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 1.3: New technologies to extend the life of elderly rail infrastructure

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Abstract of the MAINLINE Project

Growth in demand for rail transportation across Europe is predicted to continue. Much of this growth will have to be accommodated on existing lines that contain old infrastructure. This demand will increase both the rate of deterioration of these elderly assets and the need for shorter line closures for maintenance or renewal interventions. The impact of these interventions must be minimized and will also need to take into account the need for lower economic and environmental impacts. New interventions will need to be developed along with additional tools to inform decision makers about the economic and environmental consequences of different intervention options being considered.

MAINLINE proposes to address all these issues through a series of linked work packages that will target at least €300m per year savings across Europe with a reduced environmental footprint in terms of embodied carbon and other environmental benefits. It will:

- 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 consortium includes leading railways, contractors, consultants and researchers from across Europe, including from both Eastern Europe and the emerging economies. Partners also bring experience on approaches used in other industry sectors which have relevance to the rail sector. Project benefits will come from keeping existing infrastructure in service through the application of technologies and interventions based on life cycle considerations. Although MAINLINE will focus on certain asset types, the management tools developed will be applicable across a broader asset base.

Partners in the MAINLINE Project

UIC, FR; Network Rail Infrastructure Limited, UK; COWI, DK; SKM, UK; University of Surrey, UK; TWI, UK; University of Minho, PT; Luleå tekniska universitet, SE; Deutsche Bahn, DE; MÁV Magyar Államvasutak Zrt, HU; Universitat Politècnica de Catalunya, ES; Graz University of Technology, AT; TCDD, TR; Damill AB, SE; COMSA EMTE, ES; Trafikverket, SE; Cerema (ex SETRA), FR; ARTTIC, FR; Skanska a.s., CZ.

WP1 in the MAINLINE project

The main objective for WP1is is to apply new technologies to extend the life of elderly rail infrastructure. This main objective can be subdivided in the following way:

- to explore and evaluate new technologies to extend the life length,
- to develop new more accurate assessment methods to determine if and when the life can be extended with and without any interventions (as e.g. strengthening),
- to further develop new technologies that can reduce life cycle costs for repair and strengthening and minimize the necessary traffic interruption,
- to develop a guideline for the application of new technologies to extend the life length,
- to transfer existing knowledge of new technologies to Eastern Europe and developing economies.

The figure on the cover shows the Åby bridge in northern Sweden, which was used for demonstrations of new technologies regarding assessment methods

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Glossary

Abbreviation/ acronym	**Description**
ACI	American Concrete Institute
CEN	European Committee for Standardization
DoW	Description of Work
EC	European Commission
fib	International Federation for Structural Concrete
IABMAS	International Association for Bridge Maintenance and Safety
IABSE	International Association for Bridge and Structural Engineers
IALCCE	International Association for Life-Cycle Civil Engineering
IM	Infrastructure Manager
LCA	Life Cycle Analysis
LCC	Life Cycle Cost
LCCA	Life Cycle Cost Analysis
LCAT	Life Cycle Assessment Tool
RILEM	International union of laboratories and experts in construction materials, systems and structures
SIA	Swiss Society of Engineers and Architects
SB	Sustainable Bridges, EC FP6 Project
TecRec	Technical Recommendation approved as standard by UIC and UNIFE
UIC	International Union of Railways
UNIFE	Association of the European Rail Industry
WP	Work Package

1. Executive Summary

There are many traditional technologies available to extend the life of elderly rail infrastructure, some of which are being improved or developed, whilst new technologies continue to emerge.

In two earlier reports a benchmark of new technologies was given and assessment methods were presented, ML-D1.1 (2013) and ML-D1.2 (2013). In this report, ML-D1.3, an overview is given of some of the most promising new or updated technologies. Based on the findings, work in the Mainline project has focused on the following two areas for bridges, tunnels and track:

- Assessment methods
- Repair and Strengthening methods

Some of the methods are still under development and may not yet be available commercially. Hence these are presented on a “for information” basis and as something that may be introduced on a broader scale in a near future.

In the report assessment and strengthening of bridges are treated in Chapter 4 and Chapter 5. Tunnels are treated in Chapter 6 and track and earthwork in Chapter 7.

The report also includes with five appendices with details of important work that has been done in the MAINLINE project. Appendix A presents results from the assessment and full scale testing to failure of a 50 year old metallic truss bridge. Appendix B presents results from the strengthening by post-tensioning of a concrete trough bridge. Appendix C presents methods to extend life for tunnels. Appendix D proposes methods for the assessment of fatigue and Appendix E, finally, gives a fairly comprehensive list of references on how to extend the life of structures.

A Guideline for application of the new technologies is given in ML-D1.4 (2014).

2. Acknowledgements

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- Network Rail, UK (Brian Bell, John Amoore, Colin Sims, Eifion Evans)
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3. Introduction

3.1 General

The MAINLINE project has the overall aim to give railway infrastructure owners tools to reduce environmental footprint and prolong the life length of existing rail infrastructure.

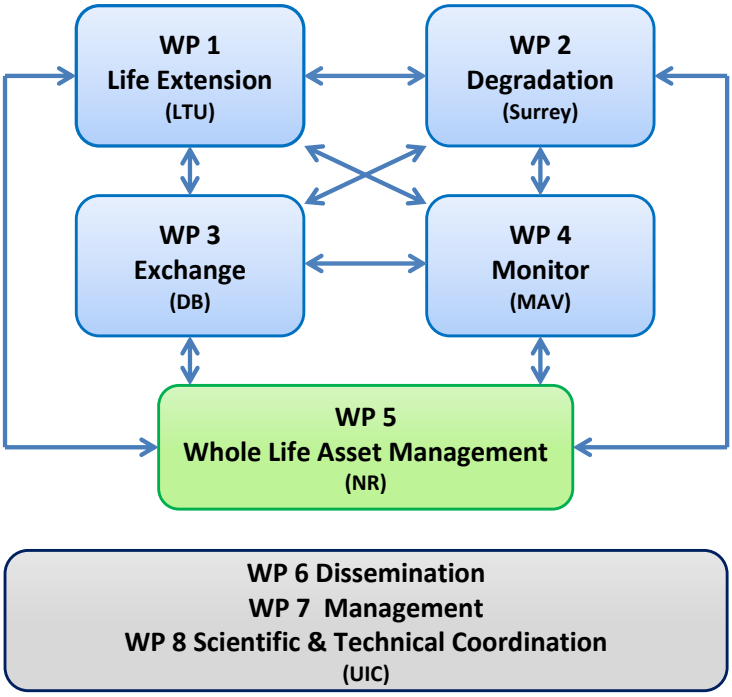


Figure 3-1 General organization of the project

The purpose of this report is to present technologies that have the best potential of achieving this aim regarding methods to prolong the life of existing elderly rail infrastructure. A chart illustrating the project is given in *Figure 3-1.*

WP1 has interacted with the other WPs helping to identify new technologies, methods and data that have the potential to improve degradation and structural models (WP2), replacement (WP3), monitoring (WP4), whole life environmental and economic asset management (WP5) and dissemination of results (WP6). Results are uploaded on the web, see http://www.mainline-project.eu/Results,7.html.

The partners in WP1 are listed in Table 3-1.

Table 3-1 Partners in WP 1

Part n°	WP1 Partners	Country
1	Union Internationale des Chemins de Fer - UIC	France
2	Network Rail Infrastrukture LTD - NR	United Kingdom
7	Universade do Minho - UMinho	Portugal
8	Luleå Tekniska Universitet - LTU	Sweden
11	Universitat Politecnica de Catalunya - UPC	Spain
19	Skanska AS - Skanska	Czech Republic
20	Jacobs/SKM	United Kingdom

3.2 Outline

In this report, new technologies are presented that have become available during recent years to extend the life of existing elderly rail infrastructure. The report is a follow up of ML-D1.1 (2013) “Benchmark of new technologies to extend the life of elderly rail infrastructure” and ML-D1.2 (2013) “Assessment methods for elderly rail infrastructure”. It will be followed by Guideline: ML-D1.4 (2014): “Guideline for application of new technologies to extend life of elderly rail infrastructure”.

Some of the technologies presented here have earlier been introduced in the EC-FP6 projects Sustainable Bridges (2007) and Innotrack (2010). Others have been reported in a questionnaire to railway infrastructure owners; see Appendix A. in ML-D1.1 (2013). Still others are treated in three parallel EC projects: Automain (2014), Smartrail (2014) and Sustrail (2014). Of these, Automain and Sustrail concentrate on track, while Smartrail has a scope more similar to that of Mainline. Smartrail focus on earthwork, transition zones, slopes and tunnels and Mainline focus more on bridges and track. Both projects work with life cycle cost assessment, LCCA.

Assessment and strengthening methods for bridges are first treated in chapter 4 and chapter 5. Tunnels are treated in chapter 6 and track is treated in chapter 7. A full scale test to failure of a metallic bridge is presented in Appendix A. Strengthening of a railway bridge in Haparanda, Sweden, is presented in Appendix B. Methods for tunnels are presented in Appendix C and some notes on fatigue assessment in Appendix D. Finally a fairly comprehensive list of references is given in Appendix E.

4. Assessment Methods for Bridges

In this chapter Section 4.1 presents reliability based assessment methods, system safety, robustness and redundancy criteria, side specific live loads, dynamic amplification factors, model updating and proof load testing. Section 4.2 discusses finite element models (FEM) and Section 4.3 treats temperature effects. Section 4.4 finally gives some notes on degradation modelling.

4.1 General

4.1.1 Background

The assessment of existing infrastructure is becoming essential as their age is increasing day after day. The assessment process can be more or less sophisticated, cumbersome and accurate, depending on the asset to be evaluated and the required information to be obtained. In the context of the MAINLINE project, where the optimum management strategies from an economic point of view are foreseen, the costs incurred in the assessment and the information obtained take mximum relevance. Therefore, different levels of assessment should be considered depending on the chosen results and the consequent cost. Recent progress is presented in the conference proceedings Chen et al (2014)

In general, the best final assessment approach is the cheapest method which shows the structure to have the required strength, CIRIA C664 (2008). Since the assessed strength will not be known until the assessment is well advanced, it will sometimes be necessary to refine the approach and use more advanced methods as the assessment progress. As indicated in the CIRIA document “It is reasonable, indeed desirable, to assess bridges as safe using simple conservative approaches. However, it is never appropriate to condemn them on the basis of such assessments without first reviewing the findings and asking the question: is this failure real, or merely the result of a conservative assessment? The result is that assessment can be an iterative process involving progressive screening".

Assessing the safety of an existing bridge for determining its load carrying capacity, evaluating its ability to support increased loading, or prolonging its service life, should evolve to become an adaptive, multi-level process that allows for the refinement of an engineer’s initial estimate of the present and future state of the bridge and its behaviour. This is, for instance, the procedure adopted in the Sustainable Bridges project (SB-LRA 2008).

At the Initial Level, the assessment is usually performed using standard methods similar to those used in design. If the bridge passes the Initial Level assessment, no additional analyses or actions are necessary and the bridge remains in operation as it is. Bridges that fail to pass initial safety checks should be re-evaluated using Intermediate Level analysis procedures, which would involve any combination of the following methods, Wisniewski et al. (2012):

- Additional more thorough inspections with possible field testing for material properties to obtain better estimates of member strengths.
- The use of Weigh-In-Motion (WIM) or other monitoring techniques data to obtain improved estimates of the applied loads.
- The use of refined structural analysis models or field measurements to obtain better estimates of the response of the bridge to the applied loads.

It is evident that increasing the level of assessment will require more resources for advanced experimental methods; theoretical analyses etc.…The decision to proceed to the next level should be justified by the saving of resources resulting from the final result. The new expenses derived from the more advanced assessment and the savings because the final decision must result in a lower strengthening or repair need. The cost of advanced assessment must always be lower than the cost of the intervention that is avoided by obtaining a pass, If the result is equilibrium then it would be better to replace the structure as that should lead to lower maintenance costs in the future. Therefore, in the framework of a Life Cycle Assessment (**LCA)** as the one proposed in the MAINLINE project, the decision on the use or not of a more advanced assessment method will be based upon the following criteria:

1.- To minimize the total cost, expressed as:

$$C_{total} = C_{cons} + C_{insp} + C_{ass} + C_{user} + C_{repair} + C_{failure} \quad (4.1)$$

C_{cons}= construction cost
C_{insp}= cost of inspection and routine maintenance
C_{ass} = cost of assessment
C_{user} = user cost
C_{repair} = cost of repair, strengthening
$C_{failure}$= cost associated to the failure of the bridge to perform a required limit state

2.- with the following constraints: To guarantee a minimum performance level (safety, service) to the user

$$S > S_0 \quad (4.2)$$

S = the actual performance level
S_0 = a prescribed minimum performance level

It thus makes sense to examine the feasibility of enhanced or advanced assessment as a mean of reducing future repair and failure costs, and consequently, the total life-cycle cost. Therefore, advanced assessment may be seen as the most cost effective way to extend the service life of an existing bridge, before anything else is done, allowing, in some cases, the continuation of the bridge in its normal service operation, without additional expenses. This justifies the inclusion of advanced assessments in the LCA of existing railway bridges. In the LCA tool developed in Mainline this is not yet included but it should be an aim for further development.

The assessment of bridges, like other structures, is still usually done using approaches which were originally developed for design. The analytical procedures used in design are often conservative. This approach does not suppose a large increment of cost since placing more material (steel, concrete) during the construction of a new bridge is relatively inexpensive compared to labour cost. However, this is not the case when retrofitting structures already in service. Alternative approaches are available which give more realistic results and the high cost of strengthening existing structures makes it worthwhile using them in assessment.

One of the reasons why design codes are conservative is because they are based on a member by member analysis, not taking into account the bridge as a system, where the failure of an element does not automatically mean the failure of the whole bridge. Tests in the laboratory, as well as experiences from the real world, show that most bridges have important reserves of strength when compared to their capacity predicted during the design or even obtained with a sophisticated analytical assessment tool. This is mainly due to the following facts:

1.- The inaccuracy of the theoretical models available to predict the behaviour of existing bridges. Although important advances in relating to the non-linear behaviour of materials and geometric simulations have been made recently, 100 % accurate modelling is not yet possible.

2.- Not taking into account resisting mechanisms that may appear during the in-service state and were not considered, for instance, the composite action between a priori non-composite elements.

3.- Actual design and assessment methods are based on a member target, identifying the failure of a single member. The failure of the whole bridge, where redundancy and robustness are important is not considered.

The reserve of strength in existing bridges is clearly shown in the examples presented in Deliverable ML-D1.2. An advanced assessment, either based on a more sophisticated numerical analysis (see 4.2 below), an up-date of the load conditions (see 4.1.4) or a load test on the bridge (see 4.1.6), could discover the possibility of a higher strength and the possibility to put the bridge again in operation with a minimum retrofitting work.

In WP5, Task 5.4 of MAINLINE, the key parameters for LCA (Life-cycle assessment) of railway infrastructure have been identified. Among them, in the required asset information, the actual asset condition and the life extension appear as two important key elements for a posterior LCA of the asset. In fact, in deliverable D5.4, they are pointed out in the concluding remarks of the parameter asset condition: "*The asset condition is highly relevant to the LCA tool since the LCAT is required to take into account the minimum safety level of asset performance. This is influenced by the asset conditions and its deterioration level. The selection of reliable asset condition parameters is fundamental to the successful operation of a deterioration-based mode*l". The condition parameters relevant to LCAT will be significantly influenced by the deterioration models (see 4.1.5 and 4.4). Therefore, the accurate knowledge of the actual condition and safety of the asset is presented as a key parameter. This accurate knowledge should be provided by means of the advanced assessment of the bridge.

When a policy of combining advanced assessment with less severe interventions is applied, we should refer to figure 4.1 as a way to refine engineering judgment. From a wider perspective, as shown in the figure, the "advanced assessment" should be also considered in the process of returning the condition of the asset to the correct performance level (point 4). In the case that only a basic assessment is carried out, the structure is believed to be in point 1. Therefore, to enhance the performance from this point 1 to point 4 will require a repair work with a total cost C_1. If an advanced assessment is undertaken, this will have an additional cost C_2. However, if the result of the assessment is that the real situation of the bridge is better than predicted (point 2), then the repair cost to bring the structure to point 4 (C_3) will be lower than C_1. Therefore the final decision to minimize the life-cycle cost should take into account the relative costs C_1 (cost of repair without advanced assessment), C_2 (cost of advanced assessment) and C_3 (cost of repair after advanced assessment). Within a LCA framework also the timing of the assessment (t_{ass}) should be considered as one of the variables. However the performance level obtained

from an advanced assessment may be lower than that obtained from the basic assessment. Within an LCA framework, the advanced assessment may be viewed as an alternative to the repair work, in the way that they both result in the up-dating of the actual bridge performance. However, in this case, contrary to the case of repair, maintenance and renewal, the updating from advanced assessment can be either positive (higher performance indicator, point 2) or negative (lower performance indicator, point 3). Also, the advanced assessment may have an important influence in the life-cycle management process. In fact, in the minimization problem expressed in equation (4.1), with the boundary condition expressed in equation (4.2), the practical solution may be completely different if it is discovered by means of an advanced assessment that the actual safety or performance level is below the minimum required as it happens in point 3 of the figure.

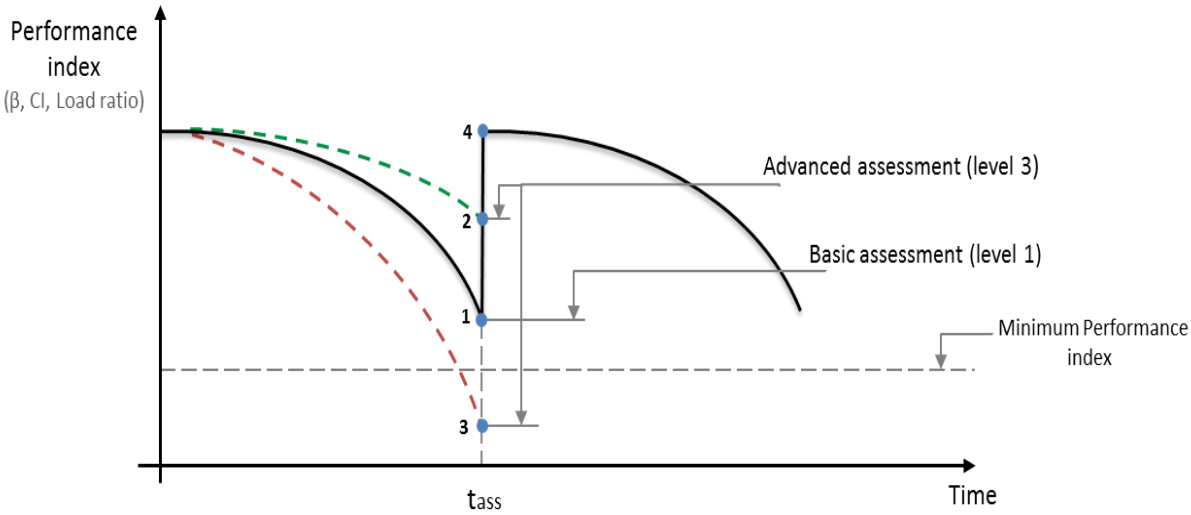


Figure 4-1 Model of life cycle condition of an asset with repair and renewal interventions taking into account the result of and advanced assessment

Life Cycle Assessment Tool should preferably quantify:

- Direct economic costs
- Availability (Delay costs/user cost/benefit from upgrade etc.)
- Environmental impact costs

For some advanced assessment methods, only direct economic costs and environmental impact costs will be considered (no user costs). This will be the case when the assessment does not require interruption to the normal traffic operation across the bridge. However, some advanced assessment techniques will require closure of the bridge meaning that disruption costs will need to be considered. The environmental impact costs (benefits) come from the fact that knowledge of actual capacity of the asset may derive in large economic savings coming from no need of repair/strengthening/demolition.

Deliverable ML-D1.4 provides a catalogue of maintenance strategies to extend the service life of predefined existing types of assets. Proper maintenance ensures that the service life of the assets may be extended considerably. Such maintenance should be carried out with due attention to the total cost and environmental impact. Among the maintenance strategies, the one of “doing nothing” can be considered as the most economical effective and environmental friendly. Such strategy can be the result of an advanced assessment which reveals the asset still in good condition to continue its function without any up-grading.

New methodologies for a more accurate assessment would involve one or a combination of the following tools and techniques (ML-D1.2):

- Direct application of reliability-based assessment methods
- Consideration of system safety, redundancy and robustness criteria
- Site-specific loads and impact factors
- Material and diagnostic load testing, including Structural Health Monitoring (SHM) and model updating
- Proof load testing

In the following a short description of these new technologies is presented. The application of some of them on a 50 year old metallic truss bridge is illustrated in Appendix A.

4.1.2 Direct application of reliability-based assessment methods

The application of these methods is possible if permitted by the IM. when the standard deterministic or semi-probabilistic methods declare the bridge as not satisfying relevant limit states and before any remedial repair or strengthening is undertaken. With the application of this method, the reliability index, related to the bridge safety, is used as the performance index and a minimum target value is indicated as providing the minimum safety required for the structure. Therefore, with the application of this method, the performance index is directly obtained and no subsequent analyses are needed.

A complete description about how probability-based assessment may be carried out can be found in the deliverable SB-LRA (2008) from SUSTAINABLE BRIDGES and the deliverable SR-D2.2 (2014) from SMARTRAIL An example of application of this technique is shown in Appendix A.2.

4.1.3 Consideration of system safety, robustness and redundancy criteria

The importance of considering structural robustness and redundancy in the assessment processes is highlighted by a number of historical events that led to catastrophic collapse following local failures in critical members. As the standard assessment methods are based on member failure criteria, the new proposed technology uses the measure of system safety and robustness as a modifying factor in the rating equation.

Although the concepts are well understood, researchers and engineers have been struggling to come up with consistent non-subjective definitions of robustness and redundancy. Robustness is defined as the capability of the structure in a damaged state affecting a local or member component to continue to carry load independent of hazards that provoked the initial damage. In other words robustness may be defined as "*the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause*." (EC 0 (2002) and EC 1-7 (2006)). Redundancy is defined as the capability of an originally intact structure to continue to carry load after the failure of one element. Consequently the two terms robustness and redundancy are closely related and the main difference is that robustness is related to loads and the load-carrying capacity while redundancy deals with changes in the geometry (loss of elements) in a structure. The concept of disproportionate collapse must always be kept in mind. A complete study on the robustness of structures is available in COST-TU0601, Cavaco et al. (2013 a,b), Anitori et al. (2013).

Significant discussion exists on the advantages and disadvantages of different methodologies to evaluate robustness. Three different levels of analysis can be used to quantify robustness: risk analysis, probabilistic analysis and deterministic analysis (ML-D1.2).

Risk based approaches are the most complete framework to calculate the robustness of structures. They provide the most comprehensive model which accounts for the probability of structural collapse and simultaneously account for the economic, political and societal consequences of collapse. The model can be summarized by the following expression:

$$R = \sum_{D,H} P(C|D) \cdot P(D|H) \cdot P(H) \cdot Cost(C) \tag{4.3}$$

where $P(H)$ is the probability of occurrence of an abnormal hazard event which is related to the exposure of the bridge to damaging events , $P(D|H)$ is the conditional probability of local damage given the event H H and this is related to the vulnerability of the structural member to the hazard, $P(C|D)$ is the conditional probability of collapse C of the structure as a result of the local damage D which according to many researchers is related to structural robustness, R is the total risk of the structure, and *Cost (C)* is the cost of collapse in which the consequences of collapse are given monetary values in order to establish common units to add the tangible and intangible consequences. The summation extends to all the domains of damage D and hazards H.

The approach follows closely the well established procedures that had been used in seismic hazard analysis studies and the reliability analysis of bridge systems under the effect of traffic overloads. However, the implementation of the general methodology proved to be extremely difficult for several reasons including: a) lack of accurate models for many critical natural and man-made hazards, b) difficulty of modelling the response of bridge structures to extreme events, c) difficulty of performing a reliability analysis for complex structural systems, d) determination of acceptable levels of reliability.

A risk measure of robustness was proposed by Baker et al. (2008) defining robustness as the ratio between direct, R_{Dir}, and indirect, R_{Ind},, risks associated with unexpected events:

$$I_{Rob} = \frac{R_{Dir}}{R_{Dir} + R_{Ind}} \tag{4.4}$$

This approach is very general, but extremely complex, in particular due to the need to clearly define the frontier between direct and indirect consequences, as well as, the quantification of these consequences.

The risk approach and the robustness index shown in eq. 4.4 represent the most complete and rigorous approach to robustness, although the practical calculation of these measures is still in a preliminary phase. For example, the effort of some researchers (Imam 2012; Janssen 2012) in the last years has shown that the data on the consequences of collapse are still too few to build a prediction tool for assessing the consequences of failures. It has been also found that even enumerating the full range of consequences is not an easy task.

According to Ghosn and Moses (1998) and Liu et al. (2000), redundancy is defined as the capability of the system to continue to carry load after the failure of one main member. Redundancy is measured by means of three parameters. Two measures are related to the

intact configuration of the structure and are related to structural collapse and loss of structural functionality. The third measure is calculated for a damaged configuration of the structure and permits to evaluate the capability of the system to carry some emergency load after the damage in one main member. The measures can be calculated for bridge superstructures considering vertical load capacity or for bridge substructures considering lateral load capacity. In addition to the definitions, Ghosn et al (1998, 2000) calibrated criteria for evaluating the redundancy of specific structures based on the reliability analysis of typical configurations. The measures of redundancy advanced by Ghosn et al (1998, 2000) are practical and can be implemented in structural codes. Based on this approach, a proposed framework to include redundancy in the assessment of existing railway bridges is presented in Wisniewski et al (2006) and in the advanced assessment of existing highway bridges in Casas et al. (2012) by means of the calculation of system factors.

Cavaco et al. (2013 a) proposed an alternative approach considering that a single robustness indicator, R_d, must be defined for all levels of damage. The proposed indicator can be defined as:

$$R_d = \int_{d=0}^{d=1} f(x)dx \qquad (4.5)$$

where *f* is the normalized performance, given by the ratio between the structural performance on the intact and damage states, and *d* is the normalized damage, given by the ratio between actual and maximum possible damage.

This approach for robustness allows the consideration of several damage scenarios as performance indicators. Damage should be considered with a broader sense, i.e., damage can vary from a simple degradation state to a more serious damage as a column or a beam failure. Errors during the design or the construction stage can also be seen as types of damages. The structural performance can assume many forms (load factor, reliability index,…), and can be related to service limit states or to ultimate limit states.

In any case, all measures of robustness as presented in Deliverable ML-D1.2 provide a relative measure of robustness that may help on evaluating which structure is more robust than another. However, they do not provide a guide on how the inherent structural robustness may be taken into account when facing an assessment of an existing structure and, even more important, which is the required minimum level of robustness and redundancy.

Advanced assessment Codes and/or Guidelines should include clear definitions of redundancy and robustness and quantifiable measures as well as criteria in order to enable the engineer to make appropriate decisions regarding the probability of collapse of the bridge being evaluated.

In order to evaluate the performance of bridges subjected to extreme events and unforeseen hazards, the safety assessment of bridges is evolving from a member-oriented approach to a structural system approach where the interaction of the members is paramount for ensuring the survivability of bridges. A systematic approach for evaluating the reliability of a bridge system would require as input probabilistic models of the hazard, the assessment of the vulnerability of the bridge members, the ability of the system to survive the direct damage induced by the applied hazard and the capacity of the damaged system to continue to carry some traffic load. Ideally, a comprehensive risk-based approach must be taken in order to assess the bridge safety by accounting for the consequences of exceeding the required performance criteria. However, many of the elements necessary to execute such a comprehensive approach cannot

be assembled in a quantitative format that is amenable to an analytical evaluation of risk and reliability. For this reason, recent research has focused on establishing methods and criteria for the evaluation of the robustness of bridge systems which is independent of the type of structural hazard that initiates the damage process. Structural robustness is then used as a characteristic of the structural system that is relevant to different extreme events and hazard types. In fact according to Eq. (2.3) improving the performance of a bridge system that shows low levels of structural reliability, or high P(C) values, can be achieved by one of three ways:

1) Reducing its exposure to the relevant hazard and lowering P(H), For example this can be achieved by placing barriers around critical bridge members or columns to protect them from potential impacts or reducing the access for potential malicious activities.
2) Reducing the vulnerability of the bridge members to particular hazards by reducing P(D/H). For example, this can be achieved by wrapping members with steel or FRP jackets to reduce their vulnerability to direct impact and increasing their ductility to improve their capability of withstanding seismic motions.
3) Enhancing the robustness of the system. This can be achieved by adding members to change the configuration of the system and ensure the presence of alternate load paths. Other approaches to improving the robustness may consist of providing adequate mechanisms for load transfer through improving the ductility of bridge members and providing adequate detailing and connections.

As mentioned above, current research is focusing on developing methods for assessing the robustness of bridge systems and improving the bridge design and safety evaluation process by accounting for their structural robustness properties. This focus is justified because structural robustness is a property of the structural system that is appropriate for different types of hazards. The goal of current efforts is to provide engineers with the necessary tools for the consideration of bridge system safety and structural robustness in bridge engineering practice. Because probabilistic methods cannot be used for the practical assessment of bridge safety on a regular basis, researchers and code writers have developed deterministic methods of analysis which are calibrated to lead to similar conclusions concerning a bridge's safety. In either case, whether probabilistic or deterministic methods are being used, a necessary step for considering structural robustness is to define appropriate non-subjective measures of robustness and develop acceptance criteria.

In Cavaco et al. (2013 b), it is explained how most of the proposed measures of robustness are relative, in the sense that they may help identify which structure is more or less robust than another. However, a target or threshold value that defines the border between what is robust or not, does not exist. However, when dealing with life-cycle assessment and looking at the best maintenance/repair option for a group of assets, this is not a problem, since optimal decisions can be taken just on a relative basis.

Although Eq. (4.3) is presented using a probabilistic formulation, simplified deterministic calibrated methods should also be provided to allow engineers who are not trained in probabilistic methods, to perform direct evaluations using widely available tools. Alternatively, the code could help engineers by providing guidelines that will help engineers determine the level of robustness without the need to perform special analyses. Thus, the alternative methods can be divided into: a) application of system factors to design and evaluation equations; b) Direct deterministic analysis method and c) probabilistic analysis methods.

System factors: This is the easiest and least time-consuming approach, it allows the practicing engineer to evaluate the robustness of the structure by means of characterizing it among a set of standard topologies covered by the code for bridges with well-known behavior.

The prescriptions of detailing, local resistance and other influencing parameters should follow the identification of the structure in the code. The final output is a system factor that account for inherent robustness. The system factor is then used in the assessment equation at member level. Example of this procedure can be found in AAHSTO LFR (2003) and Casas et al. (2012).

Deterministic direct analysis method: When the structural topology is not covered by the code or a higher level of refinement is required, a numerical analysis should be carried out. Although this analysis should be as simple and fast as possible it must include post elastic behavior of the materials and be able to describe properly the overall response of the original and damaged structure. An example on how the robustness can be evaluated is presented in appendices A.3 and A.4. An example of application of the inherent robustness to the advanced assessment of a railway bridge, using a deterministic approach, can be found in Wisniewski et al. (2006). The same example with a simplified probabilistic approach is presented in Wisniewski et al. (2009). Other simplified methods to consider the system behavior and redundancy of existing bridges in an advanced assessment are also presented in SB-LRA (2008), SB4.4.1 (2007), Wisniewski et al. (2009) and Wisniewski et al. (2013).

Reliability based method: Reliability criteria should be provided by the code as the basis for performing the analysis. The level of complexity is the highest, including the simulation by numerical methods and the consideration of uncertainties in the structural parameters. Examples can be found in SB-LRA (2008).

At the present moment, only 2 bridge Codes (Canada and USA) provide practical and clear indications on how to take into account the redundancy effects into the assessment process. In the first case, it is done by the definition of a specific target reliability index; in the second, the concept of system factor is used. This second approach seems the most effective for a practical use on an advanced assessment process. This was the approach also considered in the SUSTAINABLE BRIDGES project. In the deliverable SB-LRA (2008) are presented the guidelines and the method to follow for the consideration of system performance and redundancy to a specific bridge assessment, based on the concept of the redundancy factor (Deterministic direct analysis method). Practical applications are presented in Wisniewski et al. 2006, Wisniewski et al. 2009, showing how taking into account their redundancy and robustness characteristics, bridges that would be condemned to a strengthening or replacement could still be assessed as safe and therefore kept in service without further actions.

Future generations of bridge codes should allow a multi-level analysis of bridge redundancy and robustness so that simple tools such as system factors can be used for less important structures and more advanced tools such as nonlinear analyses or reliability analyses are used for critical structures. These analyses should be based on objective measures of redundancy and robustness and the acceptability criteria that the codes should set must be based on providing uniform levels of risk. Achieving this goal requires considerable effort by joint teams of researchers, bridge engineers, code writers and bridge owners in order to develop models for the quantification of the consequences of failure and for setting risk-tolerance benchmarks.

4.1.4 Site-specific live loads (WIM) and dynamic amplification factors

The loading to which bridges are subject is known to be an area in which significant savings may be made due to necessary conservatism of bridge loading standards that are broadly applied. Using measured data, the load effect, or effects, on a particular bridge, or a range of bridges, can be estimated with confidence.

Knowledge of the current loadings to which railway bridges are subjected is imperative for accurate bridge evaluation. It has been over 20 years since thorough measurements of dynamic wheel loads have been taken in the field. Loading spectra describe the most probable range of loading for a type of freight and are important for fatigue evaluation.

The live loads and dynamic amplification factors in the design codes are given for the design of new structures and can therefore be very conservative in some circumstances leading to structures failing their assessments. Consequently, it is often beneficial to use site-specific live loads and dynamic amplification factors when assessing existing railway bridges.

To collect information on the site-specific live loads, Weigh-in-Motion (WIM) systems are commonly used. WIM is the process of converting an instrumented track or bridge into a scale for weighing passing trains. Such systems provide information on:

- Static and dynamic axle loads (or bogie loads depending on type of system)
- Axle distances (or bogie distances depending on type of system)
- The speed of the train
- The direction of the train

In chapter 4 of the Sustainable Bridges background document SB4.3.2, (2007) and Gonzalez (2011), a method is presented by which one is able to determine site specific characteristic train loads from “Bridge Weigh in Motion ((BWIM)) measurements.

A typical output of a WIM system is presented in Figure 4.2, showing a typical histogram of axle load obtained using BWIM in a railway bridge in Sweden.

In many cases, the assessment of bridge al Ultimate Limit States (ULS) is of interest. To this end, an accurate estimate of the maximum traffic effect within a predefined period of time is necessary. Based on the site-specific train loads obtained using WIM techniques as in figure 4.2, a simple but enough accurate method is presented in Deliverable ML-D1.2 to derive such maximum effects. The method is based on a methodology developed for the case of highway bridges but that could be adapted to the case of railway bridges, see Gonzalez (2014). In the case of railway infrastructure, because the probability of trains passing on bridges with more than 1 track is very low, even when the volume of train runs is high, the one-train load governs. Eurocode for actions in railway bridges allows the reduction of maximum train load when the bridge presents 3 or more tracks. Also, the BWIM data can be used to calculate the probability of two trains on a 2 track bridge. This probability will be highly dependent on the period of time considered. Of course, at ultimate the bridge must be capable to carry the weight of the train on every track. The question is: Which weight? The characteristic load in the code? The average weight? It will probably do with the average weight but here a probabilistic assessment will be able to predict a sufficient safety without causing extra high costs.

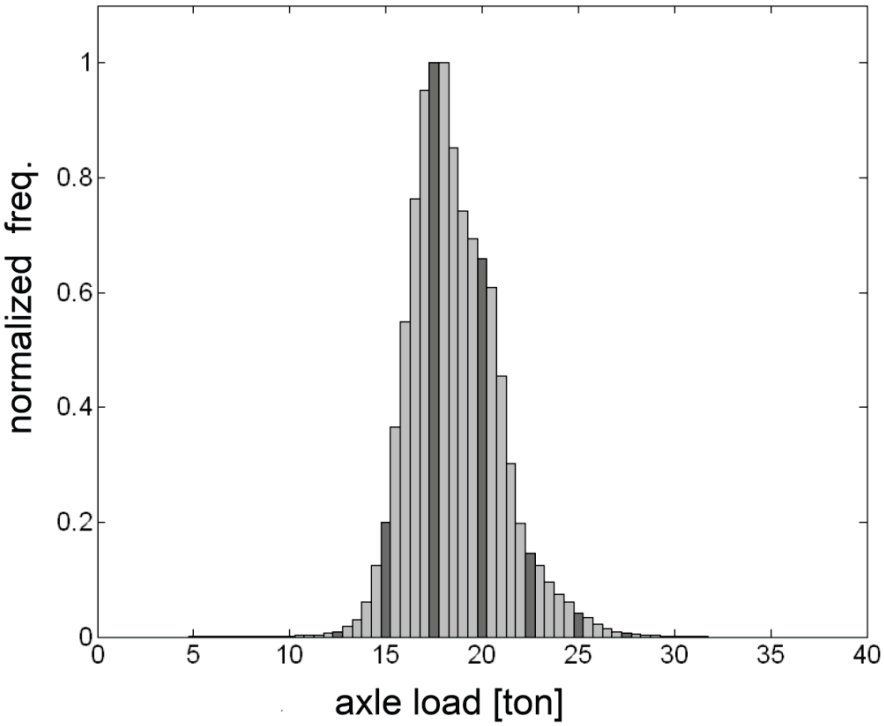


Figure 4-2 Histogram of the axle loads, obtained from over 7500 train passages (Gonzalez 2011

In the SR-2.2 (2014) are presented other topics related to railway loading and alternative methods to use of WIM data to define maximum loading scenarios.

The dynamic amplification factors (DAF) prescribed in design/assessment codes are sometimes based on theoretical analysis and/or dynamic load tests of existing bridges and tend to be conservative. There is considerable discrepancy among the values recommended by different codes due to the complexity of the Vehicle Bridge Interaction (VBI) problem. They typically suggest a dynamic amplification which is function only of a few general parameters that ignore many of these significant bridge and truck dynamic characteristics. Thus, the DAF values are conservative and they produce maximum dynamic effects that might not necessarily correspond to the maximum static effects. This level of conservatism could be acceptable for new construction due to the low marginal cost of adding capacity and uncertainty about future traffic loading growth. However, more accurate assessment of the capacity of existing structures may prevent needless expense in bridge rehabilitation.

By measuring displacements or strains, site-specific dynamic amplification factors (DAF) can be calculated. This can be done either from (a) tests with trains crossing at normal running speeds and tests with the same trains crossing at a crawling speed (typically less that 10 km/h) or (b) by filtering the measured signal to separate the static from the dynamic content. The latter method is more delicate and requires more experience in signal analysis. Moreover, a total separation of the dynamic response can be hard to achieve by filtering on some bridges. Consequently, this method is only recommended when it is observed from measurements that the dynamic response constitutes oscillations around the static response. The DAF is then calculated by dividing the maximum dynamic response with the maximum static response.

The dynamic behaviour of the bridge under traffic loads consists in absorption, storage, dissipation and release of energy that is stored in the structure due to dynamic traffic action. For elastic bridge behaviour, this energy stored in the bridge element consists in vibrations leading to increase of deflections and internal forces. Elastic bridge behaviour is considered for fatigue and service limit state. However, for the ultimate limit state, elastic-plastic structural behaviour must be accounted for, and formulas like those commonly given in design codes are then fundamentally wrong since they refer to elastic structural behaviour only (Brühwiler & Herwig, 2008). The use of the elastic formulas is mostly on the safe side but for an accurate assessment the elastic-plastic behaviour must be taken into account.

4.1.5 Model updating. Incorporation of data from inspection and monitoring

Up-dating of models for resistance, loading and structural response to get more accurate theoretical models in the analysis is also considered as a new technology in an advanced assessment.

Model updating can be also carried out via diagnostic load testing. This type of test provides useful information when structural models including finite element methods can not accurately predict the behaviour due to uncertainties in member properties, boundary conditions and influence of secondary members. As an example, in Olaszek et al. (2013) the results of 3 diagnostic load tests in different bridge structures are presented, emphasizing their diagnostic potential for assessment. In the cases presented, it is shown how the experimental results differ considerably from the expected ones and as a result, up-dated models are obtained based on the results of the tests.

The goal of structural monitoring is to gain knowledge of the integrity of in-service structures on a continuous real-time basis. Scheduled maintenance and periodic inspections offer only limited knowledge of structural condition, and these methods are costly in terms of extensive labour and downtime. However, advances in sensing technologies, material and structural damage characterization, and monitoring diagnostic technologies enable the integration of distributed sensors for real-time inspection and damage detection. Thus, the essence of structural health monitoring is the development of autonomous systems for the continuous monitoring, inspection and damage detection of structures with minimum labour involvement.

In this sense, two emerging measuring technologies are of main interest in bridges: acoustic emission and distributed optical fibre. Recent experimental studies have shown how intensity analysis of hits of acoustic emission can detect and quantify the current condition of the bridge as well as taking into account the degree of existing damage prior to conducting a load test. Also the b-value analysis can provide an early warning of damage accumulation. Finally, AE source triangulation enables the location and pattern of cracks (ElBatanouny et al. 2014). Another technique that is able to detect cracking before being visible and to locate the cracks and their width is the distributed optical fiber system OBR (Optical Backscattered Reflectometer). The technique has the feasibility of measuring strain and temperature continuously along the fiber, therefore detecting the crack when it appears in the concrete element. The bonding of a continuous fiber along the element allows the detection of a crack wherever it appears (Villalba and Casas 2013, Rodríguez et al. 2014). The technique of Smart film with crisscross enameled copper wires glued on the surface of a concrete structure can also monitor the initiation time, length propagation, shape and location of cracks (Zhang et al. 2014).

But the monitoring systems and methods are just a tool to assist the advanced assessment of the bridge. What is of interest is how the results gained from the monitoring can be incorporated into a bridge assessment framework and translated into reliable answers about the bridge condition and performance in the future.

One challenge in dealing with SHM has been the processing of the large amount of data extracted and interpreting it into meaningful information that can be used for decision making. This is also pointed out in ML-D4.2 (2014). Accordingly, SHM has enjoyed significant research efforts aimed at the development of algorithms and methodologies for system identification, damage detection, and updating of finite element models. One of the main problems related to the use of SHM data in the damage detection is to find out a real-time strategy to conduct structural assessment without the need to define a baseline period in which the monitored structure is assumed healthy and unchanged. This can be achieved by means of machine-learning algorithms know as cluster analysis. They are able to find groups in data relying only in its intrinsic features (Sohn and Kim 2008). A change in the structural response, due to a possible damage in the structure, is reflected in these intrinsic features and derives on the aggrupation of data from the SHM in clusters different from the previous ones. This derives in an on-line and real-time damage detection.

Only recently, research that treats SHM under uncertainty has emerged. However, these studies have mainly focused on information related to the load effects that SHM provides. Okasha and Frangopol (2012) have proposed an approach in which the SHM information can be, in fact, used to update the structural parameters of the structure that are in turn used in updating the lifetime reliability of the structure. The information provided by monitoring a bridge can be used for updating the PDF (Probability distribution function) of its time to failure through a Bayesian process.

Ultimately, optimal decisions are needed for ensuring the continuous safety of structural systems under multiple objectives, uncertainty and constraints and only a proper integrated framework would yield such decisions.

For a given complicated bridge structure, the first step is to build a Finite Element (FE) model of its system and components. These models can be used for at least two purposes:

(a) To perform the life-cycle performance (reliability) analysis. Even though, performance indicators can be developed without using FE analysis (FEA).

(b) To be used with the SHM to update the resistance parameters. Traditionally, SHM data has been used to update FE models of structures for the purpose of obtaining a FE model that captures the performance of the structure more accurately.

A well-known way to incorporate SHM data into a life-cycle assessment is by means of Bayesian updating techniques. However, judgment based only on SHM data obtained over a period of time may lack information on events that are encountered outside this period and, therefore, it is crucial to combine SHM with prior estimates of these quantities. The classical estimation approach treats the parameters of the PDF (probability distribution function) deterministically and makes not possible to incorporate prior information of the uncertain variable. Instead, the Bayesian estimation approach treats the parameters as random variables and, in this way, makes it possible to user prior knowledge.

A Bayesian updating framework (SB-LRA, 2008) can be used to consider both the original and posterior sources of uncertainty in a consistent manner, resulting in a more reliable indication of the actual properties of the materials, improved estimation of the resistance of the bridge

members, and a more accurate understanding of the stress and load distribution throughout the structure.

The treatment of SHM data is usually associated with monitoring of extreme events (i.e. load effects of very heavy trains). The literature of papers linking the themes of Bayesian updating and extreme value modelling is sparse, in part due to computational difficulties, some of which have recently been overcome by techniques such as Markov chain Monte Carlo (McMC) (Bocchini, et al., 2013). Extreme value distributions do not lend themselves easily to Bayesian updating; the main problem is that there is no conjugate distribution. For example, the Weibull distribution or the Gumbel distribution are the most used extreme values distributions for model the traffic extreme events. The Bayesian-updating of extreme value distributions cannot lead to explicit posterior distributions. Hence, a simulation procedure is the best way to determine the posterior distribution. The Metropolis-Hastings (MH) algorithm has been suggested for this purpose.

4.1.6 Proof load testing

The evaluation of structures requires information related to its properties and real boundary conditions. This information cannot always be known with the desired accuracy, especially in existing structures, among other reasons because of incomplete documentation, unknown effects due to deterioration and uncertainty in the modelling of the structure. In these cases, the information can be obtained by non-destructive testing or partially destructive of the constituent materials and accurate measurements of the geometry of the existing structure. Even in extreme cases the structural safety cannot be determined by analytical means, so that in certain cases it is beneficial to determine the structural safety through the execution of a load test in situ.

The non-destructive load tests on real structures are considered as one of the best methods to assess the carrying capacity, because the concept is intuitively acceptable and the results are conclusive.

The load tests are an alternative or a complement to the assessment of bridges through analytical calculations. A proof load test evaluates the ability of the bridge to support its own weight and dead load, plus a certain overload. In order to obtain an adequate safety margin, against a possible overload, the bridge must be tested by the imposition of a greater load than the expected service load.

In many cases, the cost of replacement of a structure can be avoided by testing the safety of the structure through a load test and, additionally, based on the results, the maintenance and repair of the structure can be more efficiently planned.

It is appropriate to carry out a proof load test if:

- The analytical methods produce an unsatisfactory load value of service, or
- The analytical method is difficult to perform due to the deterioration of the structure or the deficiency or absence of the necessary information for its application (drawings, material properties, etc.).

It is worth noting that a number of countries (e.g. UK) specifically recommend that proof load testing (i.e. load testing to the design or assessment load or above) should not be undertaken. This is because if the test is not properly performed and controlled, due to the high level of load in the bridge, a possible cracking or damage could be produced. For this reason, proof load is limited by the elastic behaviour of the materials, never going beyond this limit.

For masonry arch bridges proof loading may be dangerous as we currently have few ways of measuring any parameters (however, acoustic emission is one) that give meaningful information about capacity or when the elastic range may be exceeded nor are there many monitoring techniques that can warn that damage is beginning to occur. There is also a big difference between “load testing” and “proof load testing”. Load testing using service loading is an old standard technique which is accepted and used quite often. .Proof load is designed in the way that imposing a certain level of load in the bridge means that a certain lower load level can be carried out with a pre-defined level of safety. Because the high failure load of masonry arch bridges that means that for these levels of load the bridge does not show any sign of distress or damage. Nonetheless, using loads above the design load is not allowed on masonry bridge by NR. However, other countries as Germany have used proof load testing in railway masonry arch bridges with success.

During the performance of the load test it is necessary to take several precautions, such as good planning, gradual implementation of the test load and control of the responses of the structure in order not to exceed the elastic limit of the material.

In general, load tests are performed to evaluate the load capacity of existing bridges, whose test loads are the maximum that the bridge can withstand without damage. Nevertheless the load proof does not necessarily have to be the top resisted by the bridge, but it must be of such magnitude that allows classifying the bridge as safe.

Since bridges are subjected to very high loads during the proof load tests, there is always the possibility that the bridge can permanently be damaged. However, this scenario is extremely unlikely. This can be controlled by a proper monitoring, using for instance acoustic emission among other techniques. In fact, acoustic emission has the advantage when compared to other measuring techniques that it may anticipate any malfunction or damage of the bridge, by measuring a warning increment of the number of hints or a decrease on the parameter “b” in a b-value analysis (ElBatanouny et al. 2014). It is observed that the b-value reaches its minimum near the peak load and reaches maximum during micro cracking while it tends to decrease when micro cracks coalesce and start forming major and visible cracks (Vidya-Sagar et al. 2012). A complete description of the application of AE to an existing real bridge is presented in ARCHES-D16 (2014). It is shown there how the AE monitoring is an excellent tool to follow the response of the bridge to the increasing load and provides a robust criteria to stop the loading process (Olaszek et al. 2010).

The use of piezoceramic transducers (Song et al. 2007) has been proved also as a suitable monitoring to detect the existence of cracks and their growth in concrete, much earlier than other monitoring techniques (LVDT, visual inspection).

Despite some countries specifically do not allow to use proof load tests, other countries recognize them as alternative and complementary assessment tools. For instance in Canada, CAN/CSA-S6-06 gives guidance on bridge load testing. In general two types of tests are considered, static and dynamic. The static load tests are meant to capture the overall structural behaviour of more complex bridges and confirm the load distribution between their components and members. The dynamic load tests are performed to establish dynamic characteristics and

behaviour of bridge structures. In particular, the dynamic load tests might be used to determine bridge specific dynamic amplification factors, which can be further considered in the load carrying capacity evaluation. Furthermore, in CAN/CSA-S6-06, proof loading is considered as an alternative method of load rating for bridges which cannot be accurately evaluated by analysis, or if the structural response of a bridge to live loads is questionable. A condition inspection and preliminary load rating of critical components in the bridge must be carried out prior to any load test. If during the test, the measured change in bridge behaviour is indicative of nonlinear load-deformations and inelastic strains, then the test must be immediately stopped and the maximum applied load may be assumed to be representative of the ultimate bridge capacity. When there is no apparent damage or change in the bridge behaviour during the test, but the test is stopped due to limitation of the test equipment, then the bridge capacity may be determined by an extrapolation of the test results. The extrapolation is based on comparing estimated initial strains due to the dead load and an acceptable level of maximum strain.

In the USA, the AASHTO LRFR (2003) also provides a reliability-based method for including the results of proof load testing during the safety evaluation of bridges and encourages the direct use of reliability methods in the rating of special bridges as compared to using the specified reliability-calibrated load factors.

The Code DAfSTb-Richtlinie (DAfStb, 2006-09) issued by the German Committee on reinforced concrete provides guidance on the proof-loading testing of reinforced concrete buildings.

Proof-load testing is used to update the information about the condition and actual capacity of the bridge based on the fact that the bridge has survived an external load that is perfectly known. However, many bridges that have never seen a load test are perfectly operating under normal traffic. Surviving a service load history that is stochastic in nature provides evidence of strength that may be comparable to what might be learned from a proof load test. This is what is called as service-proven bridge. A proof load test enables the lower tail of the resistance distribution to be truncated at the level of the maximum load carried (figure 4.3). For a service-proven bridge, the magnitude of the maximum load carried is un-known, however, it can be determined statistically by using weigh-in-motion data as presented in Wang et al. 2011.

Full scale tests to failure of old bridges, which are to be taken out of service, is another possibility. It can give valuable results regarding the capacity of the kind of bridges that is tested. An example of this is given in Appendix A where the test of a steel truss bridge is presented. A typical concrete railway trough bridge was earlier successfully tested in the project Sustainable Bridges SB-D7.3 (2008), Puurula et al (2012, 2014). Recently also a prestressed bridge was tested to failure, Bagge et al (2014). In all cases it was shown that the bridges had a much higher load-carrying capacity than what was predicted by ordinary code methods.

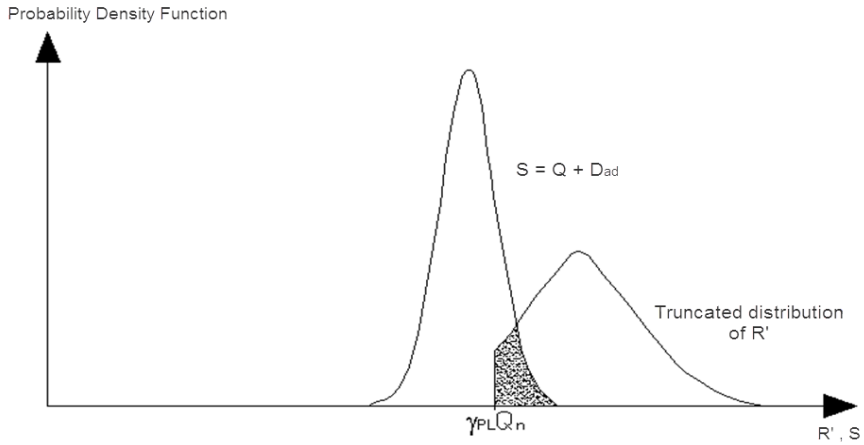


Figure 4-3 Truncation of PDF of Resistance as a result of proof load test

4.2 FEM models

An accurate and advanced assessment requires the most accurate response models to be used. The finite element method (FEM) has become widely used for design and assessment of bridges. Finite element (FE) analyses provide the possibility for more accurate studies of the structures than what is possible with more traditional methods used for structural assessment. In such an analysis, the three-dimensional geometry of the structure and its non-linear response due to e.g. material plasticity, cracking and second order effects can be taken into account.

