safety, transport safety and operational safety retification of defects within warranty	
Q6. Are interventions related to a condition or a safety assessment?				
Q7. Rate degradation in terms of costs (Operations & Maintenance and Renewal) (1-10, 1 being the most costly)				
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)		no	no	
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)		Ril 853.8001 - safety risks and defect category 1-4 and assessment categories 1-4	Ril 853.8001 - safety risks and defect category 1-4 and assessment categories 1-4	
Q10. Do you have accessible inspection or monitoring data for this mechanism?		no, but in same cases (setting pins, inclinometer, extensometer)	no, but in same cases (setting pins, inclinometer, extensometer)	
Q11. What key paramter(s) is/are recorded through monitoring or inspection?				
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).		Ril 853.8001	Ril 853.8001	

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	TUNNELS			
	Unlined (please give approx. number in network)	**Masonry lining (please give approx. number in network)**	**Concrete Lining (please give approx. number in network)**	**Metallic lining (please give approx. number in network)**
Q1. Describe experienced degradation mechanism(s)	deformations cracks breaks wetness weathering background washout	deformations cracks breaks wetness weathering background washout	deformations cracks breaks wetness weathering background washout	We haven't got this type of tunnel
Q2. What is the primary agressor for this degradation?	water movements in the cover layer tube break in cover layer	water movements in the cover layer tube break in cover layer	water movements in the cover layer tube break in cover layer	
Q3. How is this degradation monitored or inspected?	Tunnel inspections according to D5 and D11 instruction books	Tunnel inspections according to D5 and D11 instruction books	Tunnel inspections according to D5 and D11 instruction books	
Q4. How fast does an imminent failure develop over time?	Exponential	Exponential	Exponential	
Q5. Rate degradation in terms of costs (O&M and renewal) (1-10, 1 being the most costly)	deformations 2 cracks 2 breaks 1 wetness 5 weathering 7 background washout 7	deformations 2 cracks 2 breaks 1 wetness 5 weathering 7 background washout 7	deformations 2 cracks 2 breaks 1 wetness 5 weathering 7 background washout 7	
Q6. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	5	5	5	
Q7. What documents are used to assess this degradation mechanism? (guidelines, codes, internal documents)	Qualifications	Qualifications	Qualifications	
Q8. Do you have accessible inspection or monitoring data for this mechanism?	Inspectional data, peridoc checkings	Inspectional data, peridoc checkings	Inspectional data, peridoc checkings	
Q9. What key paramter(s) is/are recorded through monitoring or inspection?	Clearance gauge Longitudinal profile Geometry Moisture content	Clearance gauge Longitudinal profile Geometry Moisture content	Clearance gauge Longitudinal profile Geometry Moisture content	
Q10. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	Periodic	Periodic	Periodic	We haven't got this type of tunnel

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	TUNNELS(where possible please give indication of the number/quantity of assets in each category)			
	Unlined	**Masonry lining**	**Concrete Lining**	**Metallic lining**
Q1. Describe experienced degradation mechanism(s)	Weathering of rock faces	Material durability	Material Durability	Material durability
Q2. What is the primary agressor for this degradation?	Water ingress	Mainly water ingress and some loading	Water ingress and freeze thaw	Water ingress
Q3. How is this degradation monitored or inspected?	Detailed tactile examination	Detailed tactile examination	Detailed tactile examination	Detailed tactile examination
Q4. How fast does an imminent failure develop over time?	within five years	over 50 years but further evaluation is required	over 50 years but further evaluation is required	over 50 years but further evaluation is required
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)	Degradation in unlined areas as highlighted in the Unlined Tunnel Geotechnical Risk assessments (UTGRA)	Whole life cots model for intervention policy for specific tunnels based on degradation of defects monitored by the TCMI (Tunnel Condition Marking Index)	Whole life cots model for intervention policy for specific tunnels based on degradation of defects monitored by the TCMI (Tunnel Condition Marking Index)	Whole life cots model for intervention policy for specific tunnels based on degradation of defects monitored by the TCMI (Tunnel Condition Marking Index)
Q6. Are interventions related to a condition or a safety assessment?	Safety	Safety	Safety	Safety
Q7. Rate degradation in terms of costs (Operations & Maintenance and Renewal) (1-10, 1 being the most costly)	5 (relative to the other lining types i.e. masonry, concrete, metallic)	1	5	5
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	5 (again relative as above)	2	8	8
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	UTGRA's (Unlined Tunnels Geotechnical risk assessments	In NR - Back analysis of TCMI (tunnel condition marking Index)	In NR - Back analysis of TCMI (tunnel condition marking Index)	In NR - Back analysis of TCMI (tunnel condition marking Index) TCMI under development for this lining type.
Q10. Do you have accessible inspection or monitoring data for this mechanism?	Yes	Yes	Yes	Yes
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	Condition and condition drivers	Condition and condition drivers	Condition and condition drivers	Condition and condition drivers
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	Periodic	Periodic	Periodic	Periodic

Appendix A.4 - Track

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	TRACK (where possible please give indication of the number/quantity of assets in each category)			
	Plain Line	**Sleepers**	**Ballast**	**Switches and Crossings**
Q1. Describe experienced degradation mechanism(s)	defects in rails and and track geometry	longitudanal cracks, cross cracks, crimp cracks, composite cracks, head cracks, break outs, Treibrisse, shrinkage cracks		frog material disruption, break-out of material in areas of peak of frog and wing-rail switch side abrasion of gauge line, break-out of gauge line
Q2. What is the primary agressor for this degradation?	traffic	quality gap between fabrication, transport and storage	no drainage, to high grade of fine fraction, defects in subsoil	traffic
Q3. How is this degradation monitored or inspected?	regular inspection (site visit) in relation to operational demands (loading)	regular inspection (site visit) in relation to operational demands (loading)	regular inspection (track geometry, site visit) in relation to operational demands (loading)	regular inspection, in relation to operational demands (loading)
Q4. How fast does an imminent failure develop over time?	development of failures is immenent to loading	development of failures is immenent to loading	development of failures is immenent to loading	development of failures is immenent to loading
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)	see TSI (and some internal codes) then repair is necessary	see TSI (and some internal codes) then repair is necessary	see TSI (and some internal codes) then repair is necessary	see TSI (and some internal codes) then repair is necessary
Q6. Are interventions related to a condition or a safety assessment?	due to safety assessment	due to safety assessment	due to safety assessment	due to safety assessment
Q7. Rate degradation in terms of costs (Operations & Maintenance and Renewal) (1-10, 1 being the most costly)	1 Vegetation / 2 Replacement of rails / 3 Renewal of rails / 4 Inspections of switches and crossings / 5 Replacement of switch blades / 6 Tamping / 7 Rail grinding / 8 Track inspection	cracks that occur after warranty		defects of frog are mor expensive than defcets of swiches
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	effects of elastic components for rail defects (without frog) Is preventive track level control useful (preventive use of vibration tamper)?	effects of (new) additives on the service life	quality test could be done more specificly	1 reason of break-out of switches
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	DB Ril 821.xxxx	DB Ril 821.2003, DB Ril 821.2018	DBS 918 061, DB Ril 821.2003	DB Ril 821.2005
Q10. Do you have accessible inspection or monitoring data for this mechanism?	internal asset tool : SAP R3 / IIS	extra ordinary failures in sleepers and sleeper defects are documented	data base for reclamation is planned	in some cases
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	track geometrie, material condition	material condition	material condition	track guiding parameters and condition of material
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	regular inspection, in relation to operational demands (loading)	regular inspection, in relation to operational demands (loading) (frequency between 3 und 6); DB quality control of fabricators	regular inspection, in relation to operational demands (loading) (frequency between 3 und 6); DB quality control of fabricators	regular inspection, in relation to operational demands (loading)

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Sub-Task 2.1: Degradation and performane specification for selected assets