Assessment of structural safety and functionality of existing bridges is a step-level procedure, as described in SB-LRA(2007). FE analysis can be helpful at intermediate level assessment for structural (system) analysis, in combination with resistance models. In bridge design, 3D linear FE analysis corresponding to this level is commonly used. However, FE analysis is particularly useful for advanced assessment. Here, non-linear analysis is usually needed. This requires skilled and experienced structural engineers and is considerably more time consuming. On the other hand, a non-linear (FE) analysis is considered to have the highest potential for discovering any additional sources for load carrying capacity in reinforced concrete railway bridges, SB-LRA(2007).

The use of FEM for analysis of building and civil engineering structures is treated in several text- and handbooks, e.g. Crisfield (1991), Rugarli (2010) and Blaauwendraad (2010). Design and analysis of concrete structures are treated in e.g. Fib (2008) and Rombach (2004). In Sustainable Bridges (2007), SB-D-4.7(2007), assessment of railway concrete bridges with non-linear FE analysis is treated. Guidelines for non-linear finite element analysis of concrete structures are provided for girder members in Rijkswaterstaat (2012) and for modelling of shear and torsion in bridges in Broo (2008).

Structural idealisation

The most important step when assessing an existing structure using FEM is the definition of the structural model and its properties. The purpose of the structural analysis is to model the behaviour of the structure to a sufficient level of accuracy. Consequently, it is desirable and

necessary to make simplifications in the structural model. The geometry, material and loading, as well as the extent of the structure and its boundaries may need to be idealised. Different structural models reflect different aspects of the structural behaviour differently well, and sometimes several models with different levels of detailing are needed. Often, structural models for assessment need to be different from models used in design.

The purpose with the analysis decides how the structural model should be defined and what idealisations that can be made. There are many aspects to consider when setting up a structural model, such as:

- What should the model be capable of describing?
- What structural phenomena should be reflected? (Is it static or dynamic? Does it involve large deformations, non-linear or time dependent material response, etc.?)
- Can the problem be simplified by using beam or plate theory, or a two-dimensional stress state?
- What is the extent of the model and how should the boundaries be defined?
- How should the material response be idealised and what material parameters should be used?
- How should interaction between different parts or materials be modelled?
- How can important details, such as supports, connections and stiffened areas be simplified?
- How should the actions on the structure be applied?
- What element types and FE mesh density is needed?
- What solution method should be used?

Verification and evaluation of FE analysis

The verification of the FE model and analysis results is a very important step. Since advanced FE analyses are complex with many possible error sources, a rigorous quality control is needed. The structural engineer can usually rely on the fact that the FE program with its elements, material models etc. has been verified by the programme developer. However, the modelling method, i.e. the possibility to reflect the desired response with a specific structural idealisation, combination of FE elements and solution method, is usually up to the structural engineer. The developer can provide examples, but often it is necessary to go to the literature, and comparison to tests may be needed. To confirm that the actual FE model is correct needs always to be checked by the user, and comparisons to simplified calculations should always be made.

An FE analysis usually provides large amounts of results. Consequently, the results need to be post-processed and interpreted bearing the structural idealisations made in mind. It is of great importance that the structural engineer has a thorough understanding of the structural response in the model as well as the behaviour of the real structure.

Linear FE analysis

In design, FEM is commonly used for structural analysis to determine the action effects, normally in terms of sectional forces and moments. These action effects are then used in combination with local resistance models to design each cross-section or structural element. In order to simplify the analysis and to be able to use the superposition principle for evaluating the effect of load combinations, linear analysis is generally adopted. Guidelines for linear FE analysis of concrete structures can be found in Fib (2008), Rombach (2004), and for plate structures in Blaauwendraad (2010). The question of how to redistribute sectional forces and moments for design of reinforcement in concrete slabs, and how this is connected to the structural idealization and FE modelling is treated in Pacoste *et al.* (2012).

In reality, most bridge structures have a non-linear response under loading up to failure; concrete bridges normally have a pronounced non-linear response already for service loads due to cracking. In ultimate limit states, the use of linear analysis can normally be justified since the structures have good plastic deformability. Theoretically, the design is based on the lower bound theorem of plasticity. Consequently, since the design is based on a force (and moment) distribution that fulfils equilibrium, the load carrying capacity will be sufficient when the structure has sufficient plastic deformation capacity. In serviceability limit states, the use of linear analysis for concrete structures is based on the assumption that redistribution of the action effects due to cracking is limited.

In the design situation, the linear FE analysis is used to choose a force (and moment) distribution that fulfils equilibrium, and to design the structure to resist this action effects. For example, the reinforcement in a concrete slab is designed to resist the moments from a linear analysis of the slab.

In the assessment situation, the structure is already designed. The original bridge design was most likely not based on detailed 3D linear analysis, but rather made with simplified analysis methods typical for the time of the design. This means that the bridge will most likely have larger capacities in some parts or sections and smaller in others, compared to the design based on linear elasticity. Both designs may fulfil equilibrium, but can have different requirement on the plastic deformation capacity; due to the non-linear structural response, it is not necessary that the linear analysis require smaller plastic deformations.

This means that there is a big risk of underestimating the capacity of the bridge when using linear FE analysis for assessment in the same way as it is used for design. The critical section, limiting the capacity according to the linear analysis, may not be as critical in reality due to the non-linear response. If linear FE analysis is used for assessment, redistributions from the linear force distributions must be allowed, and conservative estimations of redistribution widths according to e.g. Pacoste *et al.* (2012) should be avoided. Comparison to other structural models must be included and knowledge of the structural model used in original design is very desirable.

Non-linear FE analysis

In a non-linear FE analysis the structural response is simulated in a more realistic way, with the possibility to take the material non-linearity into account e.g. due to steel yielding and concrete cracking or non-linear geometric effects. The stress redistribution in statically indeterminate structures is reflected, providing a more correct distribution of the load effects. Moreover, since a realistic material response is included, the resistance of the structure is determined by failure occurring in the overall structural analysis. This way, intermediate results in terms of cross-sectional forces and moments are not needed, avoiding the inconsistency of combining linear system analysis for determination of action effects with non-linear local analysis for determination of resistance. Non-linear FE analysis can be used for all types of bridges and for service limit state SLS as well as for ultimate limit state ULS.

In the Eurocodes, EN 1992-1-1 (2004), non-linear analysis is specified as the most advanced level of structural analysis, and general guidelines are given for such analysis. In research, non-linear FE analysis is long since regularly used to obtain a better understanding of structural behaviour. However, non-linear analysis is still not often used in engineering practice. It is more time consuming than simplified analysis methods and the demands on expertise of the structural engineers performing the analyses are higher. On the other hand, in many cases it has shown great potential in revealing additional load carrying capacity and to provide great cost savings in assessment of existing bridges, Plos (2002), SB-D4.5(2007) and Broo et al. (2009).

Non-linear FE models can be made with different levels of detailing, leading to analyses reflecting the structural response and resistance to different levels of accuracy:

- For structural analysis of entire bridges, models built up of “structural” finite elements like beam and shell elements are often useful. Models on this level of detailing are well established and verified to reflect the response due to bending and normal forces in a good way; they are commonly used by researchers and engineers and the results are regarded as reliable. However, failure due to e.g. shear, anchorage or local buckling must be checked with separate resistance models.
- For analysis of structural members or smaller structures, more detailed FE models can be used. Here, shell elements can be used to build up sections consisting of parts with plate response, while continuum elements are needed in case of solid sections. In some cases, the model can be simplified to two dimensions assuming plane stress or plane strain. On this level of detailing, also failures due to e.g. local buckling in steel girders and shear failure in reinforced concrete members can be reflected.
- To reflect e.g. anchorage or detailed crack pattern in reinforced concrete members or details, the bond between the reinforcement and surrounding concrete must be included. A predefined bond-slip relation according to fib (2013) can be used to model the interaction along the reinforcement bars. To include the splitting effect and reflecting the influence of confinement and reinforcement yielding on the bond properties, a 3D bond model according to e.g. Lundgren (1999) is needed.

In case all possible failure modes are not reflected by the non-linear analysis, on the chosen level of detailing, these failure modes need to be checked by separate resistance models according to e.g. Eurocodes. The fib Model Code for Concrete Structures 2010, fib (2013), provides resistance models on different levels of detailing. This enables more sophisticated resistance models for e.g. shear and punching to be used together with the enhanced non-linear system analysis.

Non-linear FE analyses are used in a more enhanced level of assessment. On this level testing and monitoring can provide relevant input data concerning actual loads, in-situ material properties and actual behaviour of the bridge, and the bridge model can be updated based on on-site measurements, Schlune *et al.* (2008), ASCE (2011).

Structural assessment of ULS capacity based on non-linear FE analysis requires special considerations regarding the safety format. In a non-linear analysis reflecting both the overall stress distribution and local failure in the same model, the system and resistance analysis become integrated. The structural analysis becomes similar to an experimental test where the structure is subjected to increased loads until failure is reached. The verification of the structural safety is here made on the global level rather than by comparing load effects and resistance locally. The fib Model Code, fib (2013), provides safety formats for non-linear analysis, ranging from probabilistic method, over global resistance methods to the partial factor method. However, the model uncertainty given for complicated to model failure modes, like e.g. shear failure, may be un-conservative, Schlune (2011).

4.3 Temperature effects

Besides the correct assessment of variable-in-time loads as the live load or traffic load, in some cases, the accurate assessment of temperature effects is of interest, mainly for statically non-determined bridges. This is particularly relevant in the case of concrete bridges and for the bridge assessment versus the serviceability limit states (cracking, deflection, vibration). In fact,

the cracking due to temperature effects may produce a reduction on the sectional stiffness leading to possible adverse effects. The temperature effect is less relevant in the case of assessment versus ultimate limit states, except in the case of fatigue assessment. This is due to the relaxation of thermal stresses when releasing some of the imposed boundary conditions or decreasing the stiffness that occur when increasing the external load level. At the limit, when the bridge is close to failure, it becomes a mechanism, and therefore, the imposed strains due to temperature effects, do not produce internal forces in the bridge elements.

4.3.1 Influence of thermal actions

Due to variations in the surrounding environment, the temperature in a bridge structure will vary in both time and space. The most influential factors governing the temperature variation in a bridge are the ambient air temperature, solar radiation influx, wind speed and amount of clouds in the sky. The variations in ambient temperature during a year mainly govern the average temperature in the bridge, while the other factors have a larger impact concerning more short-term temperature variations.

A varying temperature in a bridge will produce movements with the magnitude of the movements depending on the material and geometry of the bridge. If the movements are restrained, strains and stresses may be induced, which can affect the safety and performance of the structure. Cracks and other problems may occur which may influence the structural behavior and the durability of the bridge.

When designing or assessing a bridge for thermal effects, the temperature distribution has historically been considered in similar ways in Europe (CEN 1996). The distribution can be divided into parts according to Fig. 4.4; an average temperature, horizontal and vertical temperature gradients/differentials, and a non-linear residual temperature gradient/differential.

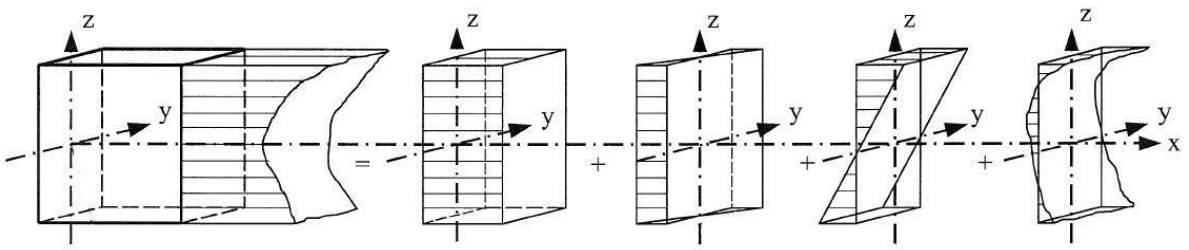


Figure 4-4 The components of a temperature distribution (EN 1991-1-5 2003)

The average temperature component, which governs the longitudinal movements, has been coupled to the ambient air temperature in most national codes (CEN 1996). In the design process the average temperature is determined depending on the geographical location of the bridge. The main differences between the various national building codes that preceded the Eurocode lie in the application of the temperature differentials. The differences in design approach between countries in Europe could be a considerable factor for the assessment of existing bridges, since the use of various differentials may give differences in the structural response.

In several national codes such as the German, Swedish, Danish and Italian codes that preceded the Eurocode, only a linear temperature differential was described and used for

design. The design values of thermal actions covered in the German code DIN 1072 was for different bridge types and construction materials, average temperature variations, linear temperature differential, difference in temperature between different bridge members and movements of the bearings due to temperature variations (CEN 1996). The non-linear temperature component was not considered, since these effects are strongly dependent of the detailing of the bridge and was therefore deemed more important to be included in the detailing rules.

In other countries such as the United Kingdom, the temperature differential parts of the distribution were considered in a slightly different fashion (Emerson 1973). The linear part was combined with some of the non-linear part, which gave a design distribution more similar to the real temperature distribution. In Spain, the preceding national code included temperature differentials which varied depending on the location and type of bridge by the use of calibration coefficients. The British and Spanish national codes had the highest level of detail for the national codes described here.

In the Eurocode (EC 1-5 2003), the temperature differential may be considered in two different ways. The first approach is based on the linear differential, similar to the method described in the German national code. The design values were calculated from long-term climate data and measured temperatures in a bridge in Germany (Soukhov 1994, CEN 1996). The data was analyzed using extreme value theory, and the characteristic values were established. No geographical variations exist for the design values, although various studies have shown that such effects may be found (Larsson and Thelandersson 2011, Lucas et al 2003).

The second approach for temperature differentials that can be used for design is based on the method in the British national code. The temperature differential is described with a multiple straight line, with different values depending on bridge type and used construction materials. A similar approach is also used in the American code (AASTHO 1989).

Temperature differences between different parts and members in a bridge may also give large effects. In the Eurocode (EC 1-5 2003) a recommended value of a temperature difference of 15 °C between main structural elements is given. This value should however be used with care in analyses, since it may give very large effects. If applied in a non-suitable manner, the additional stresses/section forces may be overestimated and lead to unnecessary placement of reinforcement and other measures.

4.3.2 Evaluation of thermal actions for concrete bridges

For assessing the temperature influence of existing railway bridges the most common methods are the finite difference method (Emerson 1973) and finite element modelling. In most studies the finite element method has been used for both calculating the temperature distribution and the resulting stress field (Elbadry and Ghali 1983, Sveinson 2004, Larsson 2012).

Most research concerning resulting thermal stresses and sections forces has been focused on the behavior of the bridge in the longitudinal direction. The two major effects from temperature variation are elongation/retraction and curvature. For normal concrete bridges the elongation is considered by the use of expansion joints and/or movable supports. If ignored, the effects of longitudinal elongation could be severe, producing cracks and leading to structural failure.

Curvature occurs in a bridge due to temperature differences in the vertical or the horizontal direction (Elbadry and Ghali 1983). For a simply supported bridge the curvature will lead to

bending of the bridge, internal restraint and self-equilibrating stresses. The curvature itself is related to the linear temperature differential, while the non-linear part of the temperature distribution will produce the self-equilibrating stresses. With different support conditions, e.g. when the bridge has more spans or is not simply supported, the structure will be restrained causing continuity moments and statically indeterminate reactions. These effects have to be added to the self-equilibrating stresses to obtain the total response in the structure. According to Elbadry and Ghali (1986) this can be achieved by the use of the general displacement or force methods.

The restraint causing the large thermal stresses is significantly reduced if cracks are present in the structure (Jokela 1983). The cracks reduce the moment of inertia and thus the stiffness, leading to a reduction of the stresses. This must be considered when using a linear-elastic model for evaluating the effects of thermal actions, since these effects otherwise may be overestimated.

Other important studies of longitudinal effects in concrete bridges have been performed by Prakash Rao (1986) and Thurston et al (1984). Prakash Rao found that differences in temperature between members (walls and slabs) in a bridge could cause severe effects and lead to cracking. It was also found that it is important to consider both positive and negative temperature gradients when assessing temperature effects. Thurston et al. developed a method for calculating thermal stresses in cracked beams and bridges. The background to the development was the intention to use partially prestressed concrete bridges, to allow controlled cracking to take place. The results found in the investigation showed that cracks in critical sections reduced the thermal moments with up to 46 % for a large temperature load consisting of a gradient of 40 °C. This confirmed that cracks in a concrete bridge can reduce the induced restraint and thus reduce the effects of thermal actions. Casas (1983) demonstrated that the temperature effects in the case of prestressed (full prestressed and partially prestressed) concrete bridges built by the balanced cantilever method were almost negligible when carrying out a non-linear analysis up to failure.

Some studies also exist of the bridge behaviour in the transverse direction and at cross-sectional level. One of the first studies was performed by Elbadry and Ghali (1986), where a bridge cross-section was considered to be a plane frame. A non-linear temperature distribution applied to this section gave high enough bending moments to produce tensile stresses of over 4 MPa at the bottom surface of the bridge deck, which would contribute to cracking in the bridge.

A more extensive study on transverse effects in a box-girder bridge was performed and presented by Sveinson (2004). Cross-sections from several positions along a bridge were studied with applied linear and non-linear temperature differentials over the various members in the bridge sections. According to the results significant transverse stresses and bending moments were induced by the temperature differentials when the frame action of the cross-section was considered. A larger effect was observed for temperature differentials applied over the individual members than for temperature differentials applied over the whole cross-section. The total stresses in the cross-sections increased with 25-50 % due to temperature effects when compared to the stresses from only self-weight and traffic loads.

In Larsson and Thelandersson (2012) the effect of thermal actions on a cross-sectional level was investigated with a FE-model, with focus on long-term simulations and geometrical variations. A four-sided concrete section from an arch bridge was studied as reference, with the length, height and member thicknesses varied in the further studies. In this study it was found that the largest stresses in the section occurred on the inside of the thinner members, independent of the location and orientation of the section. A thin top slab and thicker vertical

walls gave the largest effects in the top slab, and thinner walls with thicker horizontal slabs produced large stresses in the walls. This indicated that in order to reduce the impact of thermal stresses, it is preferable to have similar thicknesses in both the horizontal and the vertical members of a box-girder bridge.

Thermal effects have been analyzed by applying fixed temperature differentials obtained from extreme value analysis of long-term situations. In the background document of the Eurocode (CEN 1996) the procedure used for obtaining the design situations is described. Long-term measurements of air temperature, solar radiation and other climatic effects have been used to calculate extreme values for average temperature, linear temperature differentials and quasi-linear temperature differentials described in the previous sections. The design situations have been used in the calculation models to estimate the resulting thermal stresses and section forces. The validity of this approach has been studied by e.g. Emerson (1973), CEB (1985) who found that large thermal effects occur when large temperature differences occur. A drawback with this approach is that it may be difficult to capture all possible situations leading to large effects, since only the worst temperature situations are analyzed and not the worst stress situations.

An attempt to determine which temperature and climate situations who give the most unfavorable situations was presented in Larsson (2012). Input data and results from long-term temperature simulations and corresponding thermal stress simulations were analyzed to find which combinations of climate factors that gave the most unfavorable stresses. It was found that a large influx of solar radiation combined with clear skies and a large difference in air temperature gave large thermal stresses, which was expected. However, another situation was also found, where a rapid increase in temperature during winter time also produced large stresses. This situation could be the cause of cracks due to thermal effects in locations where the effects of solar radiation are limited. The results in Larsson (2012) also showed that instead of using fixed values of air temperature and/or temperature differentials it is possible to use a shorter period of climate data to simulate thermal effects. A short-time period gave similar results to the long-term simulations, and may give more reasonable results than using fixed values in every situation. This fact could be of paramount importance in the assessment of existing bridges.

4.3.3 Evaluation of thermal actions for steel and composite bridges

The effects of thermal actions in steel and composite bridges have not achieved as much focus as thermal effects in concrete bridges. The main reason for this is the more direct effects that can be seen in concrete structures from thermal actions, such as cracks on one side of the bridge only due to solar radiation affects, with also the consequences of thermal effects being less severe for steel bridges.

The most common assessment method concerning thermal effects in steel and composite bridges is the finite element method. The same principles concerning the average temperature and temperature differential as for concrete bridges is applicable for steel and composite bridges, with slightly altered values depending on the difference in material properties. It should how ever be noted that investigations exists where the design values given in e.g. Eurocode is deemed to be underestimated (Lucas et al. 2003)

For a steel bridge the main factor is the longitudinal elongation and retraction. Due to a much higher heat conductivity in steel than in concrete, the effects of changing temperature is more

direct and the effects arise more quickly. If the bridge is restrained from moving in the longitudinal direction, large stresses and forces will occur that may lead to risks for failure is the steel components and/or in other parts of the structure. (CEN 1996)

A major concern for steel structures is the damage from fatigue. Studies of fatigue due to temperature variations in integral abutment steel bridges have been performed during the latest years by Hällmark et al (2010), Dicleli and Albhaisi (2004). Here it was shown that low cycle fatigue from temperature variations could significantly reduce the lifetime of such a bridge. However, it was also shown in the study by Hällmark et al. that the risk for failure due to low-cycle fatigue is quite low for bridges with a length below 50 meters. Large effects on the amount of fatigue were also found dependent on the surrounding soil stiffness.

4.4 Degradation Modelling of Concrete Structures

As presented in figure 4.1, advanced assessment may be performed at any time along the service life of the structure. Because deterioration and damage occurs while carrying out the assessment, the actual bridge characteristics and material properties should be considered in the assessment. Therefore, the accurate modelling of the remaining strength, ductility, etc is of main interest and the structural effects of the deterioration processes have to be taken into account. The main cause of material deterioration both in concrete and steel bridges is corrosion. Corrosion in steel bridges and performance profiles due to this process are the main issue of several deliverables of the MAINLINE project, as steel bridges was considered one of the main assets to be studied in the project, see ML-D2.1 (2012) to ML-D2.4 (2014).. Therefore, corrosion in steel bridges will not be considered here. However, because corrosion in concrete bridges and tunnels is also an important source of deterioration along life-cycle analysis, the way to model this degradation process in the advanced assessment of a concrete bridge (or tunnel) is considered in the present chapter.

As mentioned, corrosion of steel reinforcement is the most common cause of deterioration of reinforced concrete bridges, Bell (2004). Many of the existing structures show significant corrosion damages; from cover cracking or even cover spalling. Wang (2010) analyzed the impacts of climate change on the deterioration of concrete structures, and showed that the deterioration is expected to become even worse than today. In addition, the demand for load-carrying capacity often increases over time. Thus, there is a growing need for reliable methods to assess the load-carrying capacity and remaining service life of existing buildings. Early research has mainly been concerned with the causes of reinforcement corrosion, se e.g. Tuutti (1982); more recent research also includes the structural effects, e.g. Duracrete (2000).

Corrosion of reinforcement affects the structure in two ways: (a) volume expansion that generates splitting stresses in the concrete, which may crack and spall the concrete cover and affect the bond between reinforcement and concrete, and (b) area reduction and ductility change of the reinforcement bars. The effect on the bond can become dangerous if corrosion takes place in anchorage regions, such as splices, cut-off regions, or at end anchorages. The area reduction of the reinforcement bars will reduce both bending moment and shear capacity, and the effect on the ductility decreases the deformation capacity. All these effects reduce the safety of the structure; therefore they are important to understand and control.

A methodology to analyse the mechanical behaviour and remaining load-carrying capacity of corroded reinforced concrete structures was proposed in Zandi Hanjari (2010). The methodology is shortly described below. It is based on the assumption that the usual method of structural analysis for concrete structures should be applied also to corroded reinforced concrete structures. The effect of corrosion is modelled as a change in the geometry and

material properties of the concrete, reinforcement and their interface through the following steps.

1. If corrosion caused the concrete to spall off, the effect on both the concrete cross-section and the cover loss can be taken into account by modifying the geometry used in the analysis. In compression regions where corrosion leads to cracking of concrete, lower strength and stiffness than for the virgin concrete should be assigned to cracked concrete. The behaviour of concrete around corroded stirrups can be simulated by adapting lower tensile strength. The method of adjusting compressive and tensile strength of cracked concrete is described in Zandi Hanjari *et al.* (2011).
2. Reduction of the effective reinforcement area by both uniform and pitting corrosion is the most obvious effect to take into account. The actual area of a uniformly corroded bar can be calculated by assuming that corrosion has penetrated evenly around the bar. However, pitting corrosion affects the reinforcement locally; therefore, measurement or estimation of the pitting configuration is needed to be able to calculate the residual bar area, see e.g. Val and Melchers (1997). Finally, the ductility of corroded reinforcement can be calculated using practical models in which the residual ductility is confined to empirical correlations with area loss of the corroded reinforcement, see e.g. Cairns *et al.* (2005).
3. Corrosion affects the interaction of reinforcement and concrete. Therefore, the bond-slip relationship should be modified accordingly. The modification could be done according to the method proposed in Lundgren *et al.* (2012), where the level of corrosion corresponds to a certain amount of slip. This procedure can be applied to models at structural level where the bond-slip between the concrete and reinforcement is modelled by one-dimensional bond-slip relation. For simpler structural analysis models, such as beam-element analysis, where the bond-slip is not directly accounted for in the model, the anchorage length can be calculated by the procedure described in Lundgren *et al.* (2012). The basic 1D bond-slip differential equation is numerically solved in a Matlab-routine, resulting in an anchorage length required to anchor the yield force. Either the capacity of the reinforcement is then adjusted in the anchorage region, or the anchorage is checked manually. It could be noted that also a more advanced level of modelling bond is available; Lundgren (2002), Berra *et al.* (2003), and Zandi Hanjari *et al.* (2013) have used detailed finite element modeling to investigate the bond mechanism for corroded bars in concrete, in particular the effect of splitting stresses induced in the concrete by the volume increase of the corrosion products. However, this type of detailed three-dimensional (3D) modeling of the region around all the reinforcement bars is today mainly suitable for research purposes, as it is considered impractical for analysis of complete structures. It could also be noted that while earlier research almost solely has treated accelerated corrosion, recent results presented in Tahershamsi (2013) indicate that the reduction in bond capacity was smaller for naturally corroded specimens; thus, it appears to be safe to apply methods developed and verified for accelerated corrosion.

The methodology is exemplified in an assessment of “Gröndalsviadukten”, a bridge in Stockholm built in 1967 and illustrated in Figure 4.5. The bridge was built in phases; the first phase consisted of columns, cross heads and the starter sections of the main beams, with the remainder of the main beams being cast as a second phase.. This meant that large amounts of reinforcement were spliced at each cast joint. Today, the bridge exhibits systematic damage in the form of spalled concrete on the bottom side of the main beams at these cast joints.

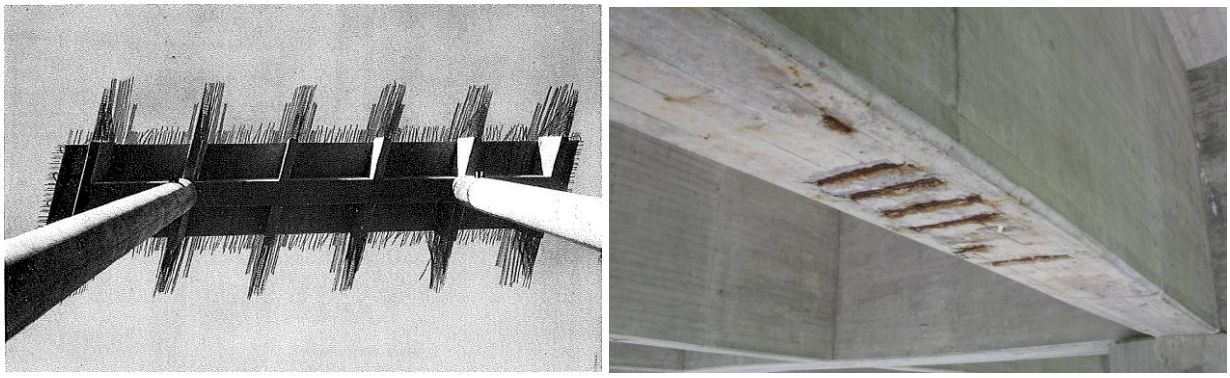


Figure 4-5 Left: Photo from the time of construction in the 1960ies, showing the large amount of reinforcement spliced at the cast joints. Right: Example of a damage with spalled cover at a cast joint at the bridge today.

At the assessment of the bridge, sufficient capacity could be shown if the structure was assumed to be undamaged. In view of the visible damage, this was however considered to be an unrealistic assumption. In next step, no anchorage at all was assumed in the damaged regions; the capacity was then not sufficient. Accordingly, a more detailed investigation on the anchorage in the damaged sections was needed.

For this particular case, the first two points in the list above were of minor importance compared to the third; i.e. the loss of cross-section due to spalling was not included as it was on the tensile side of the beams, and the main reinforcement bars were not assumed to have started to corrode yet (the sign of corrosion is mainly on the stirrups. For these reasons, the main focus was on the third item; i.e. related to the anchorage of the main bars. The model in Lundgren *et al.* (2012) was applied, using the following basic assumptions:

- Design values of strength for both reinforcement and concrete.
- As the model is based on the bond-slip model in CEB-FIP Model Code 1990, CEB (1993), an assumption regarding bond conditions need to be done. It was assumed to be ”all other bond conditions”; thus roughly half the capacity of “good” bond conditions.
- Transverse reinforcement was assumed to be corroded with a decreasing the diameter from 10 to 9 mm; this should preferably be correlated with measurements
- Anchorage was assumed to be affected of spalling in the bottom one-two layers of reinforcement; this corresponds to an assumption that the spalling cracks are not situated higher up in the cross-section and should also preferably be correlated with measurements.
- As the corrosion level was unknown, the maximum anchorage length for varying corrosion penetration levels was chosen; in Figure 4.6 this would for this example result in 1472 mm.

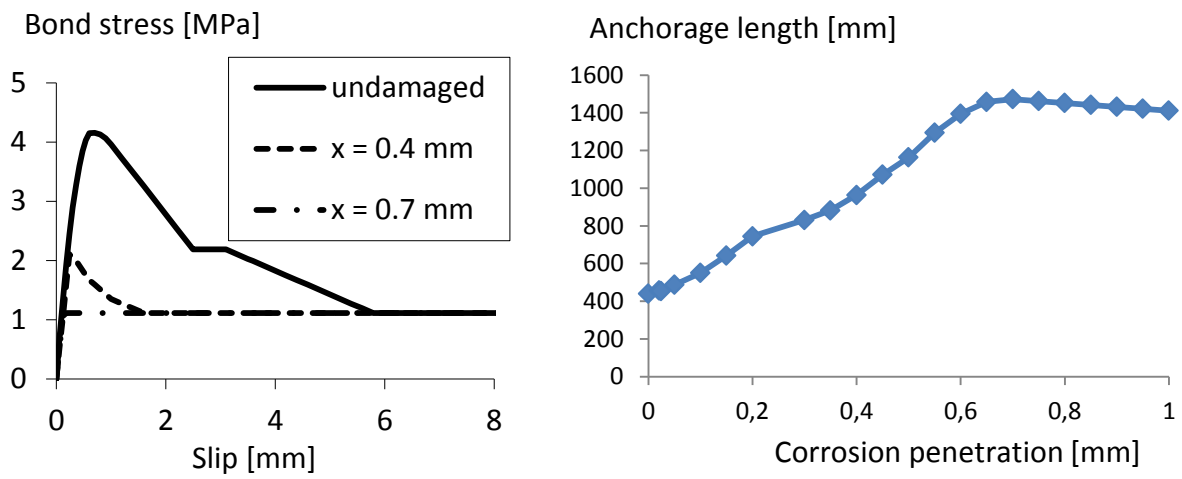


Figure 4-6 Left: Bond versus slip for some varying corrosion levels, x is corrosion penetration from each side of a reinforcement bar. Right: Anchorage length depending on corrosion penetration resulting from numerical solving of the basic 1D bond-slip differential equation along a bar, using the bond-slip relation shown in the left graph as input.

The anchorage length was used to calculate the tensile capacity of the reinforcement in the splice; the stress increase was assumed to be linear. The effect on structural level is exemplified in Figure 4.7, where the bending moment in one beam in one span of the bridge is shown. As can be seen, the capacity could be shown to be sufficient.

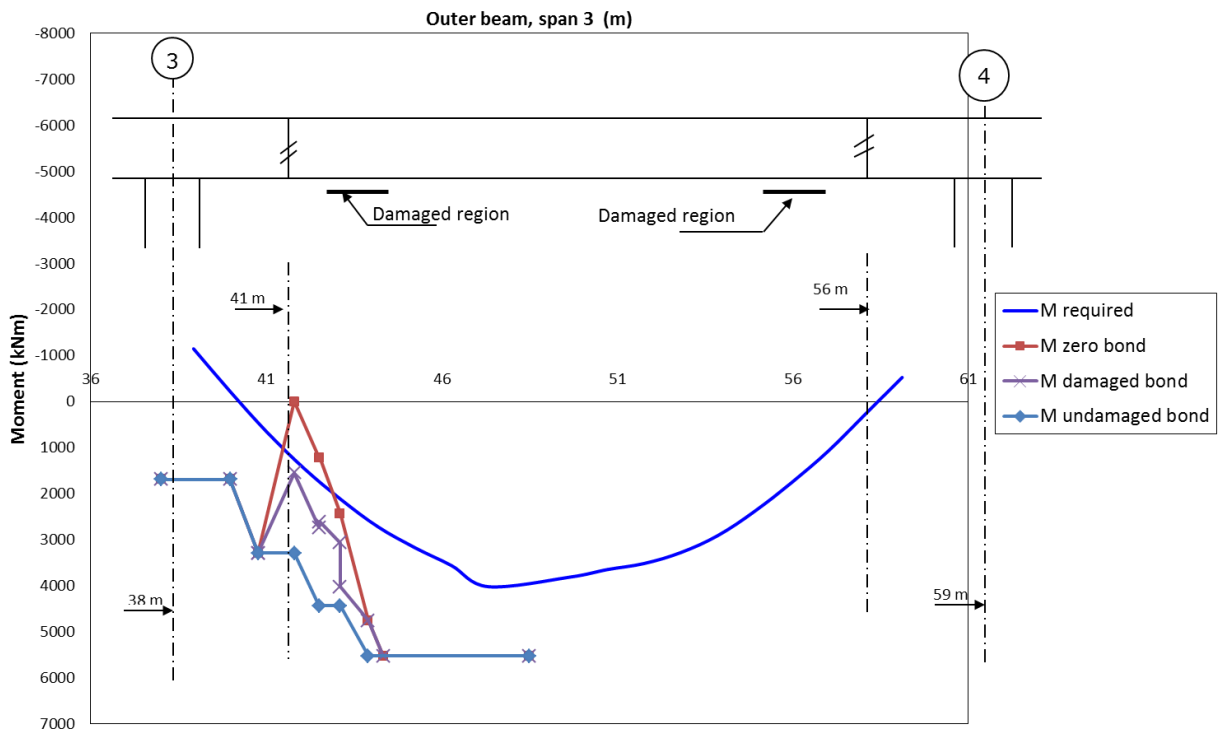


Figure 4-7 Bending moment in one span in one of the beams of the bridge.

The assessment process above made it possible for the IM Trafikverket in Sweden to continue to use two bridges at Gröndal and Blommersberg in the Essinge Route through central Stockholm without extensive repair/replacement. They hereby saved 3M€ to be compared to the assessment cost of 0,2 M€, Lundgren et al (2014).

5. Strengthening with prestressing /post-tensioning

There are many ways to strengthen structures. The most common ones are presented in the Guidelines ML-D1.4 (2014) and SB-STR (2007). Here we will focus on prestressing which is a method which is finding more and more applications.

5.1 General

There are many ways to strengthen structures. The most common ones are presented in ML-D1.4 Guideline (2014) and in SB-STR (2007).

Prestressing and/or post-tensioning is a strengthening method that can be used to improve the performance of various structural members. The American Concrete Institute (ACI) defines prestressed concrete as: "*Structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads*" *(ACI 2013).* According to the International Federation for Structural Concrete (fib) prestressing
"*tendons may be:*
- internal to the concrete, and
- pretensioned, or
- post-tensioned – in this case they may be bonded by grouting, or temporarily or permanently unbonded;
- external to the structure but totally within the external outline of the structure.

Prestressing may be used for any type of structure for
- new construction;
- repair and strengthening of existing structures", fib Model Code 2010 (2012)

Collins and Mitchell (2001) explained that the tensioning of prestressing reinforcement in the concrete results in a self-equilibrating system of internal stresses (tensile stresses in the steel and compressive stresses in the concrete) which improves the response of the concrete to external loads. While concrete has a high compressive strength it is weak and brittle in tension, and hence its response to external loads is improved by applying a pre-compression.

5.2 Historical perspective

Lin and Burns (1982) described the development of structural materials into prestressed concrete as three parallel lines depending on the characteristics of the material. In the first line there were the materials with a high compressive strength, starting with the use of stones and resulting in high strength concrete, after implementing bricks and regular concrete. In the second line there were the tension-resisting materials; from bamboo, ropes, iron- and steel bars, to high strength steel and fibre reinforced polymers (FRP). The third line comprised materials used to resist bending, such as; timber, structural steel, reinforced concrete, fibre reinforced concrete and eventually prestressed concrete. A reproduction of the Lin & Burns development chart is shown in Fig. 5.1.

A historical review on prestressing of structures was given by e.g. Gasparini (2006), where he discussed how prestressing was used already in Egyptian boats from 1500 b.c. as twisting of ropes created a prestress, preventing hogging in the hulls. For prestressed concrete, a historical review can be found in Billington (1976), where Eugene Freyssinet was appointed as

the first person to successfully produce prestressed concrete as he in 1928 started to use high strength steel wires instead of normal strength reinforcement for prestressing.

Classification of prestressing

Prestressing can be classified in several ways depending on the characteristics and application of the method. Some of the most common classifications are;

- Pre-tensioned or Post-tensioned systems
- Bonded or unbonded systems
- External or internal systems
- Linear or circular systems
- Partial or full prestressing

The most common classification is based on the sequence of casting the concrete and stressing the tendons, and includes pre-tensioning and post-tensioning. While pre-tensioning means that the tendons were tensioned before concrete casting, post-tensioning means tensioning after concrete hardening. Thereby, strengthening of existing concrete structures implies mostly post-tensioning.

The Post-Tensioning Institute (PTI) classifies the different types of post-tensioning as unbonded or bonded, and external or internal systems (PTI 2006). Bonded systems denote that the tendons are bonded throughout their length to the surrounding structure, while unbonded systems comprise end-anchored tendons. End-anchored systems can however be both unbonded and bonded, where the bond is accomplished by subsequent grouting.

External prestressing systems are located on the outside of the structural surface, while the internal prestressing can be found internally in the structural member.

Circular prestressing is mostly applied on circular structures like columns pipes, silos and tanks, where the tendons are wound around in circles. Linear prestressing comprises all structures that are not circular but it does not necessarily have to be straight tendons, bent and curved tendons are also considered as linear.

The joint ACI-ASCE Committee 423 (1999) defines partial prestressing of concrete as *"An approach in design and construction in which prestressed reinforcement or a combination of prestressed and non-prestressed reinforcement is used such that tension and cracking in concrete due to flexure are allowed under service dead and live loads, while serviceability and strength requirements are satisfied"*. Fully prestressed concrete, on the other hand, is defined as concrete with prestressed reinforcement and no flexural tension allowed under service loads

5.3 Advantages of prestressing

Prestressing generates positive effects to a structure in all phases, from design to the end of its service life. Incorporating prestressing in the design enables the design of more slender structures with lower weights, resulting in e.g. longer spans. Collins and Mitchell (2001) explained that concrete one-way slabs can be made up to 60% longer if they are prestressed and for this reason, more than 50% of all bridges are now constructed of prestressed concrete. Smaller cross-sections result in reduced material consumption, which finally lead to more economical structures. The main advantages in the design and construction phases are;

- More slender structures

- Longer spans
- Reduced costs

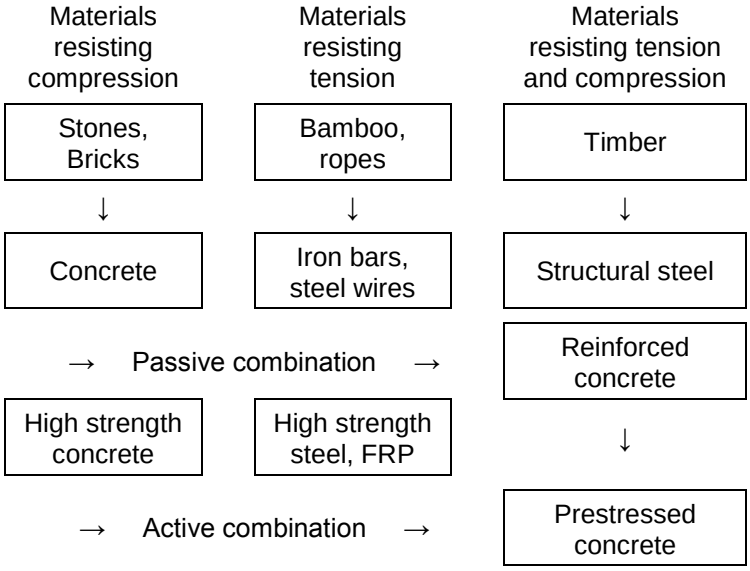


Figure 5-1 Development of building materials. Adapted from Lin and Burns (1982).

The service limit state effects are highly dependent on the prestressing design, where the main advantages appear in the form of reduced cracking and crack widths, but smaller deformations can also be achieved due to a higher stiffness of the structure. As a prestressed concrete structure remains uncracked for service loads, the corrosion of reinforcement is reduced, which increases the durability.

- Increased cracking load
- Reduced crack widths
- Smaller deformations

In the ultimate limit state, the advantages of prestressing, compared to a non-prestressed structure, are represented by higher maximum load capacities, regarding both flexure and shear. The positive aspects described for the service limit state are, however, still effective.

- Higher load carrying capacity
- Increased ductility

5.4 Post-tensioning applications

The post-tensioning manual (PTI 2006) describes a wide range of applications for post-tensioning in all facets of construction from buildings and bridges to parking structures, slabs-on-ground, ground anchors, highway pavements, as well as rehabilitation and retrofit applications.

5.5 Strengthening of bridges

Prestressing or post tensioning can be applied to strengthen bridges in a number of different ways. The bridge girders can be reinforced by introducing a system of external tendons, anchored at the ends of the girders. The tendons can be either straight or curved to obtain the maximum effects, and the prestressing bars or tendons can be made of either high-strength steel or composites, see Ng and Tan (2006), Grace and Abdel-Sayed (1998), Matta et al. (2009) and Bennitz et al. (2012).

Prestressed high-strength steel or composite strips, plates and bars can also be bonded to a structure for a higher structural load capacity. These reinforcement components can either be bonded directly to the structures surface, or be installed as near surface mounted reinforcement (NSMR) in post-produced grooves in the concrete surface. Czaderski and Motovalli (2007) described the strengthening of a 40-year-old concrete bridge girder by prestressed carbon fibre reinforced polymer (CFRP) plates, bonded to the bottom edge of the girder. An unstrengthened beam was tested and compared to beams strengthened by non-prestressed and prestressed CFRP. The results showed that the deflections were smaller, while the ultimate load capacity increased for the girder with prestressed CFRP. The maximum load increased by 24% for the CFRP-strengthened non-prestressed girder, while the corresponding capacity increase for the CFRP-strengthened prestressed girder was 45%. Further information regarding surface bonded prestressing can be found in e.g. El-Hacha et al. (2003), Garden and Hollaway (1998), Tarantfillou et al. (1992) and Al-Emrani and Kliger (2006). Prestressed NSMR has been investigated in Nordin and Täljsten (2006), De Lorenzis and Teng (2007) and Badawi and Soudki (2009).

Most prestressing systems contain mainly longitudinal horizontal tendon layouts, but vertical solutions are also possible for shear strengthening purposes, see Rahai and Shokoohfar (2010), and transverse internal prestressing of concrete bridge slabs has been investigated by Noël and Soudki (2013), Nilimaa (2013) and Nilimaa et al. (2014).

A strengthening method for concrete beams has been tested by Valerio (2009). Vertical holes are drilled into the web of the beams from the soffit in the shear spans, high-viscosity epoxy resin is injected from the bottom and then FRP or steel bars are embedded into place. It is called deep embedment of steel or fibre-reinforced polymer (FRP) bars and was found to be feasible and very effective for reinforced concrete (RC) and prestressed concrete (PSC) beams of any size.

An example of strengthening of a bridge slab by post-tensioning is given in Appendix B.

6. Tunnels

6.1 Introduction

A general handbook on tunnels is Maidl et al (2013, 2014).

Railway tunnels are almost impossible to replace so they must be maintained on the assumption that they have a virtually infinite life. They can be either lined or unlined and have a variety of shapes as illustrated in Figure 6.1.

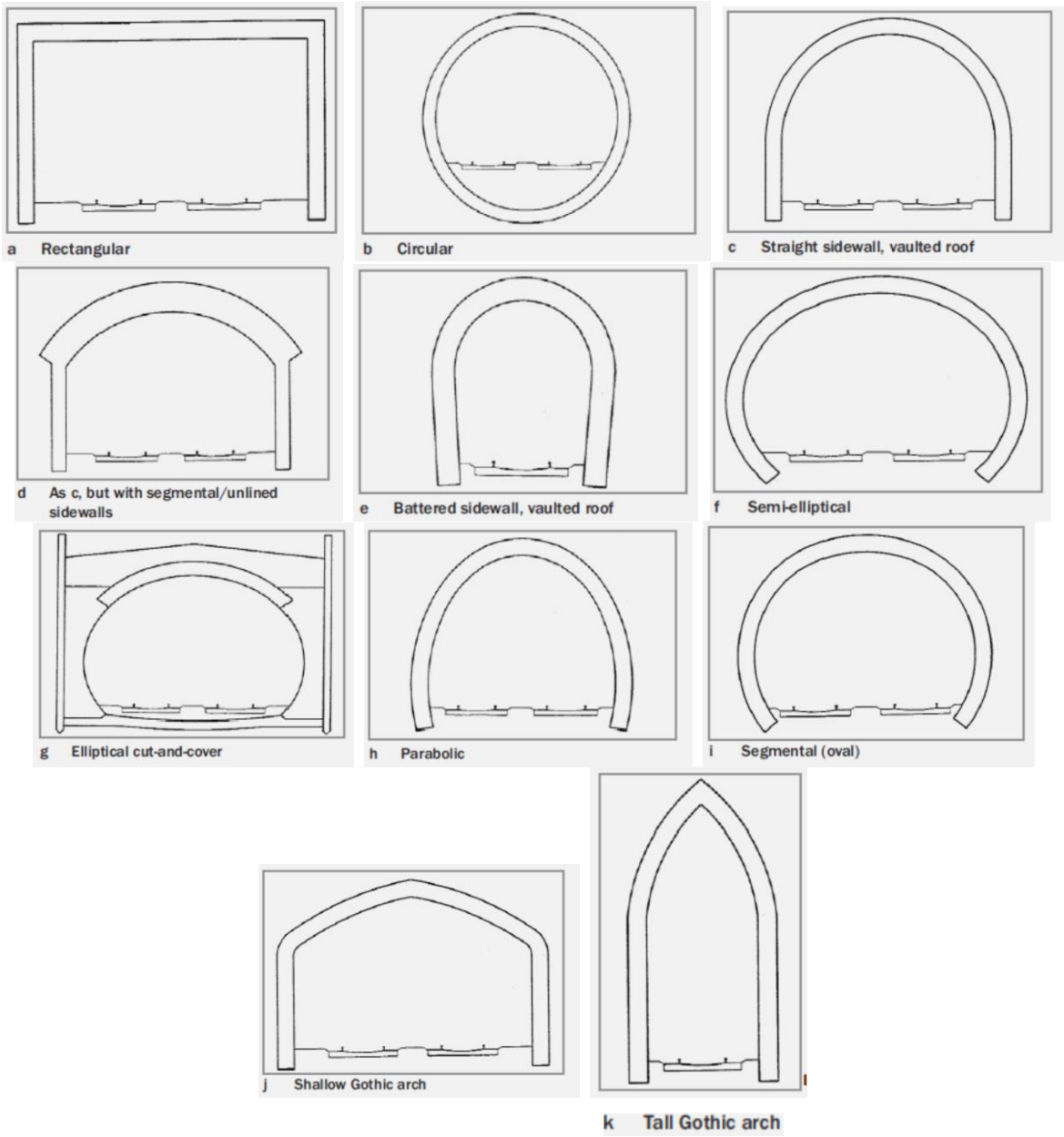


Figure 6-1 Typical UK tunnel cross sections (Railtrack 1996)

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

As most railway tunnels date back to the 19th century they were inevitably constructed using hand tunnelling techniques, which usually required the construction of vertical access shafts from ground level. In some cases these shafts were left open to provide ventilation but others were backfilled and are referred to as buried or hidden shafts whose location is often unknown. Construction details for a typical tunnel shaft are shown in Figure 6.2.

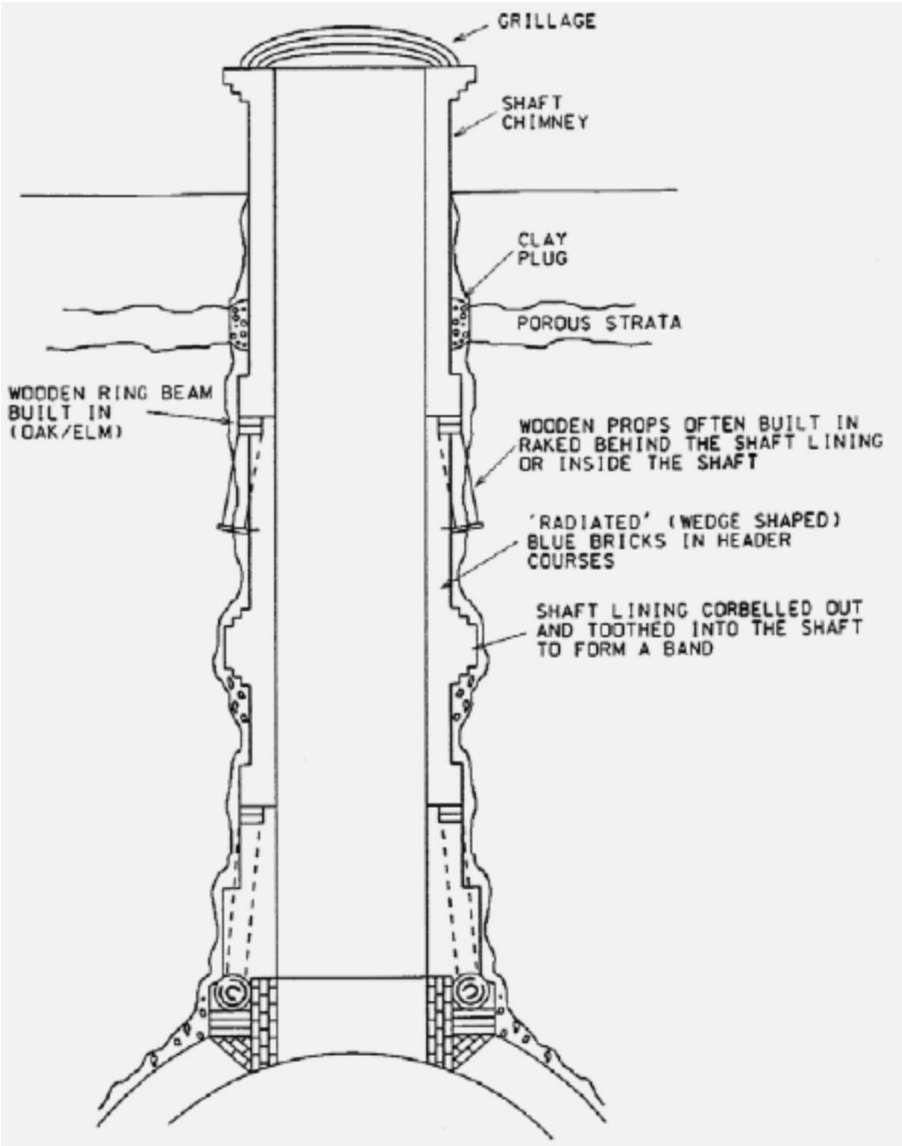


Figure 6-2 Construction details for a typical UK tunnel shaft (Railtrack 1996)

Shafts create a discontinuity in the tunnel lining and can act as outlets for ground water, which may locally accelerate the rate of lining deterioration. Shafts, and particularly buried shafts, have been known to collapse, sometimes with fatal consequences to people at ground level as the following extract from an official inquiry shows.

> *"On 13th April, 15 days before the accident, some brickwork fell from the roof of the tunnel at a place over which an old and unknown constructional shaft had been filled in. Immediate steps were taken to stop rail traffic and arrangements were made to*

strengthen the tunnel at this point, but before the protective work had been completed, the roof collapsed in the early morning of the 28th April. The contents of the shaft poured into the tunnel, thus forming a crater on the surface of the ground, where a pair of semidetached houses in Temple Drive collapsed and the end wall of a third one fell outwards. I regret to state that the five occupants of the first two houses lost their lives, but fortunately two others living in the third house were rescued, suffering only from minor injuries and shock." (MOT 1953)

In considering the risks at ground level from tunnel shafts it is necessary to take account of the zone of influence, as illustrated in Figure 6.3.

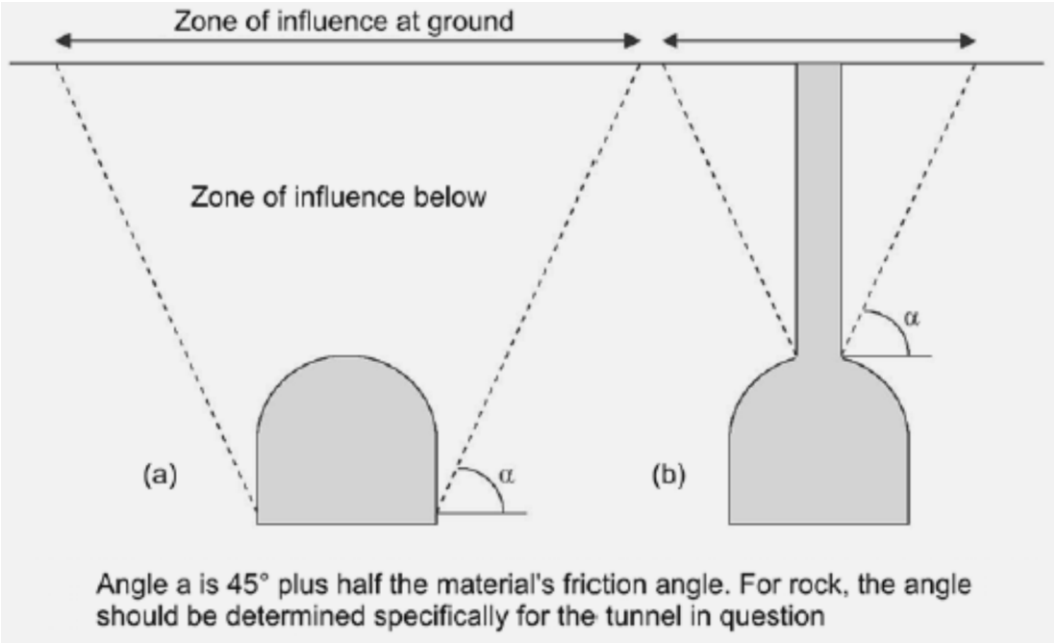


Figure 6-3 Zone of influence of a tunnel (Network Rail 2004)

As part of the process of determining the need to undertake life extension of a tunnel it is sometimes necessary to undertake a structural assessment. This will follow similar lines to that adopted for bridges constructed of the same basic engineering material and can include the advanced techniques described in chapter 4 of this report.

6.2 Life extension of unlined tunnels

The deterioration of unlined tunnels is usually limited to failures of the rock through which the tunnel is driven, often caused by ground water permeation or the effect of frost. This can vary from the regular loss of small pieces from friable rock to the dislodgement of large blocks of stone which could be large enough to derail a train as shown in Figure 6.4. The potential for the latter type of failure is largely dependent on the orientation of the bedding planes within the rock mass.

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

- Scaling of loose material or the removal of loose blocks
- Provision of protection shelters
- Localised rock bolting
- Rock bolting and rockfall protection mesh
- Localised application of sprayed concrete to support weak areas of rock

- Installation of supporting ribs
- Installation of full lining, usually using sprayed concrete

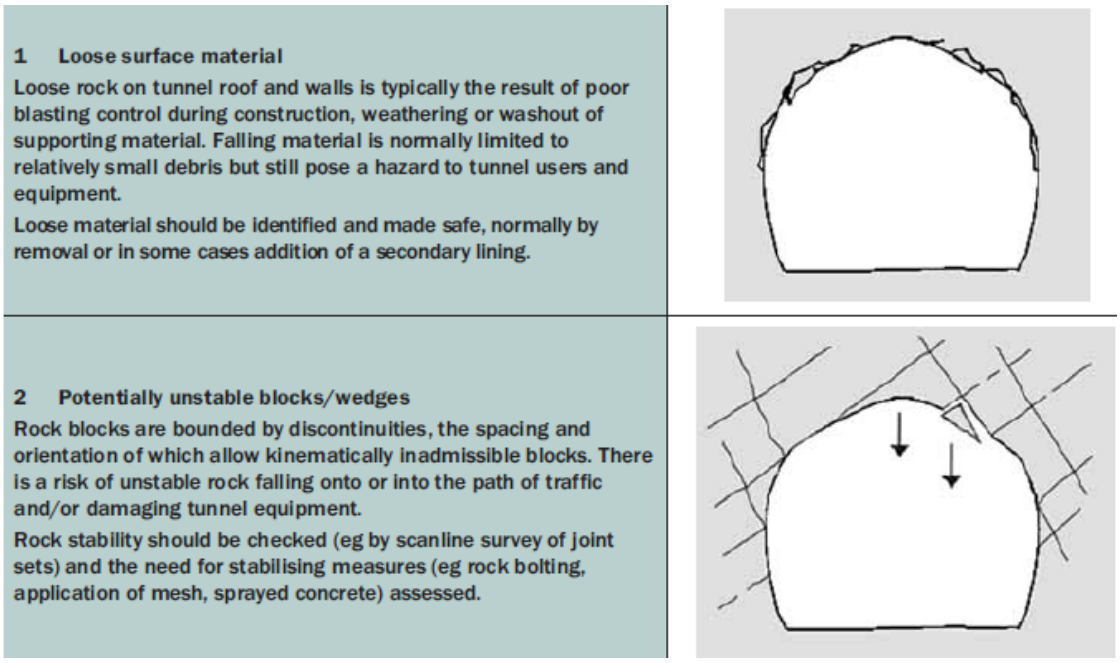


Figure 6-4 Defects in unlined tunnels (CIRIA C671)

6.3 Life extension of lined tunnels

6.3.1 Masonry

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

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

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

Traditional repair/life extension techniques usually consist of:-

- Patch repairs (see Figure 6.5)
- Crack repairs
- Ring separation repair
- Grouting

- Underpinning
- Invert repair

Figure 6-5 Traditional method of relining/replacing damaged brickwork using temporary metallic sweeps and wooden lagging to support new brickwork.

Novel products are now available on the market and are briefly described below.

- Electro osmosis can be used to firstly dry out wet patches in a tunnel wall and then, by reversing the polarity of the system, to draw grout into voids to both structurally strengthen masonry and inhibit further water ingress. When using this technique it is not a good idea to try to fully waterproof a tunnel, since that can lead to the build-up of highly dangerous hydrostatic pressure behind the lining. Instead water should be directed to suitable outlets, such as low level weep pipes, where the water can be successfully managed. A description of the method can be found at http://www.structural.net/article/electro-chemical-dewatering-system
- New support systems have been developed, which can be left insitu permanently. One example is shown in case study 2.

6.3.2 Insitu concrete linings

Insitu concrete linings usually suffer from similar deterioration mechanisms as other forms of concrete structure (principally carbonation or chloride induced corrosion) and the repair methods also generally follow traditional lines. Advice on how to ensure quality repairs can be found in numerous publications and research project outputs; one example is the Rehabcon manual available at http://www.cbi.se/objfiles/1/MANUALmaindocum_1522366004.pdf

More recently the use of both steel and carbon fibre to strengthen tunnel linings has started to become accepted, particularly in Japan. Details of one steel system can be found at http://www.nssmc.com/en/tech/report/nsc/pdf/n9209.pdf and details of carbon fibre strengthening (illustrated in Figure 6.6) can be found at http://www.wtec.org/loyola/compce/04_07.htm).

Figure 6-6 Applying carbon fibre strengthening sheets to the Daiichi - Yamaki railway tunnel (http://www.wtec.org/loyola/compce/04_07.htm)

6.3.3 Concrete segments

Tunnels lined with concrete segments can deteriorate in a number of ways, which can indicate different root causes as shown in Table 6.1 (after CIRIA C671), see also Figure 6.7.

Table 6-1 Deterioration of concrete tunnel segments

Deterioration noticed	Possible cause
Cracking within segment parallel to cross joints	Concrete shrinkage
Spalling of concrete within segment	Casting defects, impact damage, rebar corrosion, excessive loading
Spalling to edges of segment	Out of plane construction
Diagonal cracking within segment	Incipient compression failure
Lipping of joints between segments	Construction defect
Circumferential crack within segment	Overloading leading to possible bursting failure

Figure 6-7 Edge spalling of a concrete tunnel segment

Where excessive loading is suspected a detailed engineering assessment will be needed before deciding on the correct remedial action. For deterioration that is not structurally significant conventional concrete repair techniques are usually sufficient, but in more serious cases techniques such as those illustrated in Figure 6.8 will need to be considered.

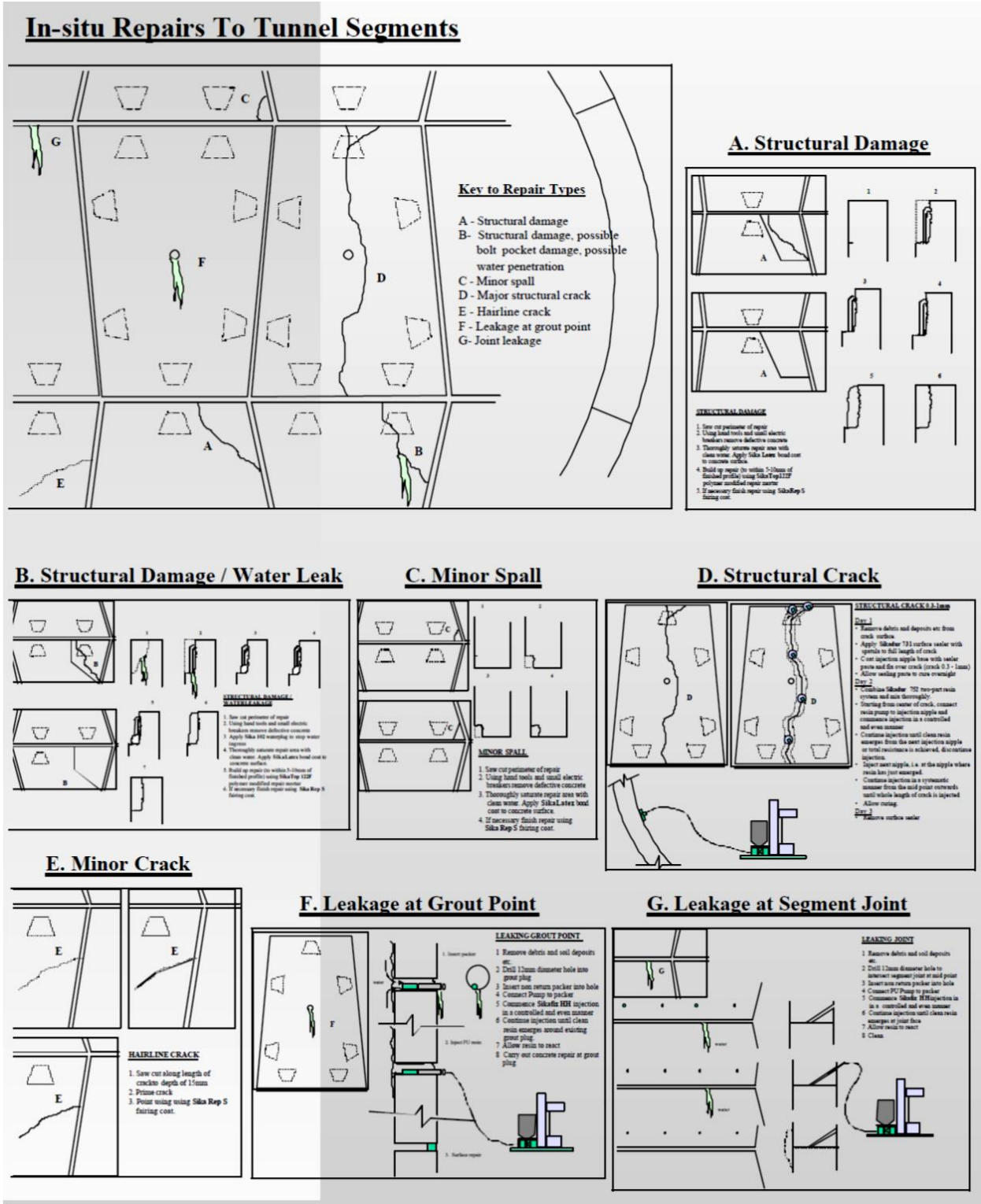


Figure 6-8 Repair techniques for deteriorated concrete segments

http://www.hammersmithgroup.com/datasheet/01/ref-10.pdf

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

6.3.4 Metallic segments

Metallic segments can be manufactured out of either steel or cast iron and are usually only to be found in circular tunnels. The most common defects are cracking (principally at bolt holes) and corrosion. Steel linings can be repaired by welding or bolting new steel into position but cracked cast iron should be repaired using the stitching technique illustrated in Figure 6.9. Corrosion principally affects steel linings and should be treated using conventional surface coating techniques.

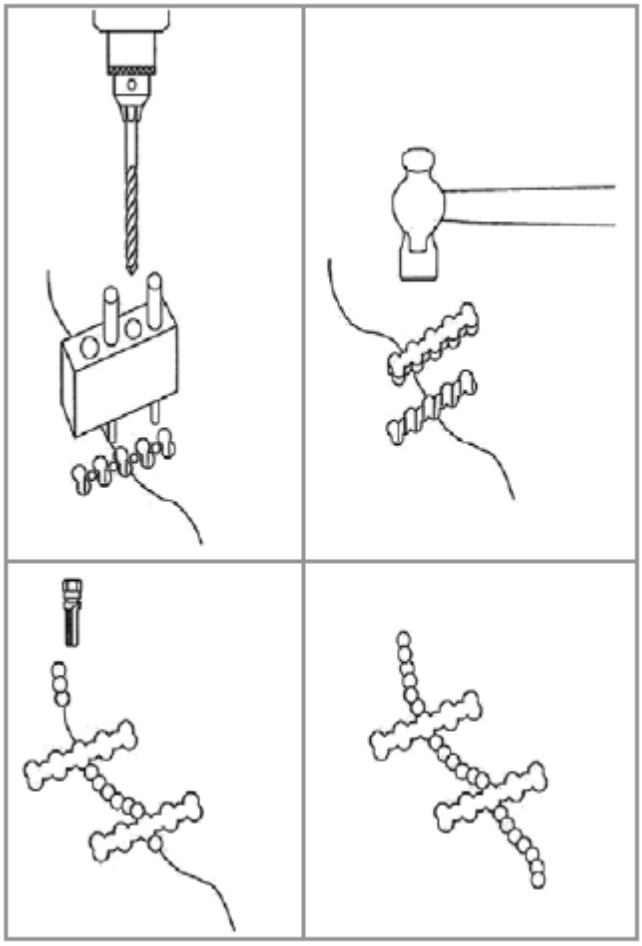


Figure 6-9 Method for stitching cracks in cast iron

http://www.locknstitch.com/Metal_Stitching.htm

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

6.4 Life extension of tunnel shafts

Shafts in tunnels can be found in a number of different configurations as shown in Figure 6.10.

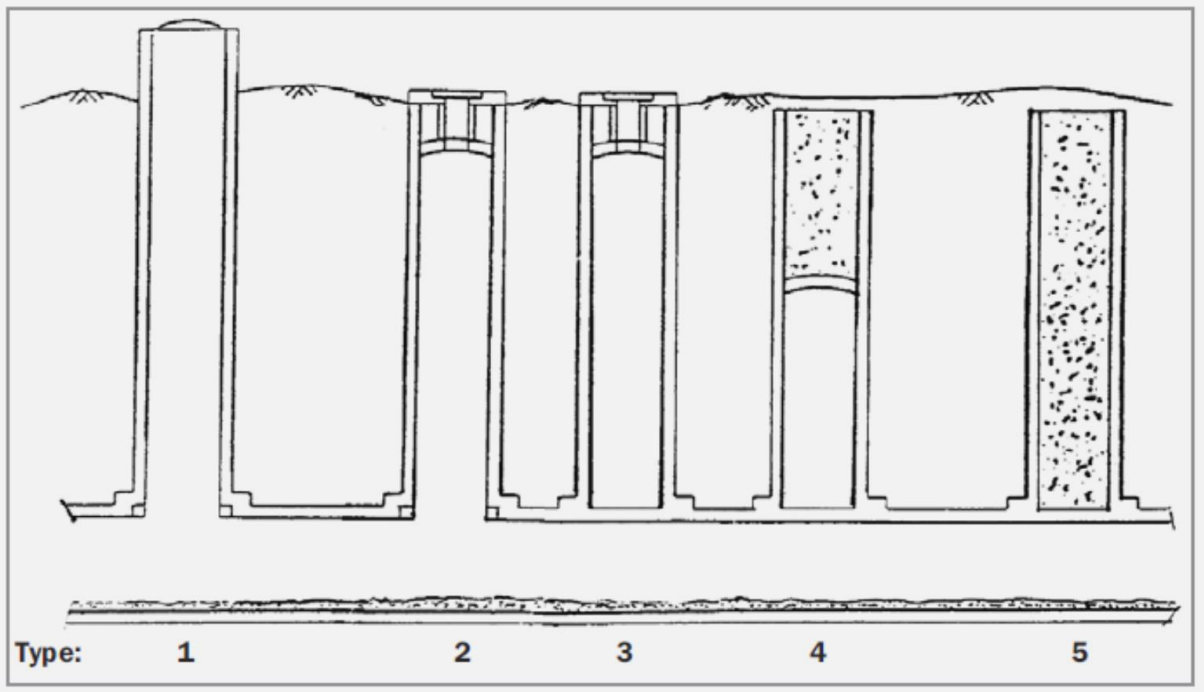


Figure 6-10 Possible states of construction and ventilation shafts (Network Rail 2004)

As explained above blind or hidden shafts pose the greatest risk as their location may be unknown and hence they may never have been inspected or maintained. The deterioration of tunnel shafts is generally similar to the deterioration of tunnels constructed out of similar materials; hence the repair methods are similar as well, although access can present some difficulties. When undertaking repair work in a tunnel shaft it is advisable to seal the bottom of the shaft (often referred to as the eye) to ensure that any material or tools that are accidentally dropped do not present a danger to those using the tunnel.

As shafts inevitably pass through the water table water ingress can be a serious problem and shafts often contain drainage gutters (sometimes referred to as garlands) at appropriate positions and down pipes to conduct the water to tunnel level. These are usually manufactured out of steel and are thus very susceptible to corrosion and need to be regularly maintained so as not to create a hazard. Modern composite gutters may be used to replace traditional steel ones as they should give a longer service life and the lighter to install.

Filling redundant or seriously deteriorated shafts is often considered, particularly as ventilation requirements are considered to be reduced following the general withdrawal of steam locomotives. However this is not an action that should be undertaken without a full consideration of the downstream effects of such a course of action, bearing the following in mind (Network Rail 2004):-

- shaft filling is an expensive operation regardless of the material used
- it could be assumed that maintenance will stop and the shaft, with its fill, will revert to solid ground. Unfortunately this is not the case and, after filling, it will be impossible to inspect or work within the shaft
- shaft lining on its supports because shear support between the lining and the fill is often negligible

- filling an open shaft is likely to reduce ventilation in the tunnel, which may increase the general level of dampness and increase the likelihood of build-up of gases (although it may also reduce frost damage to the tunnel lining
- open shafts are an important alternative method of access for emergency services and equipment. They also provide emergency escape and evacuation routes. Filling open shafts removes this valuable facility.

It is also possible to cap shafts, preferably at ground level. The most common method of covering a shaft is the construction of a prefabricated reinforced concrete cap to span the shaft void. The size of the cap should sufficiently exceed the internal diameter of the shaft and be adequately supported, preferably at or below rock-head if present. Where this is not possible, the cap may be supported on competent ground with extra support measures, for example, plugging, grouting (inside and outside the shaft lining) piling and diaphragm walling etc depending on the situation.

Where a cap is to be constructed at the base of a shaft, it should be designed to support any shaft fill material and provisions for water drainage should be included.

Shaft caps should include access covers to help inspection of shaft condition or, where the shaft is backfilled, to allow the surface level of the fill to be checked for signs of settlement.

Shafts may have been infilled with materials that decay to produce potentially hazardous gases, particularly domestic or industrial refuse. Covers to such shafts can be sealed using gypsum, cement or resin-based products to restrict the emission of gases to the atmosphere and prevent their migration to and accumulation in confined areas. It may be necessary to install vent pipes to prevent excessive gas build-up, and these may need to be fitted with flame arrestors and where appropriate protected by lighting conductors.

The location of caps should be recorded and their central position permanently marked on the ground surface and within the tunnel so that they can be easily found and identified at a later date. (CIRIA C671)

Three case studies are presented in Appendix C.

7. Track and Earthwork

7.1 Summary

Overviews of practice and research are presented in e.g. Lothar (2007), IHHA (2009) and Innotrack (2010). Track-wheel interaction is treated in Lewis and Olofsson (2009). General information on the history of tracks is given in Pike (2010). Many experiences as a railway engineer in the U.S. is shared by Selig (2014). Recent trends are given in Veit and Marschnig, (2011, 2012).

A typical double track and its parts are illustrated in *Figures 7.1* and *7.2* Below new and improved methods are summarized in a table. The different methods are then presented in some more detail in the following subsections.

Figure 7-1 Double track and embankment in Torp, Sweden, Innotrack (2010)

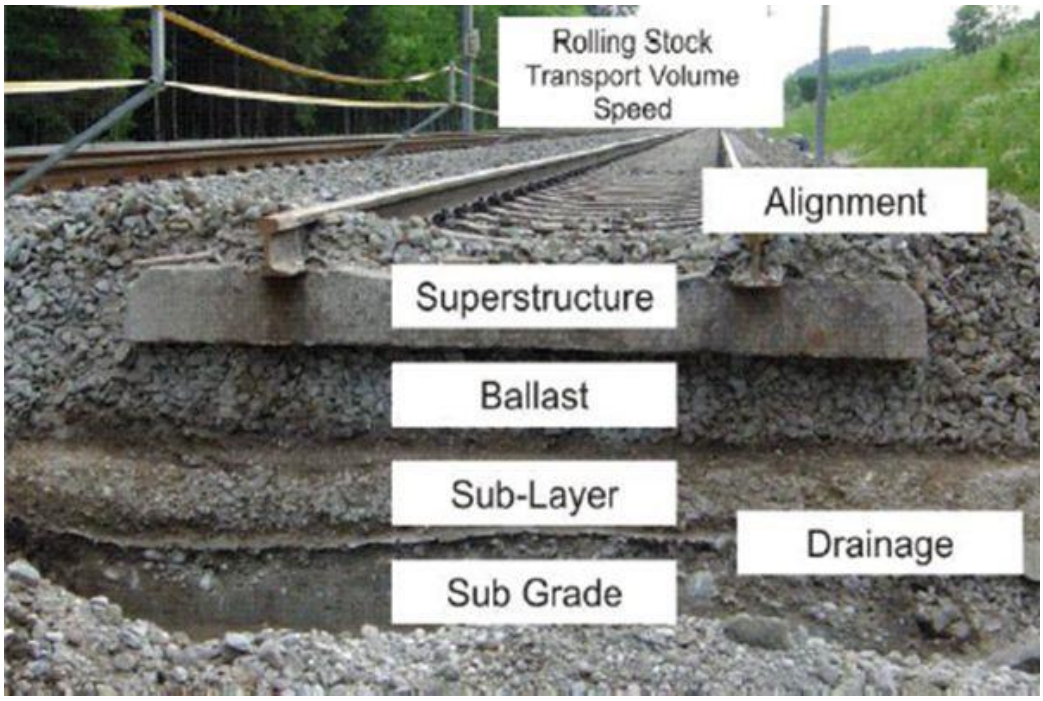


Figure 7-2 Track definitions, ML-D5.6 (2014)

Work on track systems is or has been going on in the European projects Automain (2014) during 2011-2014, Interail (2012) during 2009-2012, Smartrail (2014) during 2011-2014 and Sustrail (2014**)** during 2011-2015**,** In Sustrail D1.2 (2012) examples are given on track conditions for three routes in Bulgaria, Spain and the U.K.

It can be seen from the Questionnaire in appendix A in ML-D.1 (2013) that not many Infrastructure Managers yet use Life Cycle Cost Assessment in the planning of maintenance and repair of their rail infrastructure. There is a lack of data and methods and here the MAINLINE project with its Life Cycle Assessment Tool (LCAT) may give assistance, see ML-D5.7 (2014).

Table 7-1 Summary of technologies to improve track – pros and cons

New Technology	Pros	Cons
Better inspection and assessment methods	Improves safety and economy	Implementation may need some additional work
Grinding of rail with optimized procedures	Improves safety and economy	Takes time and and may cost money in the short run
Improved Switches and Crossings	Fewer failures	May initially cost more
New strategies for tamping and ballast cleaning	Improves ballast properties	May be more expensive in the short run
Improved sleepers to delay degradation	Last longer	Quality control is necessary

7.2 Better Inspection and Assessment Methods

Inability to detect rail cracks at an early stage of growth hinders the planning of mitigating actions such as grinding. It may also entail that cracks are allowed to grow too long before removal, which leads to higher grinding costs and more operational disturbances, and also to a shorter rail life. In severe cases this may even be a safety issue.

Inspection methods and equipment to detect rail cracks were studied in Innotrack (2010), see Ekberg and Paulsson (2010). The work continued in Automain (2014), Interail (2012) and PM'n'IDEA (2012). In Automain 2.3 (2014) five inspection technologies were developed:

- infrastructure mounted video cameras
- high speed inspection of switches using lasers
- SIM (Switch Inspection Measurement) wagon
- in-service track geometry recoding
- instrumented washers and alternative approaches to securing track components

In addition, three higher level enabling technologies were also considered:
- the development of a suitable Man Machine Interface
- modular components to facilitate rapid track maintenance, ideally incorporating facilities to support automatic inspection

- the potential for self-powered wireless sensors to be used to monitor or measure various aspects of the infrastructure

The report Automain 2.3 (2014) also includes a concept design for a self-inspecting switch and supporting systems

7.3 Grinding of rail with optimized procedures

Grinding is used to increase rail life and reduce cost, see Zarembski (2005). Grinding costs are today relatively high. Two reasons for this are poor logistics planning and lack of network grinding strategies. Innotrack (2010) has delivered a guideline and a Technical Recommendation on optimized grinding procedures. This document includes not only technical specifications (e.g. profile tolerances), but also logistical and strategic considerations.

High speed grinding (HSG) is a relatively new proprietary method based on the principle of circumferential grinding. Cylindrical grinding stones are pulled over the rail at an angle, inducing rotation as well as an axial grinding motion. The grinding stones are mounted on grinding units hauled by a carrier vehicle. In this way the speed can be increased to 60-80 km/h compared with traditional grinding with a much lower speed. Fast ultrasonic testing (search for head checks) can support fast grinding procedures.

In Automain 4.2 (2013) the following conclusions are drawn regarding optimization of grinding:
• Long enough maintenance windows are necessary to minimise track possession time needed for grinding.
• The use of improved conventional grinding with 64 stones shows that reduction in track possession time in the order of 50 % is possible. This reduction depends on the layout of the track.
• HSG and twin HSG present good opportunity for the reduction of track possession time, in comparison with conventional grinding over 67% reduction in track possession time is possible.
• The grinding cost is in the order of 5 -10 % of the total LCC for the rail and the most significant cost element is the cost of track replacement. This makes decision on time to replace track crucial from LCC perspective.
• An improved conventional grinding machine will have about the same order of cost as the High Speed Grinder, but will most probably give earlier replacement of rail than the High Speed Grinder so the LCC-cost is slightly higher.

7.4 Improved switches and crossings

Switches & crossings (S&C), also named turnouts, are discontinuities in the track systems. They impose dynamic loads on track and rolling stock and are prone to mechanical failures. Two types of improvements may be possible: (1) more robust switches with new types of components less susceptible to failure and (2) improved maintenance procedures.

Through numerical simulations calibrated from in-field measurements recently several measures to optimize the mechanical characteristics of S&C have been proposed, by Nissen (2008) and Innotrack (2010). Work is now going on in Automain (2014) and Sustrail (2014).

An interesting example is shown in **Error! Reference source not found.** where Performance ndicators (PI) for switches are studied, Stenström (2012). Figure 7.3a shows that two subsystems of the S&C, namely the switch control system and the switch motor system, are deviating considerably from the other subsystems with respect to number of work orders and delays. The corresponding active repair times can be seen on the right hand side of the figure as box plots. On each box, the central mark is the median, the edges of the box are the 25th

and 75th percentiles, and the whiskers extend to 1.5 IQR (interquartile range). Outliers are left out. The median time to repair of the frog, i.e. the switch crossing point, is over 200 minutes, while other systems take about 50 minutes.

The subsystems of the S&C are further broken down to the component level in Figure 7.3b. Connectors and point drives, which are part of the switch control system and switch motor, are found to have a high risk ranking. In addition, the frog point and wing rail of the frog have high active repair times.

Performance measurements can give large savings and bring business safety by more proactive management, while there are additional costs associated with measuring. It is therefore important to thoroughly analyse what, where, when and how to measure. Thus, there exists a need to study the railway infrastructure PIs used by different IMs, to find out which ones are the most important, which are required and which are not required.

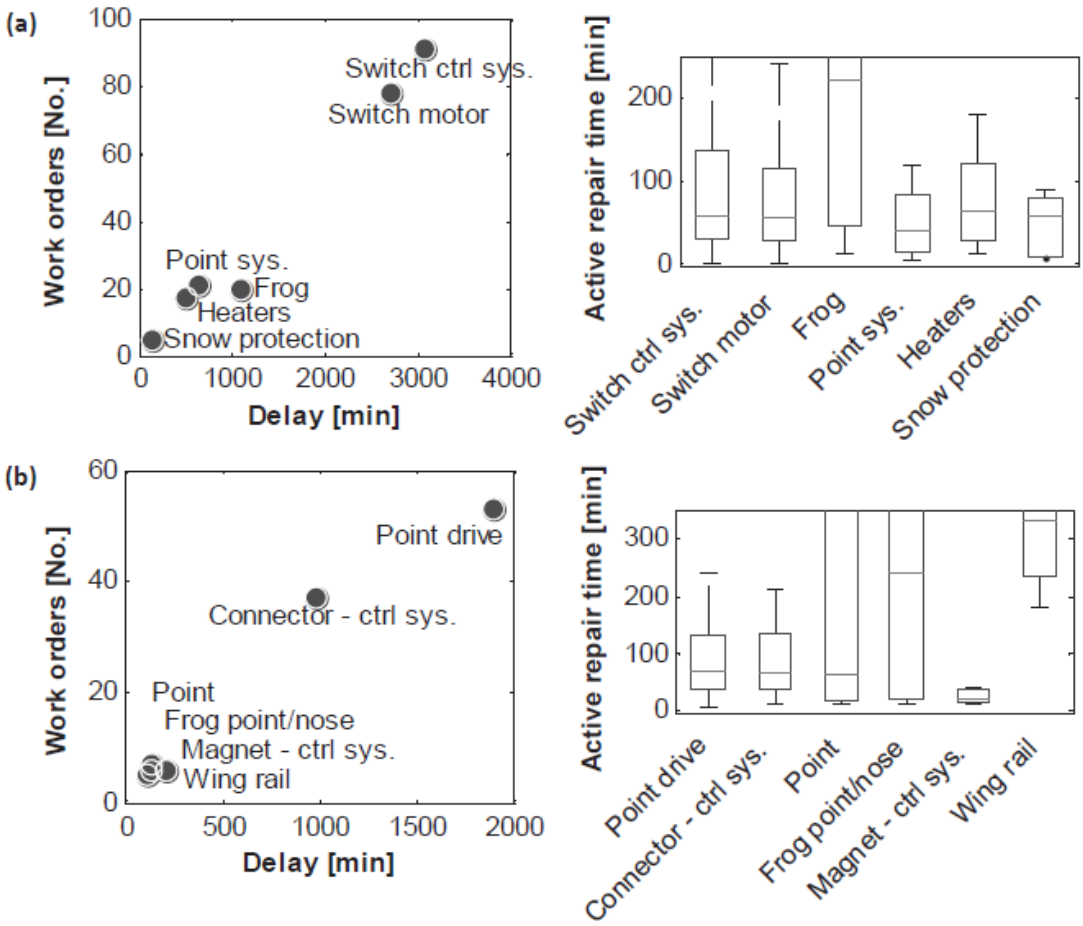


Figure 7-3 Example of (a) subsystems and (b) components which cause most work orders and longest delays in switch systems, Stenström (2012). It can be seen that most of the trouble is caused by the control system and the motor in (a) and the point drive and the connector in (b)

For maintenance of switches and crossings Automain 4.2 (2013) found that

• Value stream mapping together with simulation makes a time saving of 50 % in crossing replacement possible. However, it requires two welding teams compared to one in current practice.

• In the study on optimal maintenance window in-between regular departures, a maintenance work of 120 minutes was simulated with regular time tables at different frequencies. Results indicates that 40 minutes train frequency could be considered as an optimal window size regarding train service and maintenance cost. It gives 35 % saving compared to a train frequency of 20 minutes.

These models and simulations may be implemented after suitable modifications as relevant to the IMs for achieving an anticipated 40% maintenance possession time reduction.

7.5 New strategies for ballast tamping and cleaning

Over time, ballast settle and the track may be in need of an alignment. This can be done with a ballast tamper or tamping machine which is used to pack (or tamp) the ballast under the railway sleepers. With modern machines the track as a whole can be levelled, aligned and tamped, in order to achieve a smoother ride for passengers and freight and to reduce the mechanical strain applied to the rails by passing trains.

The ballast also becomes worn with time, and loses its angularity, becoming rounded. This hinders the interlocking or tessellation (from Latin tessera = square) of pieces of ballast with one another, and thus reduces its effectiveness. Fine pieces of granite, like sand, are also created by attrition, known simply as "fines". Combined with water in the ballast, these fines stick together, making the ballast like a lump of concrete. This hinders both track drainage and the flexibility of the ballast to constrain the track as it moves under traffic.

Ballast cleaning removes the fine material in the worn ballast, screens it and replaces the "dirty" worn ballast with fresh ballast. The advantage of ballast cleaning is that it can be done by an on-track machine without removing the rail and sleepers, and it is therefore cheaper than a total excavation. It can be carried out with e.g. a ballast cleaner which is a machine that specializes in cleaning the railway track ballast (gravel, blue stone or other aggregate) of impurities.

In total, over the whole life of track, tamping is the most frequent measure in maintenance. Rail exchange or a ballast cleaning is far more costly than tamping (factor up to 10 times more expensive). Together tamping and cleaning of ballast are usually the processes that are most costly in maintaining a railway track. The strategy for how often these maintenance procedures are carried out have a vast influence on the overall cost and quality of the track, Veit and Marschnig (2012).

In Automain 2.4 (2013) a case study showed that:
• Adequate maintenance windows lead to reduced track possession time that will be required on the long run. Around 5 hours would be optimum since the benefit of further increasing the maintenance is very small.
• Optimum possession length is required to reduce the impact of necessary non-value added tasks.
• The behaviour of the track becomes unreliable if the tamping cycle becomes too large or in the absence of one owing to increased number of spot failure with high variation in track possession time.
• Improvement of tamping speed gives the highest reduction in track possession time. 10% improvement in the tamping activities gives 11% reduction in the track possession time while 40% improvement gives about 35% reduction.
• The number of isolated geometry failure over time follows a power law process. Following this, an optimum strategy from track possession point of view is to have a tamping interval of 6 years.

Stone blowing is a relatively new process of track geometry adjustment that involves adding crushed rock to the surface of ballast under the lifted tie to shim the track. This is an alternative to tamping in which the ballast particles are rearranged to fill the void under the lifted tie. To

begin the stone blowing process, the existing geometry of the track is measured. Then, the vertical adjustment at each tie required to achieve the desired geometry is calculated. Next, the volume of stone to be blown beneath each tie to achieve this adjustment is determined. The stone is then placed by lifting the ties, inserting blowing tubes, and blowing the stone under the tie. Train traffic will then seat the ties as the blowing stone particles settle in. Stones used for blowing are to be selected to optimize their use for shimming., Selig (2014)

7.6 Improved sleepers to delay degradation

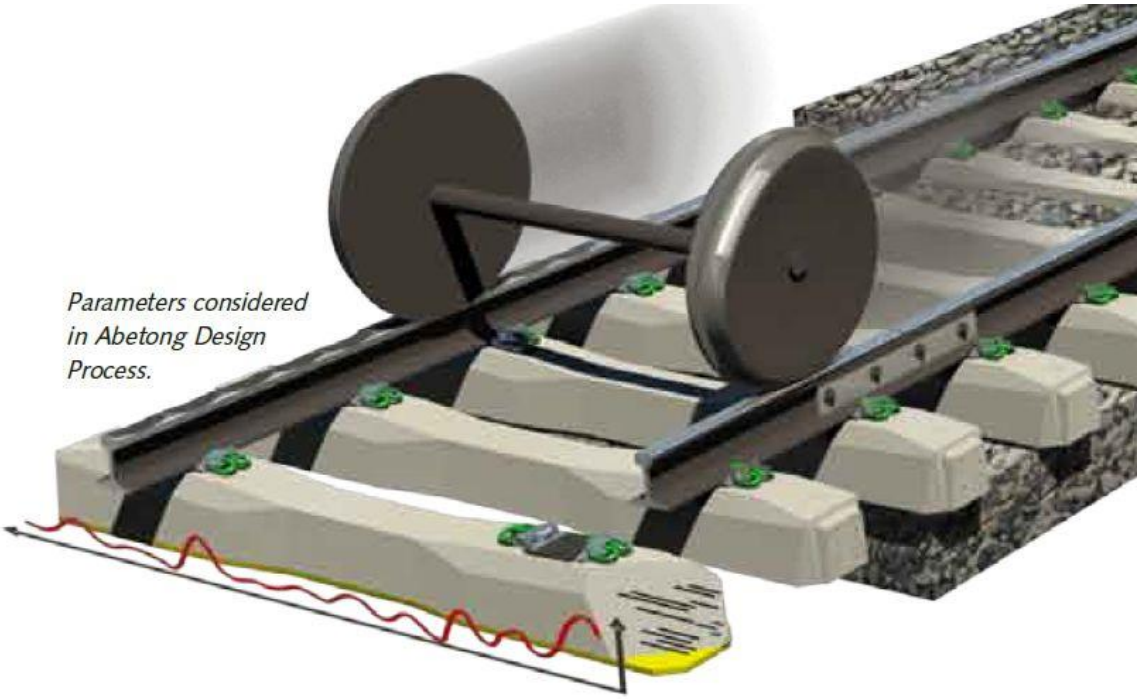


Figure 7-4 Example of parameters to be considered in concrete sleeper design. Abetong (2011).

The design of prestressed sleepers, see Figure 7.4, has been studied by Gustavsson (2002) and Charmec (2012). In a recent design guide, Bolmsvik et al (2011), conclude that the conditions at the wheel–rail interface and of the sleeper support have significant influences on the bending moments generated in the sleeper. Hence, regular and controlled maintenance of rolling stock, rails and ballast bed, are prerequisites to optimize the pre-stressed concrete sleeper.

An innovative new sleeper design is to enlarge the contact area between the sleeper and the ballast by applying a polyurethane layer on the bottom of the sleeper, so called under sleeper pads, USP. This also gives an additional elasticity to the track, Schneider et al (2011), Veit and Marschnig (2012) and Charmec (2012).

When prestressed concrete sleepers are produced it is important: (1) that cement of the correct quantity and quality is used and (2) that the curing process is properly controlled so that the temperature does not get too high and hence lead to delayed ettringite formation (DEF) after some years. This has happened in several countries time after another, see Tepponen and Eriksson (1987) and Famy and Taylor (2001). Recommendations have been given in CBI (2000), Scrivener and Skalny (2005) and EN 13230 (2009). Sleepers with limited partial cracking fulfil the load-carrying requirements. They are quite robust, and small cracks do not seem to influence the load carrying capacity or the fatigue resistance and it is first when the cracking is very severe that the load carrying capacity is reduced significantly, Thun et al

(2008). However, this may finally be the case with delayed ettringite formation and then most of the damaged sleepers have to be replaced. So, it is important to keep an eye on the production so that proper cement and a good hardening process are used.

7.7 Earth Work

A general presentation of geotechnical problems and solutions regarding stabilization and scour protection in Earthwork is given in Fell et al (2005) and in Selig (2014). Generally it can be said that predicted climate changes may cause intensified periods of rain and drought in various places on the earth. This will demand that traditional designs using classic 1000 year values need to be revised. Some special areas are discussed below, see Table 7.2. Other areas are treated in Smartrail (2014) as transition zones.

Table 7-2 Summary of technologies to improve earthwork – pros and cons

New Technology	Pros	Cons
Slope stability analysis for progressive failures	- More accurate predictions	- More complicated calculations
Lime-cement columns	- Stabilizes the ground	- Need special equipment to apply
Vertical drains	- Stabilizes the ground	- Need special equipment to apply

Sub grade (natural ground) - Degradation processes consist mainly of settlement due to creep, and erosion leading to lack of shearing strength and bearing capacity failure. A recent method to analyse the risk for progressive landslides in long natural slopes was presented by Bernander (2011). The problems with progressive landslides are especially important in countries with strain-softening clays as e.g. Scandinavia and Canada.

A fairly new method to stabilize the sub-grade is to use the so called lime–cement columns, see e.g. Larsson et al (2012).

Cuttings and Embankments - Degradation processes consist mainly of erosion and creep deformations leading to scour and loss of stability. A method to improve the embankment for the Ådal line in northern Sweden is presented in Müller (2010). The line is founded on sulphide clay and the stability was improved by using preloading in combination with prefabricated vertical drains. Additional information on vertical drains is given in e.g. Hansbo (1997).

Retaining/sea walls - Degradation processes consist mainly of erosion and creep deformations leading to scour and loss of stability.

8. Conclusion

The most cost-efficient way to extend the life of elderly infrastructure is to maintain it properly and to know and understand how it functions. This is a main responsibility of the Infrastructure Manager (IM) and cannot be outsourced without losing control over the assets, their value now and in the future and how and when to maintain/repair/upgrade/strengthen them.

Regarding bridges there is much to be gained using adequate assessment methods. The methods prescribed in our Eurocodes are often designed to give safe structures under all construction conditions. Regarding existing structures the codes are often conservative as there are fewer uncertainties regarding actual geometry and material properties than in not yet built structures. New probabilistic methods are useful and additional help may be gained from finite element models and studies of redundancy and robustness. Proof load testing, carried out in a careful way, may also give information that can be used to calibrate models. Full scale tests to failure of old bridges, which are to be taken out of service, is another possibility. They can give valuable results regarding the capacity of the kind of bridges that is tested. An example of this is given in Appendix A where the test of a steel truss bridge is presented. It is shown that the bridge had a much higher load-carrying capacity than what was predicted by ordinary code methods.

One simple assessment process presented in chapter 4.4 made it possible for the IM in Sweden to continue to use two bridges through central Stockholm without extensive repair/replacement. They hereby saved 3M€ to be compared to the assessment cost of 0,2 M€.

Strengthening of structures is another important possibility to extend life. Some examples are given in chapter 5 on the use of post-tensioning and other examples are given in ML D1.4 (2014).

Methods for tunnels are presented in chapter 6 and for track and earth work in chapter 7.

Appendices – A-E Table of Contents

A. Test to failure of Åby Bridge

A.1 General

A.1.1 Introduction

Bridges have often been replaced on theoretical assumptions that they have reached the end of their lifespan. After safety considerations, cost is the next most important factor when it comes to replacing bridges. Recently the environmental aspects have also become a factor in decision-making. The Swedish Traffic Administration, Trafikverket, has declared that it intends to increase its work with Life Cycle Analysis (LCA), TRV Climate (2012). For bridges, this will lead to a greater number being assessed to determine their actual capacity and thus determine what remedial action is most effective, whether it is repairing, upgrading or replacing the entire structure.

The assessment of an existing bridge can be performed with different levels of accuracy and effort. Generalized load-models might often be sufficient in order to verify if the load capacity is good enough or at least serve as an initial estimation. Conservative assumptions may, however, lead to an unnecessary high safety level and a low loadbearing capacity. This may be increased also by high safety factors regarding materials. Together this may give such a low capacity that it is concluded that the bridge needs to be replaced. The opposite, overestimating the capacity can however be catastrophic. Failure of a bridge results in major delays and possible human casualties if the failure is brittle.

In 2012, the Swedish authorities owned 3842 railway bridges and 145 tunnels and over 13,642 km of railway tracks, Du & Karoumi (2013). A substantial part of the bridge stock in Sweden is older than 50 years, as shown if Figure A.1-1, at the same time, loading and the traffic intensity on our existing bridges are constantly increasing. This is not a unique situation for Sweden but rather a problem for most countries in the world.

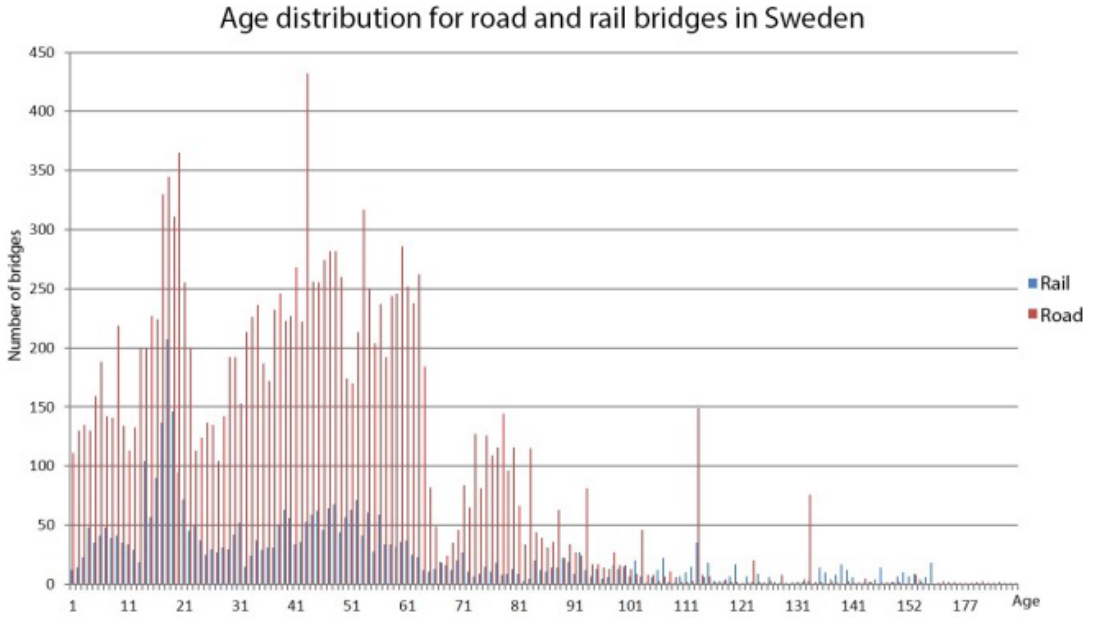


Figure A.1-1 Age distribution for bridges in Sweden (Du & Karoumi (2013) based on data from Trafikverket)

Figure A.1-2. The Aby River bridge in its original position

An increase in traffic, both in regard of weight and intensity significantly reduces the lifespan of the steel bridges due to fatigue. The load models and estimation of the fatigue capacity is not as straight forward and clear as for Ultimate Limit State and Serviceability Limit State which leads to an uncertainty which is needed to be taken into consideration with proper safety level.

Even if old bridges theoretically have served their lifespan due to fatigue or insufficient load capacity as a result of increased loads and increased traffic they are not necessarily in need of being exchanged. With the help of new knowledge together with refined calculations and inspections it might be possible to prolong the lifespan of these bridges. In order to ensure the continued safety of the bridge it is often required to monitor the structure by preforming measurements.

The bridge over Aby River in Figure A1-2 is one of these bridges that conservative calculations and bad performance of transition zones motivated a replacement in 2012. In order to gain knowledge of its structural behavior, the old bridge was moved to temporary supports close to its original position in order to be tested for both static and dynamic loading.

The overall aim of this study is to identify critical elements in the Aby River-bridge and to develop a method for assessing these. This is followed by the identification of the measurements that characterize its behavior in order to create a method for non-destructive assessment of similar bridges in order to be able to evaluate if, and what measures are needed in order to ensure the continuing safety of those bridges. The reason for the particular interest of this bridge is that there is an almost identical bridge over Rautasjokk, located on the iron ore line in the northern parts of Sweden. If the measurements from the Aby River-bridge can provide information that the bridge over Rautasjokk doesn't needs to be replaced, great savings can be made.

A.1.2 Previous work

Before the Aby-River Bridge was taken out of use in the autumn of 2012, measurements were performed while it was still in service. Maximum strain ranges of $\Delta\varepsilon = 270*10^{-6}$ were recorded corresponding to stress ranges of $\Delta\sigma$ = 55 MPa, see Häggström et al. (2014), Morena (2013). Since the live measurements were less comprehensive than the final tests, it served as a step towards planning the full scale tests. As train loads are known it is possible to estimate the

dynamic response of the bridge as well as transforming the static load scenario to the shape of train sets, Blanksvärd (2012, 2013).

Simulations of the intended load case were performed prior to the test using a Finite Element Model created in ABAQUS. The model was made as a shell model with the limitations of not assigning any constraints at the joints; therefore all connections are fully rigid.

Photographic strain measurements were performed at the joint between the longitudinal stringer beams and the crossbeams, in order to evaluate the degree of constraint in the connection between the stringers and the crossbeams which according to calculations were the critical hotspot which lead to the exchange of the structure. The evaluation of these results is reported in Elhag (2013).

A.1.3 Geometry and material

The bridge is a 33 metre long steel truss railway bridge built in 1951 and designed according to trainloads of type F46 which corresponds to 25 tons axle load, which also is the present load on the railway. The location for the bridge is in a rural environment approximately 50 km from the coast. Girders and connections in the bridge are partially riveted and partially welded. The steel used in the superstructure is described in Table 1 with material properties according to TRV BSV (2013). Compared with the steel materials used today the variation of material properties from the 1950s is far greater, Larsson & Lagerqvist (2009). It is therefore conservative to use the design values for modelling when estimating the structural behaviour which is the reason for the higher values being chosen.

Table A.1-1. Material properties used for evaluation

Part	**Material**	f_{yk} MPa	f_{uk} MPa	f_{yk} MPa Used for modelling
Stringer beams, verticals, diagonals	S1311	240	360	345
Main truss, Cross girders	S1411	270	430	345

A.1.4 Experimental work

Test setup

After the bridge was taken out of service it was put on temporary supports and loaded by two jacks attached to a girder that distributes the load as four equally distributed point loads. In order to be able to archive the force needed to load the bridge to failure the jacks need to be attached to solid rock. During the phase of drilling the intended precision was not reached which is the reason to why the point loads aren’t symmetrical around the centre of the bridge.

Measurement program

This project is a unique opportunity to gain knowledge of the structural behavior of this kind of bridges. Since the full scale testing was performed under a limited amount of time, the program for measurement was made as comprehensive as possible. The limitations for the test equipment were 145 channels and 141 were used during the tests. In addition to this, photographic strain measurements were made, see Figure A.1-3..

The different sensors were divided between 72 strain gauges, 46 LVDT's, 8 temperature gauges and the photographic strain or photometric measurements, Aramis-system, GOM (2009). Discussion will be limited to the measurements taken for the global analysis of the main truss.

The measurements consist of 18 different predefined load series with the last three undertaken with the rail removed and the final one to failure. Figure A.1-4 shows how the load was varied over the different series of measurements. The sensors placed on the main truss for the global analysis are illustrated in Figure A.1-5 and further described in Table A.1-2.