	TRACK			
	Plain Line	**Sleepers**	**Ballast**	**Switches and Crossings**
Q1. Describe experienced degradation mechanism(s)	1. Constructional quality	1. producing quality	1. producing quality	1. wear of components
	2. deterioration of track geometry	2. sleeper skewing	2. contamination	2. quality of instalment
	3. deterioration of structural state	3. deterioration of bores	3. appearance of vegetation	3. state of joining and intermediate joints and welds
	3.a. appearance of rail faults	4. concrete sleeper: cracking, scabbing wooden sleeper: jamming/scoring	4. slacking of stones, wear of grains, becoming round	4. deterioration of connectors (bonding units)
	3.b. appearance of welding faults	5. concrete sleeper: break wooden sleeper : rot	5. mudding	5. deterioration of geometrical state
	3.c. deterioration of fasteners			
	4. deterioration of substructure			
Q2. What is the primary agressor for this degradation?	1. Traffic load (load, speed)	1. Traffic load (load, speed)	1. Traffic load (load, speed)	1. Traffic load (load, speed)
	2. Weather load (heat fluctuation, wet)	2. Weather load (heat fluctuation, wet)	2. Weather load (heat fluctuation, wet)	2. Weather load (heat fluctuation, wet)
Q3. How is this degradation monitored or inspected?	- Periodical mechanical track geometry measurement	- Periodical track inspection (visual track inspection)	- Periodical track inspection (visual track inspection)	- Periodical turnout inspection (by manual device)
	- Periodical instrumented (manual,- mechanical) rail diagnostic inspection			- Periodical track inspection (visual track inspection)
	- Periodical visual rail inspection			
	- Periodical track inspection (visual track inspection)			
	- periodical travelling the line on vehicles			
Q4. How fast does an imminent failure develop over time?	geometry: exponential (depending on the initiating fault and rate of load)	structure: no instrumented inspection applied. Therefore we have no analysing experience.	structure: no instrumented inspection applied. Therefore we have no analysing experience.	(depending on the load)
	structure: no instrumented inspection applied, except rail. Therefore we have no analysing experience.			
Q5. Rate degradation in terms of costs (O&M and renewal) (1-10, 1 being the most costly)	1. / 9	1. / 9	1. / 8	1. / 4
	2. / 6	2. / 5	2. / 3	2. / 7
	3.a/ 5	3. / 5	3. / 4	3. / 5
	3.b/ 6	4. / 6	4. / 2	4. / 4
	3.c/ 6	5. / 3	5. / 3	5. / 4
	4. / 2			
Q6. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	1. / 9	1. / 9	1. / 9	1. / 5
	2. / 8	2. / 6	2. / 7	2. / 8
	3.a/ 5	3. / 6	3. / 7	3. / 7
	3.b/ 5	4. / 2	4. / 7	4. / 5
	3.c/ 7	5. / 5	5. / 7	5. / 8
	4. / 4			
Q7. What documents are used to assess this degradation mechanism? (guidelines, codes, internal documents)	Inner documents	Inner documents	Inner documents	Inner documents
Q8. Do you have accessible inspection or monitoring data for this mechanism?	track geometry: yes rail diagnostics: yes	no (only periodical track inspection minutes)	no (only periodical track inspection minutes)	yes, turnout inspection book (non digital)
Q9. What key paramter(s) is/are recorded through monitoring or inspection?	**geomety:** common parameters (gauge, sinking, distortion, on several base length, alignment, cross-level	crack, break, state of screw seats	contamination	wear of components
	Rail diagnostics: Rail iner fault, rail profile measurement (lateral, vertical wear)	dragging, rot	szemcsék átlagos mérete	guiding gauge
				parts problems
Q10. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	**Track geometry: periodical**	per half year detailed visual inspection	per half year detailed visual inspection	per half year detailed visual inspection
	per 6 months mechanical measurement if V>120km/h per 4 months	per month general visual inspection	per month general visual inspection	per month general visual inspection
	Rail diagnostics: periodical			
	per 6-24 months mechanical examination			
	per 6-24 months manual instrumented examination			
	General state: periodical			
	per half year detailed visual examination			
	per month general visual examination			
	per month travelling on vehicle			

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	TRACK (where possible please give indication of the number/quantity of assets in each category)			
	Plain Line plain line = total track superstructure	**Sleepers**	**Ballast**	**Switches and Crossings**
Q1. Describe experienced degradation mechanism(s)	see ML_WP2_12_111010_D1_DescriptionDegradationModelTrack	wooden ones: depending on gauge stability and force distribution, critical to water drainage concrete ones: critical to pad wear and poollution of ballast	degradation modell following an e-function of track riding quality index critical to sublayer quality, water draining, traffic load, radii, ballast quality (both strenght of ballast and distribution of size)	no seperate modell existing, behaviour close to track but additional parameters as number of trains in divertion, turnout in straight or curve, with/without cant
Q2. What is the primary agressor for this degradation?	see abvove	see abvove	see abvove	see abvove
Q3. How is this degradation monitored or inspected?	recording car + inspection of rails (ultrasonic, eddy current) + inspection for drainage + georadar	recording car + visual inspection of structure	recording car + estimation of pollution + identifiacation of pumping spots	recording car + inspection of geometric quality + inspection of structures (crack in frog, tomgue rail)
Q4. How fast does an imminent failure develop over time?	depends on boundary conditions	depends on boundary conditions	depends on boundary conditions	depends on boundary conditions
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)	twist, gauge, component failures	cracks and rail inclination	standard deviation of riding quality and moreover the deterioration of quality figure	all
Q6. Are interventions related to a condition or a safety assessment?	both (twist, gauge, single failure = safety)	both (gauge = safety)	both (twist, gauge, single failure = safety)	both (twist, gauge, single failure = safety)
Q7. Rate degradation in terms of costs (Operations & Maintenance and Renewal) (1-10, 1 being the most costly)	varying due to boundary conditions, biggest cost portion = ballast and substrtcture including water drainage system		varying due to boundary conditions	varying due to boundary conditions, biggest cost portion = ballast and substrtcture including water drainage system and components as frog
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	6	7	8	2
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	the investment and maintenance strategy of ÖBB for track is based in LCC calculations. Within LCC the degradation modell is considered	the investment and maintenance strategy of ÖBB for track is based in LCC calculations. Within LCC the degradation modell is considered	the investment and maintenance strategy of ÖBB for track is based in LCC calculations. Within LCC the degradation modell is considered	the investment and maintenance strategy of ÖBB for track is based in LCC calculations. Within LCC the degradation modell is considered
Q10. Do you have accessible inspection or monitoring data for this mechanism?	yes (4000 km of main lines, in a 10 year time row	yes (4000 km of main lines, in a 10 year time row	yes (4000 km of main lines, in a 10 year time row	yes (4000 km of main lines, in a 10 year time row
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	valid for all: standard deviation, MDZ figure, gauge, twist, rail inclination, rail foot distance, all boundary conditions (traffic volume, radii, type of superstructure)			
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	periodic (recording car 1 to 3 times a year, depeding on line importance) inspection more frequent	periodic (recording car 1 to 3 times a year, depeding on line importance)	periodic (recording car 1 to 3 times a year, depeding on line importance)	periodic (recording car 1 to 3 times a year, depeding on line importance) inspection very 2 to 6 months