Figure A.1-3: a) The Aramis setup for the photometric b) Cables from sensors for measurements c) Hydraulic jack and one of the load distributing beams

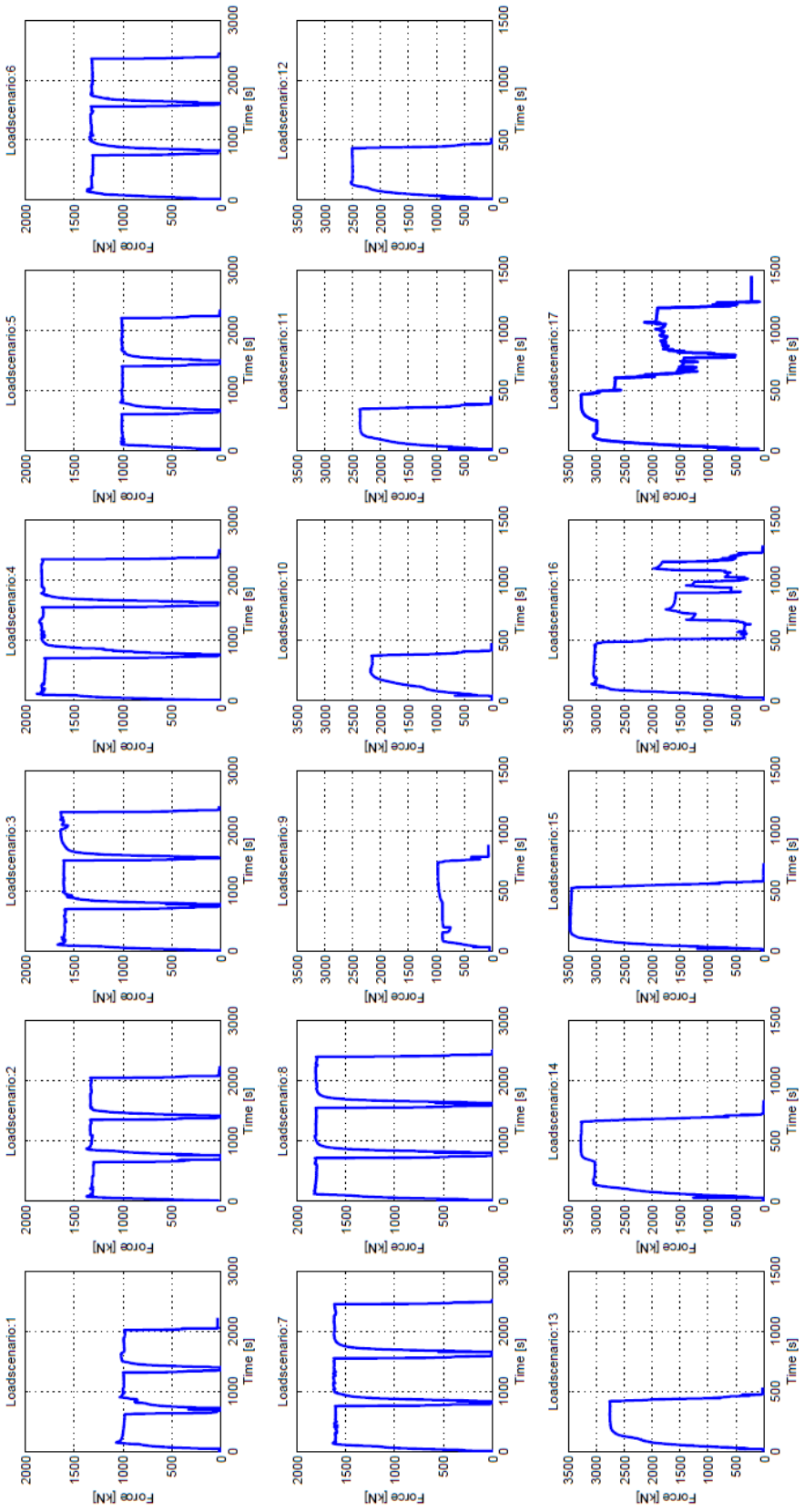


Figure A.1-4 Load scenario with loads on the vertical and time on the horizontal axis.

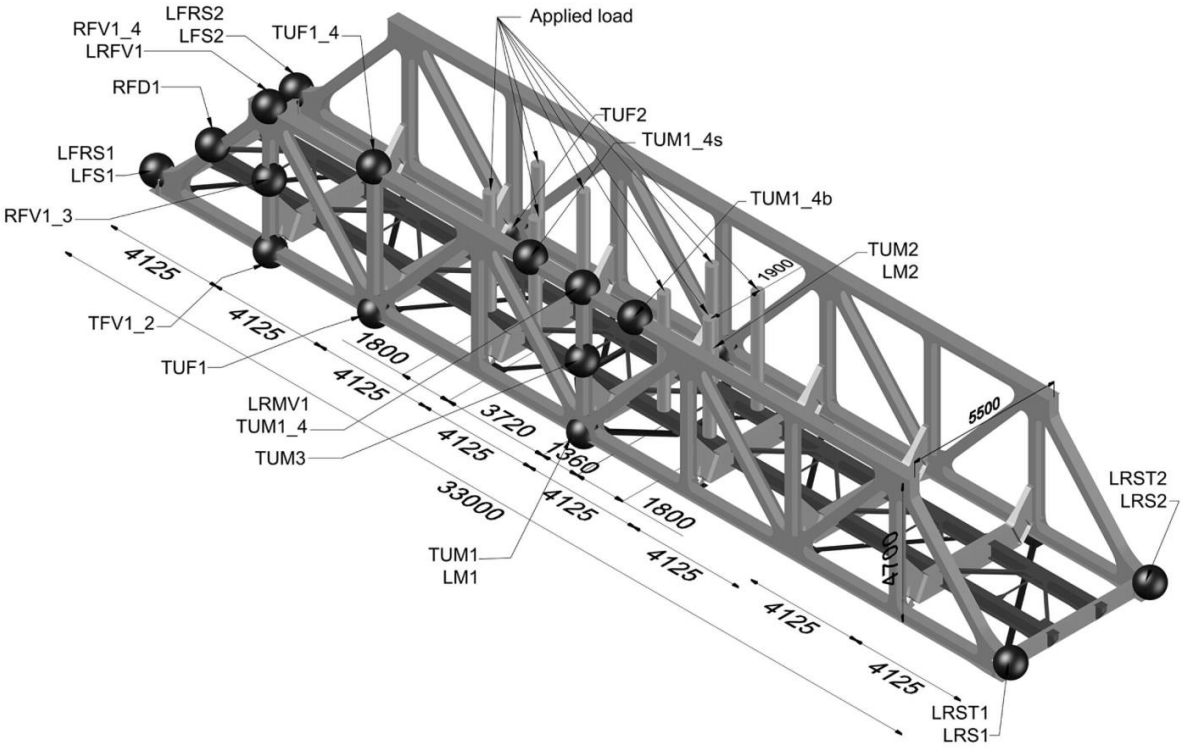


Figure A.1-5. Position of sensors for global measurements, with the description in Table A1-.2

Table A.1-2. Sensors for measuring global effects

Sensor	**Description, deflections, mm**	**Sensor**	**Description, strain gauges, μm/m**
LFRS1	Rotation at support	RFD1	Three directional
LFRS2	Rotation at support	RFV1 3	Three directional
LFS1	Settlements at support	RFV1 4	Three directional
LFS2	Settlements at support	TFV1 2	Main direction
LM1	Deflections at midspan	TUF1	Main direction
LM2	Deflections at midspan	TUF1 4	Main direction
LRFV1	Horizontal deflection	TUF2	Opposite side of TUF1
LRMFV1	Horizontal deflection	TUMI	Both top and lower flange
LRS1	Settlements at support	TUMI 4	Mid span
LRS2	Settlements at support	TUMI 4s	Main direction
LRST1	Rotation at support	TUM2	Both top and lower flange
LRST2	Rotation at support	TUM3	n the vertical at mid span

Since buckling of the top frame was the likely failure mode, the horizontal displacement was monitored; the sensor is shown in Figure A.1-6a. The positions of the both sensors for measuring the horizontal displacement are shown in Figure A1-5. Figure A1-6b illustrates the sensor measuring the global deflection at mid span. At the same point the strains were measured for the lower frame for both the upper and lower flange.

Figure A.1-6. a) Measurements of the horizontal deflection due to loading (LRFV1) b) Measurements of deflection at mid span (TUM2) and strain (TUM1) c) Measurements of settlements and rotation at support d) Strain gauge at the top frame (TUF1-4)

By measuring the strain of two or more points on a cross section it is possible to calculate the section forces caused by a specific load. Since the bridge was placed on temporary supports and loaded to failure, settlements were likely to occur. In order to be able to adjust and get correct results for deflection it was necessary to measure the settlements at the supporting points which are shown in Figure A1-.6c as well as the measurement of rotation.

A.1.5 Results and analysis

Estimations made before testing by the Finite Element Software Abaqus indicated that the global failure would occur approximately at 9MN and that it would be caused by buckling of the top frame in the main truss. Before buckling of the frame there would be some yielding and redistribution of forces in the structure. The results in Figure A.1-.8 and A.1-9 show both the static non-destructive testing and the final test to failure. For the static load scenarios one point is taken for each terrace point, which means that for load scenario 1-8 three points of measurement are taken and for 9-17 it is just one, whereas the failure test are shown in its entirety. Values from the sensors are given as a function of the total force induced by both of the hydraulic jacks. The jacks are manually controlled, kept, as far as possible, at an equal level. Due to the high loads needed in order to load the bridge to failure, no load cells could be used, but the force is accounted as a function of the oil pressure and the area of the cylinder. For the final test the stroke of the cylinders was not enough, which meant that the cables were wedged so that the jacks could be repositioned. The repositioning means that the load is consistent but that the oil-pressure is reduced to zero. Since the results are presented as a function of the total force which is based on the oil pressure these are cut out in order to create a diagram that gives a clearer picture of what is happening, Figure A.1-7. In Figure A.1-7 the expected results for the simulation are also displayed. Looking at the diagram, one can conclude that the simulation corresponds well to the measured results regarding global deformation. This indicates that the simulation has the correct stiffness of the structure but once it starts to yield it differs to some extent. The reason for this difference might be an

incorrect assumption of the material properties once it starts to yield or that the real bridge has a greater ability to redistribute forces.

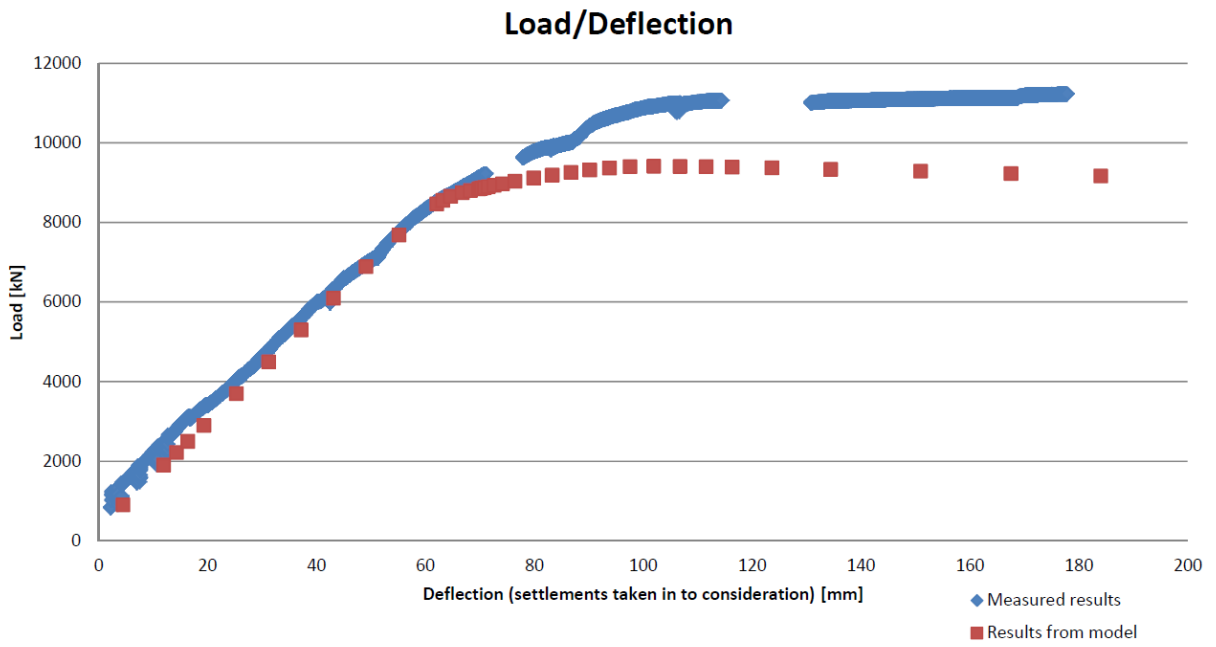
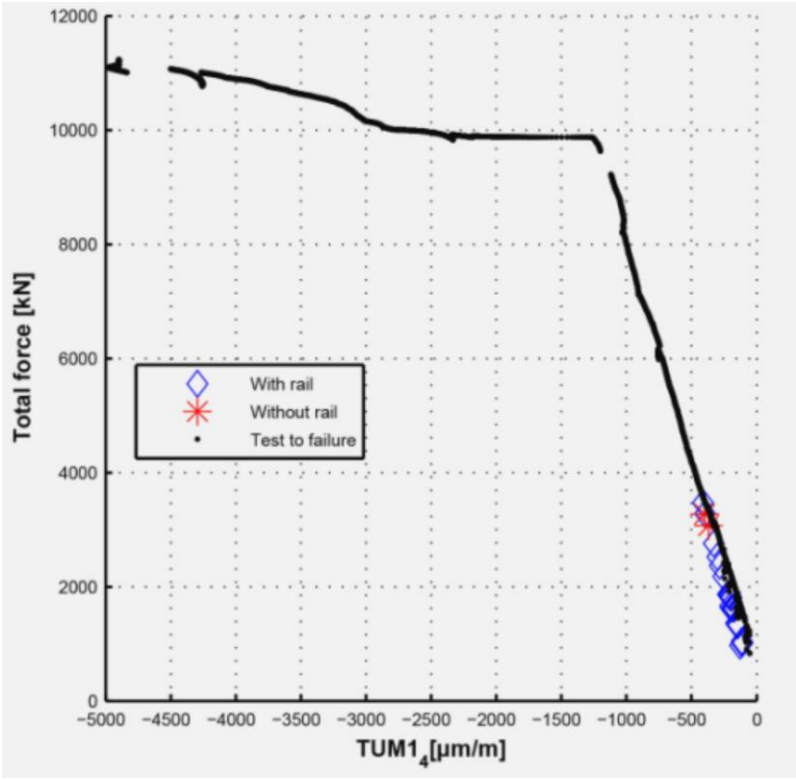


Figure A.1-7. Deflection at mid span (sensor LM2). The blue dotted line is corrected due to settlements. The figure also show the expected outcome based on the simulation (red boxes) and the parts that are cut out because of repositioning of the jacks (blue).



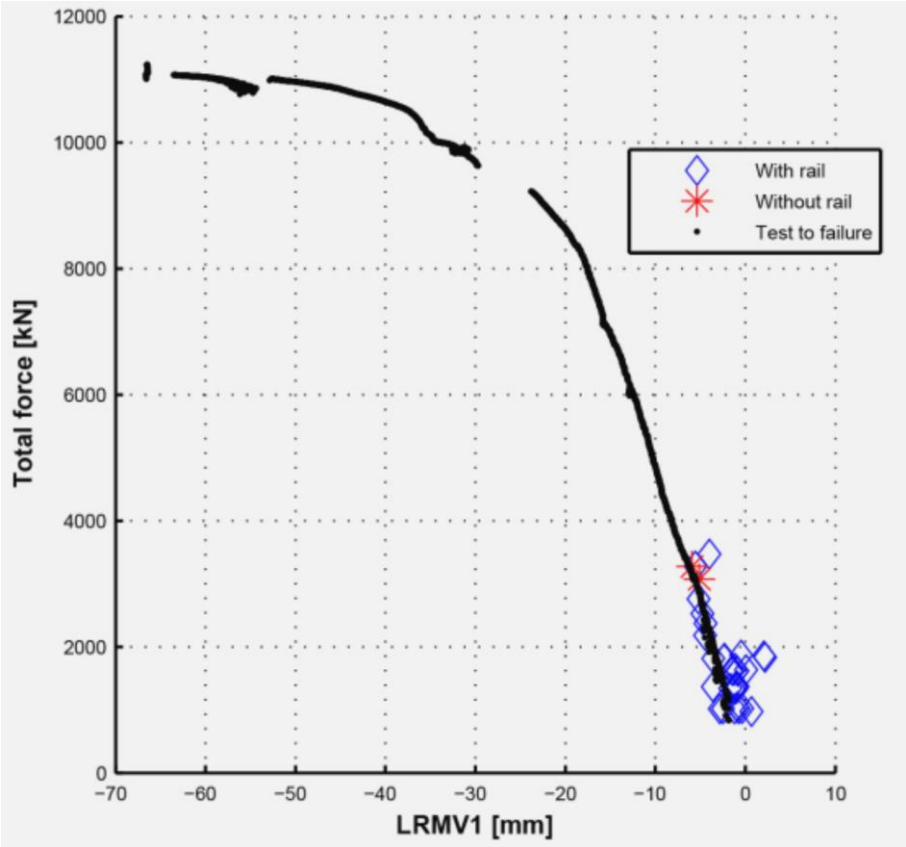
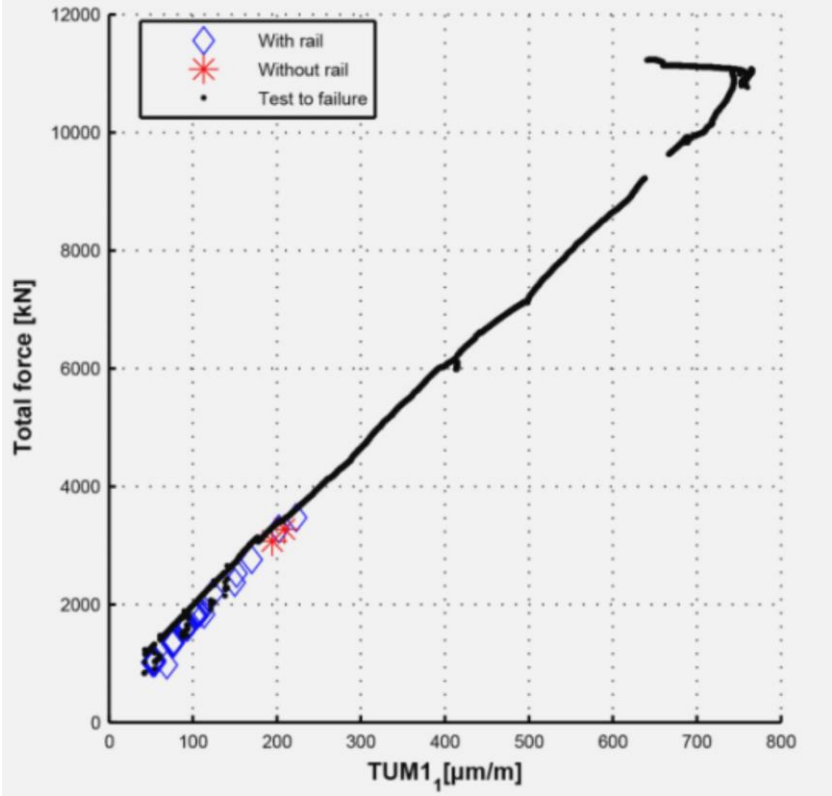


Figure A.1-8. a) Strain as a function of total force in the upper frame at mid span

b) Horizontal deflection for the top frame



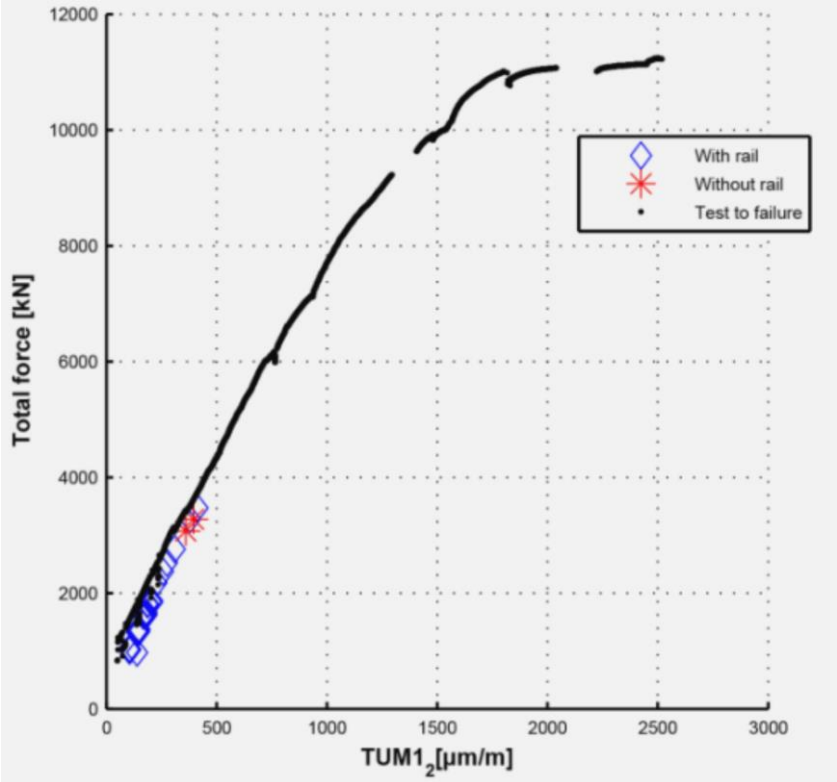


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Figure A.1-9. a) Strain as a function of total force in the upper flange of the lower frame at mid span b) Same position as Figure A.9a but for the lower flange

As the load increased the top frame eventually started to yield. By observing Figure A.1-8a it appears that the yielding starts at approximately 10MN. Since it is in compression without

constraints in the horizontal direction, buckling will occur as a consequence as seen in Figure A.1-8b and A.1-10b. By observing Figure A.1-9a and b it is clear that that both the flanges in the lower frame are in tension but that the moment induced by the concentrated loads leads to a great difference with respect to strains. It can be inferred that the lower flange yields towards the end of the test at the same time as the strains are decreased for the upper flange, which is the result of the buckling of the top frame. When buckling occurs, the forces in the top frame will not increase significantly and as the applied load is increased the lower frame will be subjected to a greater amount of bending instead of tension.

Besides the global buckling mode of the top frame there were local failures underneath one of the load distributing beams Figure A.1-12a. The web could not withstand the high concentrated force which resulted in local shear buckling as well as local buckling due to patch loading.

Figure A.1-10. a) The local failure of the longitudinal stringer beams,

Figure A.1-10. b) The global failure mode, buckling of the top frame in the truss

A.1.6 Discussion, Conclusions

The failure that eventually prevented the bridge from taking more load was the buckling of the top frame. This was also the estimated outcome according to the simulation performed on the bridge. Besides buckling of the top frame, there was local failure in the web right underneath where the load was applied. The local failure might be interesting to study from a scientific point of view, but is not relevant in respect of deciding the capacity of the bridge since the trainloads will be more distributed. It is highly unlikely that problems related to the main and ultimate limit state would limit the capacity of the bridge, but rather the issue of fatigue which was also concluded by the traffic authorities.

A.1.7 Future work

The evaluation of the measurements done on the Aby-River Bridge is an ongoing project which is not yet finished.

In order to refine the material parameters of the Finite Element Model and to be able to evaluate the results more accurately, it is necessary to know the real properties of the material. The next step towards finding the real material properties are to create specimens to be tested in the lab. The specimens will be subjected to tensile tests in order to find the static material parameters. There are also plans on making tests on the toughness of the material, in order to be able to evaluate ductility and fatigue capacity. Beyond the material testing, tests of fatigue will be performed on selected bridge parts. It is also planned to perform measurements on the Bridge over Rautasjokk. This is to verify assumptions and conclusions made on the Aby Bridge. Further it gives an opportunity to evaluate the dynamic response of the bridge, since it will be subjected to live loading.

A.2 Direct reliability based analysis

Some of the advanced methods presented in this deliverable will be applied to the Aby bridge, presented in A.1, a steel truss railway bridge that was located in Northern Sweden (figure A2-1). The bridge had to be replaced by a new bridge. Thus, previous to the replacement, the bridge was removed from its original position and placed nearby in order to carry out a test up to failure (figure A2-2).

Figure A2-1 Aby bridge in its original location

Figure A2-2 Aby bridge removed from the location and ready for load testing

The advanced assessment method that will be shown in this appendix is the application of reliability method to calculate the bridge safety measured by the reliability index.

A.2.1 Description of the bridge

Aby bridge is a steel truss simply supported structure with a span-length of 33 m. According to the design specification the steel quality is of types 1311 and 1411 (Swedish specifications) with a mean yielding strength of 220 MPa and ultimate strength 360 MPa.

More details about original drawings, with the bridge geometry and bar connections can be found in the appendix A of deliverable ML-D1.2 (2013).

A.2.2 Bridge model

The numerical non-linear model of the bridge was developed using the ABAQUS software. The full model was developed by Professor Yongming Tu from Southeast University, SEU, in Nanjing (China) during his stay at LTU. The FEM is made of shell elements taking all the connections as rigid. In figure A2-.3 is presented a perspective of the FEM model. The general mesh is presented in figure A2-4 and a detail of one of the joints is shown in figure A2-.5.

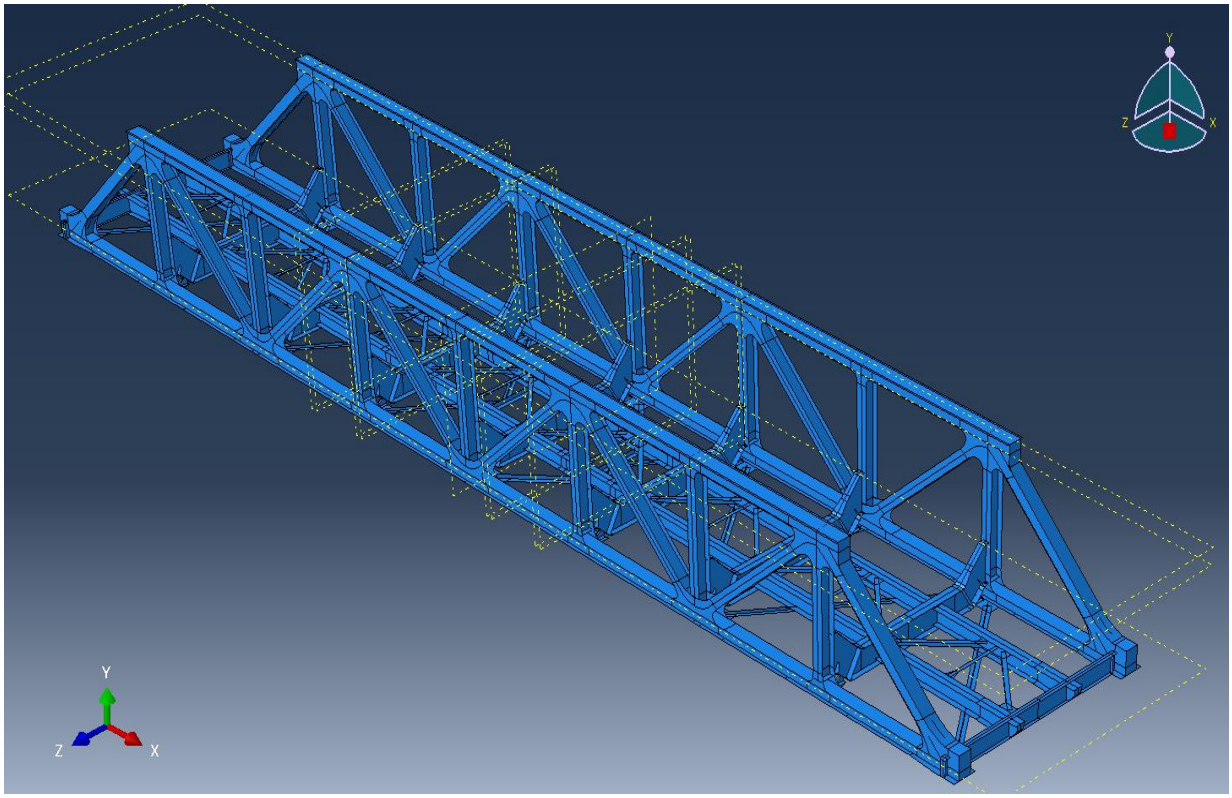


Figure AA2-3 General view of the model of Aby bridge

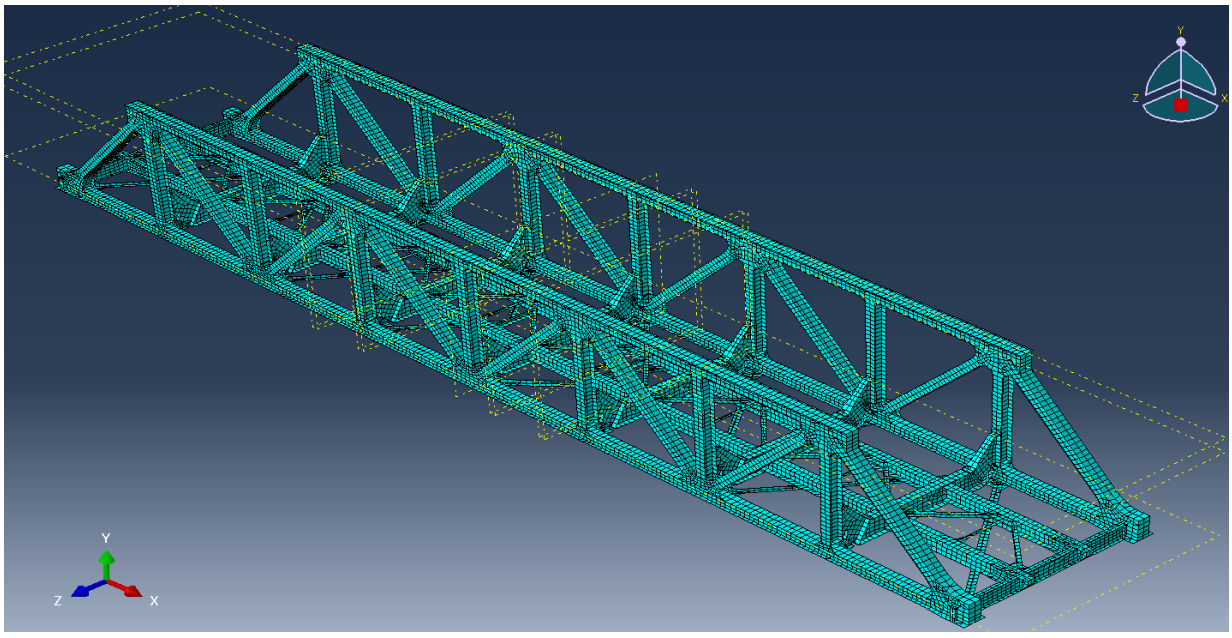


Figure A2-4 View of the FEM mesh

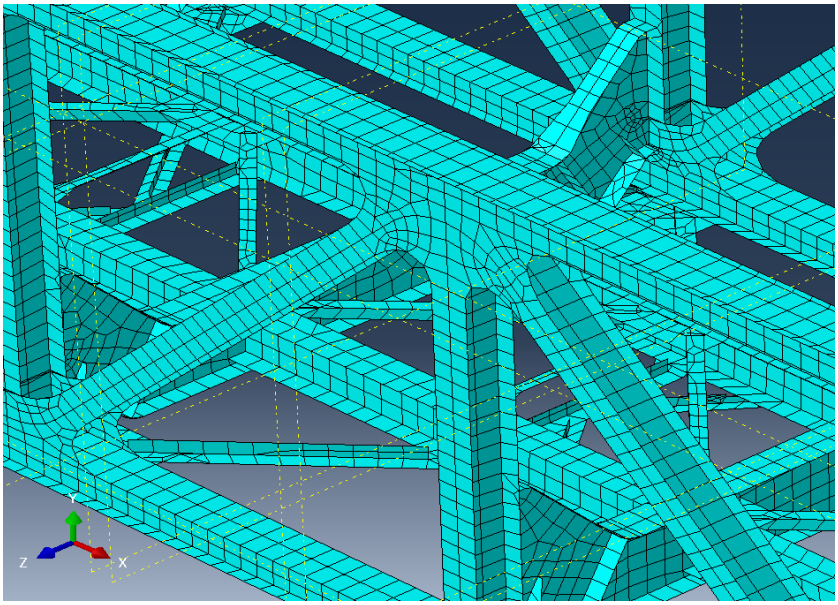


Figure AA2-5 Detail of mesh at the joints

The strain-stress relationship for the structural steel is considered as bi-linear with a hardening modulus (H) in the second part of the curve.

The following actions were considered in the analysis: self-weight of the structure, additional permanent loads and live load on the railway track including impact (UIC train load model).

A.2.3 Variability of parameters

In order to take into account the complex non-linear behaviour of this bridge, the FE model is also very complex and needs a long run time for one non-linear analysis. Trying to reduce the computing time and simplify as much as possible the assessment, the number of simulations to characterize the non-linear behaviour of the bridge from a probabilistic approach must be reduced to a minimum. The number of simulations is highly dependent on the number of variables considered as random. For this reason, the pre-selection of the variables to be considered as random was performed using some previous knowledge and engineering judgement. The variables that describe the geometry were considered as deterministic. Also the variability of elasticity modulus was considered to be small and negligible.

The random variables considered were the yielding strength and the hardening slope (hardening modulus) of the structural steel. The elasticity modulus was considered deterministic with a value equal to 210 GPa and a total correlation is assumed between the yielding strength and the ultimate strength, taking the last one as 1.636 times the value of the yielding stress. The value 1.636 is the ratio between the ultimate and yielding strengths (360 over 220) considered in the design. With all these values defined and according to the bilinear shape of the curve, once the hardening modulus is defined, also the ultimate strain can be obtained, completing in this way the full stress-strain relationship. The statistical models for yielding strength and hardening modulus are presented in table A2-1, jointly with the statistical models of the self-weight (no additional permanent loads due to ballast, etc. are considered), the railway traffic loads and the impact. As it can be noticed, for the purpose of simplicity all the random variables not related with the resistance were assumed to have Normal (Gaussian) distribution. The parameters of all the random variables were defined according to the data obtained by various authors and available in the literature

Table A2-1 Definition of random variables considered in the analysis

Random variable	Unit	Mean	COV (%)	PDF
Yield strength	MPa	220	10	Lognormal
Hardening modulus	MPa	1080	25	Lognormal
Self-weight	Kg/m^3	7800	3	Normal
Railway traffic load (concentrated)	kN	103.5 (4 loads per rail)	10	Normal
Railway traffic load (distributed)	kN/m	31.7/rail	10	Normal
Impact factor	-	1.10	25	Normal

The bridge is assessed to failure by ultimate loading under the effect of the UIC train load model (axle load (250 kN) and distributed load (80 kN/m). The values of railway traffic loads were obtained from the UIC train load model considering that the characteristic axle load (4 x 250 kN) and distributed load (80 kN/m) per track corresponds to the 98-th percentile of the PDF of the railway load assuming Normal distribution. Considering this assumption, the mean value for the axle loads (207 kN) and distributed load (63.4 kN/m) were obtained per track. This gives, finally, a value of 103.5 kN and 31.7 kN/m in each of the two rails of the track.

A.2.4 Results of the analysis

The safety assessment of the bridge was performed according to the following methodology. First a set of values of resistance random variables (yield strength and hardening modulus) were generated, using the Latin Hypercube sampling method according to the parameters presented in table A2-1. The number of simulations was decided as 100, small because the FEM model is very complex and each simulation takes a long computer time, but large enough to get accurate results as the Latin Hypercube algorithm is used. A total of 100 yield strength and hardening modulus were generated using LHM considering both variables as independent. This is a reasonable value taking into account that only 2 variables are considered as random. As the ultimate strength and strain are obtained from the former values, the analysis showed a coefficient of variation for the ultimate strength of 14 % and a mean value of 0.1329 and coefficient of variation of 29 % for the ultimate strain. The correlation coefficient obtained for ultimate strain and yield strength was in the order of 0.4. Later, 100 structural non-linear analyses up to failure were performed for each combination of generated variables (100 simulations). Finally, the results were evaluated statistically and the reliability index was calculated. In the simulations, the railway traffic loads and impact were considered as deterministic. Their variability was decided to be considered in the further calculation of the reliability index.

Figure A2-6 presents the histogram of the calculated load factor by which the railway traffic loads have to be multiplied to cause the bridge failure (maximum load). A Normal probability distribution function shows a very good fit of the obtained results. This can also be seen in the graph of the simulation results on Normal probability paper as presented in figure A2-7.

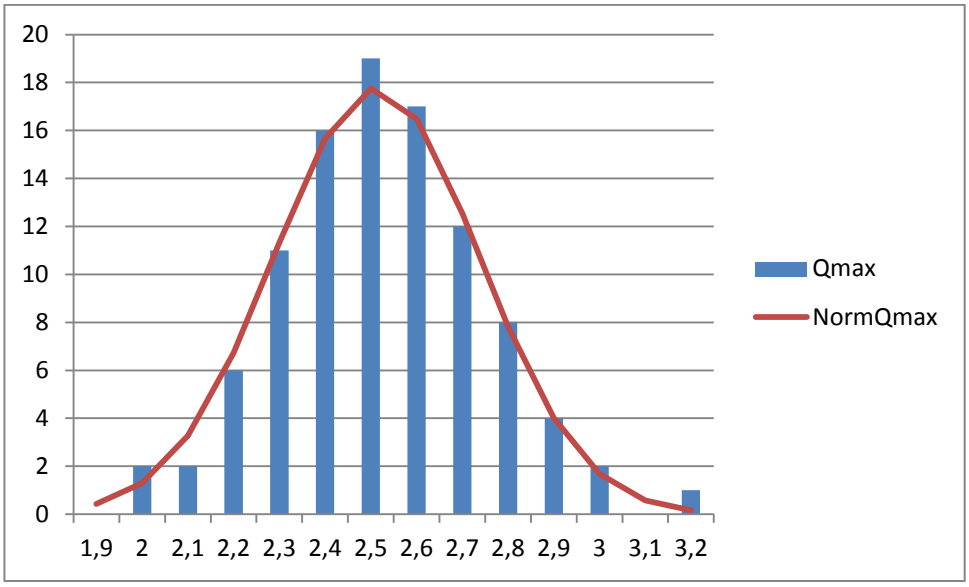


Figure A2-6 Histogram of Maximum Load Factor and Normal fitting

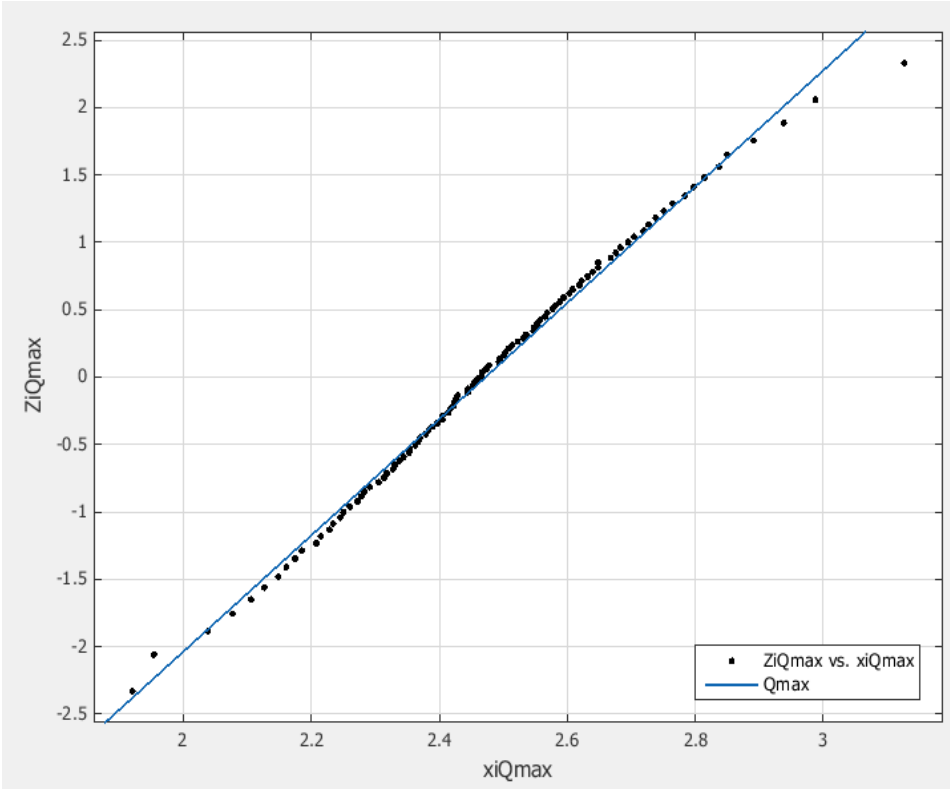


Figure A2-7 Simulated data for maximum load factor in Normal probability paper

The stress state corresponding to the maximum load for one of the simulations (the one corresponding to simulation number 95 with a yield strength of 256.76 MPa, ultimate strength of 420.15 MPa and ultimate strain 0.1945) is presented in figures A2-8 and A2-9. In the first

one, the compression and tension along the longitudinal axis are shown. Figure A2-9 shows the general Von Mises stresses.

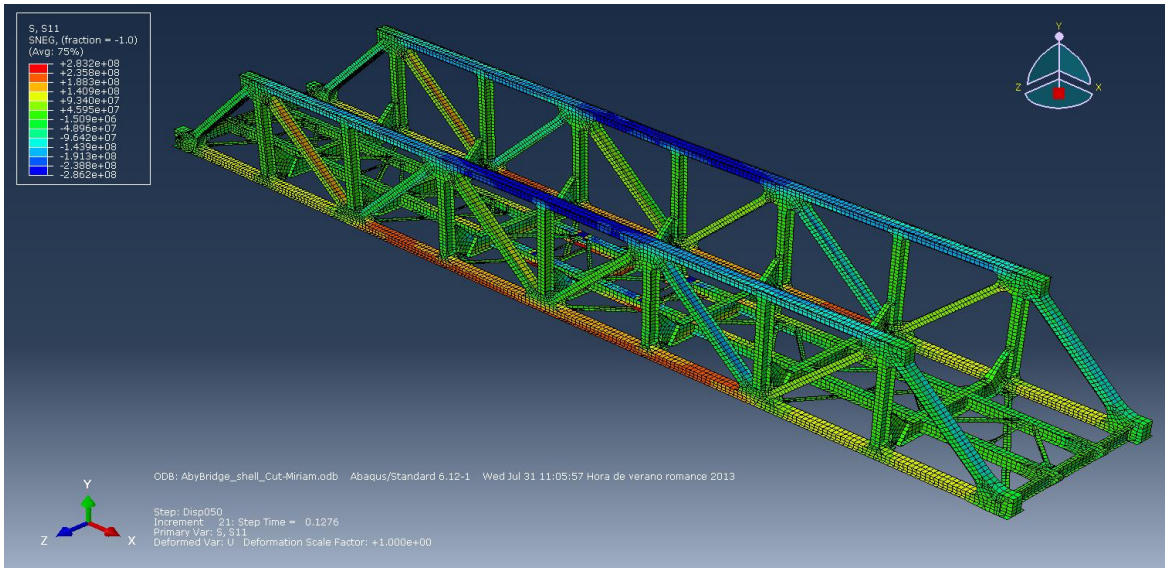


Figure AA2-8 Stresses in the longitudinal direction corresponding to maximum load

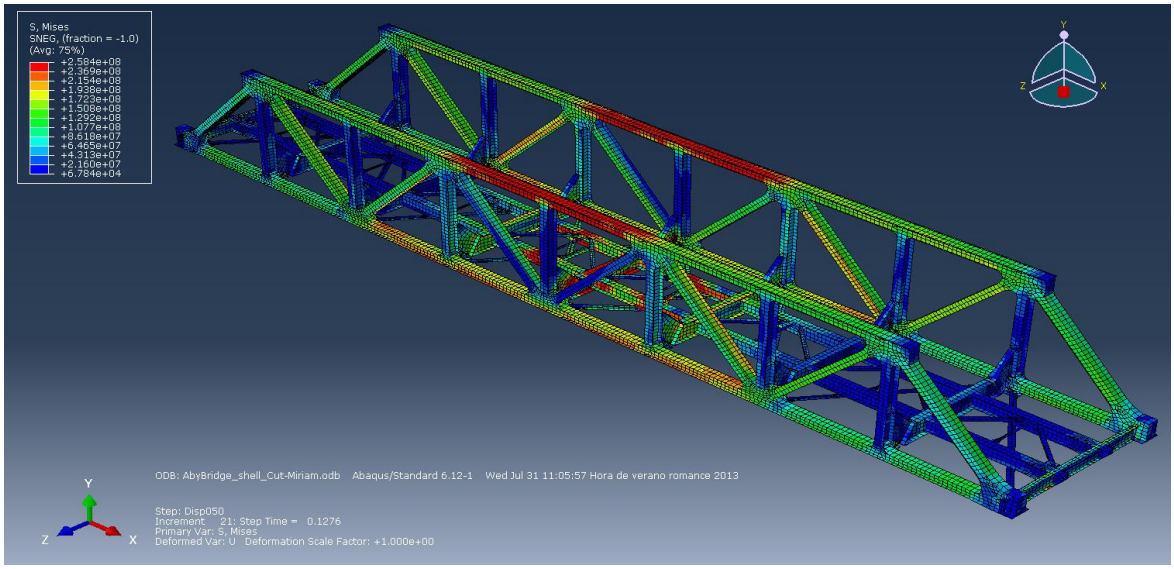


Figure A2-9 Von Mises stresses corresponding to maximum load

The deflections in the bridge corresponding to the maximum applied load are shown in figure A2-10. The load applied versus deflection at mid-span can be seen in figure A2-11.

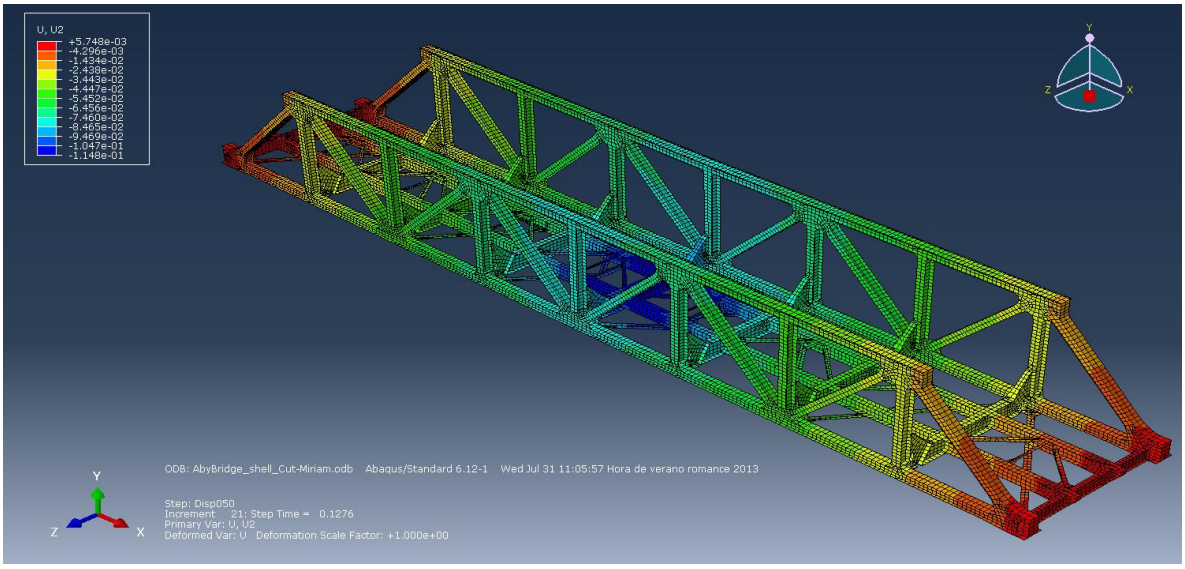


Figure A2-10 Deflection corresponding to maximum load

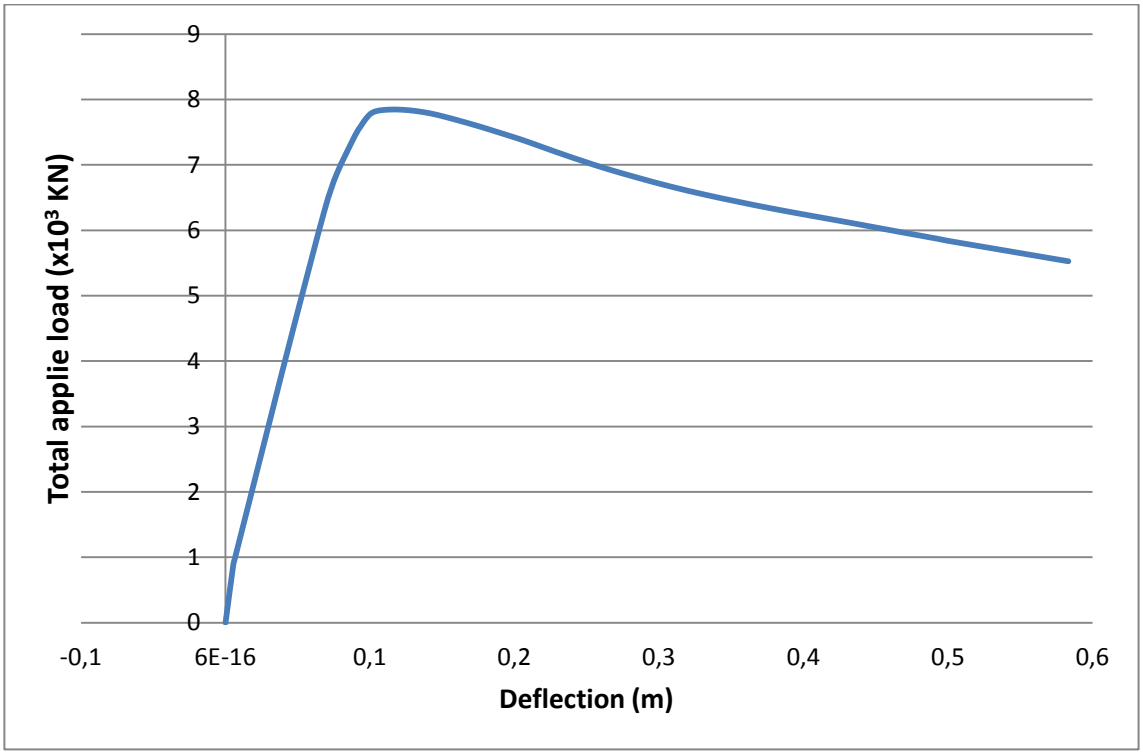


Figure A2-11 Load- deflection result for simulation N. 95

After reaching the maximum load, the bridge presents a post-peak behaviour as shown in figure A2-11, arriving to the ultimate load with corresponding stress state as presented in figure A2-12. The figure also shows the buckling on several members leading to the final failure of the bridge.

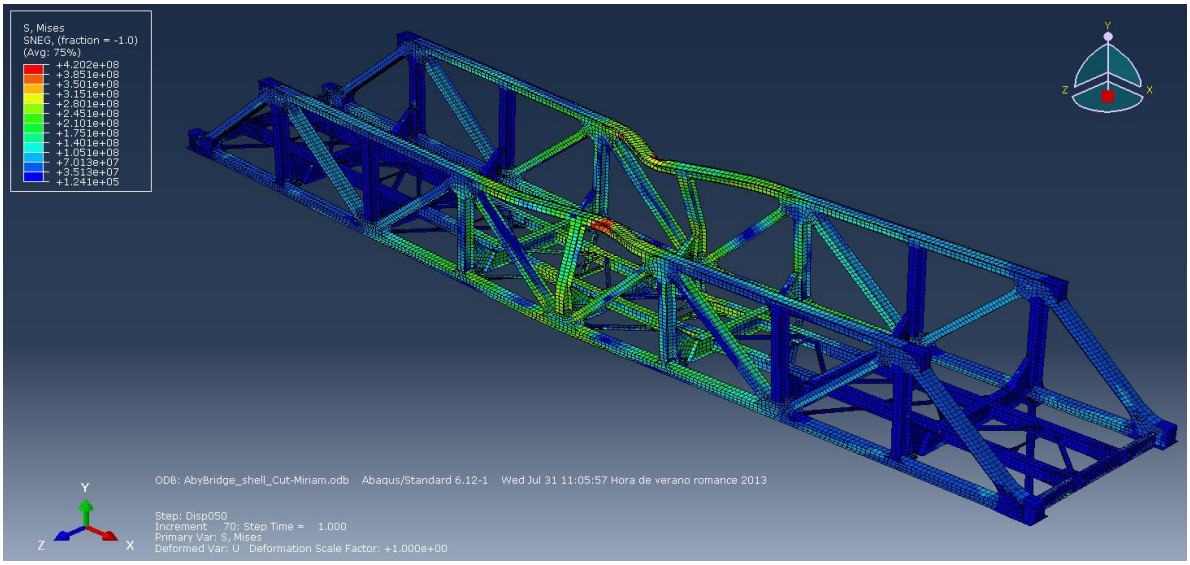


Figure A2-12 Stress (Von Mises) corresponding to ultimate load for simulation N. 95

A different failure mode is shown in figure A2-13 which corresponds to the simulation number 1, where the material values are the following: yield strength equal to 167.88 MPa, ultimate strength 274.71 MPa and ultimate strain 0.0998.