MAINLINE **WP2: Degradation and structural models to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	TRACK (where possible please give indication of the number/quantity of assets in each category)			
	Plain Line (Rail)	**Sleepers**	**Ballast**	**Switches and Crossings**
Q1. Describe experienced degradation mechanism(s)	Wear Rolling Contact Fatigue (RCF) Corrugation	Wood - age - rotten Concrete - fatigue cracking Concrete - very little if not cracking	Water due to bad drainage Crushing	Wear Rolling Contact Fatigue (RCF) Fatigue cracking
Q2. What is the primary agressor for this degradation?	Traffic - Speed - Axle load - Number of axles/trains passed (MGT)	Traffic - Speed - Axle load - Number of axles/trains passed (MGT) Weather Age	Traffic - Speed - Axle load - Number of axles/trains passed (MGT)	Traffic - Speed - Axle load - Number of axles/trains passed (MGT)
Q3. How is this degradation monitored or inspected?	Track condition monitoring car Visual inspections Ultrasonic Non Destructive Testing (NDT)	Visual inspections	Track condition monitoring car - indirect Visual inspections of ditches - indirect	Track condition monitoring car Visual inspections Measured inspections Ultrasonic Non Destructive Testing
Q4. How fast does an imminent failure develop over time?	Broken rail - Might take months, but shorter than 1 year when not taken by ultrasonic NDT Insulated Rail Joints (IRJ) - Might take months to form lipping	Is not a problem	Is not a problem	Switch blade detector - Normally hours High friction - days up to weeks Snow/ice blocking - hours up to days
Q5. What are the current trigger points (thresholds) for intervention? (e.g. visual condition worse than X, crack size larger than Y)	Cracks - according to BVF 524.331: Perpendicular 10 mm meaured by NDT and 500 mm longitudinal on rail foot			Cracks - according to BVF 524.331: Perpendicular 10 mm meaured by NDT and 500 mm longitudinal on rail foot
Q6. Are interventions related to a condition or a safety assessment?	Safety			Cracks - Safety
Q7. Rate degradation in terms of costs (Operations & Maintenance and Renewal) (1-10, 1 being the most costly)	7	3	5	9
Q8. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	8	8	5	3
Q9. What are the documents used to assess this degradation mechanism? (guidelines, codes, internal documents)	BVF 524.331 UIC report on track condition montoring			BVF 524.331 UIC report on track condition montoring UIC report on inspections of switches and crossings
Q10. Do you have accessible inspection or monitoring data for this mechanism?	For cracks yes For rail profile yes For corrugation yes For track level, alignment, cant - yes		For track level, alignment, cant - yes	For cracks - yes (beside Mn-crossings) For rail profile - no For corrugation - hardly For track level, alignment, cant - hardly
Q11. What key paramter(s) is/are recorded through monitoring or inspection?	Crack length and number Rail profile (height and side wear)			
Q12. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	Periodical - Track condition monotoring: 2 months - Ultrasonic NDT: 1 year		Periodical - Track condition monotoring: 2 months - Ultrasonic NDT: 1 year	Periodical - Track condition monotoring: 2 months - Ultrasonic NDT: 1 year - Visual/Measurede inspection: 2 months

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Appendix A.5 - Other structures

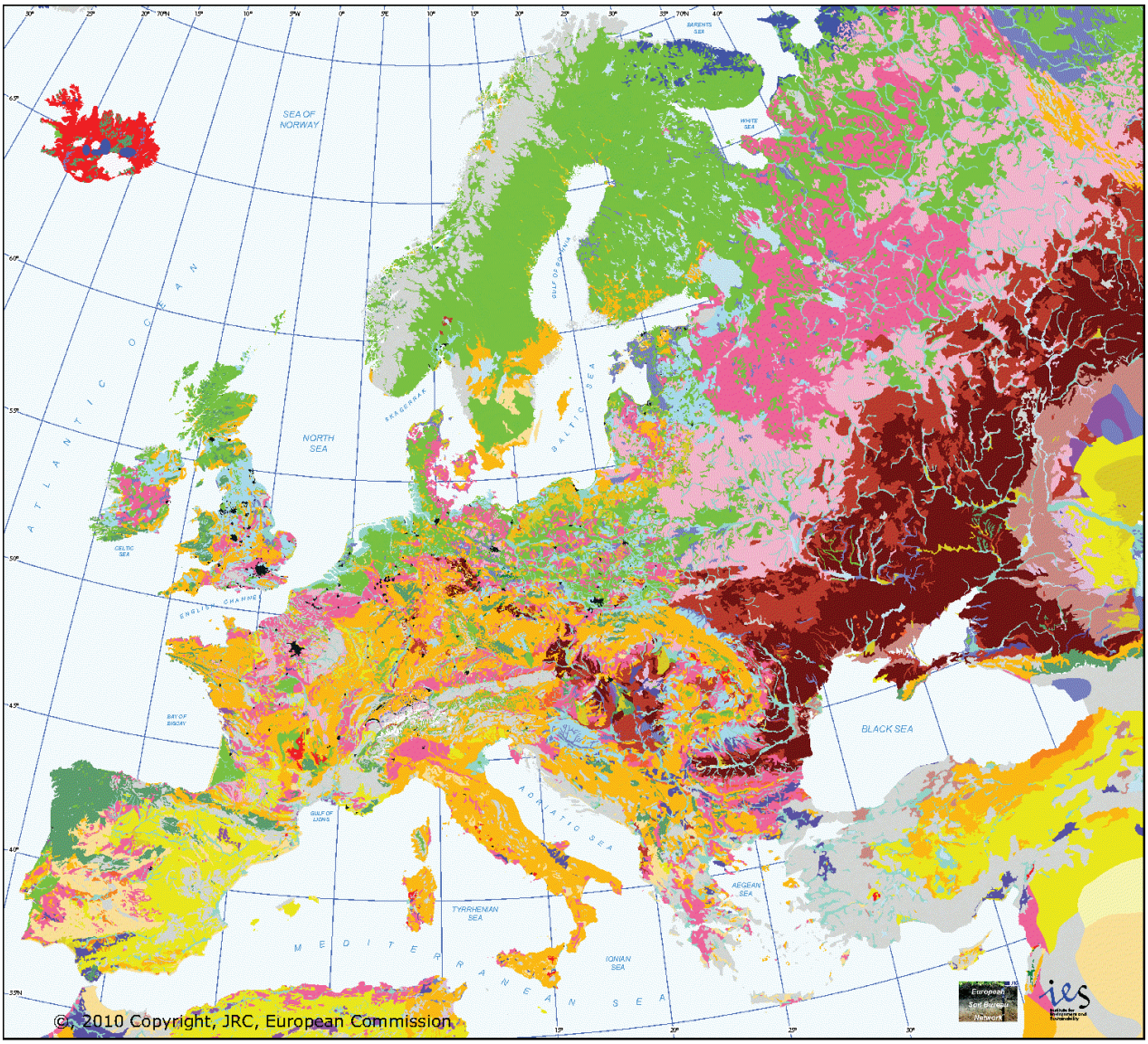
MAINLINE **WP2: Degradation and structural models** **to develop realistic life cycle cost and safety models** **Sub-Task 2.1: Degradation and performane specification for selected assets**	OTHER STRUCTURES				
	Retaining walls **(please specify material)**	**Coastal/River Defences** **(please specify material)**	**Drainage** **(please specify material)**	**Culverts** **Masonry**	**Concrete, RC concrete**
Q1. Describe experienced degradation mechanism(s)	1. Soil failure as the effect of moistering of the soil under the engineering structure 2. sliding of the engineering structure in the course of the construction of the track due to faulty technology 3. Reinforce carbonisation due to concrete surface cracks, and to loss of concrete cover 4. Mechanical damage and missing of stone, concrete and other structural materials 5. Formation of structural deficiencies due to the effect of frost	1. Soil failure as the effect of moistering of soil under the rampart 2. Mechanical damage and missing of stone, concrete and other structural materials 3. Formation of cover deficiencies due to frost 4. formation of underwashing due to water movement	1. siltation of ditches 2. sinking of the cover of ditches 3. freezing out of jointings, headers and stones due to frost 4. Cracking of concerete surfaces due to loss of concrete cover, reinforcement carbonating	1. weathering of stones 2. freeze-thaw damage of units 3. Mortar loss 4. Crack under tracks 5. Deformation of arch 6. Loss of bricks, stones 7. Separation of arch segments	1. Loss of concrete cover 2. Corrosion of reinforcement 3. Cracking of concrete 4. Water penetration 5. Freeze-thaw damage, loss of strength 6. Carbonation of concrete
Q2. What is the primary agressor for this degradation?	1. ground water, water of layers 1. frost effect 3. vibrations causes by traffic 4. the effect of melting salts used in transport 5. mechanical damages	1. water of river, lake 2. frost effect 3. water of layers	1. erodation of surrounding surfaces 2. rainwater 3. ground water, water of layers 4. frost	1. Air pollution, humidity 2. Temperature, wetness 3. Wetness 4. Traffic loading 5. Traffic loading, earth pressure 6. Freeze-thaw cycles, traffic loading 7. Traffic loading, earth pressure	1. Corrosion of steel, wetness 2. Wetness, poor quality of concrete, insufficient cover, chloride 3. Overloading, movements 4. Damaged insulation 5. Temperature, wettness 6. Air pollution
Q3. How is this degradation monitored or inspected?	1. By visual inspection 2. In some places by geodetic survey	1. By visual inspection	1. By visual inspection	1. Visually 2. Visually 3. Visually 4. Visually 5. Visually 6. Hammer tapping 7. Visually	1. Visually, hammer tapping 2.Visually 3. Visually 4. Visually 5. Visually 6. None