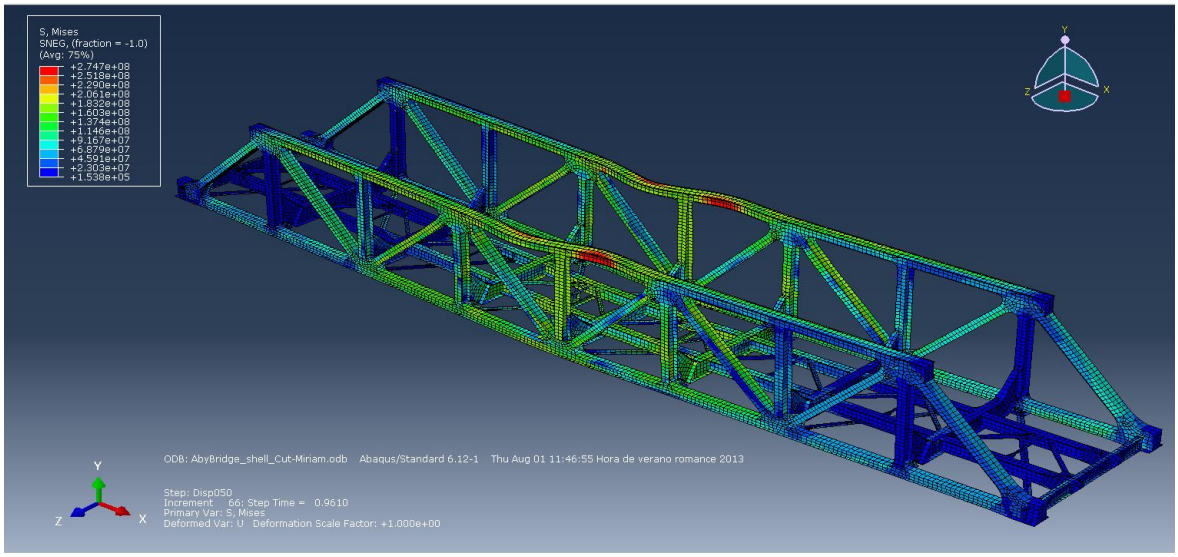


Figure A2-13 Stress (Von Mises) corresponding to ultimate load for simulation N. 1

The stress states and load versus deflection curve for this simulation N. 1 are shown in figures A2-14 to A2-16.

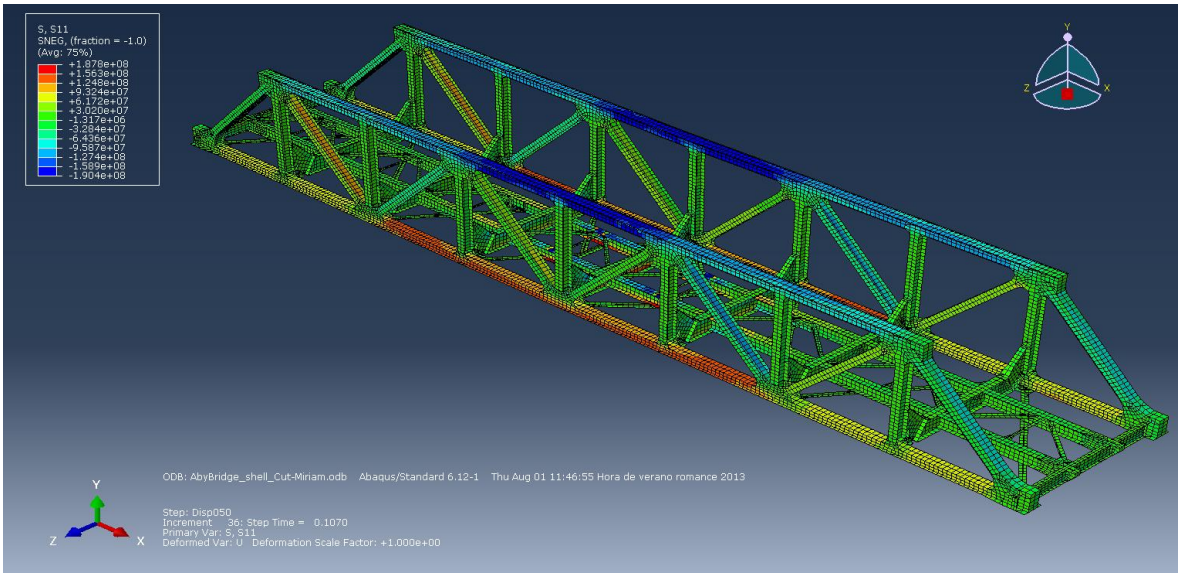


Figure A2-14 Stresses in the longitudinal direction corresponding to maximum load (simulation N.1)

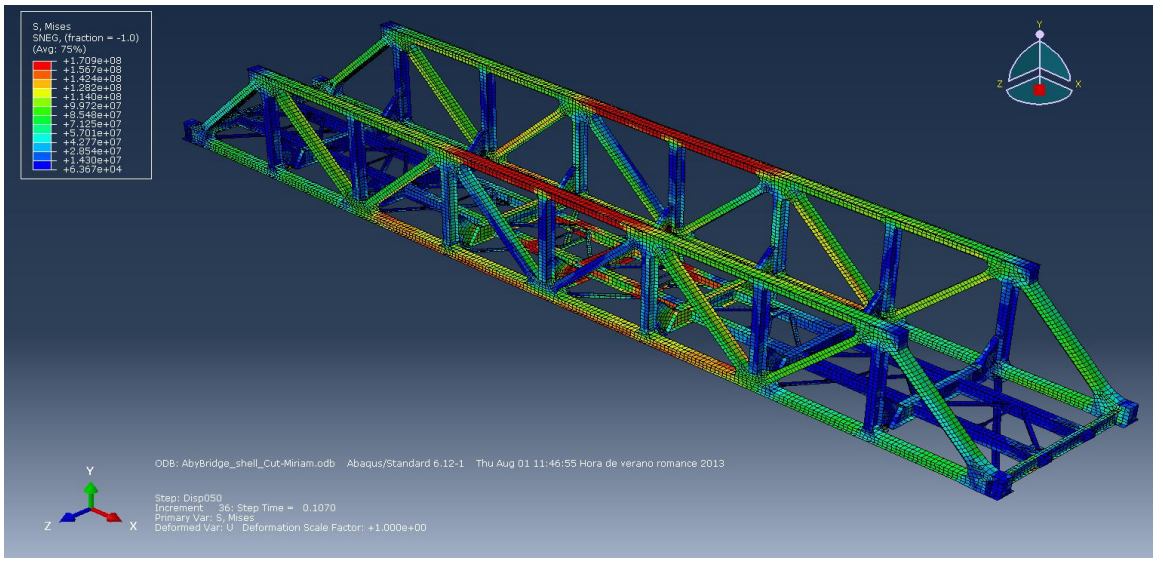


Figure 2-15 Von Mises stresses corresponding to maximum load (simulation N.1)

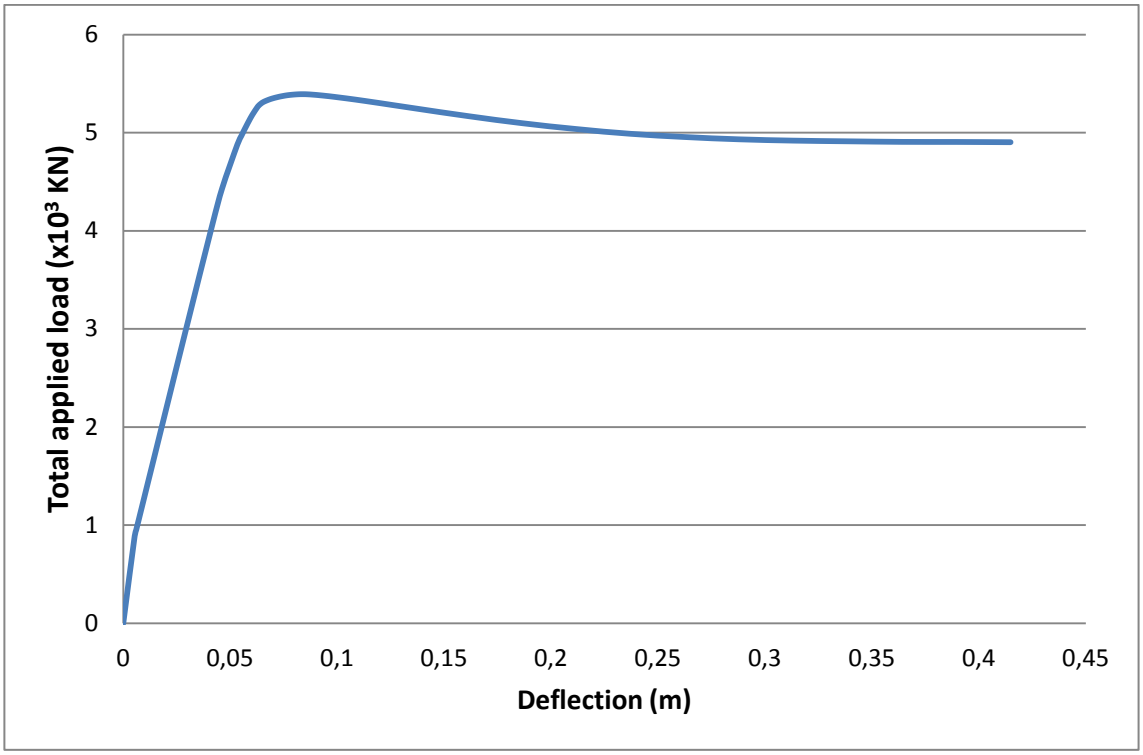


Figure AA2-16 Load vs. Deflection for simulation N. 1

Regarding the reliability-based assessment, because for the present case the random variables (namely the generalized structural resistance and the generalized action) have Normal distributions, the following equation was used:

$$\beta = \frac{R - S}{\sqrt{{\sigma_R}^2 + {\sigma_S}^2}}$$

The load factor obtained by simulation using Latin Hypercube method (see figure A2-6) is the generalized resistance (R). The mean value of this load factor is 2.513 and the standard deviation 0.224. The load factors which correspond to the real loads on the bridge are considered to be the generalized action (S). In this case, due to the fact that during the simulations the loads were applied as their mean value including impact and later incremented (by multiplying the mean load by the load factor) to reach the structure failure, the mean value of the generalized actions takes unit value. The standard deviation of the generalized action was considered to be equal to 12 %. This value is the effect of the multiplication of the railway load with a coefficient of variation equal to 10 % by the impact factor with a coefficient of variation equal to 25 %.

The reliability index obtained is 5.96. This value shows the high level of safety of the bridge when compared to target values assumed in standards and codes, even when the UIC train load is considered, which is higher than the railway live load used in the design. It should be pointed out that this high value of safety concerns the ultimate failure. However, the analysis does not take into account the fatigue resistance of the bridge, which may be the critical failure mode.

In figure A2-17 are compared the predicted failure mode by the ABAQUS model with the real failure obtained in the load test. It can be seen that the prediction is excellent, showing the buckling of the elements of the upper chord close to mid-span.

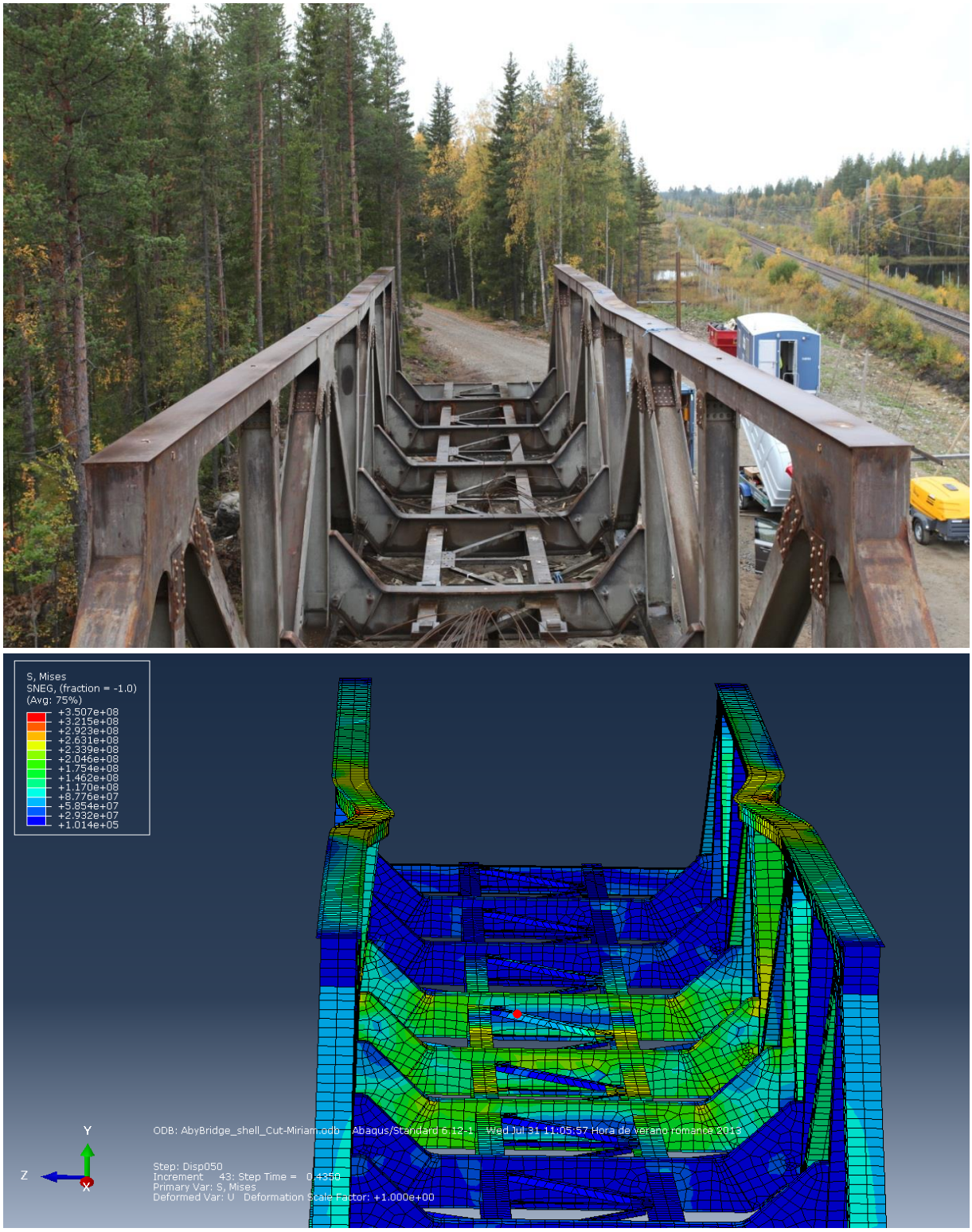


Figure AA2-17 Test to failure and numerical model of the Aby bridge (Sweden). The figure shows a comparison between actual and predicted deformations for the steel truss bridge which was loaded to failure in September 2013. The failure was initiated by instability of the two top girders. This was predicted by the reliability based assessment method using a non-linear finite element model

A.3 Redundancy assessment

This appendix describes the results of the redundancy analysis of the Aby bridge, a typical simply supported truss bridge superstructure, and its ability to continue to carry loads beyond the elastic limit. The object of the analysis is to investigate the residual capacity of the structure above the design load thanks to the inherent redundancy. This residual capacity can be furthermore used to perform an advanced assessment of the bridge.

A.3.1 . Redundancy definition

The measure of redundancy used here is that proposed by Ghosn and Moses. According to Ghosn et al. (1998) and Liu et al. (2000), redundancy is defined as the capability of the system to continue to carry load after the failure of one main member. Redundancy is measured by means of three parameters. Two measures are related to the intact configuration of the structure and are related to structural collapse and loss of structural functionality. The third measure is calculated for a damaged configuration of the structure and permits to evaluate the capability of the system to carry some emergency load after the damage in one main member. The measures can be calculated for bridge superstructures considering vertical load capacity or lateral load capacity (Figure A3-1).

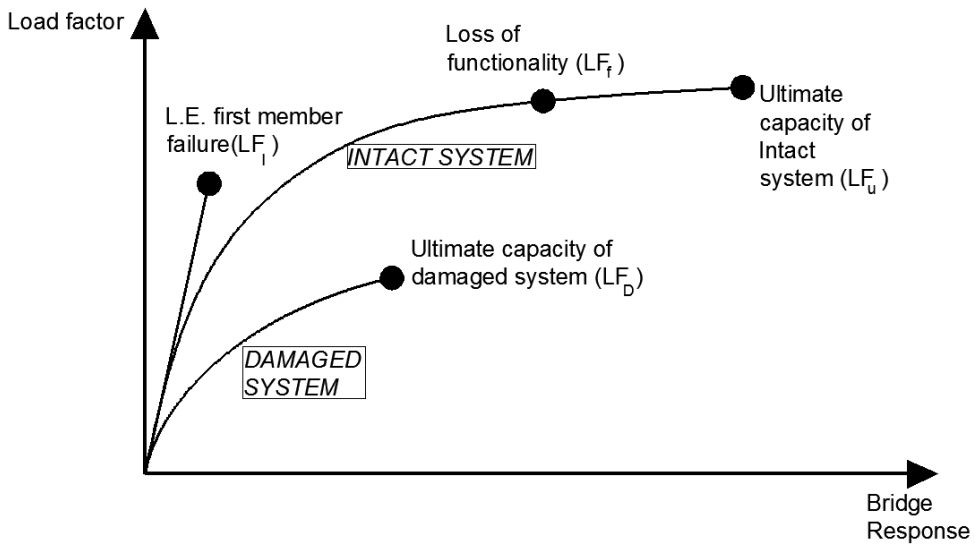


Figure A3-1. Load measures needed to calculate the redundancy of bridge systems.

These measures are:

$$R_u = LF_u / LF_1$$
$$R_f = LF_f / LF_1$$
$$R_d = LF_d / LF_1 \qquad \text{(A3-1)}$$

where:

LF_1 is the load that causes the failure of the first member;

LF_u is the load that causes collapse of the system;

LF_f is the load that causes the functionality limit state of the initially intact structure to be exceeded;

LF_d is the load factor that causes the collapse of a damaged structure which has lost one main member.

The analysis is performed with the commercial software ABAQUS. The bridge analysed is the Aby bridge, whose description and modelling is as presented in appendix A1 and A2.

In a first analysis, the originally intact bridge is loaded by the dead weight and the live load. The train load is increased incrementally and a pushdown analysis is performed up to the complete failure of the structure. In a second set of analyses, different damage scenarios are considered where in each scenario a different truss member is removed from the structure.

The analysis results show that the bridge behaves differently depending on the damage scenario, but overall the truss bridge shows a post linear capacity for most cases.

A.3.2 Analysis of the Intact Bridge

The push down analysis is performed on the originally intact bridge labeled as Model 1. The bridge is assumed to fail either when the structure reaches collapse or the maximum displacement reaches a value of span length/50 which is a very high value used to simply stop the analysis when a very large level of plasticity takes place.

The program first applies the dead load to the structure that consists of the weight of steel trusses, the girders and the rails and sleepers; then it applies the live load. The live load is then increased incrementally until the failure of the structure. According to Ghosn and Moses (1998) the acceptable level of redundancy depends on the type of failure considered. For an intact bridge, two redundancy measures can be evaluated: The first is related to the ultimate capacity of the system and the second is related to the functionality limit states. For the ultimate limit state, the load factor LF_u gives the number of live loads required to cause collapse and redundancy is measured as the ratio between LF_u and LF_1, where LF_1 is defined as the minimum value that causes the failure of the first member. LF_1 is equal to (R – D)/LL , where R is the member capacity of the most critical component, D is the dead load effect and LL is the live load effect. For the functionality limit state, the redundancy is measured as the ratio between the LF_f which gives the number of live loads needed to cause a maximum vertical displacement equal to span length/100 and LF_1 as defined earlier.

The result of the nonlinear analysis is depicted in the load deformation curve of Figure A3-2. The curve provides the maximum live load normalized to the weight of 47 Tn. (LF) versus the maximum vertical displacement. The behavior of the structure is globally linear until the point where the truss's compression chord starts to buckle although some portions of the structure have already started to yield, the yielding is still in its initial phase and thus the curve is almost linear until the onset of buckling. The buckling makes the structure deform without any significant additional increment in the applied load. After the deflection reaches a certain value, the structure begins to unload but continues to deflect until collapse. The maximum load effect LF_u obtained for this configuration is equal to 9.45 times the effect of the 47 Tn. normalizing load. The value of LF_1 is about 7.00 times the effect of the normalizing. Hence, the redundancy ratio for the ultimate limit state is equal to 9.45/7.00 = 1.35 which is greater than the reference value of 1.30 as recommended in the NCHRP report 406 for the ultimate limit state.

When the maximum displacement reaches L/100, or 33.5 cm, the maximum live load factor is about 9.0 and the redundancy ratio is equal to 9.00/7.00 = 1.28, which is greater than the reference value of 1.10 recommended in NCHRP 406 for the functionality limit.

Figures A3-3 and A3-4 show the effect of the buckling on the structure and the area of the bridge where the material yields. It is noted that the bridge is able to redistribute the load to regions far away from the area where the buckling is concentrated. This is possible when the connections between the two parallel trusses and the deck are sufficiently strong to allow for the redistribution of the load from the more heavily loaded truss to the other truss. .

According to the criteria defined in NCHRP report 406, the intact bridge structure analyzed in this example can be considered redundant.

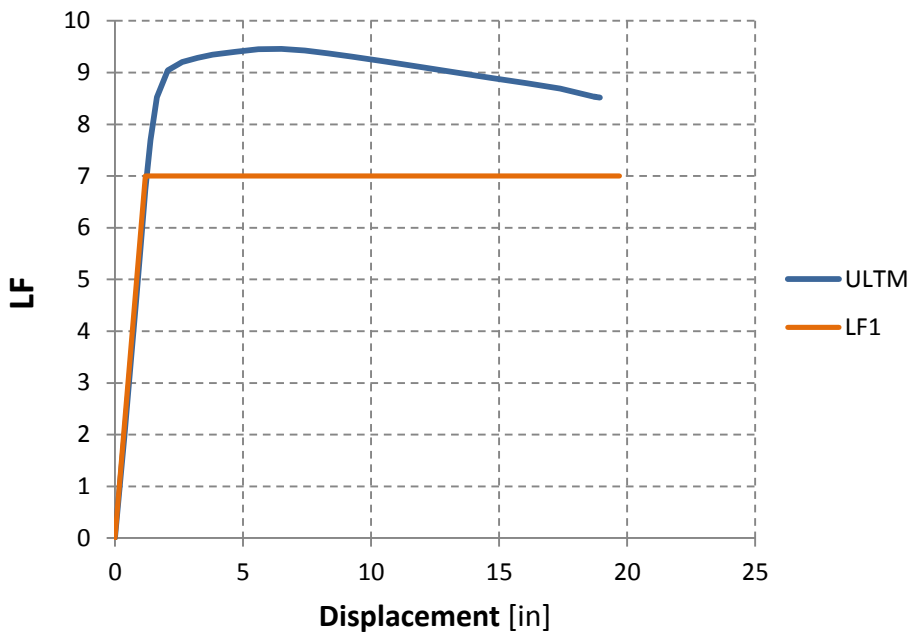


Figure A3-2 – Model 1 – Intact Structure Displacement vs Normalized Vertical Load

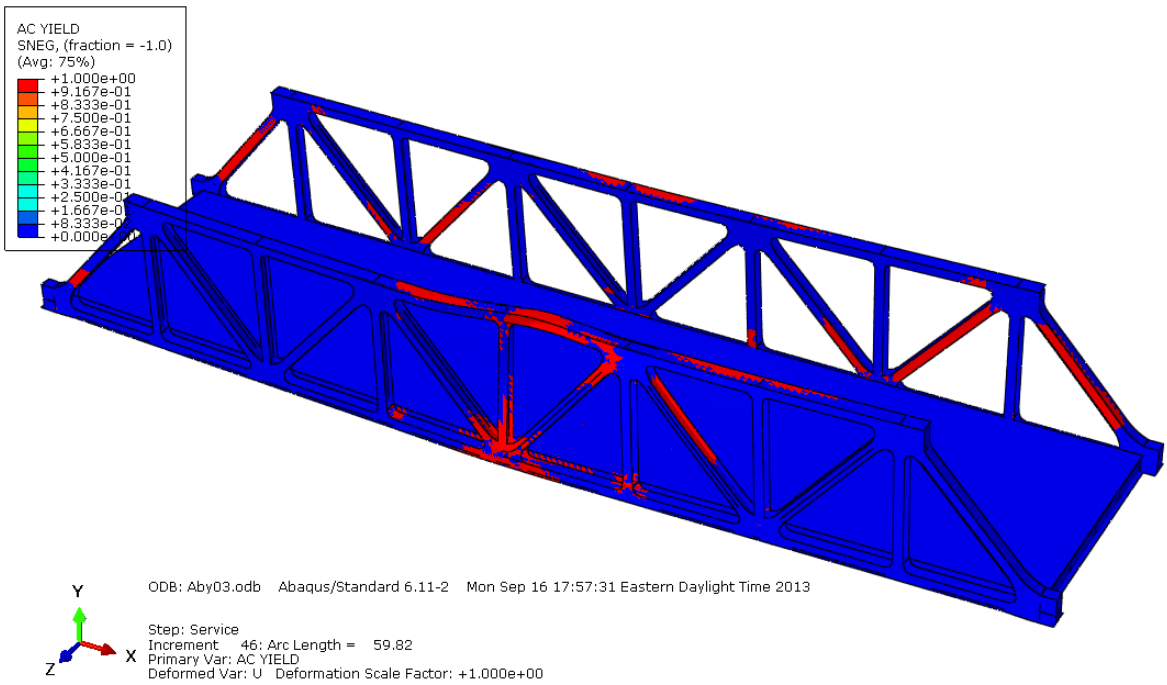


Figure A3-3 – Model 1 – Intact Structure Plastic Regions

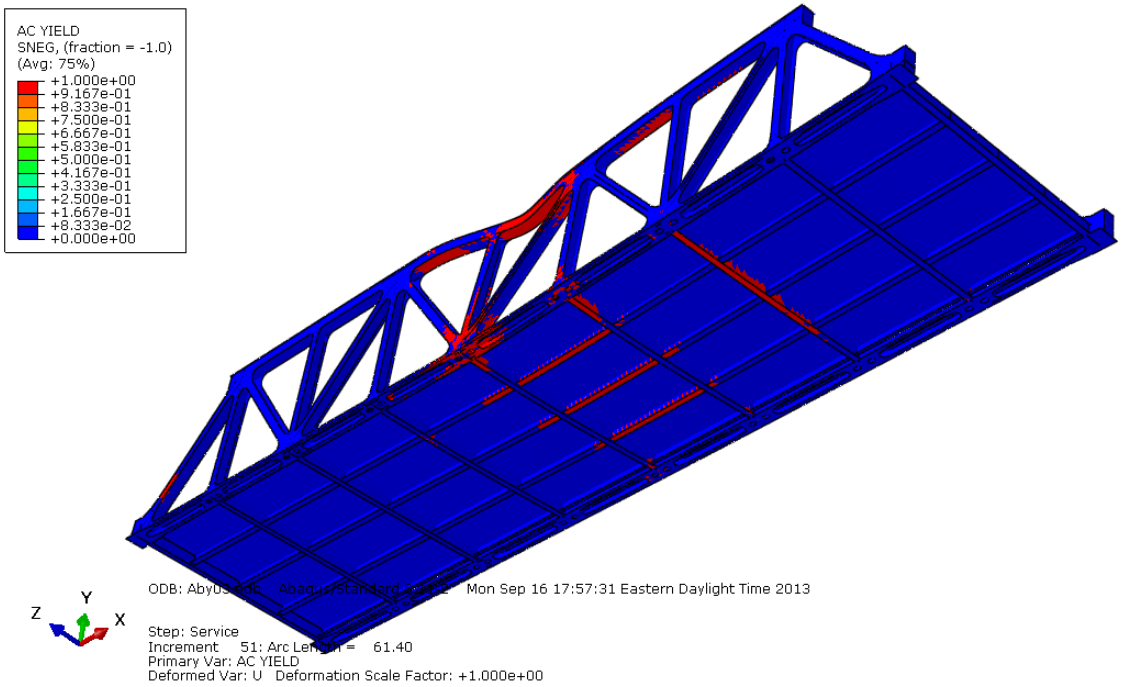


Figure A.3-4- Model 1 – Bottom View of Intact Structure Showing the Buckling Failure

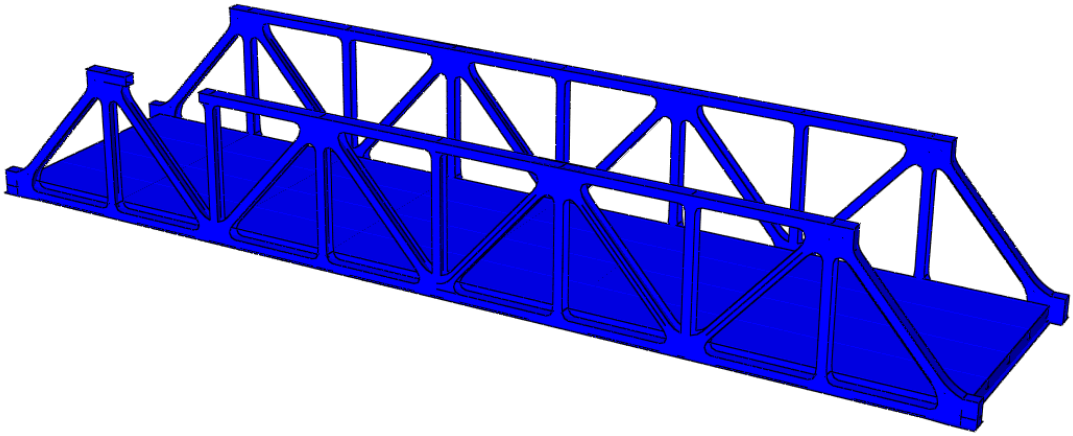
A.3.3 . Analysis of Damaged Bridge

In this section, the bridge is analyzed assuming different damage scenarios. The damage is applied to the truss that is closest to the vehicle loads, which is the truss that failed during the analysis of the originally intact bridge. Damage in all four types of truss members are considered: 1) the compression chords on the top of the truss; 2) the tension chords in the bottom part of the truss; 3) vertical members and; 4) to the diagonals. In all cases, the damage is simulated by removing an entire member and the pushdown analysis is performed on the modified models. This would simulate the fracture of tension members due to fatigue or the failure of any member due to an impact. It is noted that the analysis is performed to estimate the post-damage capacity of the system under static loading. The analysis does not consider the dynamic behavior during impact or during the release of the fracture energy. The purpose of the analysis is to verify that a damaged bridge will still be able to carry a sufficient level of traffic until the damage is noticed, the proper authorities alerted and appropriate decisions on repair or closure are taken.

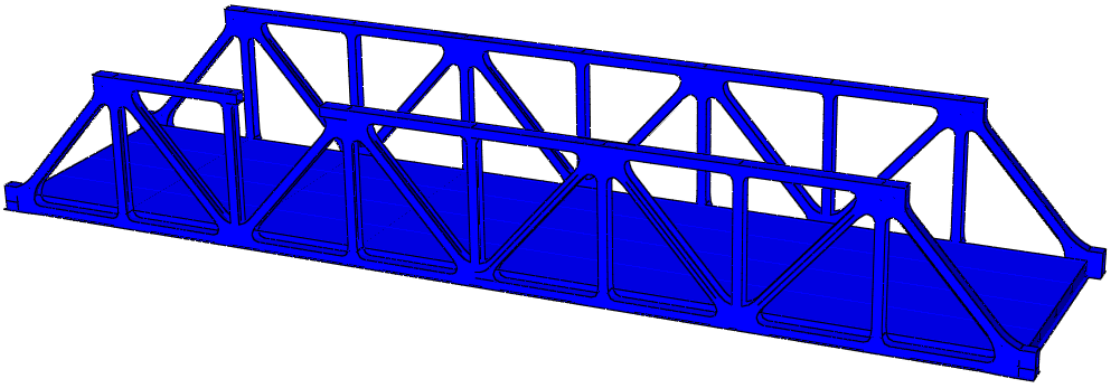
Model 2: Damage of Compression Chords

When the damage is applied at the compression chord, three cases have been considered, one for each compression member from the support through the middle span as shown in Figure A3-5. In the first case, the compression member is removed in the part of the truss closest to the support (CM01). Subsequently, in the two other cases CM02 and CM03, a different compressive chord member is removed one at a time.

CM01



CM02



CM03

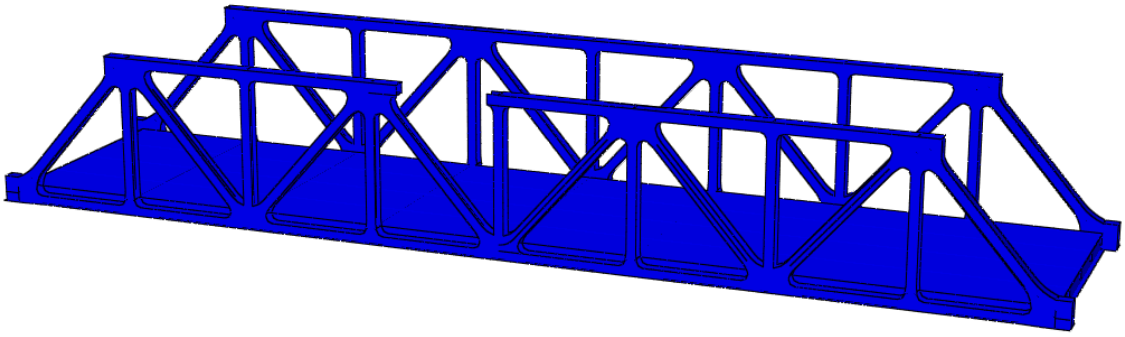


Figure A3-5- Model 2 – Damaged Structure of Compression Chords (CM01, CM02 and CM03)

The results of the analysis of the cases of compression chord member damage show that when one member of the compression chord is removed, the overall stiffness of the truss is drastically compromised and the stiffness is reduced compared to the stiffness of the intact bridge. However, despite the lower stiffness, this particular bridge configuration is able to carry significant live load after damage. In fact, the bridge is able to reach a capacity of two times the normalizing load before reaching the functionality limit state as shown in Figure A3-6. This is possible because the connections between the truss members of this bridge can carry a substantial level of moment and do not behave as pins. A typical connection for this bridge is shown in Figure A3-7. Because of the connection type, the bridge is able to redistribute the load to other elements of the structure through moments. Figure A3-8 is provided to help visualizes this aspect of the load redistribution process which shows how plasticity spreads to the members close to and far away from the damaged portion of the chord. The span/100 displacement of the damaged bridge is reached at a load increment of 2.80 for cases CM01 and CM02 and 1.50 for CM03. The ratio of these loads compared to LF_1 are 2.80/7.00 = 0.40 for CM01 and CM02 and 1.50/7.00 = 0.21for case CM03.

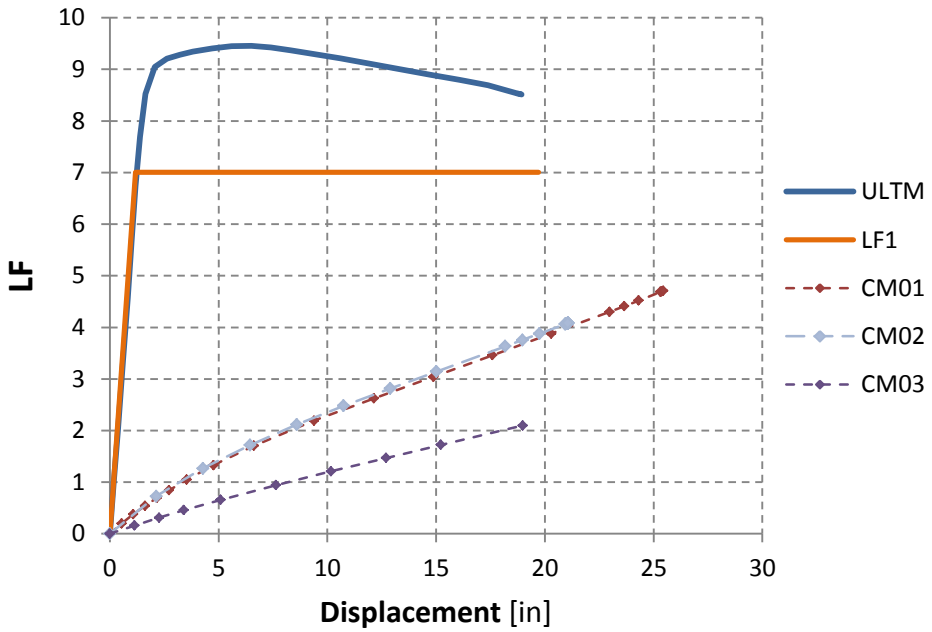


Figure A3-6- Model 2 – Damaged Structure Displacement vs Normalized Vertical Load (CM01-CM03)

Figure A3-7- Typical connection between members

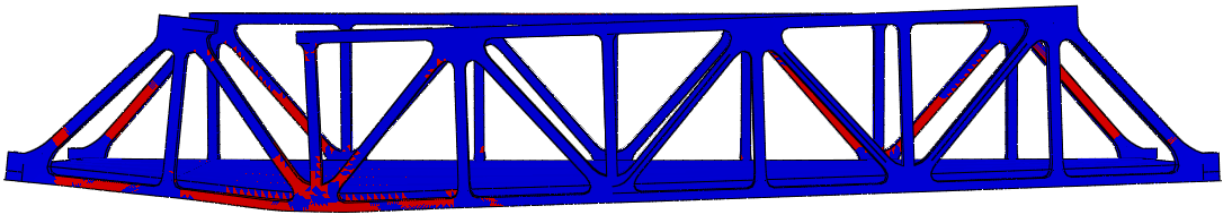


Figure A3-8- Model 2 – Top Chord Damaged Structure Moment Effect of the Connection

At the damaged limit state NCHRP 406 requires that the redundancy of the damaged bridge be equal or greater than 0.50. The normalized vertical load factor LF obtained for the damaged top chord scenarios are 2.09 for case CM03, 3.57 for CM02 and 4.68 for CM01 as shown by the load deflection curves of Figure B.6. The damage redundancy ratio varies from 2.09/7.00 = 0.30 through 4.68/7.00 = 0.66. According to NCHRP 406, the bridge is non-redundant if it is subjected to a damage of the type of CM03 which removes the middle top compression chord, while a bridge subjected to the damages of types CM01 and CM02 can be considered redundant.

For damage scenarios related to the compression chord the final failure of the bridge happens by rupture of the tension members in the vicinity of the damage as shown in Figure A3-.9.

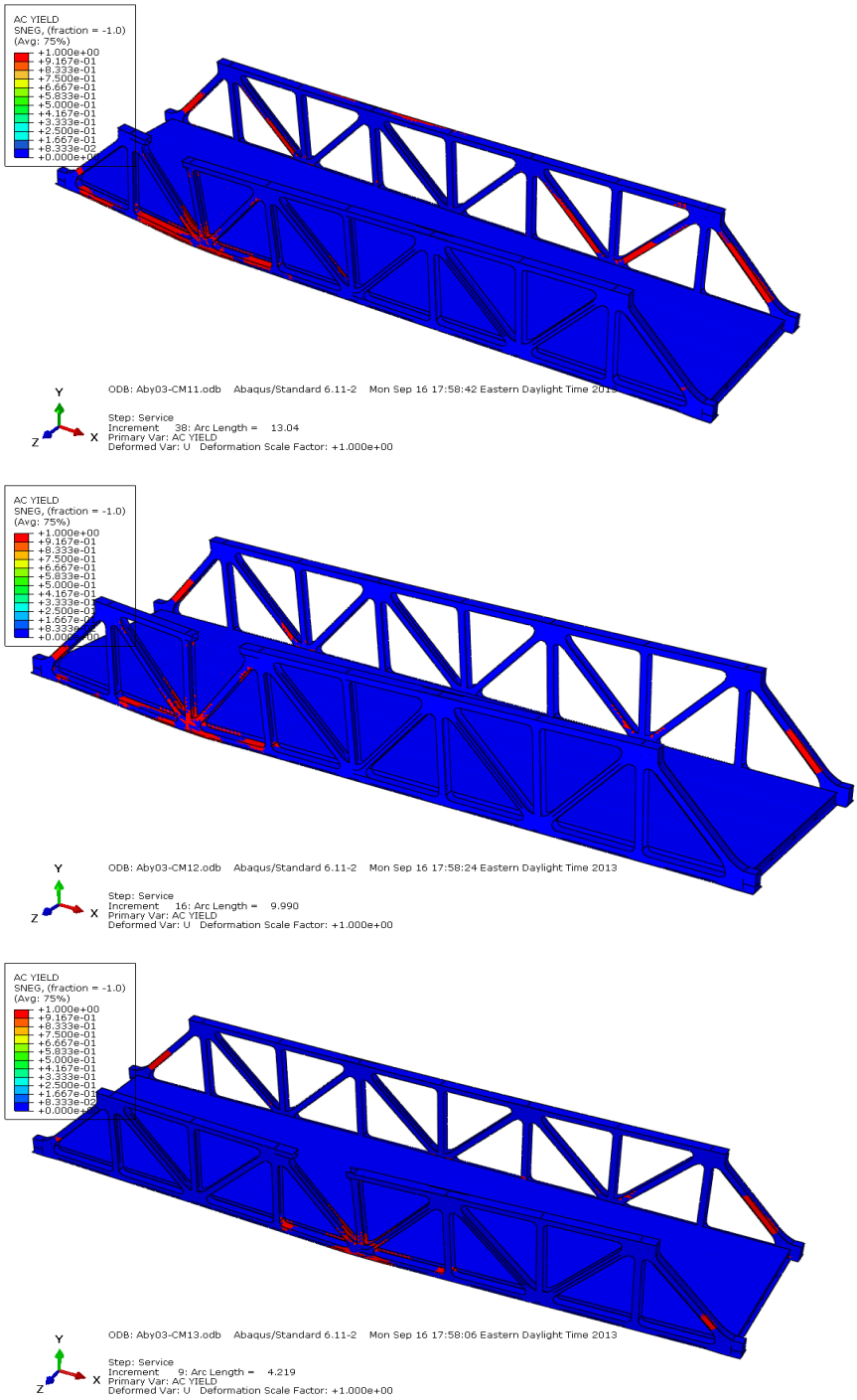


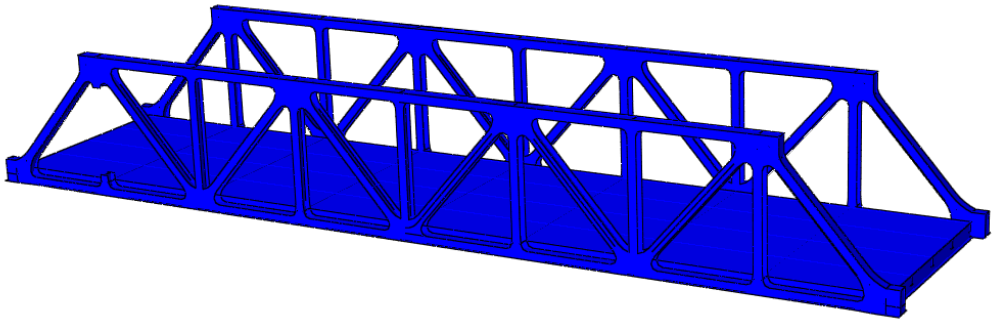
Figure A3-9- Model 2 – Top Chord Damaged Structure Plastic Failure

Model 3: Damage of Vertical Members

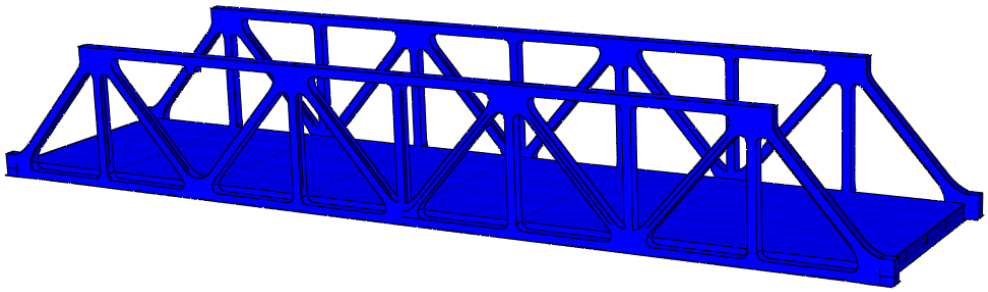
For this damage type, four different scenarios have been considered. One vertical member at a time is removed for each of the four cases as shown in Figure A3-10. In the first case, the diagonal member is removed in the part of the truss closest to the support (VM01). In the three following cases VM02, VM03 and VM04 are removed one at a time. The loading is still kept at the middle of the bridge because this happens to gives the lowest capacity for the system. The results of the analysis for this type of damage shows that when one vertical member is removed, the global stiffness of the truss is not compromised compared to that of the intact

bridge while the maximum capacity is slightly reduced as shown in the load displacement curves of Figure A3-11. The bridge finally fails due to the buckling of the compression chord; this is depicted in the failure mode of Figure A3-12.

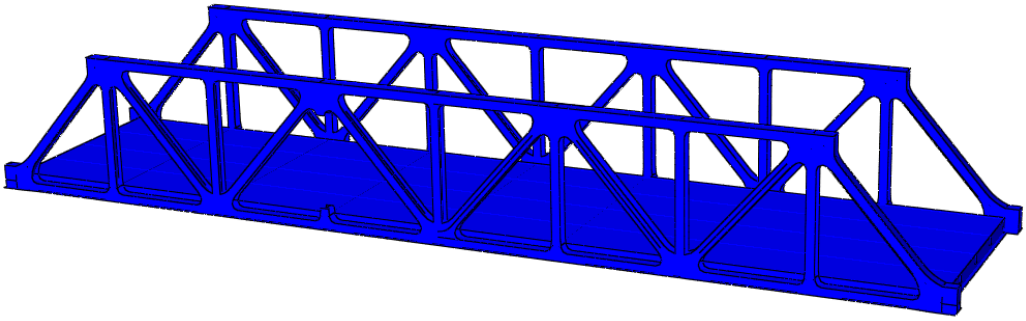
VM01



VM02



VM03



VM04

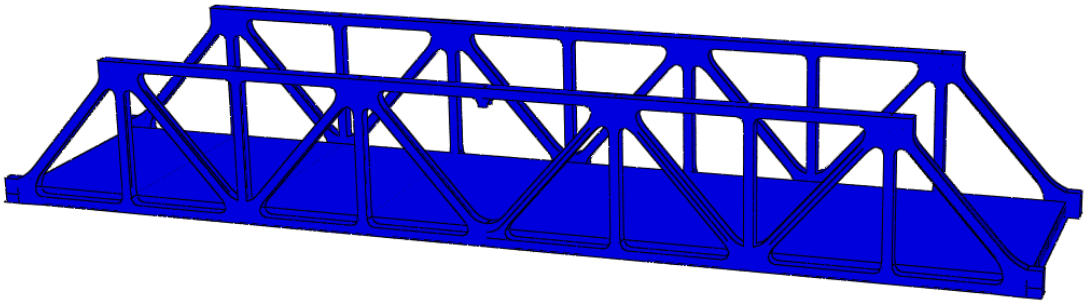


Figure A3-10- Model 3 – Damaged Structure of Vertical Members (VM01, VM02, VM03 and VM04)

When a vertical member near the support is damaged (VM01 and VM02), the behavior of the truss is practically the same as that of the intact structure. This means that the two vertical elements for the specific load condition do not contribute significantly to the distribution of the load. The redundancy ratios at span length/100 and for the maximum load for the damage limit

states are equal to the redundancy values of the intact bridge at the functionality and ultimate limit states with LF ratios are equal to 1.28 and 1.35 respectively.

When the damaged element is closer to the load location (middle of the span), the effect of the damage becomes more significant. The maximum capacity and the load at span/100 are reduced compared to the originally intact bridge. In fact, the VM03 and VM04 curves in Figure B.11 show a reduction in the capacity compared to that of the originally intact system, while cases VM01 and VM02 show similar results as those of the originally intact system

The redundancy at span length/100 ranges between 7.97/7.00 = 1.14 for the damage condition VM04 and 9.00/7.00 = 1.29 for cases VM01 and VM02. The damage bridge redundancy ratio varies from 8.90/7.00 = 1.27 through 9.45/7.00 = 1.35. According to the NCHRP 406 the structure for all vertical member cases can be considered redundant.

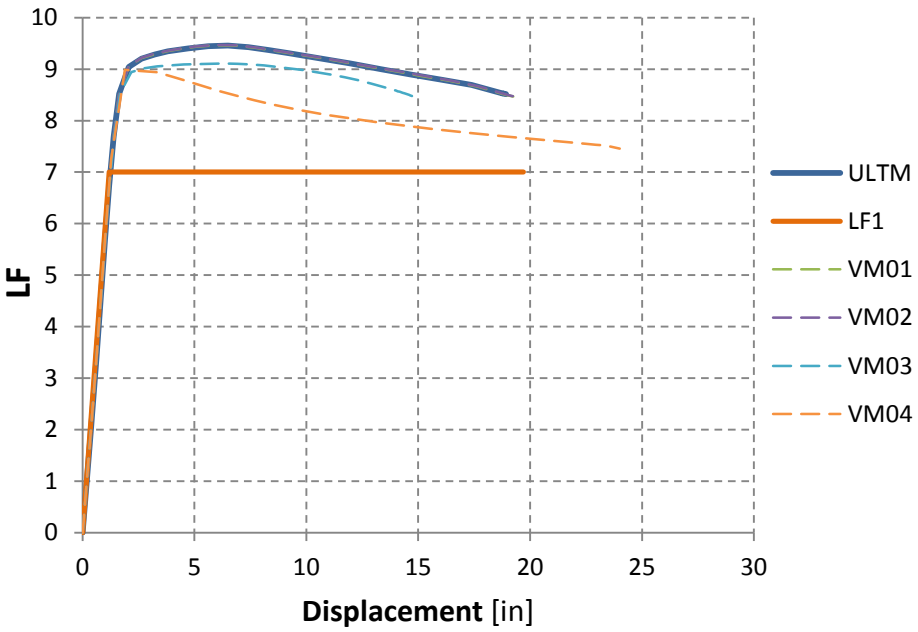


Figure A3-11- Model 3 – Damaged Structure Displacement vs Normalized Vertical Load (VM01 – VM04)

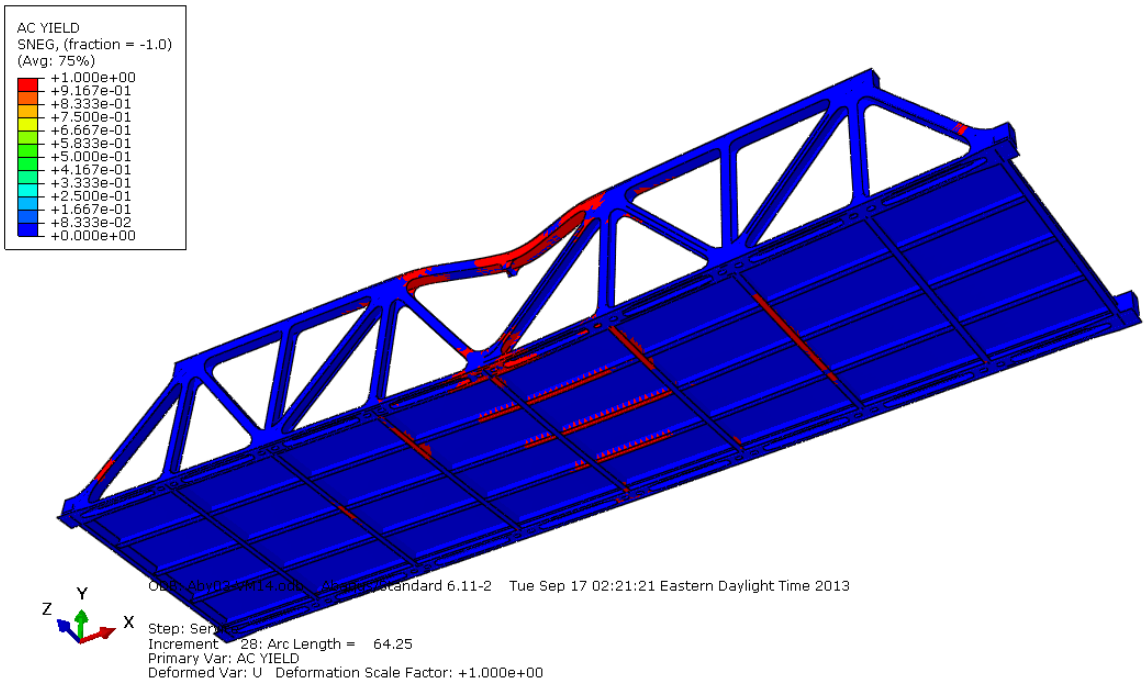


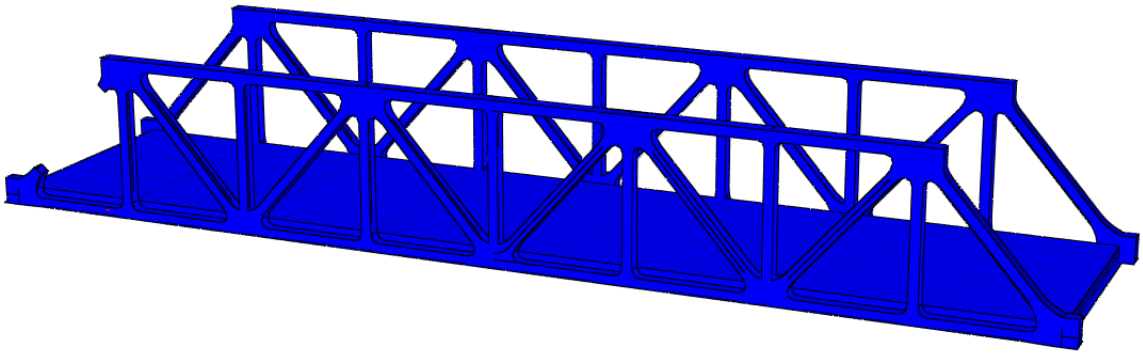
Figure A3-12- Model 3 – Bottom View of Damaged Structure Showing Buckling Failure

Model 4: Damage of Diagonal Members

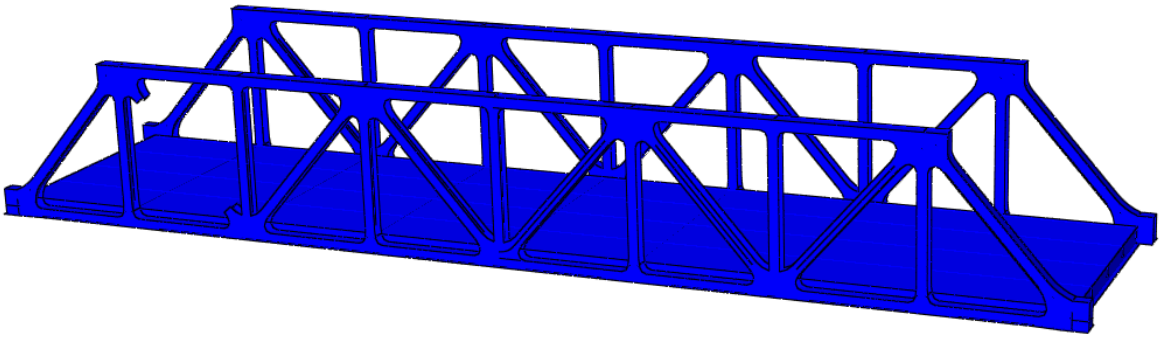
For this damage scenario, three cases have been considered. One diagonal member at time is removed for each of the four cases as shown in Figure A3-13. In the first case, the diagonal member of the truss closest to the support is removed (DM01). Subsequently, in the two cases DM02, and DM03, a new diagonal member is removed after restoring the previously eliminated member.