Q4. How fast does an imminent failure develop over time?	1. After slow displacements sudden deterioration	1 slow deterioration	1 slow deterioration	1. Development requires decades depending on stone quality 2. Development requires years or decades depending on parameters such as stone quality, presence of wettness, temperature 3. Development requires years or decades depending on parameters such as mortar quality, presence of wettness, temperature, wind intensity 4. Development requires years or decades depending on parameters such as height and quality of fill, dimensions of the arch, dynamic effects 5. Development requires years or decades depending on parameters such as height of fill, dimensions of the arch, loading effects 6. Development requires years or decades depending on parameters such as height and quality of fill, dimensions of the arch, dynamic effects 7. Development requires years or decades depending on parameters such as height of fill, earth pressure, type of foundations, subsoil, traffic loading	1. Development requires years or decades depending on concrete quality and wetness 2. Development requires years or decades depending on parameters such as concrete quality, wetness, concrete cover 3. Development requires years or decades depending on structural dimensions, loading, condition of foundations 4. Development requires years or decades depending on quality and type of insulation, quality of concrete 5.Development requires years or decades depending on quality and type of insulation, quality of concrete 6. Development requires decades
Q5. Rate degradation in terms of costs (O&M and renewal) (1-10, 1 being the most costly)	1. 1 2. 1 3. 3 4. 3 5. 2	1. 1 2. 3 3. 4 4. 2	1. 8 2. 5 3. 5 4. 4	1. 6 2. 4 3. 6 4. 3 5. 6 6. 5	1. 6 2. 4 3. 5 4. 3 5. 4 6. 6
Q6. Is there a lack of knowledge with respect to this degradation mechanism? (1-10, 1 being the degradation mechanism with the greatest lack)	1. 3 2. 5 3. 4 4. 5 5. 4	1. 3 2. 5 3. 3 4. 2	1. 10 2. 5 3. 5 4. 4	1. 2 2. 4 3. 3 4. 7 5. 4 6. 8 7. 6	1. 8 2. 8 3. 7 4. 7 5. 6 6. 8
Q7. What documents are used to assess this degradation mechanism? (guidelines, codes, internal documents)				Bridge inspectors guide (under preparation)	Bridge inspectors guide (under preparation)
Q8. Do you have accessible inspection or monitoring data for this mechanism?	Evaluation of geodetical surveys are available			Only inspection files at individual culverts Monitoring was not carried out	Only inspection files at individual culverts Monitoring was not carried out
Q9. What key paramter(s) is/are recorded through monitoring or inspection?	x,y,z coordinates of geodetical points continuity of surfaces geometry of surfaces	continuity of surfaces geometry of surfaces	continuity of surfaces geometry of surfaces	1. depth of degradation 2. visual view 3. depth of loss 4. crack length, width, location, orientation 5. Shape of arch visually 6. depth and area of loss 7. Opening of gap	1. Area with lost cover, visible signs of the initiation of loss of cover 2. Loss of effective cross-section area of reinforcement, quality of ribs on reinforcement 3. Crack width 4. Area of wetness 5. None 6. None
Q10. Is monitoring continuous or periodic? (please state the relevant time intervals/parameters).	periodical monthly, quarterly, yearly	periodical monthly, quarterly, yearly	periodical monthly, quarterly, yearly	Only periodic. Artisan's inspection is carried out every year.	Only periodic. Artisan's inspection is carried out every year.