The results of the analysis of this type of damage show that when the diagonal is removed at the support, the global stiffness is affected. In fact, for this case the truss effect is missing and the entire load is transferred through the connection at the bottom to the truss away from the load as shown in Figure A3-14. For the remaining cases, the stiffness is not compromised as much as in case DM01 although the stiffness is lower than that of the intact scenario. For these cases, the effect of redistribution of the load to a wider portion of the structure allows the bridge to reach an ultimate capacity greater than that of the intact bridge. This is possible because the failure mechanism is not localized in one portion of the truss and plasticity distributes throughout the system and is not concentrated in the region close to the load as in the intact configuration that produces buckling of the compression chord before the load redistributes to regions away from the load. The damage and the failure for cases DM02 and DM03 are shown in Figure A3-15 while the results of the analysis represented by the load versus displacement curves are shown in Figure A3-16.

DM01



DM02



DM03

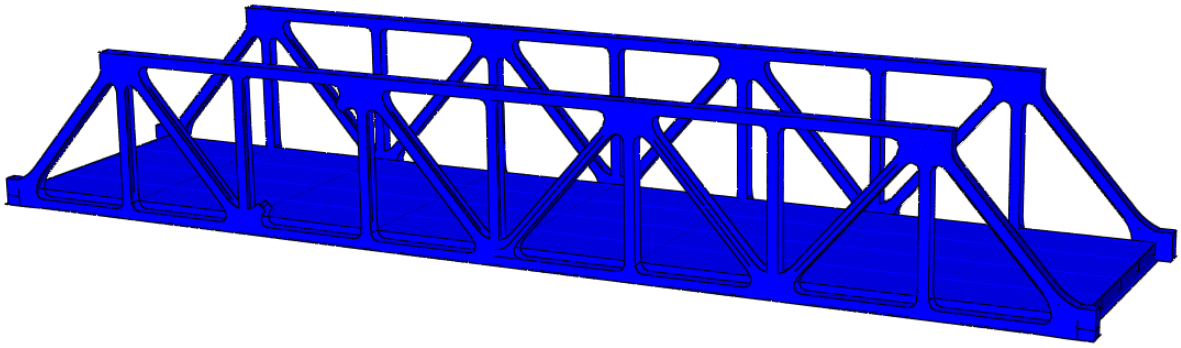


Figure A3-13- Model 4 – Damaged Structure of Diagonal Members (DM01, DM02, DM03 and DM04)

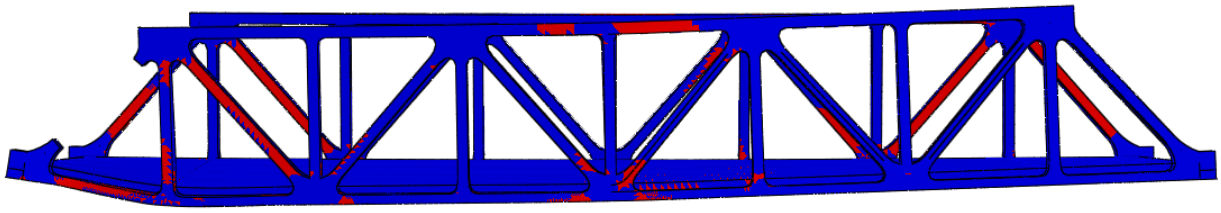


Figure A3-14- Model 4 – Damaged Structure Moment Effect of the Connection (DM01)

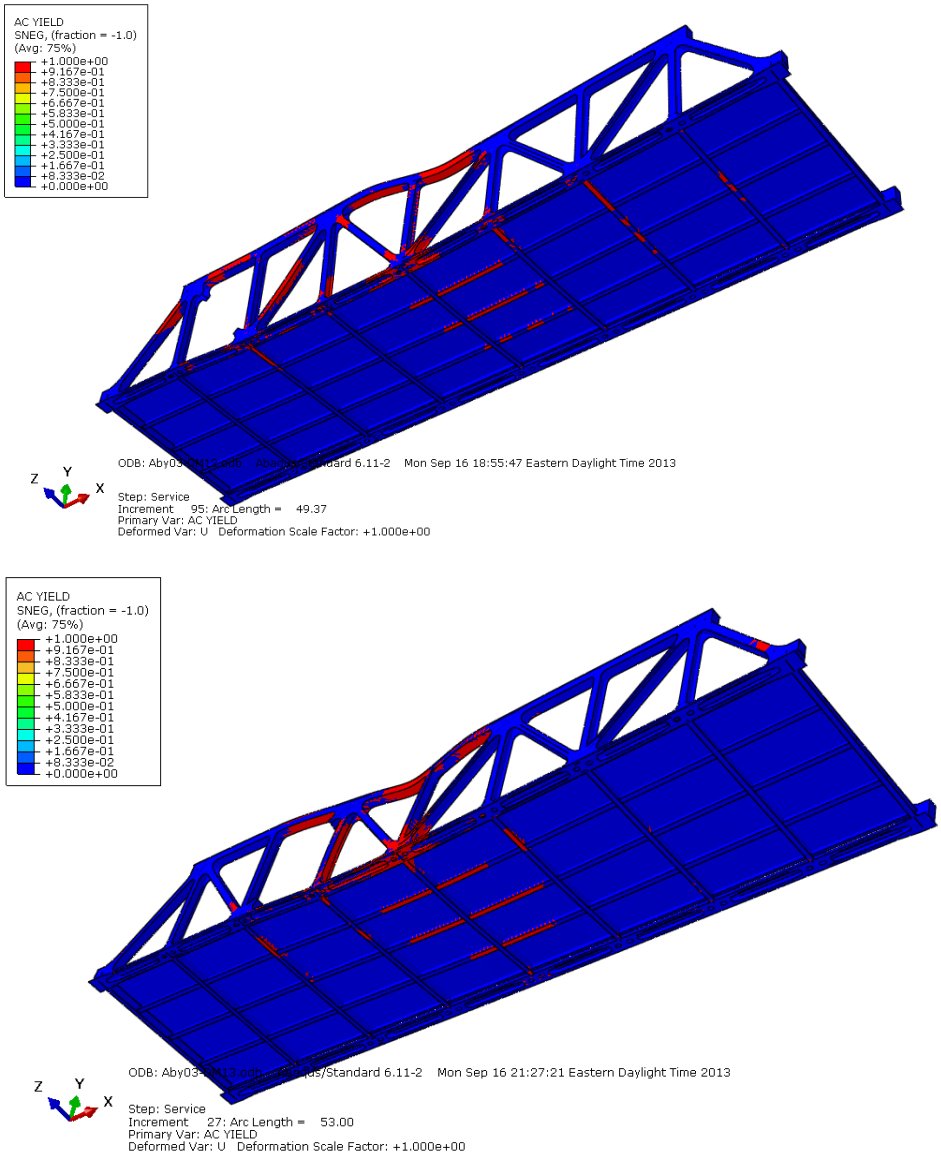


Figure A3-15- Model 4 – Bottom View of Diagonal Damaged Structure Plastic - Buckling Failure

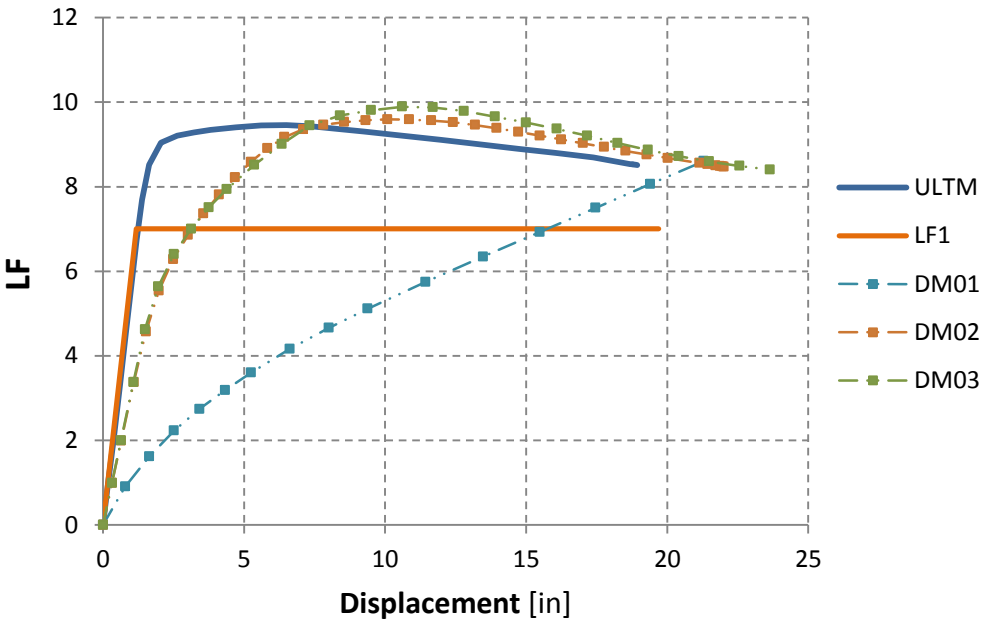


Figure A3-16- Model 4 – Damaged Structure Displacement vs Normalized Vertical Load (DM01 – DM03)

The redundancy ratio for case DM01 is equal to 6.34/7.00 = 0.91 for the span length/100. The redundancy ratio for the damaged limit state is equal to 8.60/7.00 = 1.23 that is much greater than the 0.50 criterion proposed in NCHRP 406.

At the span length/100, the redundancy ratios for damage scenarios cases DM02 and DM03 are equal to 9.46/7.00 = 1.35 and 9.78/7.00 = 1.40 respectively. The redundancy ratio for the damaged limit state is equal to 9.60/7.00 = 1.37 for case DM02 and 9.89/7.00 = 1.41 for case DM03. The bridge is considered to be redundant for these two damage scenarios according to the NCHRP 406 criteria.

A.3.4 . Comparison of Results

The analysis of different damage scenarios for the example truss analyzed showed that the redundancy ratio depends on the location where the damage develops. In fact, the ratio for different limit states varies considerably. Table A3-1 summarizes the results of the different damage scenarios.

Table A3-1 shows that the load at which the span length/100 displacement is reached varies from a minimum of 0.21 for case CM01 to a maximum of 1.40 for case DM03, while the maximum capacity ratio varies from a minimum of 0.30 for case CM01 to a maximum of 1.41 for case DM03.

It is noted that local failure of each member of a damaged truss depends on the load location. However, this analysis concentrated on the global behavior and the load was applied in the position that produced the critical damaged system capacity. If one wishes to study the local behavior of the truss members, then different loading positions should be used for the different damage scenarios. That would be needed to identify which members must be strengthened to improve the global behavior of the damaged systems.

The good performance of this bridge system is related to the type of connections used in this truss and the ability of the deck to redistribute the load. Both of these factors can drastically affect the global behavior of the system.

Table A3-1 – Redundancy Ratios for Aby Bridge (shaded cell shows low redundancy level)

Analysis Case	LFu/LF1 Ultimate limit state of originally intact bridge	LFf/LF1 Functionality limit state of originally intact bridge	LFd/LF1 Redundancy ratio for damaged bridge scenarios	LF100/LF1 for damaged bridge scenarios
ULTM	1.35	1.29	-	
CM01	-		0.30	0.21
CM02	-		0.57	0.40
CM03	-		0.67	0.40
VM01	-		1.35	1.29
VM02	-		1.35	1.29
VM03	-		1.30	1.25
VM04	-		1.27	1.14
DM01	-		1.23	0.91
DM02	-		1.37	1.35
DM03	-		1.41	1.40

A.3.5 Conclusions

The analysis of the system redundancy of truss bridges is a complex process that requires an exhaustive investigation in order to characterize all possible damage scenarios and the response of the bridge to these cases under different loading conditions.

The analysis of the sample bridge presented in this report shows that the redundancy is strongly dependent of the location of the damage. For example, the originally intact bridge provides sufficient levels redundancy for both the functionality and ultimate limit states according to the criteria proposed in NCHRP report 406.

For the damaged scenarios, the values of the redundancy ratios vary in function of the location of the damage. As an example, the damaged case redundancy ratio varies from a minimum of 0.30 for case CM03 which assumes that the compression top chord member near the middle of the bridge is damaged through a maximum of 1.41 for case DM03 which assumes that a diagonal member near the middle of the span is damaged.

The type of connections used between the truss members and the connections between the trusses and the deck affect drastically the global behavior of the truss system.

A.4 Robustness to corrosion

This appendix describes the results of the redundancy analysis of the Aby bridge, a typical simply supported truss bridge superstructure and its ability to continue to carry loads beyond the elastic limit when the different members of the bridge are subjected to an environmental degradation process (corrosion). The object of the analysis is to investigate the residual capacity of the structure above the design load and taking into account the predicted degradation scenario, thanks to the inherent robustness and tolerance to damage.

A.4.1 Robustness definition

Cavaco et al. (2010) proposed a robustness measure considering that a single robustness indicator, Rd, must be defined for all levels of damage. The proposed indicator can be defined as:

$$R_d = \int_{d=0}^{d=1} f(x)dx \qquad \text{(A4-1)}$$

where *f* is the normalized performance, given by the ratio between the structural performance on the intact and damage states, and *d* is the normalized damage, given by the ratio between actual and maximum possible damage.

This equation is based on the assumption at robustness is defined by the area under the normalized performance profile for the damage levels between 0 (intact) to 1 (total damage). In this sense, a structure for which any damage causes a complete loss of performance is considered not robust (curve A in Figure A4-1), as a structure for which no reduction in performance occurs for any damage level corresponds to full robustness (curve E in Figure A4-1). Real structures will correspond to situations between these two extremes (curves B, C and D) and the geometry of the curve will show the susceptibility of the structure to deterioration.

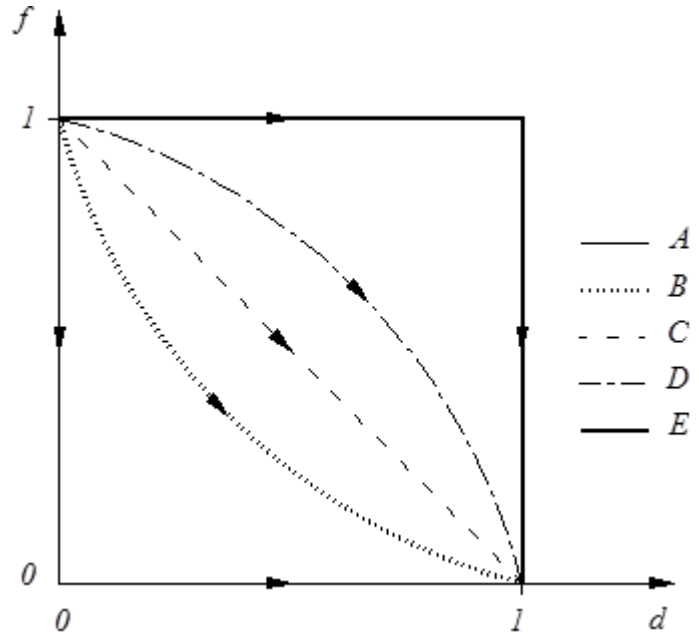


Figure A4-1.- Graphical representation of robustness index

This approach for robustness allows the consideration of several damage scenarios as performance indicators. Damage should be considered with a broader sense, i.e., damage can vary from simple degradation to more serious damage such as a column or beam failure. Errors during the design or the construction stage can also be seen as types of damage. The structural performance can assume many forms, and can be related to service limit states or to ultimate limit states.

In the present example, the proposal will be applied to the corrosion degradation of the Aby river bridge in Sweden (see appendix A for bridge description and modelling). In figure A4-2 is presented an schematic view of the bridge, labelling the different truss elements: O for the upper compression members, U for the bottom tensile members, V for the vertical elements and D for the diagonal members. Not seen in the figure, the transverse elements are designated with T and the longitudinal beams supporting the track by the letter R.

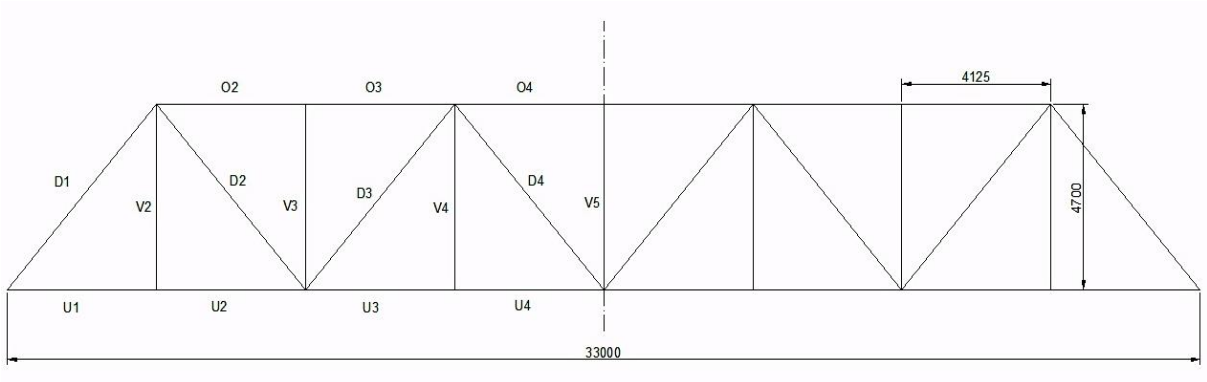


Figure A4-2. Structural configuration of Aby bridge and description of members.

A.4.2 Corrosion degradation

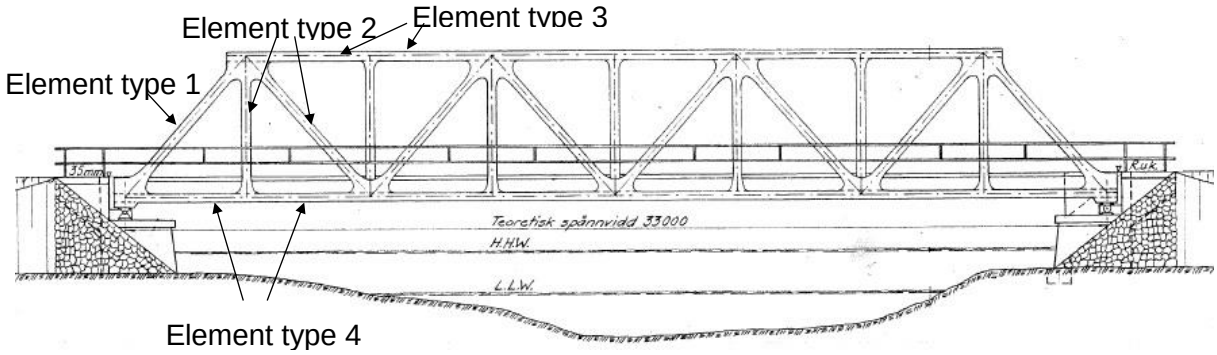
The atmospheric corrosion of the steel members will be considered as the damage scenario in equation A4-1. Atmospheric corrosion requires an aqueous film to be present on the metal surface, and as such it is a special form of general aqueous corrosion. For atmospheric corrosion to occur, four key factors are necessary, the same as for regular aqueous corrosion to occur. These four factors are; there must be an anode, a cathode, an ionic conducting electrolyte and an electrical conducting path between the anode and the cathode. If any one of these four factors is missing corrosion does not occur. When the electrolyte is missing corrosion does not occur. This is the case that occurs for atmospheric corrosion at specific times. When the metal surface is dry or does not have a sufficient layer of moisture, the electrolyte is absent. At these specific times atmospheric corrosion does not occur.

It is widely accepted that the long-term atmospheric corrosion of steel conforms to an equation of the form:

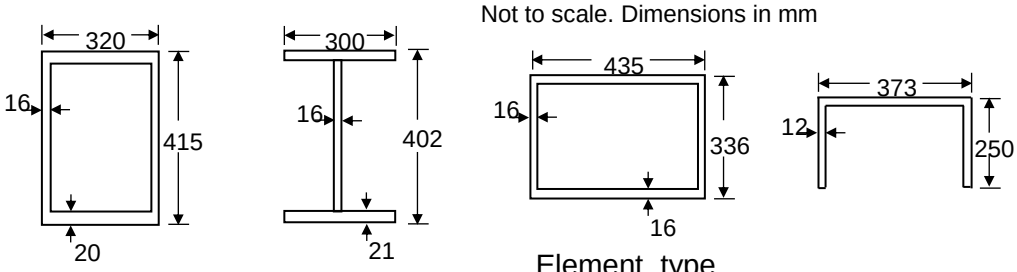
$$C = A \cdot t^{B} \qquad \text{(A4-2)}$$

The methodology used to obtain the corrosion profiles over time is presented in Kallias and Chryssanthopoulos (2014) and is based on a framework for the deterioration modelling of the coating-steel substrate system which is in line with the exposure classification recommendations of BS EN ISO 9223 (2012a). As presented in Chryssantopoulos (2013), the bridge members as defined in figure A3-3 were identified for the Aby bridge, the coefficients A and B in equation A3-2 were calculated, and the corresponding performance profiles specific for that bridge, with and without coating, were obtained as presented in the following.

According to BS EN ISO 9223, the C2 exposure corresponds to low corrosivity: Temperate zone, atmospheric environment with low pollution (SO2 < 5 µg/m3), e.g. rural areas, small towns. Dry or cold zone, atmospheric environment with short time of wetness, e.g. deserts, subarctic areas. The C5 exposure is defined as very high corrosivity: Temperate and subtropical zone, atmospheric environment with very high pollution (SO2: 90 µg/m3 to 250 µg/m3) and/or significant effect of chlorides, e.g. industrial areas, coastal areas, sheltered positions on coastline.



Not to scale. Dimensions in mm



Element type 1 Element type 2 Element type 3 Element type 4

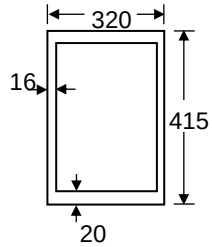
Element type	Name	Exposure classification (ext.)	Exposure classification (int.)	M21 coating life, T_L (years)	Corrosion model	
					Coef. A (mm)	Coef. B
1	EL1C2M21	C2	C2	22	0.025	0.575
	EL1C5M21	C5		18	0.200	0.575
2	EL2C2M21	C2	N/A	22	0.025	0.575
	EL2C5M21	C5		18	0.200	0.575
3	EL3C2M21	C2	C2	22	0.025	0.575
	EL3C5M21	C5		18	0.200	0.575
4	EL4C2M21	C2	N/A	22	0.025	0.575
	EL4C5M21	C5		18	0.200	0.575

Figure A4-3 Bridge element types

The obtained profiles are presented in the following pages for the different element types and exposure classifications.

Profiles designation: *EL1C2M21*
Element type: 1
Exposure classification: C2
Coating type: M21 with TL = 22 years

Not to scale.
Dimensions in mm



Element type

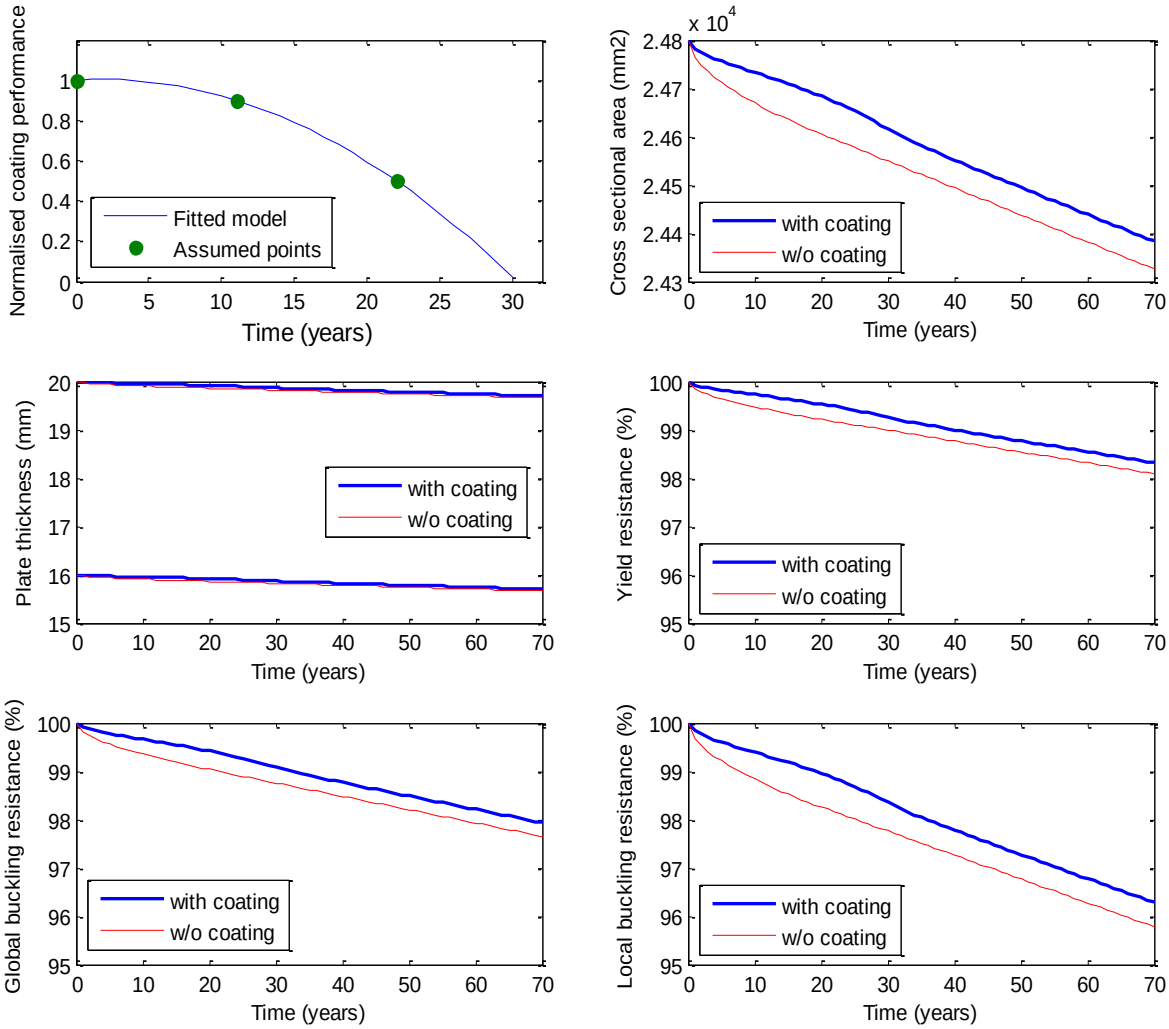
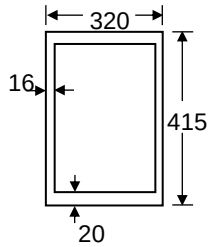


Figure A4-4.

Profiles designation: EL1C5M21
Element type: 1
Exposure classification: C5
Coating type: M21 with TL = 18 years

Not to scale.
Dimensions in mm



Element type

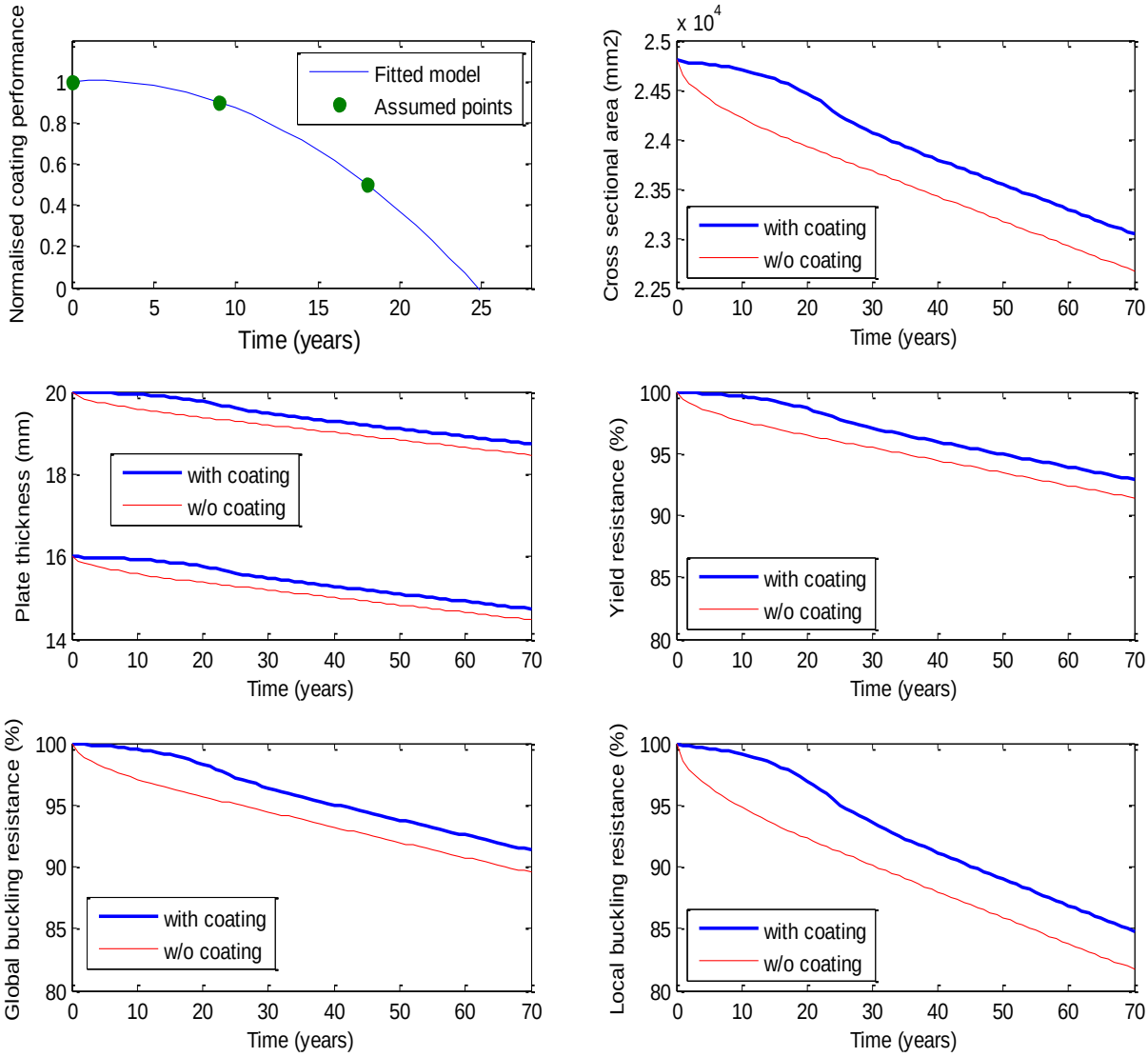
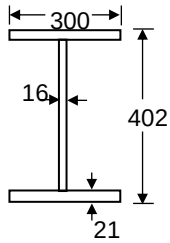


Figure A4-5.

Profiles designation: EL2C2M21
Element type: 2
Exposure classification: C2
Coating type: M21 with TL = 22 years

Not to scale.
Dimensions in mm



Element type

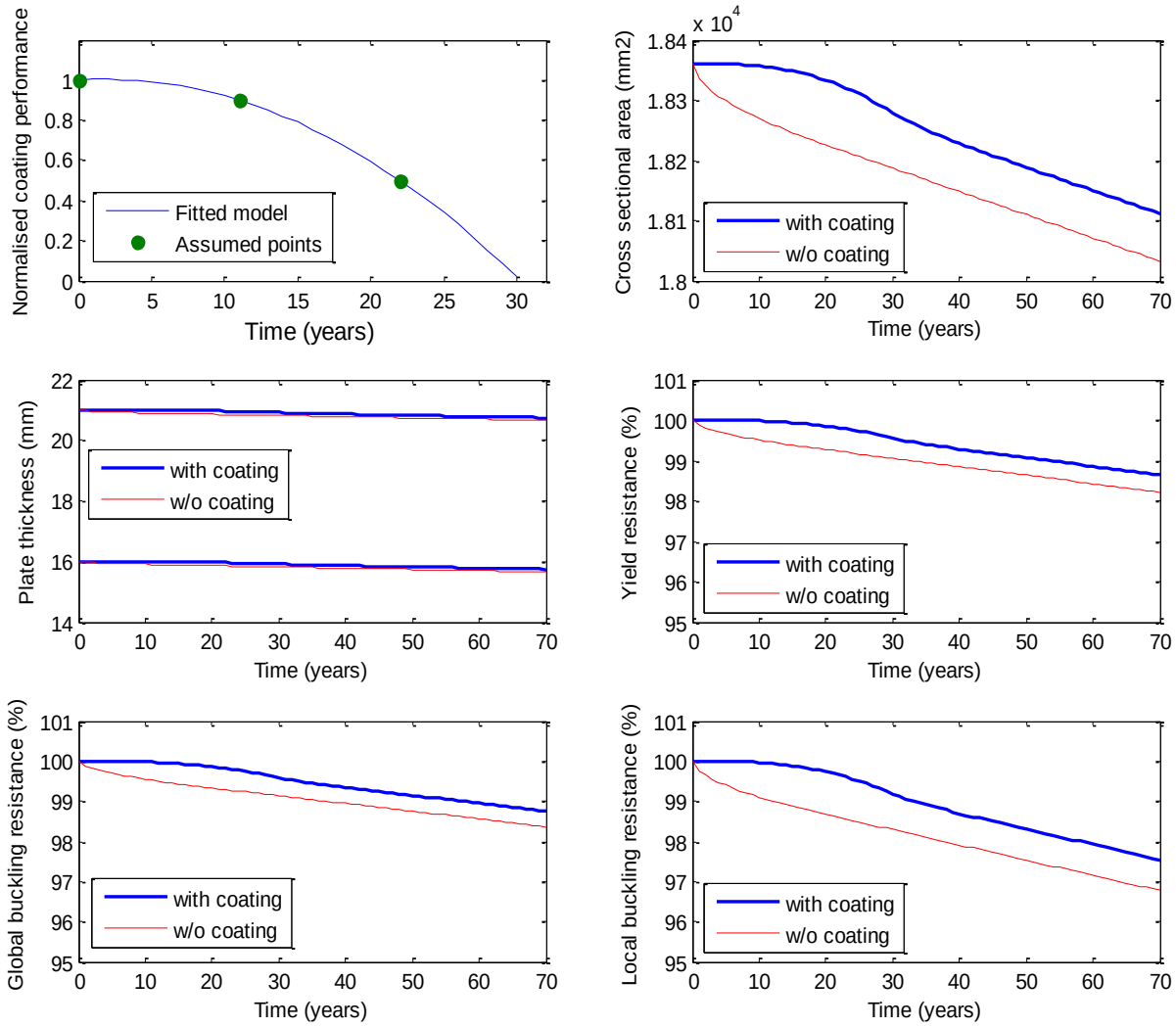
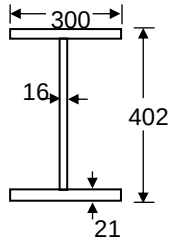


Figure A4-6

Profiles designation: EL2C5M21
Element type: 2
Exposure classification: C5
Coating type: M21 with TL = 18 years

Not to scale.
Dimensions in mm



Element type

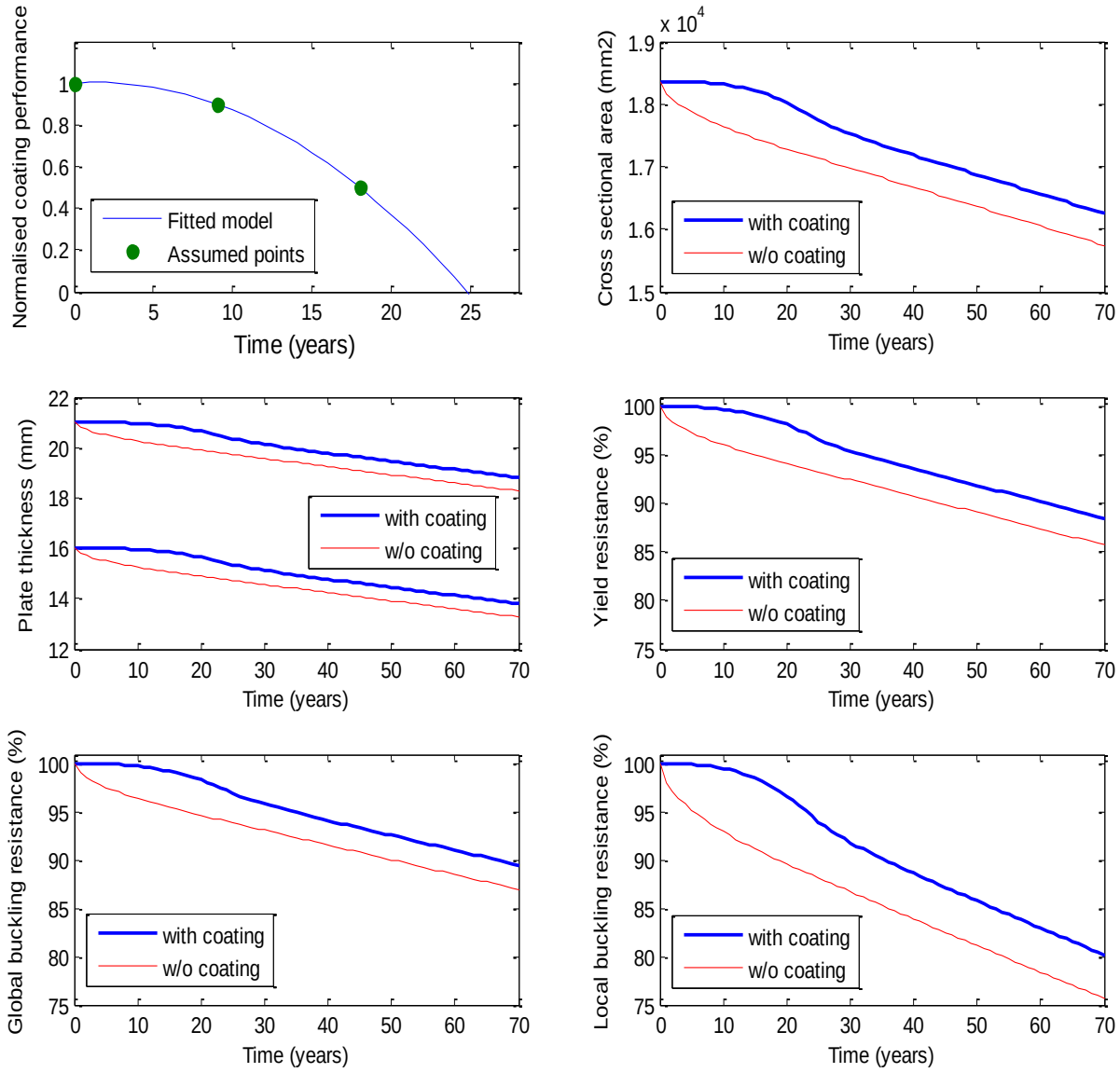
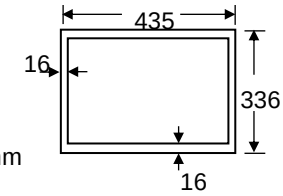


Figure A4-7.

Profiles designation: EL3C2M21
Element type: 3
Exposure classification: C2
Coating type: M21 with TL = 22 years

Not to scale.
Dimensions in mm



Element type

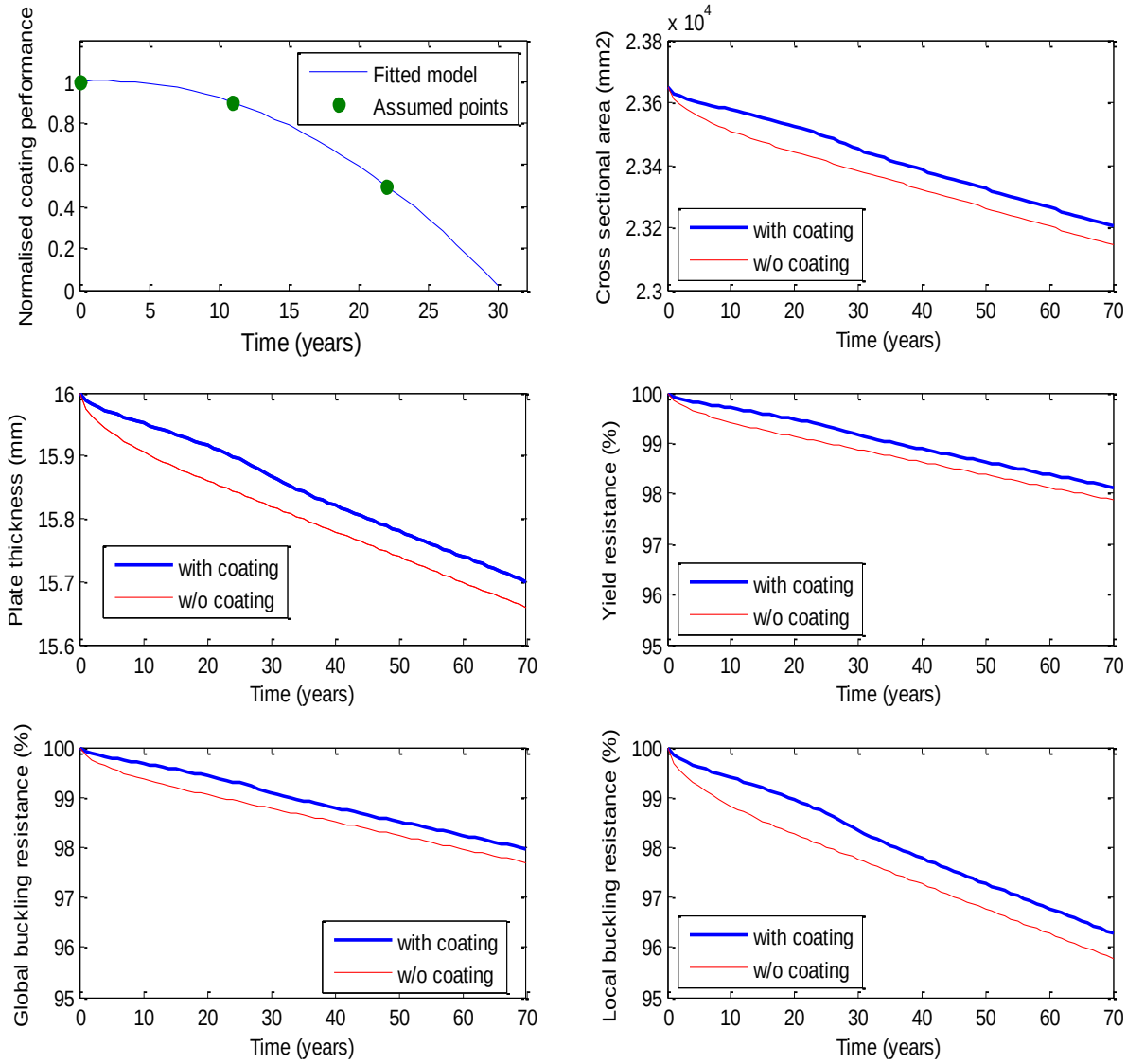
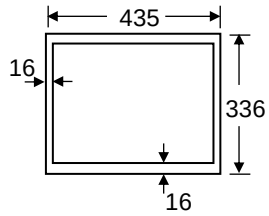


Figure A4-8.

Profiles designation: EL3C5M21
Element type: 3
Exposure classification: C5
Coating type: M21 with TL = 18 years

Not to scale.
Dimensions in mm



Element type

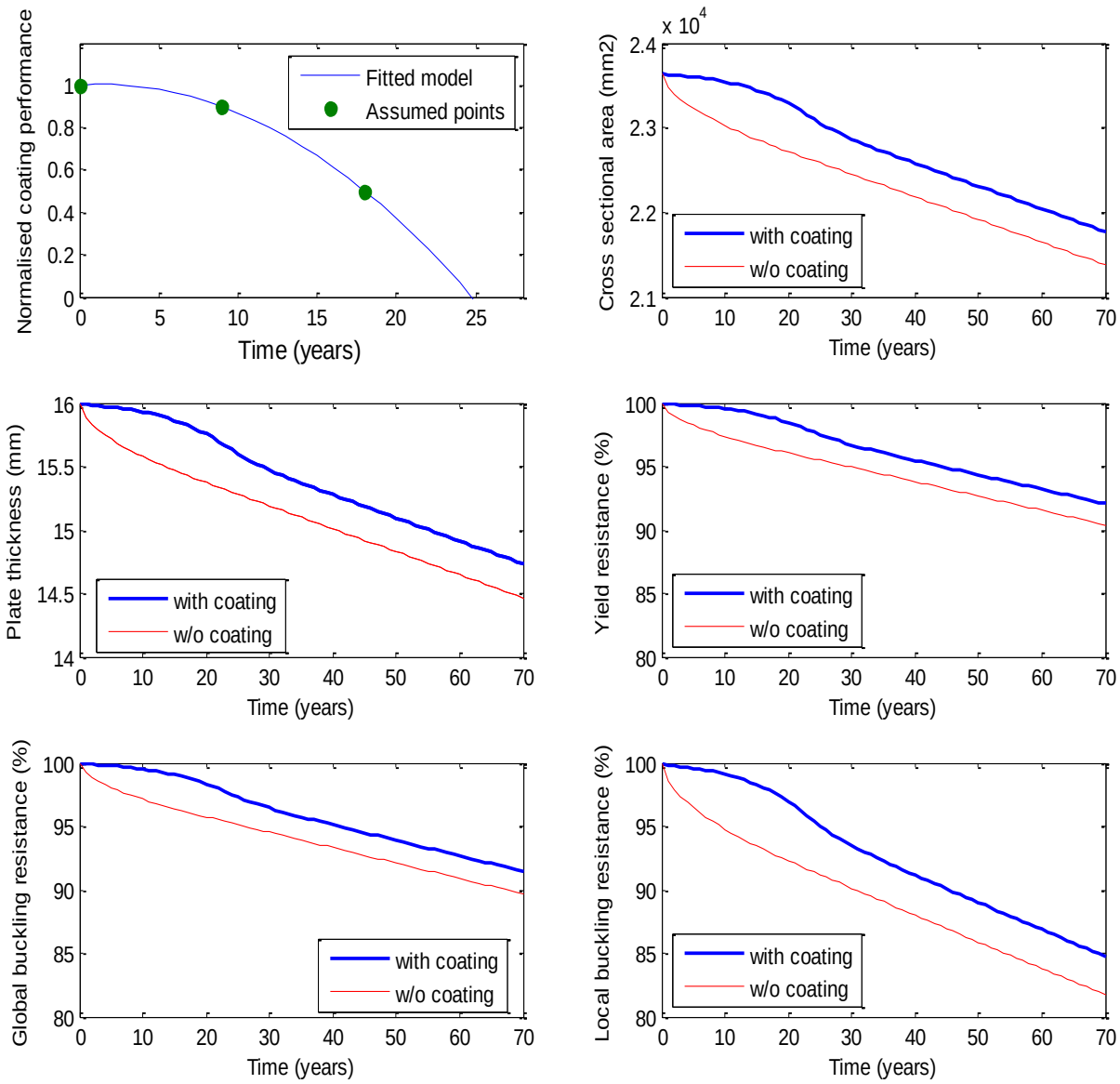
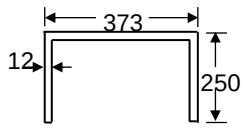


Figure A4-9.

Profiles designation: EL4C2M21
Element type: 4
Exposure classification: C2
Coating type: M21 with TL = 22 years

Not to scale.
Dimensions in mm



Element type

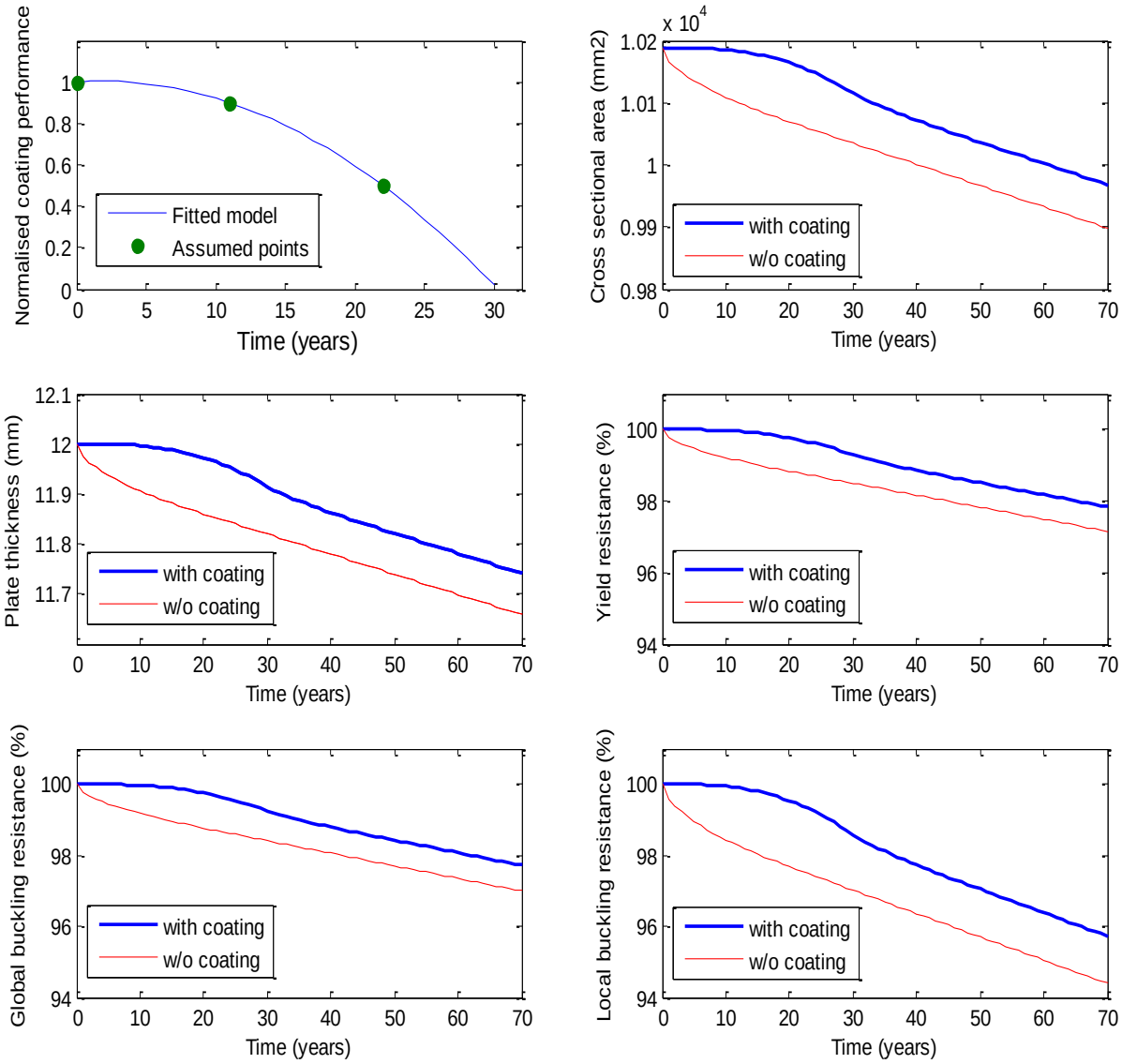
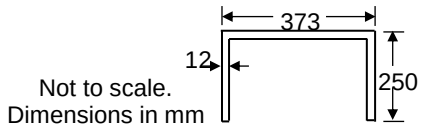


Figure A4-10.

Profiles designation: EL4C5M21
Element type: 4
Exposure classification: C5
Coating type: M21 with TL = 18 years



Element type

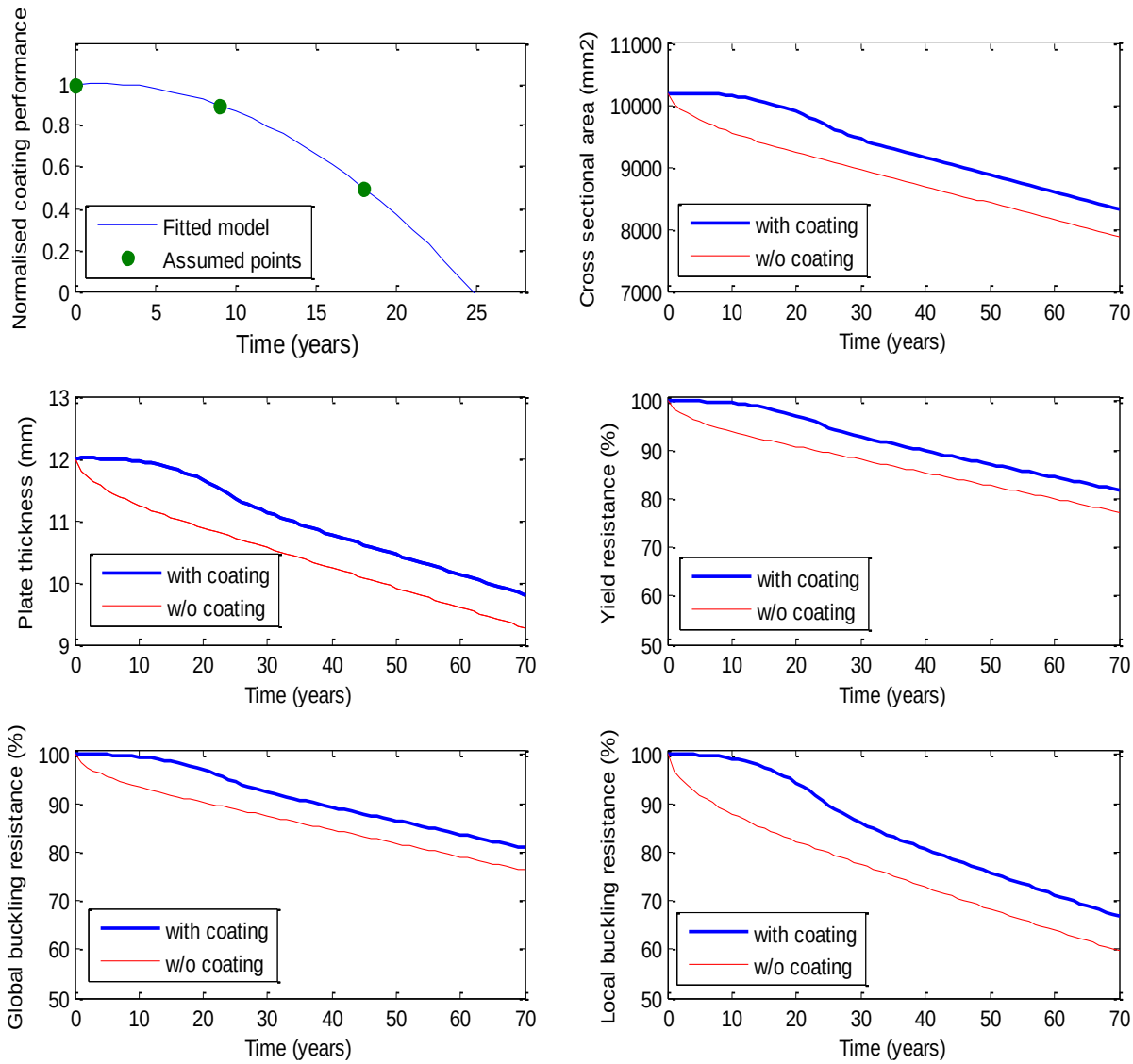


Figure A4-11

In the present study, the case without coating will be considered to evaluate the robustness. The service life period considered is 100 years, this means that the maximum damage level will be the one corresponding to a period of 100 years.

A.4.3 Robustness assessment

According to equation A4-.1, to evaluate the robustness index, it is necessary to obtain the bridge performance for different levels of corrosion degradation, from 0 (intact bridge) to 1 (maximum corrosion after 100 years of exposure without coating). In this case, a deterministic approach is adopted, defining the bridge performance in terms of the load factor. The load factor is defined as the number of times the live load has to be increased to reach the maximum failure load.

To evaluate the load factor, the same model as presented in appendix A and ML-D1.2 (2013) was used, based on a FEM model. Different corrosion scenarios have been considered, as follows (see figure A4-1 for location of elements. T are the transverse elements, not appearing in the figure):

- Corrosion affecting only to element O4 in one longitudinal carrying truss
- Corrosion affecting only to element V5 in one longitudinal carrying truss
- Corrosion affecting only to element U4 in one longitudinal carrying truss
- Corrosion affecting only to element T5 in one longitudinal carrying truss
- Corrosion affecting only to element V4 in one longitudinal carrying truss
- Corrosion affecting only to element D4 in one longitudinal carrying truss
- Corrosion affecting only to element D3 in one longitudinal carrying truss
- Corrosion affecting only to element T4 in one longitudinal carrying truss
- Corrosion affecting all elements (CGEN) in both longitudinal carrying trusses

In the case of exposure class C2, the results shown in figure A4-12 are obtained. As expected, the most sensitive member to corrosion is the upper chord, as the failure is due to buckling of this element. The bridge is almost non-sensitive to the corrosion in other key members such as diagonals and tension chord. However, even for the case of corrosion in all members of the bridge, the load capacity only decreases around 4 % for the maximum service life. The robustness index obtained in this case is $R_d = 0.98$.

The high value of robustness obtained indicates that the bridge is able to maintain the required safety level without any maintenance, or, in other words, the bridge has the ability to accommodate to degradation and can wait to maintenance interventions. This is of great interest from a life-cycle management perspective.

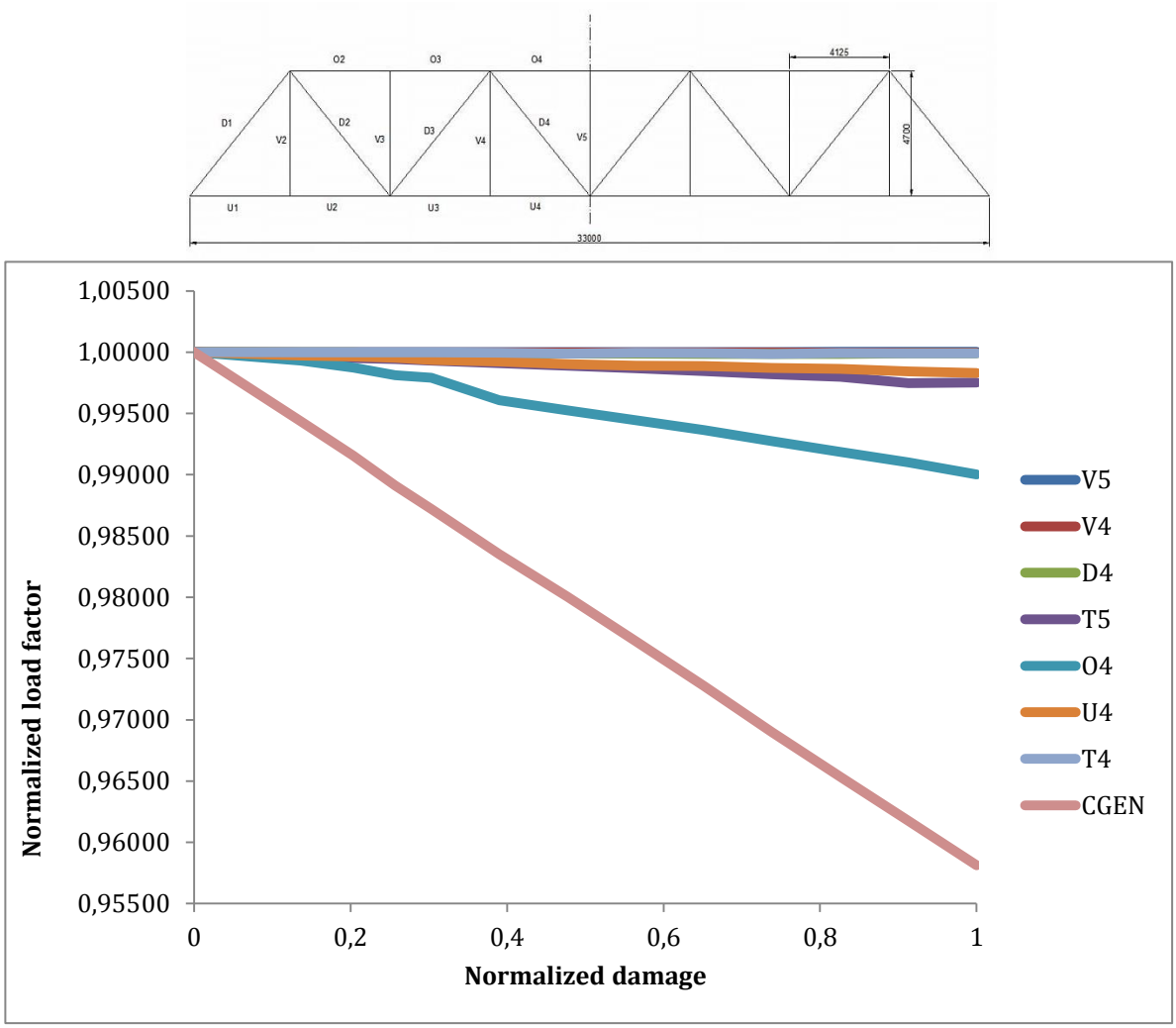


Figure A4-12.. Normalized bridge performance vs. normalized corrosion deterioration. Exposure class C2

B. Strengthening by post tensioning of Haparanda Bridge

The Haparanda Railway Bridge, Fig. B-1, was strengthened in the summer of 2012. The purpose of the strengthening project was to improve the load bearing capacity of the bridge to reflect an upgrade of the loads on the railway line. The maximum allowed axle load was increased from 250 to 300kN, and for this reason, the Haparanda Bridge required a higher transverse shear capacity of the slab. Assessment calculations indicated a 24% deficit in the shear capacity of the slab in the transverse direction (WSP Group 2008). The maximum shear capacity of the slab was 150kN/m, and the required shear capacity was 186kN/m. Thus a minimum shear capacity increase of 36kN/m was required. The strengthening method used to increase the capacity was internal post-tensioning and the design was performed according to Eurocode 2 (CEN 2008).

A horizontal strengthening system consisting of 8 unbonded post tensioning bars were installed internally at mid-height of the slab and post-tensioned in the transverse direction. The post-tensioning not only increased the shear capacity of the slab by 25%; the flexural capacity was also increased by 13%, existing cracks were reduced in width and further cracking was postponed.

The bridge was tested before and after strengthening, and the results showed that the prestressing completely counteracted the tensile strains in the main slab-reinforcement under a 215 kN/axle test-train load. The maximum tensile strains were however small, with magnitudes of only about 20 µm/m for the test-train (23 micro strain before strengthening and 20 micro strain after strengthening), which implied a higher original capacity of the bridge than calculated.

Figure B-1 The Haparanda Railway Bridge was strengthened in the summer of 2012. Eight transverse prestressing bars were installed at mid-height of the slab (left insert) and post-tensioned to upgrade the axle load capacity of the railway line from 25 to 30 ton.

B.1 Geometrical and material properties

The Haparanda Bridge is a reinforced concrete trough bridge with two parallel troughs, one for each railway track. A main road runs underneath the bridge, and because of the road direction, the superstructure is skewed by 17°, as shown in Fig. B-2. The free span of the 50 year old bridge is roughly 12.5 m, and the total width of both troughs is about 10.5 m, including the top flanges. The outer girders are 1.2 m high, while the inner girders, connecting the two troughs

are 1.3 m, and the slab-thickness is 0.4 m. The geometry and cross section of the bridge is shown in Fig. B2 and B-3.

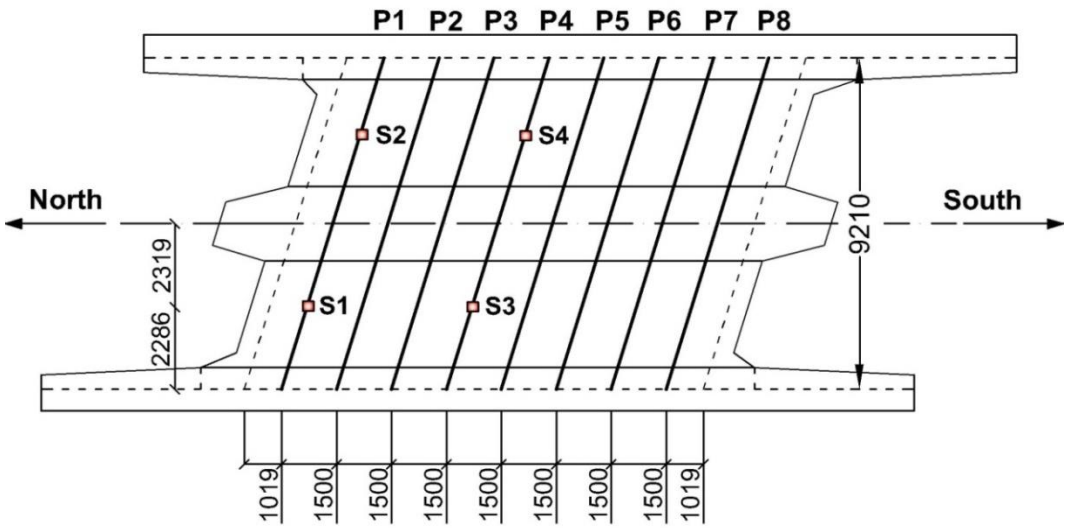


Figure B-2 Plan of the Haparanda Bridge and placement of the prestressing bars, P1–P8; S1–S4 denote strain gauges attached to the reinforcement; measurements are given in millimeters.

The main transverse reinforcement in the slab has a diameter of 19 mm, while 12 and 25 mm reinforcement were used in the beams The steel quality was denoted ks40, with characteristic yield strength of 410 MPa and a Young's modulus of 200 GPa. However, no tests were performed on the reinforcement from the structure. The concrete quality was tested on four cylinder cores in 2008 and the characteristic compressive and tensile strengths were 23.2 and 1.5 MPa, with standard variations of 6.06 and 0.47 MPa, respectively (WSP Group 2008).

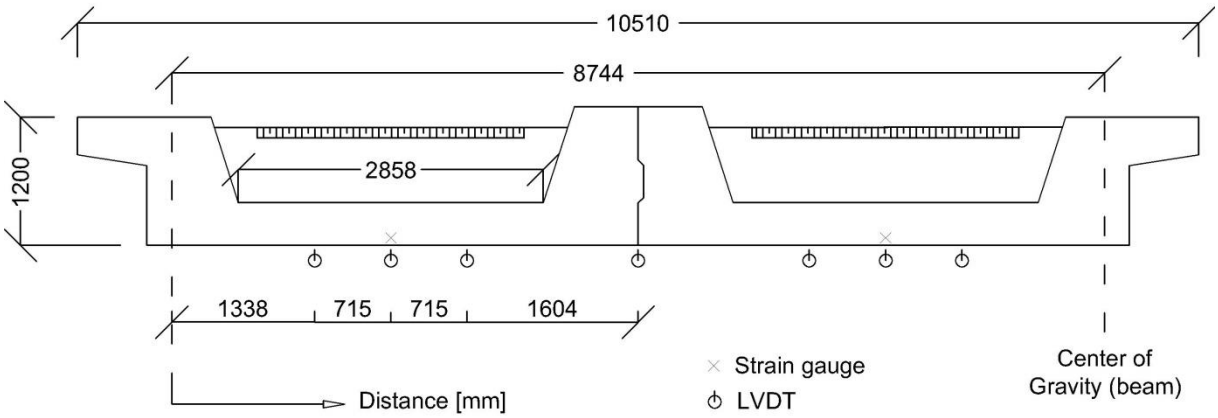


Figure B-3 Cross section of the Haparanda Bridge; measurements are given in millimeters.

B.2 Strengthening

The deck of the Haparanda Bridge was strengthened with an unbonded post-tensioning system consisting of eight post-tensioned high tensile alloy threaded steel bars with nominal diameter of 26.5 mm, characteristic tensile strength of 1050 MPa and Young's modulus of 205 GPa for the Y1050H steel (CEN 2000) n the transverse direction. The strengthening procedure was divided into four strategic working steps:

1. Transverse drilling of horizontal holes through the bottom slab at mid-height;
2. Installation of an unbonded post tensioning system with steel bars;
3. Post-tensioning of the system; and
4. Sealing of the post tensioning system.

The advantage of having an unbonded strengthening solution is that individual bars can be replaced easily if they are accidentally damaged, corroded, or no longer needed, and the level of tensioning can be adjusted at a later time, as required.

B.2.1 Drilling of horizontal holes

The core drilling method was used to produce eight horizontal holes (57-mm diameter) through the bottom slab of each of the two troughs. The holes were drilled in the same direction as the main transverse slab-reinforcement, at an angle of 73° to the concrete surface. The reason for having transverse reinforcement in this direction is to align it with the substructure. The reason for drilling the holes in the same direction as the substructure is to gain maximum compression of the tensile reinforcement in the post-tensioned state.

The slab was 400 mm thick, and the holes were located in the vertical midsection (i.e., 200 mm from the bottom surface of the slab). The main reason for positioning the holes in the midsection, rather than a lower position, which would give higher flexural capacity, was to prevent cutting of the existing internal reinforcement.

A total of eight holes with a lateral center-to-center separation of 1500mm were drilled through the structure. According to the design directions in Eurocode 2, section 8.10.3(5) (CEN 2008), this choice ensures full compressive action across the entire length of the slab. As the outer main girders have a width of 857 mm, they provide a sufficient length for dispersion of the prestressing force before reaching the slab (see Fig. B-4). The geometry described previously, including the placement of the prestressing bars, was illustrated in Fig. B-2.

B.2.2 Installation of the post tensioning system

The installation of the post tensioning system can be divided into four consecutive steps following the drilling:

1. Installation of polyethylene (PE) ducts;
2. Installation of post tensioning bars;
3. Installation of load-distributing wedges (see Fig. B-4 and B-5); and
4. Installation of the anchoring system

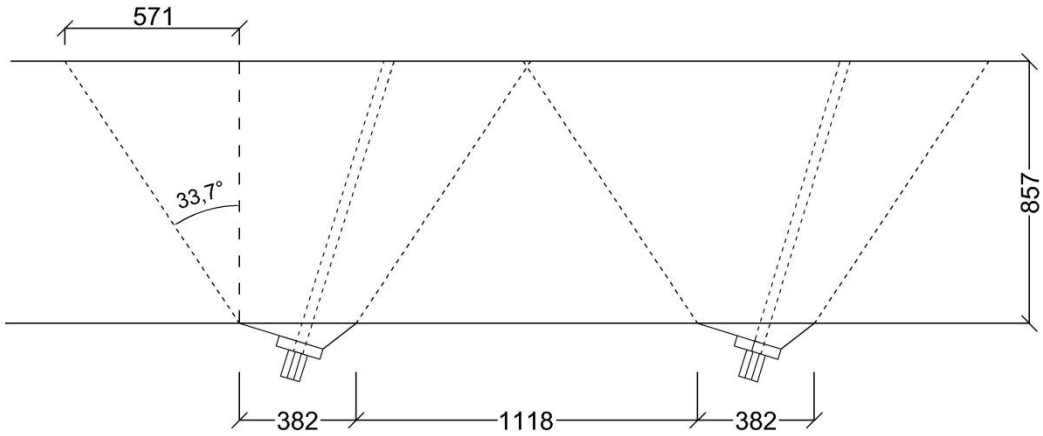


Figure B-4 Load distribution through the main girders with a width of 857 mm; measurements are given in millimeters

PE ducts

The first step in the installation of the post tensioning system was to insert ducts into the drilled holes. The ducts can be made of either steel or PE, the latter being chosen for this project. The function of the duct is to provide mechanical protection for the heat-shrunk sleeve, which surrounds the prestressing bar. The heat-shrunk sleeve provides permanent corrosion protection for the prestressing steel.

Post tensioning bars

After installation of the ducts into the transverse holes, the post tensioning bars penetrated the ducts. The post tensioning bars needed to be longer than the drilled holes, to enable anchoring and tensioning. The excess length was dictated by the tensioning equipment and the anchoring design. The post tensioning bars were installed in the center of the ducts and left unbonded.

Load-distributing wedges

A perpendicular contact between the tensioning system and the concrete structure is required in most design codes. This ensures effective stress transfer between the post tensioning bars and the concrete structure and prevents shear stress at the contact between the steel anchor and the concrete surface. Because the holes were drilled at an angle of 73°, and not perpendicular to the superstructure, a galvanized steel wedge that ensured the required perpendicularity was custom designed, see Fig. B-5. The wedges also distributed the tensioning force over a larger concrete area, and thus functioned as load distributors. This prevented local crushing or splitting of the concrete behind the post-tensioning anchors. All steel wedges were bonded to the concrete surface, thus keeping them stable during installation and preventing water leakage into the holes. The bonding was achieved by painting an epoxy adhesive onto the surface.

Eurocode section 8.10.3(5) states that the tensioning force may be assumed to disperse at an angle of 2β starting at the end of the anchoring device (CEN 2008). The value of β is assumed to be arctan(2/3) = 33.7o. The tensioning force was transferred from the bar, through the anchor plate and the distribution wedge and onto the concrete structure. The external main beam of the trough bridge, which was assumed to act as a load distribution device for the concrete slab, has a width of 857 mm. The force was dispersed over a distance of 571mm by passing through the beam. With a prestressing bar separation of 1,500 mm, the required width of the distribution wedge was given as 1,500 - 2·571 = 358 mm. Ultimately, a slightly larger width of 382 mm was chosen for the distribution wedges, as shown in Fig. B-4

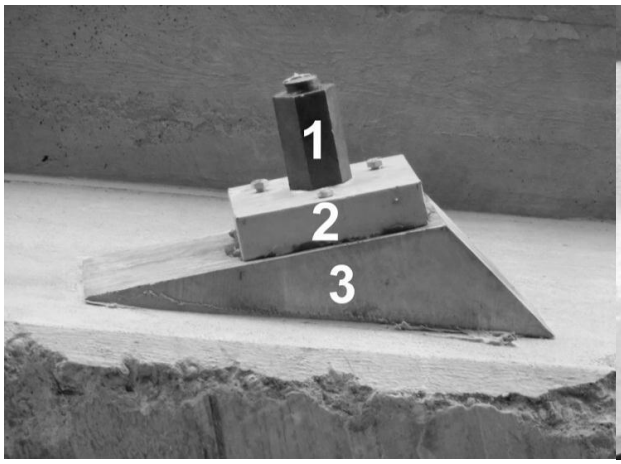


Figure B-5 One of anchoring systems, including (1) anchoring nut; (2) anchoring plate; and (3) load-distributing wedge.

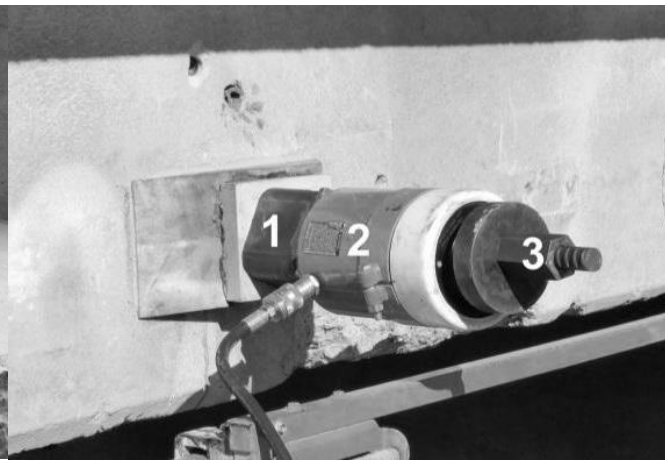


Figure B-6 Tensioning setup, including (1) steel frame; (2) hydraulic jack; and (3) extra prestressing nut.

Anchoring system

The anchoring system consisted of rectangular galvanized steel anchor plates, with dimensions of 140x165x35 mm, and anchor nuts with a length of 90 mm. These nuts anchor the prestressing bars at a certain stress level and transfer the tensioning force from the bar to the structure. Anchor plates are usually in direct contact with the concrete structure, serving to distribute the tensioning force from the anchor nuts directly onto the concrete structure. However, the bearing surface of the anchor plate must be perpendicular to the prestressing bar. This was, in part, the reason load-distributing wedges were used as a compensating layer between the anchor plates and the concrete structure. The anchor plates were all epoxy bonded to the load-distributing wedges to prevent water leakage into the holes. The load-distributing wedge and the anchoring system are shown in Fig. B-5.

B.2.3 Post-tensioning

Once the post tensioning system had been installed, the post tensioning procedure began. The eight bars were post tensioned with a force of 430 kN per bar, resulting in a total force of 3.44 MN acting on the concrete slab. A hydraulic jack was used to provide the required stress for one bar at a time, beginning with the outermost bars and proceeding inwards. The hydraulic pressure corresponding to a tensioning force of 430 kN was first calibrated against a load cell.

A steel frame was designed to ensure accurate tensioning of the bars. The steel frame had openings on both the bottom and top, and one of the side walls had an opening that provided access to the anchor nut. The bottom side of the steel frame rested on the anchor plate, with the bar running through the bottom and top openings. The anchor nut was inside the frame. The hydraulic jack was positioned on the bar, resting on the top side of the steel frame. An extra nut was screwed onto the end of the bar, which protruded from the jack. With the extra nut in place, acting as resistance at the tensioning side of the slab, and the anchor nut in place, acting as resistance on the other side, the bridge could finally be post tensioned. During the tensioning process, the anchor nut on the stressing side of the slab was continuously tightened.

After reaching the desired tensioning force and tightening the anchor nut as much as possible, the pressure in the hydraulic jack was released and the post tensioning force was transferred from the bar to the concrete structure. An elastic strain relaxation of approximately

6% occurred as the hydraulic jack was released. Therefore, all bars were overstressed, to obtain a final prestressing force of 430 kN. Finally, the extra nut, hydraulic jack, and steel frame were removed. The post tensioning arrangement is shown in Fig. B-6.

B.2.4 Sealing

The strengthening system contains mainly steel parts. Thus, if left untreated, the strengthening effect may be reduced over time by corrosion. To ensure the longevity of the strengthening system, an adequate corrosion protection solution was required. The solution adopted for this system was sealing the strengthening system after post tensioning.

First, permanent corrosion protection in the form of heat shrunk sleeves covered the prestressing bars. Second, each sleeve was protected against mechanical impact by the PE duct. The connection between the steel wedge and the concrete structure, as well as that between the steel wedge and the anchor plate was sealed by a permanent water-resistant compound (epoxy adhesive).

Finally, the anchor nuts and bar ends were sealed by welding a retention cap onto the anchor plate. As a further corrosion-protection measure, all steel wedges, anchor plates, and retention caps were galvanized. The retention cap is shown in Fig. B-1.

B.3 Monitoring

Structural movements were monitored by linear variable differential transformers (LVDT) and crack opening displacement transformers (COD). The reinforcement was exposed by chiseling, and strain gauges (SG) were welded to the reinforcement and post tensioning bars after local surface grinding of the threaded bars.

B.3.1 Strains

Four SGs were welded onto the internal, transverse, bottom reinforcement of the slabs (S1–S4), and four additional SGs were welded onto the post tensioning bars P1–P4 (S5–S8). The locations of all eight SGs are shown in Fig. B-2. S1 and S2 coincide with the longitudinal location of the prestressing bar P1, whereas S3 and S4 coincide with the longitudinal location of the prestressing bar P4.

B.3.2 Deflections

A total of 16 LVDTs (L1–L16) monitored the vertical displacements of the structure: six along a transverse line beneath prestressing rod P4 and the remaining 10 in two longitudinal lines along the midsection of the two bridge slabs.

B.3.3 Joint Opening

The Haparanda Bridge consists of two concrete troughs, which were most likely cast on two separate occasions. The distance between the troughs at the connection joint was monitored by three CODs (C1–C3). The CODs measured the differential transverse movement of the two troughs at the joint.

B.4 Test Program

The test program for the Haparanda Bridge consisted of two sets of eight tests (two static tests and six dynamic with known velocities): one before and one after the strengthening process. The protocol for the complete test program for the Haparanda Bridge is given in Table B-1. As the speed limit over the bridge was 20 km/h this was the maximum test velocity.

Table 9-1 Test program.

Test program for unstrengthened bridge				**Test program for strengthened bridge**			
Test	**Track**	**Direction**	**Velocity [km/h]**	**Test**	**Track**	**Direction**	**Velocity [km/h]**
St1:1	Main	North	0	St1:2	Main	North	0
St2:1	Secondary	North	0	St2:2	Secondary	North	0
Dy1:1	Main	North	5	Dy1:2	Main	North	5
Dy2:1	Main	South	5	Dy2:2	Main	South	5
Dy3:1	Main	North	10	Dy3:2	Main	North	10
Dy4:1	Main	South	10	Dy4:2	Main	South	10
Dy5:1	Main	North	20	Dy5:2	Main	North	20
Dy6:1	Main	South	20	Dy6:2	Main	South	20

The test load consisted of two coupled diesel locomotives (GC Td44), pictured in Fig. B-7, with an axle load of 215 kN. The axle separation on the locomotives was sufficiently large that during testing, a maximum of two axles were located on top of the bridge slab at any time. For the static tests, the locomotives were placed such that the two axles were equidistant about the midspan of the bridge.

Figure B-7 The two locomotives used as loads on the Haparanda Bridge; picture taken from the west side of the bridge.

B.5 Results and analysis

B.5.1 Strains

The strain levels were calibrated before the testing of the unstrengthened bridge. This means that the strains caused by, for example, dead loads are not included in the strain results. Fig. B-8 show the strain curve for two static tests, one before (left figure) and one after strengthening (right figure). The train loads are the only sources of strain measured in the tests before strengthening. During post-tensioning, the tensioning compressed the reinforcement. Because the reinforcement compression varied over the four measuring locations, the strain curves corresponding to different gauges have differing initial strain levels.

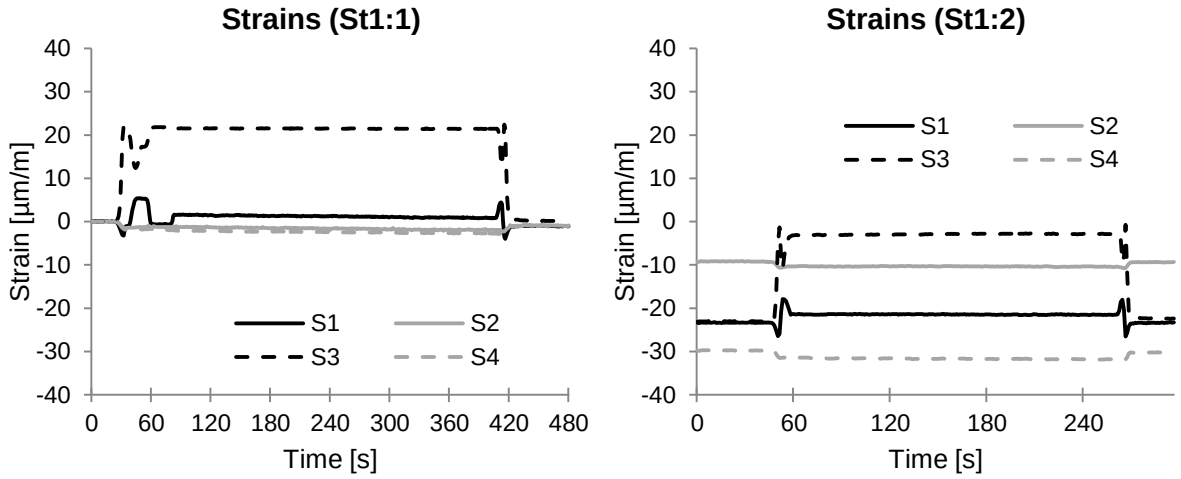


Figure B-8 Strains for the static loading of the main track, a) before strengthening (left figure) and b) after strengthening (right figure).

The strains in the main reinforcement of the concrete slab were clearly affected by the post-tensioning. At the onset of testing, all strains were calibrated to zero. After testing the unstrengthened bridge, the strains returned to their original level. The strain levels in the four SGs decreased as expected during tensioning, as this compresses the concrete slab and therefore the internal reinforcement. However, the amount of compression of the four reinforcement bars varied greatly. The reasons for the variation is not fully understood, but might have been caused by local cracking, or the fact that the four separate reinforcement bars, two of them closer to the support, were perhaps not arranged completely similarly. The strains caused by the static tests had maximum magnitudes of approximately 20 µm/m. S3 was the SG mostly affected by loading of the main track, while S4 was mostly affected by loading of the secondary track.

Although the post tensioning reduced the reinforcement strains on the bridge, the total strain magnitudes due to the loads remained constant, whether the bridge was strengthened or not. The static tests led to maximum strain magnitudes of approximately 20 micro strain for the unstrengthened bridge, whereas for the strengthened bridge the maximum was less than zero. This demonstrates that the train loads were completely counteracted by the post tensioning.

B.5.2 Deflections

The strengthening had no measurable effect on the deflection of the concrete slab, because the post tensioning was implemented close to the neutral layer. The deflection curves in all directions showed similar magnitudes before and after strengthening. However, the deflection curves after strengthening were straighter, perhaps because of the reduction in vibrations due to post tensioning. By compressing the troughs and increasing the interaction between the two bridges, the vibrations in the dynamic tests (not presented in this report) decreased significantly. With respect to the expected life span of the bridge, this decrease, combined with the lower strain, would reduce fatigue, and is therefore of utmost importance. The deflections along the transverse line showed that even before strengthening there was a good interaction between the troughs, with a maximum deflection of approximately 0.5 mm at the midpoint of the loaded trough.

B.5.3 Joint Opening

The CODs monitored the opening of the connection joint between the two troughs. No significant difference was observed in the static tests. In the dynamic tests, however, a slightly smaller spreading of the curves for the strengthened bridge was observed. The prestressing resulted in a tightening of the joint, but the reduction of the opening was not constant along the joint. C1 showed the largest compression, of approximately 0.028 mm, whereas C2 and C3 were compressed by 0.011 and 0.007 mm, respectively. As the joint opening reduces, the posttensioning force would theoretically decrease, but because of a small initial opening, long-term effects on the joint opening were neglected.

B.6 Discussion

A pronounced increase in the structural load-carrying capacity of the Haparanda Bridge was obtained by transverse post-tensioning. The post tensioning compressed the bottom reinforcement by 9.2 – 29.8 micro strain, with the greatest compression occurring along the longitudinal midsection of the bridge. The longitudinal midsection also experienced the largest stresses during loading, with an axle load of 215 kN corresponding to a maximum reinforcement strain of approximately 20 micro strain. The strain in the reinforcement decreased by 20 micro strain. Thus, the tests showed that the strains caused by the train load were counteracted by the post-tensioning.

The effect of the strengthening on the shear capacity cannot be directly deduced from the strain level in the horizontal main reinforcement. However, it is assumed that there is a direct relationship between lower horizontal reinforcement strains and higher shear capacity. This assumption is due to a higher degree of aggregate interlock and increased resistance for flexural shear cracks in a post tensioning structure. A laboratory pilot test on scaled-down trough bridges presented in Nilimaa et al. (2012) suggested that the actual strengthening effect on shear capacity, for transversely post-tensioned slabs, is greater than the design calculations predict. However, further laboratory tests are needed before any clear conclusions can be drawn.

Transverse post-tensioning has a stabilizing effect on the structure, which can be seen in the smoother post-strengthening deflection curves and decreased oscillation amplitudes in the reinforcement bars during dynamic loading, see Nilimaa (2013). The stabilizing effect is assumed to have a positive effect on the life span of the structure because of improved fatigue performance. However, pre- and post-strengthening vibration monitoring should be performed

to further investigate and validate this assertion. Further laboratory tests are required to confirm the results of this paper and investigate the reason for the difference in compression between different post tensioning bars. The design for lateral distance between post tensioning bars might also be investigated and refined through further laboratory tests. Further details are found in Nilimaa (2013) and Nilimaa et al. (2014).

B.7 Conclusions

The conclusions that can be drawn from this case study are as follows:

- The load-carrying capacity of a double-trough bridge can be increased by unbonded post tensioning along the transverse direction of the bottom slab, and the positive effect is apparently greater than that predicted in the design calculations;
- The initial degree of inter-trough interaction was high, and the two troughs acted as a single unit;
- Smoother post-strengthening deflection curves indicate decreased vibrations in the strengthened bridge;
- Less spreading of the COD results for the dynamic tests of the strengthened bridge also indicates decreased vibrations in the strengthened bridge; and
- The strengthening method is quick (the Haparanda Bridge was strengthened and tested in two days) and has no impact on the railway traffic.

C. Tunnels - Case studies

C.1 Life extension of Strood and Higham Tunnels

The Strood and Higham Railway Tunnel is some 3.7km in length and was originally built as a canal tunnel enabling boats to pass freely between the river Medway and the river Thames. The tunnel was excavated between 1819 and 1824 through reasonably competent Seaford Chalk such that many sections were left unlined. In 1830, a 100m long section was opened out in cutting as a passing basin for the canal boats before in 1844 a single track railway was added to the tunnel. Finally in 1846, the canal was infilled to permit a double railway track to be built. Over the years, the operating railway has suffered disruption because of flooding, problems associated with old shafts and from chalk falls in unlined sections of the tunnel, one of which in December 1999 caused derailment of four railway carriages. Six months later one of the original construction shafts collapsed and the tunnel was closed for four weeks, re-opening only after a 30km/hour speed restriction was applied.

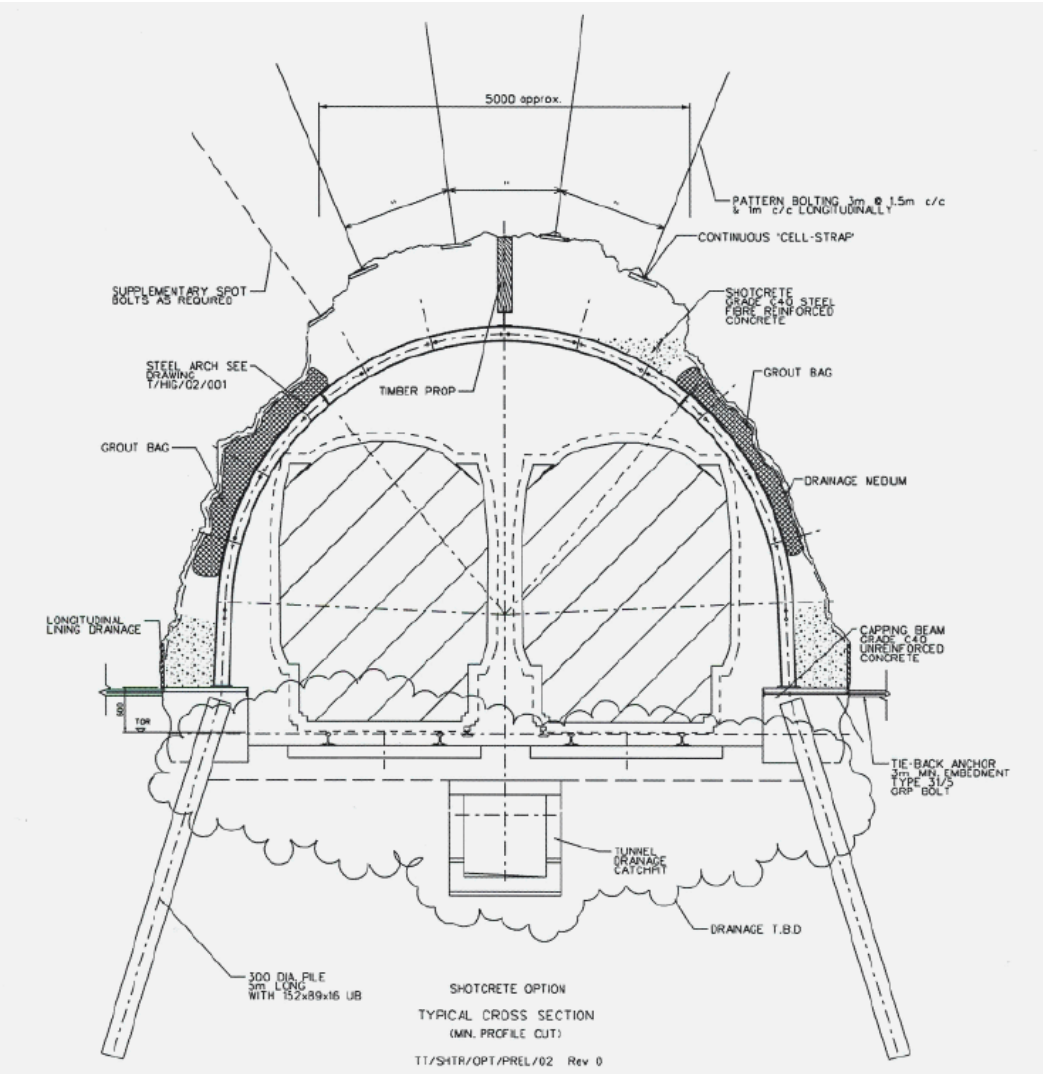


Figure C1.1 Typical cross section showing various aspects of the tunnel refurbishment works

This was clearly not a viable long-term solution and following option studies, in January 2004 a 12 month blockade began, to allow the tunnel to be fully refurbished. The refurbishment comprised the following work, illustrated in Figure C1.1:

- lining the remaining unlined sections of tunnel (including those sections covered and/or supported by Armco, shotcrete or a steel canopy arrangement) by:-
    - piling and capping beam works ahead of the main protective canopy
    - rock bolting, chalk trimming and arch/mesh erection from beneath the canopy
    - concrete lining works using a steel shutter behind the canopy
- construction of refuges typically 1.4m wide and 0.7m deep at 20m centres along the full length of tunnel
- stabilisation and infilling the remaining shafts including collapsed shaft S4
- brickworks repairs to existing lined sections where required
- complete renewal of the track

Use was made of as much of the existing rail track as possible. The works were carried out 24 hours a day for 5 days a week, with maintenance and movement of materials being undertaken on Saturday and Sunday. (Warren & Tromans)

C.2 Life extension of Holme Tunnel

Holme Tunnel, on the line between Burnley and Hebden Bridge, is 250m long had been constructed in the 1840s and by 2013 was becoming increasingly misaligned because of local ground movement which had distorted the tunnel walls and caused the installation of temporary supports. Improvement work to realign and strengthened large sections of the tunnel's walls principally using new permanent steel rings and sprayed concrete was undertaken during a 20 week closure. The project required the use of over 400 tonnes of new steelwork, over 2,400 tonnes of fibre reinforced concrete and over 650 tonnes of precast concrete. During the closure around 2km of track was also renewed.

Figure C2.1 External and internal views before refurbishment

Figure C2.2 Installing rings (left) and sprayed concrete (right)

Figure C2.3 External and internal views after refurbishment

C.3 Life extension of Whitleball Tunnel

Whiteball tunnel near Taunton in Somerset is a 1094 metre long twin-tracked railway tunnel constructed in 1842. The tunnel had significant areas of brickwork delamination and deep open joints with considerable water ingress affecting some areas through weathering, water ingress and sulphate attack. Network Rail's standard tunnel repair methods were continually being carried out but the rate of deterioration was such that a completely new approach was needed.

For the first phase of the project the Ramarch system, which offered intimate support to the tunnel lining and completely eliminated the problem of falling brickwork which was risking the safe running of the operational railway, was proposed to treat six areas totalling 355 linear meters as it provided a safe method of working and rapid installation times. The works were carried out over six 48 hour weekend possessions.

RamArch on this project was fixed in position by using 200 mm long bolts, set in place using high-speed fast setting resin although there are other systems that could be used. The design called for further bolts to be fitted around the periphery of the arch again using fast setting resin to form a very robust structure.

Figure C3.1 Installation of the RamArch system

(http://www.iss-eng.com/products/ramarch/casestudies/whiteball-tunnel-devon)

The second phase of the project involved the application of some 1500 tonnes of sprayed concrete via a system of 16 remotely controlled robotic arms to form a permanent lining. This work was undertaken during a undertaken during a 23-day blockade of the line.

Figure C3.2 Applying sprayed concrete.

http://www.amco-construction.co.uk/project/48/Whiteball-Tunnel

The final phase of the work will consist of drilling and grouting the tunnel haunches.

(From http://www.amco-construction.co.uk/project/53/Whiteball-Tunnel-Lining-Repairs , http://www.amco-construction.co.uk/project/48/Whiteball-Tunnel & http://www.iss-eng.com/products/ramarch/casestudies/whiteball-tunnel-devon)

D. Fatigue

D.1 Introduction

Fatigue is the weakening of a material that is subjected to repeated loading and unloading (cyclic loading), causing progressive and localized structural damage.

If the cyclic loads in metals are above a certain threshold microscopic cracks will begin to form at stress concentrators. They will slowly grow until they reach a critical size when, due to overstressing of the remaining uncracked material, they will suddenly develop through the full material thickness causing failure of the affected part. The nominal maximum stress values that initiate such damage may be much less than the strength of the material typically quoted as the ultimate tensile stress limit, or the yield stress limit.

Fatigue fractures are characterised by a partly smooth surface, created as the fatigue crack slowly grows and the material rubs together, coupled with a grainy surface caused by the sudden final failure of the material, as illustrated in Figure D.1

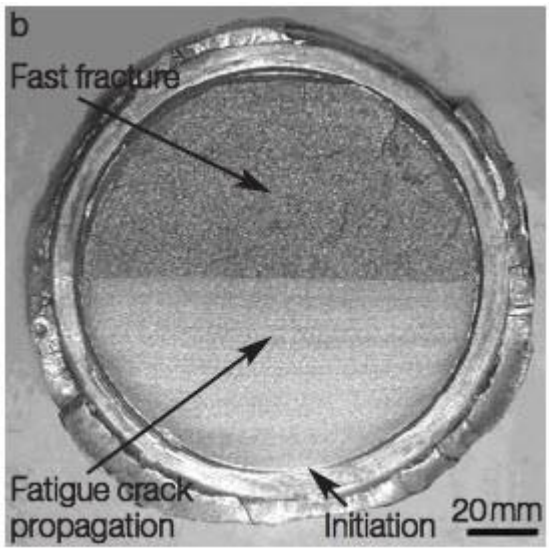


Figure D.1 A typical fatigue failure surface
http://www.keytometals.com/images/Articles/kts/Fig299_5.jpg

The shape of the structure will significantly affect its ability to resist fatigue; square holes or sharp corners will lead to elevated local stresses where fatigue cracks can initiate. Round holes and smooth transitions or fillets will therefore increase the fatigue resistance of the structure. Figure D.2 illustrates a fatigue crack growing from the square corner of a cut out notch in a main girder/cross girder connection.

This appendix is based on a report originally initiated by Trafikverket in Sweden, see Elfgren (2015).

Figure D.2 A fatigue crack in a main girder/cross girder connection.

The study of fatigue in engineering materials was started by Wöhler in 1847, which led to the formulation of the basic curve connecting the stress ranges (S) and the number of cycles (N) a material can stand. This is known as the SN (or Wöhler) curve and Figure D.3 illustrates a typical modern SN curve which uses a log-log scale introduced by Basquin in 1910.

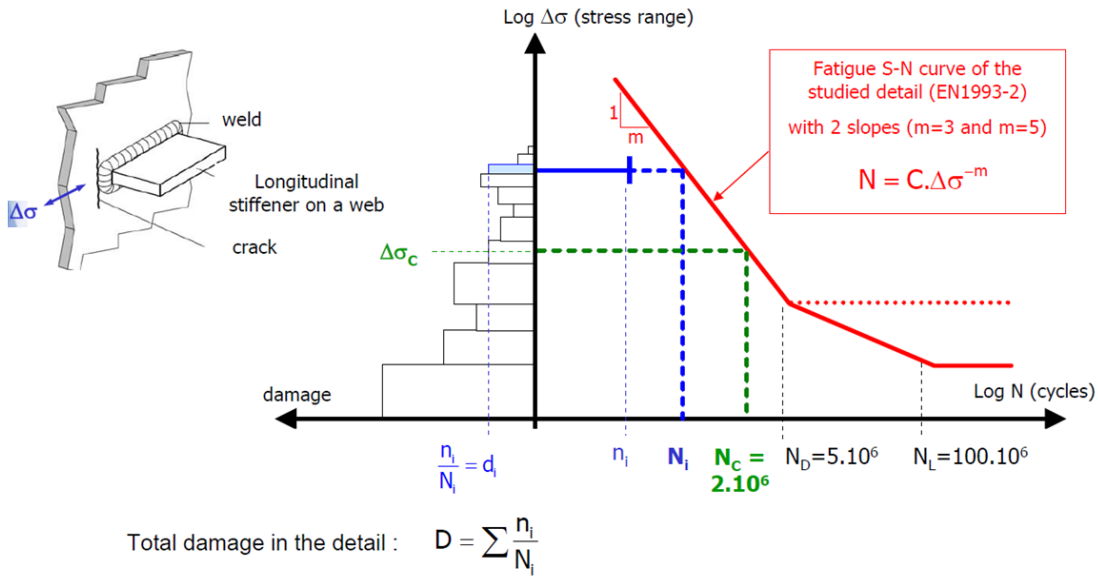


Figure D.3 Shape of the characteristic fatigue strength curve for a weld (Wöhler or S-N curve). Δσ is the stress range (S) and n and N are the number of load cycles. D is the damage according to Palmgren-Miners hypothesis. From Raoul-Devaine (2008) based on EC3 (2006), Part 2: Bridges, EN1993-2-2006.

This appendix is based on a report that was originally prepared to give a background and a proposal to a guideline for assessment of fatigue of concrete structures in codes issued by Trafikverket in Sweden. For this appendix the work has been widened to also consider fatigue of steel bridges and it presents a review of methods and principles – how they have emerged, what is used today and what development that is going on. Special questions addressed are:

- Does any good method exist that is based on partial damage hypotheses?
- What determines a load cycle?
- What is the influence of earlier loads with low axle loads?
- How is the design stress influenced by fatigue?
- How is the dynamic factor to be determined/used?

D.2 Fatigue of steel reinforcement

D.2.1 General

The fatigue behaviour of steel reinforcement is similar to the fatigue behaviour of elements in steel construction and has the following relevant parameters:.

- the *stress range* $\Delta\sigma$
  Due to stress concentrations that always are present; the maximum stress level will always be the yield stress. The stress range will thus always have its maximum value at the yield stress and any calculated mean stress has no influence.

- the *number of stress cycles n*

- *discontinuities*
  both in the cross section and the layout of the steel reinforcement, resulting in stress concentration at possible fatigue damage locations.

The fatigue life of steel reinforcement can be divided into a *crack initiation* phase, a *steady crack propagation phase* and *fracture of the remaining section*, see Figure 3.1.