13. APPENDIX B – European maps and data

The following maps are published by the European Environment Agency.

From the EEA website: 'Copyright holder – European Commission. The re-use of content on the EEA website for commercial or non-commercial purposes is permitted free of charge, provided that the sources is acknowledged.'

Figure B1 - Major soil types across Europe defined by their WRB Reference Group

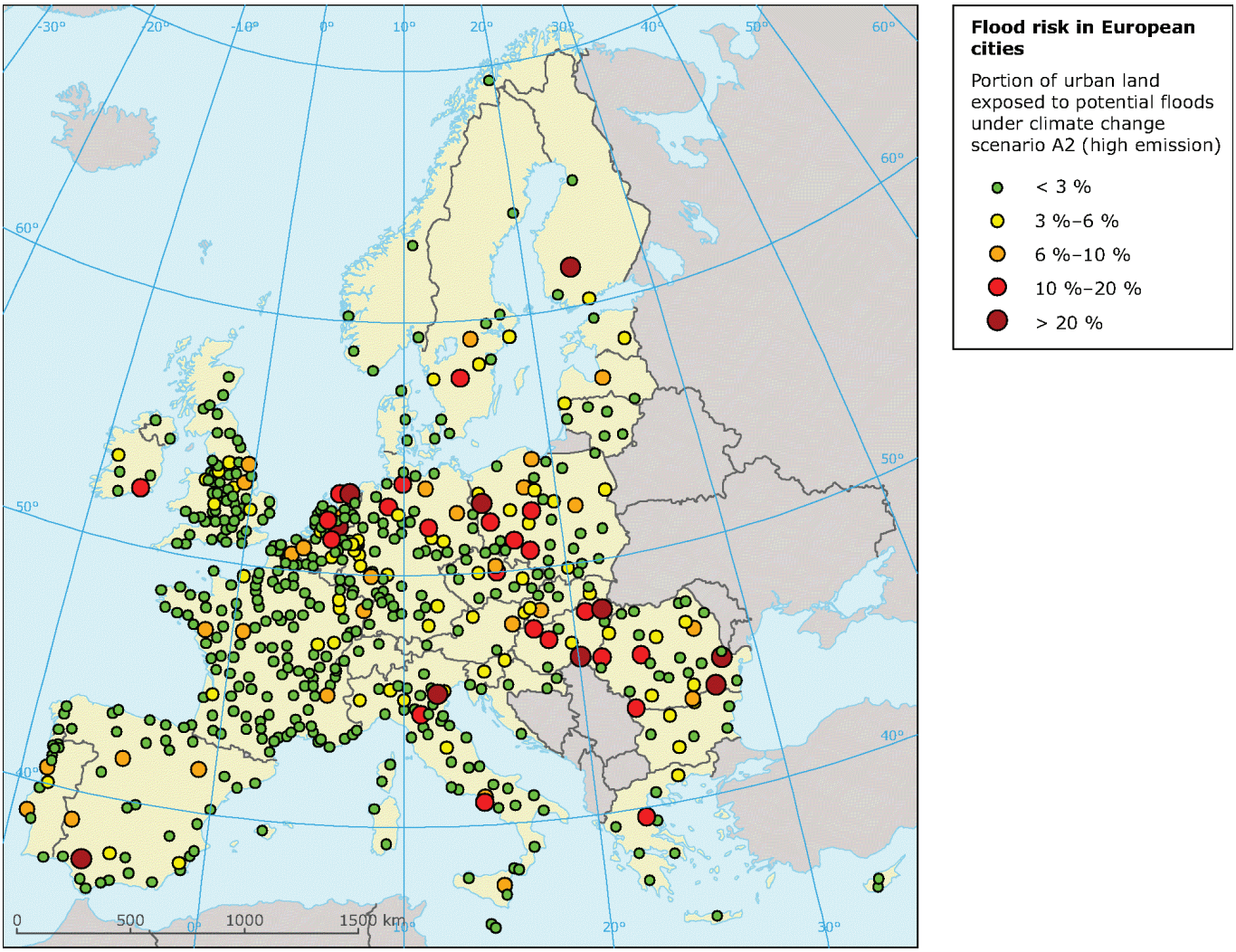


- Albeluvisols: Acid soils with bleached topsoil material tonguing into the subsoil
- Arenosols: Soils developed in quartz-rich, sandy deposits such as coastal dunes or deserts
- Cambisols: Young soils with moderate horizon development
- Cryosols: Soil influenced by permafrost or cryogenic processes
- Gleysols: Soils saturated by groundwater for long periods
- Histosols: Organic soils with layers of partially decomposed plant residues
- Andosols: Young soils developed in porous volcanic deposits
- Calcisols: Soils with significant accumulations of calcium carbonate
- Chernozems: Dark, fertile soils with organic-rich topsoil
- Fluvisols: Stratified soils, found mostly in floodplains and tidal marshes
- Gypsisols: Soils of dry lands with significant accumulations of gypsum
- Kastanozems: Soils of dry grasslands with topsoil that is rich in organic matter
- Leptosols: Shallow soils over hard rock or extremely gravelly material
- Luvisols: Fertile soils with clay accumulation in the subsoil
- Phaeozems: Dark, moderately-leached soils with organic rich topsoil
- Vertisols: Heavy clay soils that swell when wet and crack when dry
- Podzols: Acid soils with subsurface accumulations of iron, aluminium and organic compounds
- Regosols: Young soils with no significant profile development
- Solonchaks: Soils with salt enrichment due to the evaporation of saline groundwater
- Solonetz: Alkaline soils with clayey, prismatic-shaped aggregates and a sodium-rich subsurface horizon
- Stagnosols: Soils with stagnating surface water due to slowly permeable subsoil
- Technosols: Soils containing significant amounts of human artefacts or sealed by impermeable material
- Umbrisols: Young, acid soils with dark topsoil that is rich in organic matter
- Planosols: Soils with occasional water stagnation due to an abrupt change in texture between the topsoil and the subsoil than impedes drainage