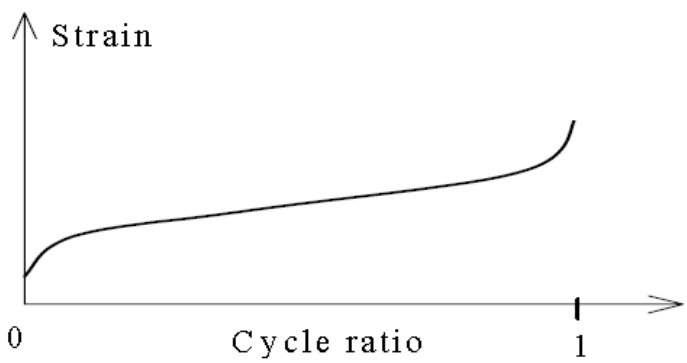


Figure D.4 Strain versus cycle ratio for fatigue of steel and concrete with initiation phase, propagation phase and fracture phase. From SB-D4.5 (2007)

D.2.2 State of the Art

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 and depends on steel quality and surface properties. For steel reinforcement characteristic values for design according to the fib Model Code 2010 (2012) are given in Tables D.1 and D.2 and in Figure D.5 The values differ slightly from the values in Figure D.4 regarding inclination (lower inclination 1/5 for ordinary reinforcement bars instead of 1/3 for steel) and breaking point (1 million cycles for ordinary reinforcement bars instead of 5 million cycles for ordinary steel). The reason for the gentler slope for reinforcement may be a positive influence from surrounding concrete. For welded reinforcement bars and for bars in marine environment the curves have characteristics that are quite the same as for ordinary steel. Reductions in reinforcement stress ranges due to friction may not need to be considered for ordinary reinforcement

Table D.1 Parameters of S-N curves for reinforcing steel (embedded in concrete), fib Model Code 2010 (2012), fib B62 (2012)

	N*	Stress exponent		$\Delta\sigma_{Rsk}$ (MPa) [e] at	
		k_1	k_2	*N* cycles*	*10^6 cycles*
Straight and bent bars with mandrel diameter $D \geq 25\ ø$					
$ø \leq 16$ mm	10^6	5	9	210	125
$ø > 16$ mm [a]	10^6	5	9	160	95
Bent bars with $D < 25\ ø$ [b]	10^6	5	9	(c)	(c)
Welded bars including tack welding and butt joints and mechanical connectors	10^7	3	5	50	30
Marine environment [b], [d]	10^7	3	5	65	40

[a] Values for $\phi = 40$ mm. For $16 \leq \phi \leq 40$ mm linear interpolation with the values for $\phi \leq 16$ mm is permitted.
[b] Most of these S-N curves intersect the curve of the corresponding straight bar. In such cases the fatigue strength of the straight bar is valid for cycle numbers lower than that of the intersection point.
[c] Values are those of the according straight bars multiplied with a reduction coefficient ξ depending on the ratio of the diameter of the mandrel D and the bar diameter ϕ for bent bars that can be taken as $\xi = 0{,}35 + 0{,}026\ D/\phi$
[d] Valid for all ratios D/ϕ and all diameters ϕ
[e] In cases where $\Delta\sigma_{Rsk}$ values calculated from the S-N curve exceed the stress range $f_{yd} - \sigma_{min}$, the value $f_{yd} - \sigma_{min}$ is valid.

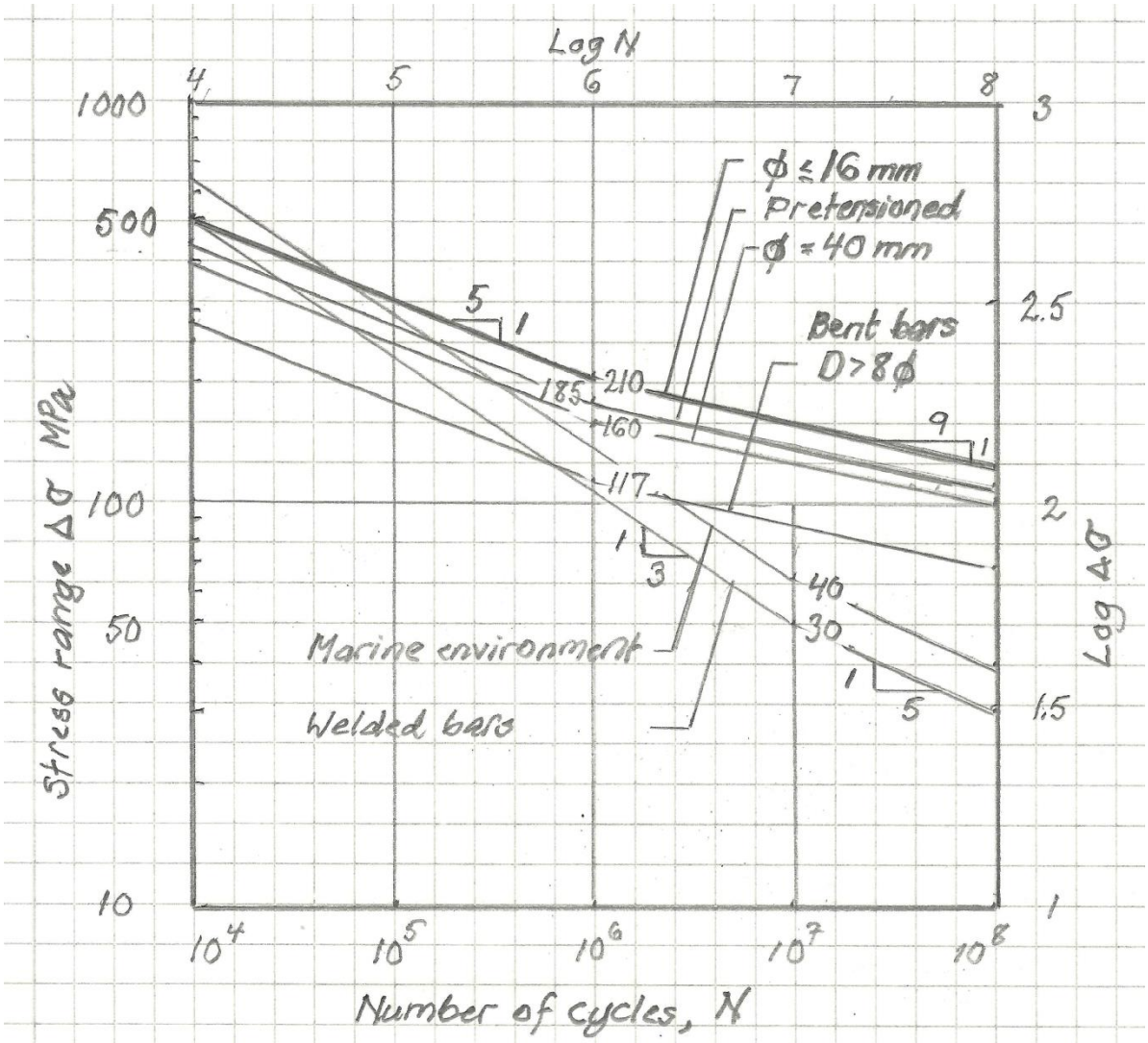


Figure D.5 Stress range (Wöhler) curves for reinforcement bars according to fib Model Code 2010 (2012).

Table D.2 Parameters of S-N curves for prestressing steel (embedded in concrete), fib Model Code 2010 (2012), fib B62 (2012).

	N^*	Stress exponent		$\Delta\sigma_{Rsk}$ [MPa] [a] at N* cycles
		k_1	k_2	
Pretensioned steel				
Straight strands and wires	10^6	5	9	185
Posttensioned steel				
Single strands in plastic ducts [a]	10^6	5	9	185
Straight tendons or curved tendons in plastic ducts	10^6	5	10	150
Curved tendons in steel ducts	10^6	5	7	120
Splicing devices	10^6	5	5	80

[a] In the cases where the S_N curve intersects that of the straight tendon, the fatigue strength of the straight tendon is valid.

The fib Model Code 2010 (2012) points out that the values are only given as guidance and where experimental values are available they may be used instead.

D.3 Fatigue of metallic bridges

Fatigue in wrought iron bridges has been studied in the U.K by Imam (2006, 2009, 2012) and in Sweden by Åkesson (1994), Al-Emrani (2002) and Larsson (2009). Their most important results are incorporated in SB-LRA (2008) with background material in SB-D4.6 (2007).
Some examples of recent work are Pipinato et al (2009, 2011, 2012) on high-cycle fatigue of riveted connections for old railway metal bridges; Akhlaghi, (2009), Fall and Petersson (2009) and Aygül (2012) on welded connections and EC 3 (2005); Brühwiler (2012a,b), Andersson et al (2013), McGormerly et al (2013) and Nilimaa et al (2013) about the MAINLINE project.

D.4 Fatigue of concrete

D.4.1 General

Overviews are given in Westerberg (1969), Gylltoft and Elfgren (1977), CEB Bull 188 (1988), Mallet (1991), Gylltoft (1994), Elfgren and Gylltoft (1997) and fib B52 (2010).

D.4.2 State of the Art

The most up to date advice is contained in fib Model Code 2010 (2012) as shown in Figure D.6.

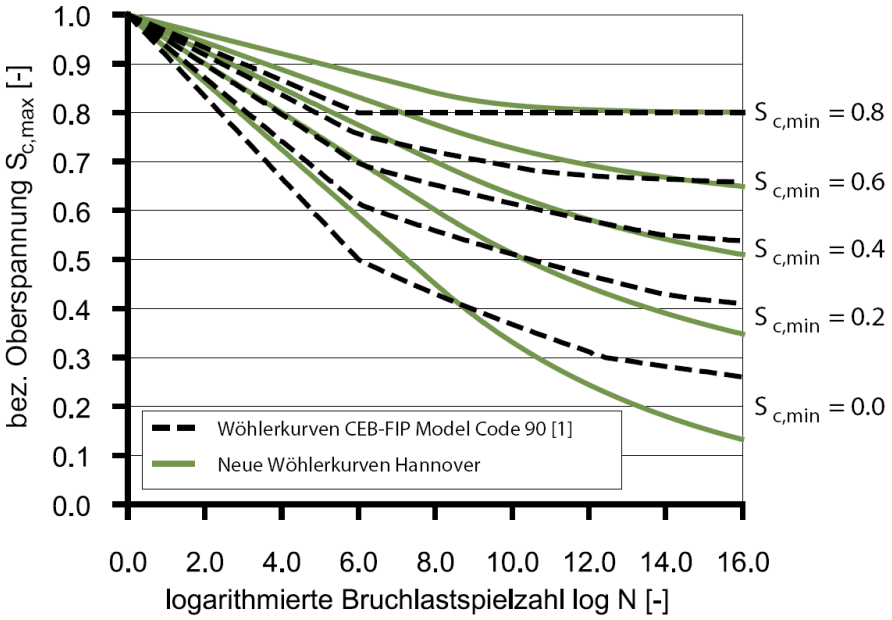


Figure D.6. Wöhler curves used in the new fib Model Code 2010 (2012) (green) and compared to CEB-FIP Model Code (1993) (dotted). Form Lohaus et al (2011) in Hannover

The curves were proposed by a group in Hannover, Lohaus et al (2011) and are somewhat less conservative for less than 8_x10^6 load cycles than earlier versions They are based also on tests on high strength concrete.

In the fib Model Code 2010 (2012) there are four levels of approximation for concrete in compression:

Level I

This is a qualitative verification that no variable action is able to produce fatigue. If the conclusion of this verification is not positive, verification according to one of the higher levels must be made.

Level II

This procedure is only applicable to structures subjected to a limited number (≤ 10^8) of low stress cycles.

$$\gamma_{Sd} \cdot \sigma_{c,max} \cdot \eta_c \leq 0{,}45\, f_{cd,fat} \qquad (7.4\text{-}4)$$

where

γ_{Ed} is a load factor = 1,1 (or 1,0 if the stress analysis is sufficiently accurate or conservative)

$\sigma_{c,max}$ is the maximum compressive stress

η_c is an averaging factor of concrete stresses in the compressive zone considering the stress gradient according to Eq. (7.4-2) in the Model Code

$f_{cd,fat} = 0{,}85 \cdot \beta_{cc}(t) \cdot f_{ck} \cdot \left(1 - \frac{f_{ck}}{400}\right) / \gamma_{c,fat}$ is the design fatigue reference strength for concrete in compression, where $\beta_{cc}(t)$ is a coefficient which depends on the age *t* of the concrete in days when fatigue loading starts (subsection 5.1.9.1 in the Model Code) and $\gamma_{c,fat} = 1{,}5$ according to section 4.5.2.3 in the Model Code.

A similar expression is given for concrete in tension Eq (7.4-5) in the Model Code

Level III

For concrete in compression see Figure D.6.

For $S_{cd,min} > 0{,}8$ the S-N relations for $S_{cd,min} = 0{,}8$ are valid.
For $0 \leq S_{cd,max} \leq 0{,}8$, Eqs (7.4.7a) and (7.4.7b) apply.

$$\log N_1 = \frac{8}{Y-1} \cdot (S_{cd,max} - 1) \qquad (7.4.7a)$$

$$\log N_2 = 8 + \frac{8 \cdot \ln(10)}{Y-1} \cdot \left(Y - S_{cd,min}\right) \cdot log\left(\frac{S_{cd,max} - S_{cd,min}}{Y - S_{cd,min}}\right) \qquad (7.4.7b)$$

with $Y = \frac{0{,}45 + 1{,}8 \cdot S_{cd,min}}{1 + 1{,}8 \cdot S_{cd,min} - 10{,}3 \cdot S_{cd,min}^2}$ where

(a) if $\log N_1 \leq 8$, then $\log N = \log N_1$
(b) if $\log N_1 > 8$, then $\log N = \log N_2$
where:

γ_{Sd} is a partial coefficient
$\sigma_{c,max}$ is the maximum compression stress
$\sigma_{c,min}$ is the minimum compression stress
$S_{cd,max} = \gamma_{ED}\ \sigma_{c,max}\eta_c / f_{cd,fat}$ is the maximum compressive stress level

$S_{cd,min} = \gamma_{ED} \ \sigma_{c,min}\eta_c / f_{cd,fat}$ is the minimum compressive stress level

γ_{ED}, $f_{cd,fat}$ and $\gamma_{c,fat}$ is given in Level II

Level IV

The fatigue damage is calculated as $D = \sum_{i=1}^{j} \frac{n_{Si}}{N_{Ri}}$ where:

D is the fatigue damage

n_{Si} denotes the number of acting stress cycles associated with the stress range

N_{Ri} denotes the number of resisting stress cycles at a given stress range

The fatigue requirement will be satisfied if $D \leq D_{lim}$. Under increasing stress levels, $D_{lim} = 1$ can be used. Under decreasing stress levels D_{lim} can be significantly smaller than 1,0 according to fib Model Code 2010 (2012),section 7.4.1.5

D.4.3 Recent research

Rempling et al. (2008, 2009) have taken the work of Gylltoft (1983) further by using a damage-plasticity approach to model fatigue in a meso-scale. The evolution of fatigue deterioration is also studied by Grigorou and Brühwiler (2013).

Examination of a fatigue failure criterion based on bond deformation proposed by Balázs (1991, 1996) has been carried out by Thun (2006, 2011). Based on this he suggests how the criterion may be used to predict the number of load cycles to failure for existing structures under cyclic tensile loading.

The criterion has successfully been used to describe bond failure between re-bars and concrete (using specimens where reinforcing bars were positioned centrally in concrete prisms). The hypothesis of the criterion is that the deformation at peak load during a static test corresponds to the deformation where the failure process in a fatigue test begins. The growth in deformation during a fatigue test can, according to the model, be divided into three phases, see Figure D.6. At the beginning of the first phase the deformation rate is high but stagnates after a while. The second phase is characterized by a constant deformation rate. These two phases can be described as stable. During the third phase, the failure phase, the deformation rate increases rapidly leading to failure within a short time. The deformation criterion for fatigue failure is that the deformation at peak load, $\delta(f_{peak})$, during a static test corresponds to the deformation at the changeover between phases two and three during a fatigue failure, see Figure D.6. When $\delta(f_{peak})$ has been reached, only a limited number of cycles is needed until failure occurs. Since there is a difference between the number of cycles at failure and at initiation of phase three the criterion could be considered as safe, Balázs (1991). The criterion has recently been successfully applied by Hoehler (2006) on concrete cone break-out of anchor bolts with cast in place headed studs.

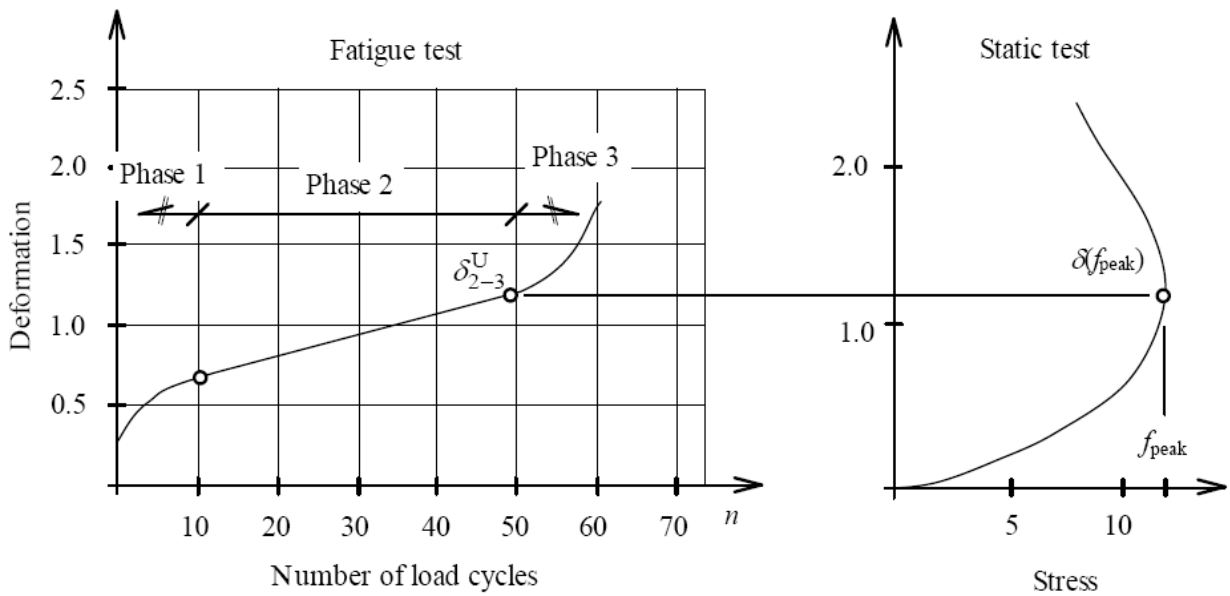


Figure D.6 Deformation fatigue process under repeated loading: (a) fatigue test with deformation as function of number of load cycles n and (b) static deformation diagram-stress diagram. From Thun (2006, 2011) based on Balácz (1991). For concrete in tension the scales may be of the order mm/100 for the deformations and MPa for the stresses.

D.5 Development of Codes

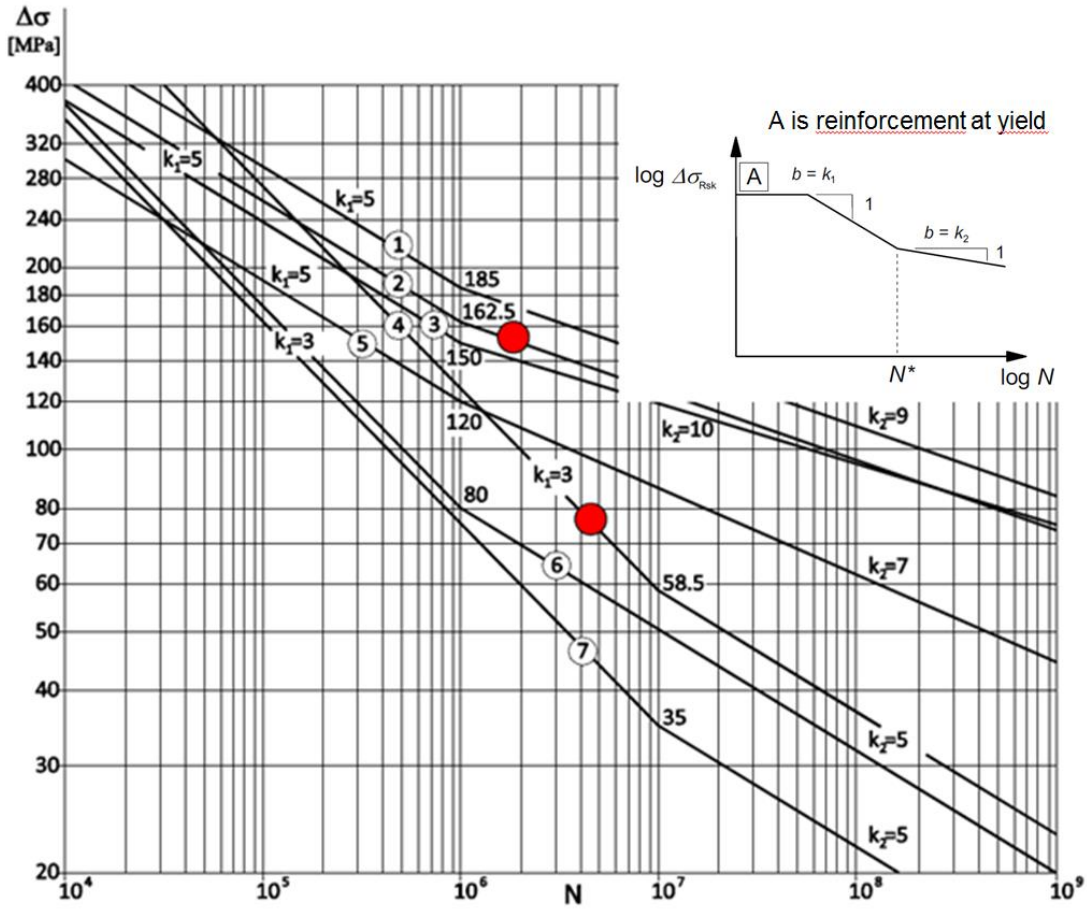
D.5.1 General

Current fatigue provisions rely on a comparatively narrow knowledge basis when compared to most other domains of structural concrete. Fatigue damage mechanisms for reinforced concrete are not yet well understood and codes are often based on pure experimental data with little scientific background. Appropriate damage accumulation theory is still lacking, and thus a worst case scenario of fatigue action effect is considered in codes, see e.g. SB-D4.5 (2008).

This conservative approach is acceptable for the design of new structures, but for existing structures it is inappropriate and may lead to unnecessary and costly strengthening. To reduce uncertainties in current engineering methods, knowledge about the fatigue behaviour of concrete bridges must be improved and realistic methods for both the examination of existing bridges and for the determination of their remaining service life need to be developed.

D.5.2 Model Codes and Euro Codes

Reinforcement
The treatment of fatigue in Euro code EC2 (2004-2006) is based on the CEB-FIP Model Code (1993). S-N curves are illustrated in Figure D.7.



Steel reinforcement	*S-N curve n.*	N*	k_1	k_2	Δσ(N*) [MPa]
Straight bars	2	10^6	5	9	162.5
Welded bars and meshes	4	10^7	3	5	58.5
Jointing devices	7	10^7	3	5	35
Prestressing steel					
Pre-tensioning	1	10^6	5	9	185
Post tensioning					
single strands in plastic ducts	1	10^6	5	9	185
straight tendons or curved tendons in plastic ducts	3	10^6	5	10	150
curved tendons in plastic ducts	5	10^6	5	7	120
Jointing devices	6	10^6	3	5	80

Figure D.7 Stress range (S) versus number of cycles (N) for steel reinforcement according to EC 2 (2002-2006). From Croce and Mlakatas (2010).

A comparison between the characteristic stress ranges for deformed ø 16 mm reinforcement bars for three codes fib Model Code 2010 (2012), Swedish code BBK 04 (2004) and EC 2 (2004-2006) is given in Figure D.8.

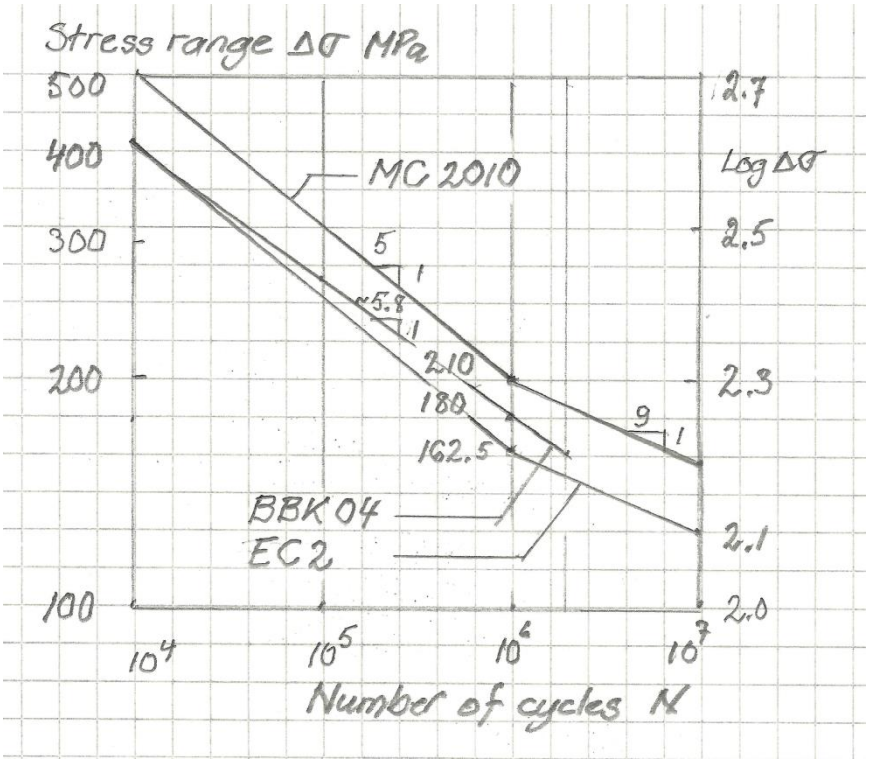


Figure D.8. Comparison of characteristic stress ranges for deformed ø 16 mm reinforcement bars according to fib Model Code 2010 (2012), Swedish code BBK 04 (2004) and EC 2 (2004-2006). The stress range for N = 10^6 load cycles is 210, 180 and 162,5 MPa respectively..

For the breaking point $N = N^* = 10^6$ load cycles for straight bars the European code EC 2 (2004-2206) instead of the Model Code value 210 MPa, uses a lower value 162,5 MPa. The French application follows the Model Code, whereas the German application of EC2 has the value 175 MPa instead of 162,5 MPa, see DIN EN 1992-2 (2010), Fingerloos and Zilch, (2008). The Swedish value in BBK 04 (2004) lies in between with 180 MPa. For bars with larger diameters than 16 mm the differences between the codes are smaller. The differences may depend on a conservative approach for design in the committee developing EC2, whereas the fib-committee may also have taken the function of existing structures into consideration.

For railway bridges, the International Union of Railways (UIC) has issued a special code - UIC 774-1 (2005) - based on ERRI D216/RP1 (1999) and ERRI D216/RP3 (2000).

Concrete

For concrete the European code is based on the earlier CEB-FIP Model Code (1993) which has now been improved to the slightly less conservative fib Model Code 2010 (2012). All codes for concrete are based on stresses calculated from linear elastic assumptions, which gives too high stresses and leads to rather conservative designs.

D.5.3 Codes for existing structures

In Sweden there has been some versions of codes: BV Bärighet (1996), BV Bärighet (2000), TRVK Bärighet (2014) and TRVR Bärighet (2014)

Hendy et al (2011) have given proposals for Eurocodes for Bridges.

In Switzerland, recently a code for assessment of structures states that a closer investigation should be performed if $\Delta\sigma_{sd,fat,act}$ is larger than 150 MPa, SIA 269 (2011). Examples of procedures are given in SB-D4.5 (2007), Herwig (2008), Brühwiler et al (2012) and Brühwiler (2012). Cremona et al (2013) have looked into calibration of partial safety factors. Wiesniewski et al (2012) have compared different codes for assessment.

Based on the information presented in this report, a proposal for a method to assess concrete railway bridges is given in chapter D.7.

D.5.4 Specific questions

- Does any good method exist that is based on partial damage hypotheses? – The methods based on Palmgren-Miner's sum is the best we have today

- What determines a load cycle ? – The stress range in that part of the bridge that is studied. In Figure D.9 it can be seen that the whole train can be seen as one big load cycle, while the bogies can be seen as smaller ones.

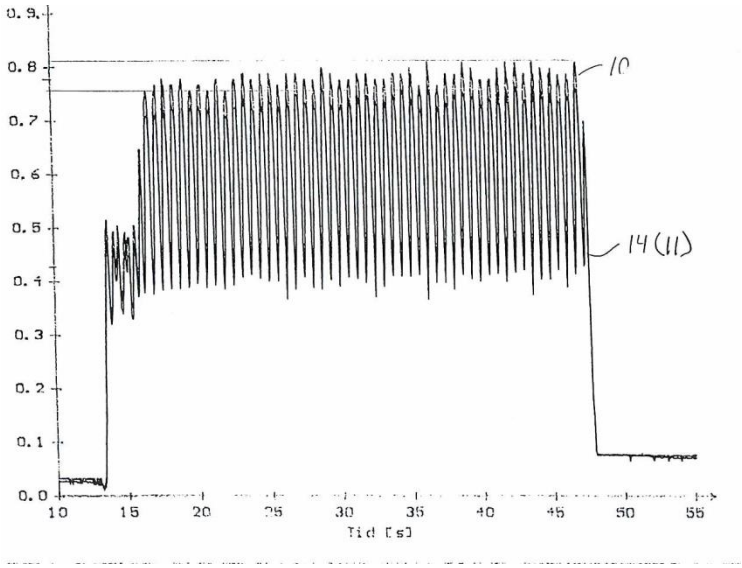


Figure D.9. The figure shows deflection measurements of a bridge at Tjäruträsk km 1214+750 m on the Iron Ore line in Sweden. It can be seen that there are several small load cycles for the different bogies and one bigger one for the whole train. The distance between the bogies are 3,5 m. It can be seen that there are several small load cycles for the different bogies and one bigger one for the whole train. Paulsson et al (1996, 1997).

- What is the influence of earlier loads with low axle loads? – The influence of load cycles giving steel stress ranges below ca 80 MPa usually have no influence on the fatigue capacity.

- How is the design stress influenced by fatigue? – The design stress can be lowered by the influence of fatigue. Fatigue loading influences the life length of the structure and the life length will be lowered by higher stress ranges according to Palmgren-Miner's rule

- How is the dynamic factor to be determined/used? – Beside code formulae and finite element models the best way is to make measurements to study the influence for a specific bridges. An example is given in Simonsson (2002).

- Design of reinforcement carrying loads to the upper parts of concrete structure ? – It depends on how the loads are carried and if it can be shown that truss action can help to carry the loads. Regarding bond length, analogies can be drawn with fasteners, see e.g. Elfgren et al (1982, 1987), fib B58 (2011)

- Splicing of reinforcement. Closed stirrups? – As long as the bars are anchored in compressed concrete there is no need to keep the stirrups closed. However, if the concrete is tensioned and/or cracked it is important that the shear flow can be absorbed by the structure.

D.6 Application to concrete trough bridges

As part of investigations into the capacity of a typical Swedish concrete trough bridge a series of full scale tests were undertaken, as shown in Figure D.10, during which the bridge was subjected to 6million cycles of a load of 360kN as a result of which the only visible damage was hair line cracking in the bottom of the slab. The bridge was subsequently loaded to failure and the load/deformation graph is shown as Figure D.11.

These tests proved that the bridge had a far greater fatigue resistance than code predictions would suggest.

Figure D.10 Full Scale Test of a 29 year old Railway Trough Bridge at Luleå University of Technology. Paulsson et al (1996, 1997).

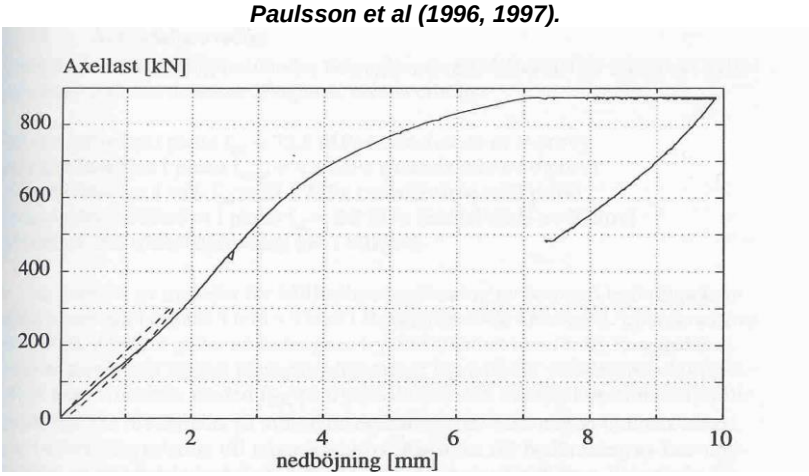


Figure D.11. Load-deformation graph from the final test of the Lautajokk Bridge in Figure 6.1. Before the final test the bridge had been loaded with 6 million cycles with an axle load of 360 kN. Paulsson et al (1996, 1997).

D.7 Summary and conclusions. Recommendations

Recommendations for assessment procedures are given in this chapter. Present codes are too conservative; the approach in SB-LRA(2008) and the new Swiss assessment code SIA 269 (2011) give higher values.

- Present codes are mostly written for the design of new structures. When assessing existing structures it is possible to ascertain actual properties and to use them instead of using very conservative estimates. Possible reinforcement fatigue damage can be assessed with partial damage methods in the same way as is done for steel structures and with similar failure stresses.

- Earlier traffic. The influence can be checked with a damage hypothesis

- Load cycle definition. Depends on the structure and what part of it that is studied. For ballasted bridges often two bogies for adjacent wagons can be identified as one load cycle

- Material properties. Code reductions in reinforcement stress ranges due to friction may not need to be considered.

- Dynamic factors can be reduced from standard code values after evaluations and/or measurements on the structure in question

- The need for closed stirrups and reductions of capacity due to splicing of reinforcement bars can be reduced if the reinforcement is situated in compressed concrete.

- More research is needed to calibrate design and assessment methods to real full scale tests on bridges. Here new measurement technology makes it possible to check real strain and stress ranges, which may be considerably smaller than the ones obtained from conservative design models.

D.7.1 Recommendations for Assessment - General

The recommendations given below are mainly based on work in the European project Sustainable Bridges (2008) and summarized in the guide SB-LRA (2008) and in Elfgren (2015) with background material in SB-D4.5 (2007) and Herwig (2008).

It is very seldom that structures fail due to concrete fatigue. In cracked reinforced structures most of the stresses are taken by the reinforcement bars and they are also the parts that are most susceptible to fatigue.

A rational methodology for the assessment of fatigue safety is based on studying the following three areas taking advantage of the fact that the bridge already exists:

- **Evaluation of the bridge structure** and reinforcement detailing – D.7.1.1
- **Inspection** of the existing bridge and study of the past performance – D.7.1.2
- **Fatigue safety check** – D.7.1.3, D.7.1.4

In the following, each of the three study areas will be discussed.

D.7.1.1 Evaluation of the bridge structure and reinforcement detailing

Grouping types of reinforcement into fatigue categories in accordance with Tables 3.1 and 3.2 in the main report - or Tables 7.4-1 and 7.4-2 in fib model Code 2010 (2012) - allows recognizing types of reinforcement with low fatigue strength. Fatigue vulnerable reinforcement details include the following:

− All **welded** reinforcement is principally fatigue vulnerable, including welded wire mesh, tack welding to reinforcing steel bars, prestressing steel or ducts. Also, butt welds are more fatigue resistant than other load-bearing welds of bars.

− **Mechanically connected** reinforcing bars show also significantly reduced fatigue strength.

− If the **radius of curvature** of bars is smaller than the minimum values according to specifications, reduced fatigue strength is to be expected.

− Areas of **high concentration of bars** or a complex process of placing of the reinforcement might have led to a fatigue vulnerable spot since it was very difficult to obtain the required quality when pouring and working the concrete. Hence, load transfer between bars may not be optimal giving rise to stress concentrations.

− Anchorages for and coupler between **prestressing** elements show in general rather low fatigue strength although there might be some differences between the various prestressing systems. Anchorages and couplers (of bonded tendons) are fatigue vulnerable if they are not located in areas where the stress ranges are small.

− The fatigue strength of bars is also reduced when **corrosion** is probable or can be observed, paying special attention to pitting corrosion. However it is difficult to describe this reduction in terms of fatigue strength.

D.7.1.2 Bridge inspection and monitoring

It can be assumed that there is no fatigue damage **if the concrete is un-cracked**. If the bridge was already exposed to significant rail traffic (more than 100’000 trains inducing more than 1’000’000 load cycles) this state can be assumed to remain stable only when traffic (axle) loads are not increased.

Measurements on a crack (variation of the crack opening) can be made. This allows a rough estimation on the real stress range in the reinforcement under known loading and the effect can be extrapolated to the effect under the fatigue load. The scope of this procedure is to find out if the stress range due to the fatigue load remains below the fatigue limit in the real structure.

Appropriate **monitoring techniques** (including detailed inspections) allow for determining the probability of detection of a fatigue damage indicator (usually a crack). Detailed information on monitoring can be found in SB-MON (2007). Recent developments as the method with photographic strain measurement is presented in e.g. Sas et al (2012)

D.7.1.3 Fatigue safety check

D.7.1.3.1 Concept

The fatigue safety of a structure is deemed to have been proved if the following condition is satisfied:

$$R_{d.fat} \geq E_{d.fat}$$

where

$R_{d,fat}$ examination value for the fatigue resistance (including a partial safety factor)

$E_{d.fat}$ examination value of the fatigue action effect

No partial safety factors for the fatigue action effect are used when proving fatigue

safety. The partial safety factor for fatigue resistance γ_{fat} may be set to 1.15 for taking account of differences between the actual load bearing system and the system used for

the calculation, simplifications and inaccuracies in the strength model, plus inaccuracies

in the cross section.

In the following, fatigue safety is proved separately for

- **the reinforcing steel** and
- **the concrete**.
- Additionally, a method is suggested using the **overall structural response** of an element.

D.7.1.3.2 Reinforcing steel

Determination of fatigue effect (fatigue stress):

The determining parameters when calculating the examination values for the effect of fatigue are the stresses due to fatigue loading only. The stress range $\Delta\sigma$ (Q_{fat}) is derived from the absolute value of the difference between the maximum stress σ_{max} (Q_{fat}) and the minimum stress σ_{min} (Q_{fat}) induced by the effect of fatigue load in the relevant unfavorable positions.

Under bending moment the stresses are calculated at a cracked cross-section, assuming that plane sections remain plane and linear-elastic material behavior. The behavior of concrete under tension is ignored.

Under fatigue loading, the bond behavior of reinforcing and prestressing steel is different.

This may be considered by increasing the stress in the reinforcing steel by a factor k depending on the steel area and bar diameters, Eq. (6.24) in SB-LRF (2007).

$$k = \frac{A_s + A_p}{A_s + A_p \sqrt{\xi \left(\frac{\phi_s}{\phi_p}\right)}}$$

with A_s und A_p being the area of reinforcing and prestressing steels respectively, ϕ_s is the largest bar diameter and $\phi_p = \sqrt{A_p}$ being the equivalent diameter of the prestressing steel.

For beams and slabs with shear reinforcement, the stresses in the main and shear reinforcement are calculated using a lattice model. The angle of inclination of the concrete compression struts to be assumed when calculating the stress differences in the reinforcement is as follows:

$$\tan \propto_{fat} = \sqrt{\tan \propto} \leq 1$$

Here α is the angle of inclination of the compression strut at ultimate limit state. The reason to choose a larger inclination for fatigue is experimental evidence that the crack inclination tends to rise during fatigue, SB-D4.5 (2007)

Determination of fatigue resistance:

Common types of reinforcement are divided into fatigue categories ($\Delta\sigma_{s,fat}$) according to their fatigue strength (see Table D.3). The number designating the fatigue category is the fatigue strength in MPa for 2 million load cycles. As an alternative the values given in fib Model Code 2010 (2012) or test results can be used.

Table D.3 Fatigue category for reinforcing and prestressed steel. SB-D4.5(2007)

Fatigue category $\Delta\sigma_{s,fat}$ [N/mm^2]	Description	Comments
Reinforcing steel		
170	1. Bars: Ø ≤ 20 mm, straight 2. Vertical stirrups: Ø ≤ 16 mm, bent in accordance with standards	For bent bars, the values are reduced as follows: $\Delta\sigma_{s,fat,red} = k_{\varnothing} \times \Delta\sigma_{s,fat}$ where $k_{\varnothing} = 0.35 + 0.026\frac{d_i}{\varnothing_S}$ where d is the diameter of the bar bender roller
140	Bars: 20 mm ≤ Ø < 40 mm, straight	
70	1. Butt welds and welded intersections between bars (e.g. reinforcing lattice). 2. Mechanical joints between bars.	If no results from specific studies are available, the standard value shown should be used.
Prestressing steel and prestressing systems		
200	Single strands and single-layer tendons in plastic ducts	
170	1. Multi-strand and HiAm cables in plastic ducts. 2. Straight tendons without ducts (e.g. in elements fabricated by the long-line method)	Tendons may be bonded or unbonded friction has a significant effect on fatigue behaviour. Where prestressing is carried out using unbonded tendons, the anchorage is the determining factor.
110	Multi-strand and HiAm cables in steel ducts.	
80	Anchorages, cable couplers	For prestressing systems that comply with the standards.

Verification:

Proof with respect to the fatigue limit: Initially, the fatigue safety of reinforcing steel is verified with respect to the fatigue limit. If all stress ranges due to service loads remain below the fatigue limit $\Delta\sigma_{sd,D}$ for the entire service life, fatigue failure will not occur:

$$\Delta\sigma_{sd,D} \geq \Delta\sigma_{sd}(Q_{fat})$$

The nominal fatigue resistance may be taken as: $\Delta\sigma_{sd,D} = 0{,}8 \cdot \Delta\sigma_{sd,fat} = 0{,}8 \cdot \Delta\sigma_{s,fat}/\gamma_{fat}$

For a normal bar in the top row of Table D-3 with ø ≤ 20 mm we obtain
$\Delta\sigma_{sd,fat} = \Delta\sigma_{s,fat}/\gamma_{fat}$ = 170/1,15 = 147,8 MPa and
$\Delta\sigma_{sd,D} = 0{,}8 \cdot \Delta\sigma_{sd,fat}$ = 0,8·147,8 = 118,3 MPa.

Proof with respect to equivalent stress range*:* If fatigue safety has not been verified with respect to the fatigue limit, proof of operational resistance shall be performed. When using this method, the aim is to demonstrate that the equivalent stress range does not exceed the fatigue strength:

$$\Delta\sigma_{sd,fat} \geq \Delta\sigma_{ed}(Q_{fat}) \geq \lambda \cdot \Delta\sigma_{sd}(Q_{fat})$$

The equivalent stress range $\Delta\sigma_{ed}$ is the product of a correction factor λ and the stress range induced by the fatigue load Q_{fat}: The correction load factor λ compares the fatigue effect of the load models with that of the effective fatigue load. This factor depends on traffic composition, traffic volume, load geometry, service life, and number of load cycles, fatigue load and static system. It is given in EC 2 (2003-2006) and reproduced below.

Proof with respect to a spectrum of stress ranges: If the proof with respect to equivalent stress range cannot be verified, a more detailed verification can be made with the Palmgren-Miner summation. The fatigue damage is then calculated as

$$D = \sum_{i=1}^{j} \frac{n_{Si}}{N_{Ri}}$$

where:

D is the fatigue damage

n_{Si} denotes the number of acting stress cycles associated with the stress range

N_{Ri} denotes the number of resisting stress cycles at a given stress range

The fatigue requirement will be satisfied if $D \leq D_{lim}$. Under increasing stress levels, $D_{lim} = 1$ can be used. Under decreasing stress levels D_{lim} can be significantly smaller than 1,0 according to fib Model Code 2010 (2012),section 7.4.1.5

D.7.1.3.3 Concrete

Proof of fatigue safety by calculation is not required for normal stresses in concrete if inspection shows that the concrete is in good condition.

Nevertheless, it is useful to determine and evaluate the level of fatigue solicitation. A simple proof of compression fields consists in showing that the maximum compressive stress due to permanent loads as well as frequent temperature and rail traffic fatigue loading are smaller than 50% of the characteristic value of compressive strength of concrete. Besides the compressive stresses due to bending may be calculated using a stress block.

For deck slabs without shear reinforcement, fatigue failure of concrete may occur only when the stress level is higher than 40% of the ultimate load (determined e.g. with nonlinear analysis) of the structural element. Also, the nominal shear stress is not the relevant parameter to describe fatigue failure. As a consequence, a fatigue safety check with respect to the ultimate load may be conducted.

D.7.1.3.4 Proof with respect to ultimate load

No fatigue failure of the structural element will occur if the following conditions are fulfilled:

- under predominant bending fatigue $0{,}5\ F_{ult} \geq F_{fat,max}$

- under predominant shear fatigue: $0{,}4\ F_{ult} \geq F_{fat,max}$

where

F_{ult} is the ultimate load of the structural element. It is determined by means of a nonlinear structural analysis using nominal values of material properties and considering partial safety factors (resistance coefficients).

$F_{fat,max}$ is the corresponding maximum fatigue load due to permanent loads as well as frequent temperature and rail traffic fatigue loading.

D.7.1.4 Remaining fatigue life assessment and action

D.7.1.4.1 Introduction

The fatigue life calculation for reinforced concrete elements is generally based on a

5% fractile value for the fatigue resistance. Due to simplifying assumptions on the traffic loads and the action effect model for the reinforcement stress ranges, the calculated remaining fatigue life may be much shorter than the real fatigue life.

In reinforced concrete elements with distributed reinforcement, the fatigue resistance is governed by the reinforcement if the concrete is in good condition. It may be expected that the reinforcement exhibit the same scatter for the fatigue resistance as observed in fatigue tests on naked bars. This has been confirmed by several tests on reinforced concrete elements.

This high scatter allows that the reinforcement bars break one after the other with a relative high interval between two reinforcement bar fractures, Schläfli (1999).

The remaining intact (un-separated) reinforcement bars after the fracture of the first reinforcement bar(s) show the same ultimate resistance as un-cycled bars and are able to yield. This was confirmed through the analysis of several tests on reinforced concrete elements, Herwig (2008). Therefore, the ULS resistance of the bridge element is only marginally affected after the fatigue fracture of the first reinforcement bar(s).

The disadvantage of the high inaccuracy of the calculation of the remaining fatigue

life can be avoided by the use of the high potential of surveying of the bridge condition through “monitoring”.

D.7.1.4.2 Crack growth calculation of weakest rebar

A damage model for a reliable and not overly conservative prediction of the remaining fatigue life of the steel reinforcement based on LEFM (Paris law for stable crack propagation) and the well-known parameters of the S-N curves is presented in SB-D4.5 (2007). Usually traffic models are used with characteristic trains, given by its ratio of total traffic.

D.7.1.4.3 Fatigue safety concept

The fatigue safety concept takes into account the high uncertainty of fatigue life predictions and the fact that fatigue failure is announced long in advance by the fracture of the first reinforcement bar.

The fatigue safety concept consists on fatigue analysis and condition survey. Due to the relative small live load of ballasted reinforced concrete bridges and their young age, no signs of fatigue damage (excessive cracking, large crack openings) can be observed in many bridges yet. However, the increase of maximum allowable axle loads may accelerate fatigue damage in the future. The fatigue safety concept is applied if fatigue safety check of Section A7.1.3is not fulfilled.

When the calculated fatigue life ends, which is still a lower bound, there are two possibilities to proceed: (1) the bridge is strengthened or replaced, or (2) a continuous monitoring system is introduced. The introduction of monitoring is advisable only for bridge elements with distributed reinforcement, such as slabs, containing a high redundancy for the fatigue resistance of the reinforcement bars. SB-D4.5 (2007) gives information on how to proceed when monitoring is indicated.

D.7.1.5 λ-values for reinforcing and prestressing steel in railway bridges according to Eurocode 2

The following is a commented excerpt from Appendix NN in EC 2 (2006).

(101) The damage equivalent stress range for reinforcing and prestressing steel shall be calculated according to Equation (NN.106)

$$\Delta\sigma_{s,equ} = \lambda_s \cdot \phi \cdot \Delta\sigma_{s,71} \qquad \text{(NN.106)}$$

where:

$\Delta\sigma_{s,equ}$ steel stress range due to load model 71(and where required SW/0), but excluding α according to EN 1991.2, being placed in the most unfavorable position for the element under consideration. For structures carrying multiple tracks, load model 71 shall be applied to a maximum of two tracks

λ_s correction factor to calculate the damage equivalent stress range from the stress range caused by $\phi \cdot \Delta\sigma_{s,71}$

ϕ dynamic factor according to EC 2 (2006)

(102) The correction factorλ_s , takes account of the span, **annual traffic volume, design life and multiple tracks**. It is calculated from the following formula:

$$\lambda_s = \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4} \qquad \text{(NN 107)}$$

where:

$\lambda_{s,1}$ is a factor accounting for **element type** (e.g. continuous beam) and takes into account the damaging effect of traffic depending on the length of the influence line or area.

$\lambda_{s,2}$ is a factor that takes into account the **traffic volume**.

λ_{s3} is a factor that takes into account the **design life** of the bridge

$\lambda_{s,4}$ is a factor to be applied when the structural element is loaded by **more than one track.**

(103) The factor $\lambda_{s,1}$ is a function of the length of the influence line and the traffic. The values of $\lambda_{s,1}$ for standard traffic mix and heavy traffic mix may be taken from Table NN.2 of this Annex. The values have been calculated on the basis of a constant ratio of bending moments to stress ranges. The values given for mixed traffic correspond to the combination of train types given in Annex F of EC2 (2002-2006).

Values of $\lambda_{s,1}$ for a length of influence line between 2 m and 20 m may be obtained from the following equation:

$$\lambda_{s,1}(L) = \lambda_{s,1}(2m) + \left[\lambda_{s,1}(20m) - \lambda_{s,1}(2m)\right] \cdot (\log L - 0{,}3)$$

where

L is the length of the influence line in m

$\lambda_{s,1}(2m)$ is the $\lambda_{s,1}$ value for *L*=2 m

$\lambda_{s,1}(20m)$ is the $\lambda_{s,1}$ value for *L* = 20 m

$\lambda_{s,1}(L)$ is the $\lambda_{s,1}$ value for 2m < *L* < 20 m

Table D1-4 (NN.2 according to EC). $\lambda_{s,1}$ values for simply supported and continuous beams

	L [m]	s*	h*
[1]	≤2 ≥20	0,90 0,65	0,95 0,70
[2]	≤2 ≥20	1,00 0,70	1,05 0,70
[3]	≤2 ≥20	1,25 0,75	1,35 0,75
[4]	≤2 ≥20	0,80 0,40	0,85 0,40
Simply supported beams			

	L [m]	s*	h*
[1]	≤2 ≥20	0,95 0,50	1,05 0,55
[2]	≤2 ≥20	1,00 0,55	1,15 0,55
[3]	≤2 ≥20	1,25 0,55	1,40 0,55
[4]	≤2 ≥20	0,75 0,35	0,90 0,30
Continuous beams (mid span)			

	L [m]	s*	h*
[1]	≤2 ≥20	0,90 0,65	1,00 0,65

	L [m]	s*	h*
[1]	≤2 ≥20	0,85 0,70	0,85 0,75

[2]	≤2 ≥20	1,05 0,65	1,15 0,65
[3]	≤2 ≥20	1,30 0,65	1,45 0,70
[4]	≤2 ≥20	0,80 0,35	0,90 0,35
Continuous beams (end span)			

[2]	≤2 ≥20	0,90 0,70	0,95 0,75
[3]	≤2 ≥20	1,10 0,75	1,10 0,80
[4]	≤2 ≥20	0,70 0,35	0,70 0,40
Continuous beams (intermediate support areas)			

s* standard traffic mix
h* heavy traffic mix
[1] reinforcing steel, pre-tensioning (all), post-tensioning (strands in plastic ducts and straight tendons in steel ducts)
[2] post-tensioning (curved tendons in steel ducts); S-N curve with k_1=3, k_2=7 and N*=10^6
[3] splice devices (prestressing steel); S-N curve with k_1=3, k_2=5 and N*=10^6
[4] splice devices (reinforcing steel); welded bars including tack welding and butt joints; S-N curve with k_1=3, k_2=5 and N*=10^7
Interpolation between the given L-values according to Equation NN.103 is allowed
Note: No values of $\lambda_{s,1}$ are given in Table NN2 for a light traffic mix. For bridges designed to carry a light traffic mix the values for $\lambda_{s,1}$ to be used may be based either on the values given in Table NN2 for standard traffic mix or on values determined from detailed calculations.

(104) The $\lambda_{s,2}$ value denotes the influence of annual traffic volume and can be calculated from Equation (NN.109)

$$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 \cdot 10^6}} \qquad (109)$$

where:

Vol is the volume of traffic (tonnes/year/track)
k_2 is the slope of the appropriate S-N line

(105) The $\lambda_{s,3}$ value denotes the influence of the service life and can be calculated from Equation (NN.110)

$$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{Years}}{100 \cdot}} \qquad (110)$$

where:
N_{Years} is the design life of the bridge
k_2 is the slope of the appropriate S-N line

(106) The $\lambda_{s,4}$ value denotes the effect of loading from more than one track. For structures carrying multiple tracks, the fatigue loading shall be applied to a maximum of two tracks in the most unfavorable positions. The effect of loading from two tracks can be calculated from Equation (NN.111).

$$\lambda_{s,4} = \sqrt[k_2]{n + (1-n) \cdot s_1^{k_2} + (1-n) \cdot s_2^{k_2}} \qquad (111)$$

$$s_1 = \frac{\Delta\sigma_1}{\Delta\sigma_{1+2}} \qquad s_2 = \frac{\Delta\sigma_2}{\Delta\sigma_{1+2}}$$

where

n proportion of traffic that crosses the bridge simultaneously (the suggested value of n is 0,12)

$\Delta\sigma_1$, $\Delta\sigma_2$ is the stress range due to load model 71 on one track at the section to be checked
$\Delta\sigma_{1+2}$ is the stress range at the same section due to the load model 71 on any two tracks
k_2 is the slope of the appropriate S-N line

If only compressive stresses occur under traffic loads on a track, set the corresponding value sj=0

Examples on applying the methods are given in Elfgren (2015).

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