Source – European Environment Agency online data and maps:

http://www.eea.europa.eu/data-and-maps/figures/the-major-soil-types-of-europe

Figure B2 - Climate change impacts - exposure to flood risk under the climate change scenario A2 over 100 year return period

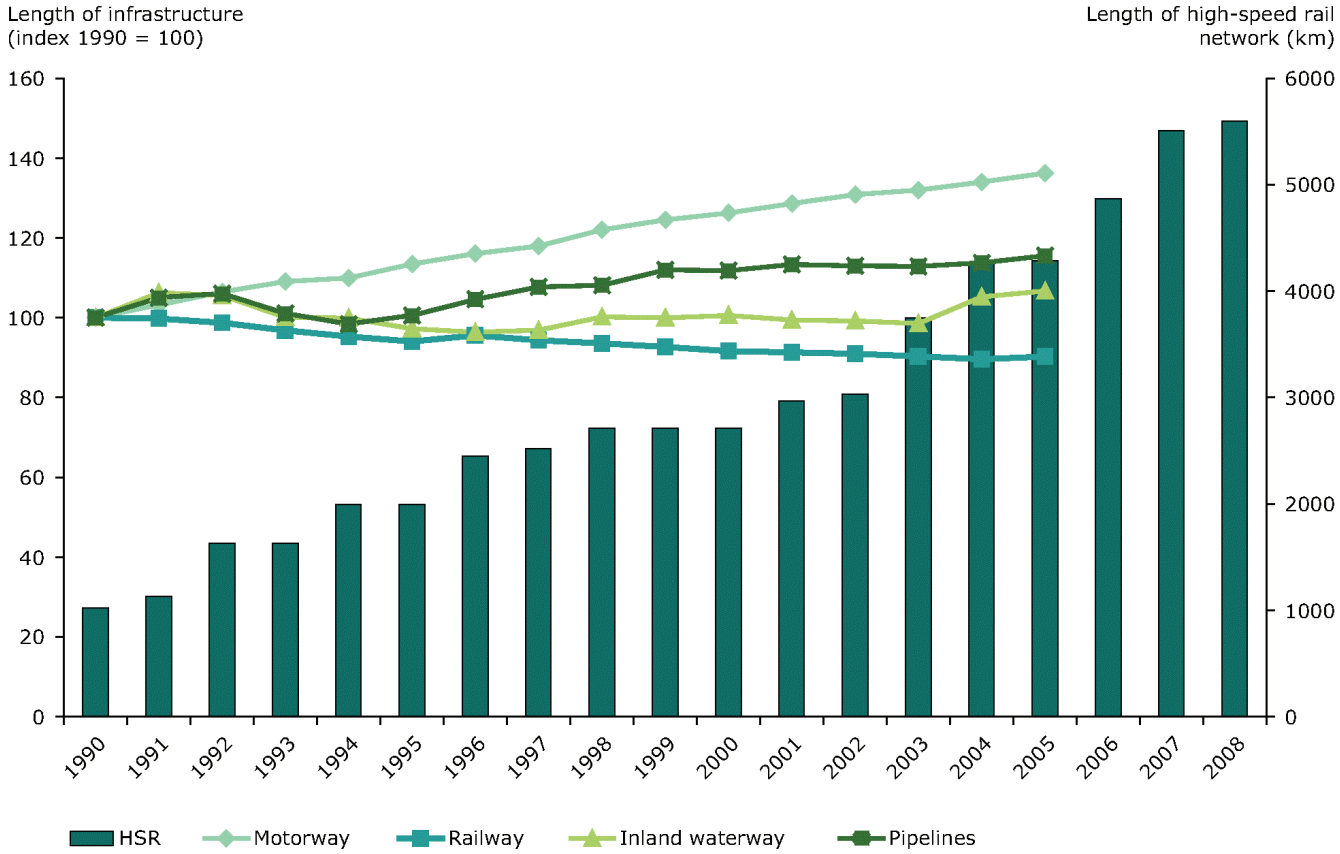


This is a non- ction scenario (high emissions). It shows that several major European cities are potentially exposed to flood events.

Source – European Environment Agency online data and maps:

http://www.eea.europa.eu/data-and-maps/figures/climate-change-impacts-2014-exposure-to-flood-risk-under-the-climate-change-scenario-a2

Figure B3 - Length of land transport infrastructure in the EEA-32 (32 member countries of the European Environment Agency)



Source: European Environment Agency online data and maps:
http://www.eea.europa.eu/data-and-maps/figures/term18-length-of-land-transport-infrastructure-in-the-eea-32