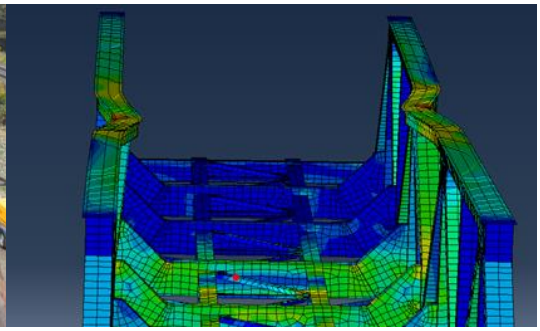
MAINLINE

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 1.2: Assessment methods for 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; DB Netz AG, 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; SETRA, FR; ARTTIC, FR; Skanska a.s., CZ.

The figure on the front cover shows a comparison between actual and predicted deformations for a steel truss bridge which was loaded to failure at Åby River in Northern Sweden in September 2013. The failure was initiated by instability of the two top girders. This was predicted by an advanced reliability based assessment method using a non-linear finite element model, see further Section 6.1.

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Glossary

Abbreviation / acronym	Description
ADTT	Average Daily Truck Traffic
AE	Acoustic Emission
BLCCA	Bridge Life-cycle cost analysis
CPF	Cumulative Probability Function
DAF	Dynamic Amplification Factor
DoW	Description of Work
EC	European Commission
ECCS	European Convention for Constructional Steelwork
FE(A)	Finite Element (Analysis)
HFC	High Frequency Content
JRC	Joint Research Centre
LCA	Life Cycle Assessment
LCAT	Life Cycle Assessment Tool
LCC	Life Cycle Cost
LCCA	Life Cycle Cost Analysis
LRFR	Load and Resistance Factor Rating
McMc	Markov chain Monte Carlo
MH	Metropolis-Hastings
PDF	Probability Distribution Function
R & D	Research and Development
RHR	Rock fall Hazard Rating system
RSHI	Rock Slope Hazard Index
SHM	Structural Health Monitoring
SOAP	Stereo Oblique Aerial Photography
SSHI	Soil Slope Hazard Index
TCMI	Tunnel Condition Monitoring Index
VBI	Vehicle Bridge Interaction
WIM	Weigh in Motion
WP	Work Package

1. Executive summary

1.1 Scope

From the MAINLINE Deliverable “D1.1: Benchmark of new technologies to extend the life of elderly rail infrastructure”, the results from a bridge questionnaire that was circulated between twelve Infrastructure Managers (IM) was extrapolated. It turned out that in the next 10 years it may be expected to strengthen some 1,500 bridges, to replace some 4,500 bridges and to replace the deck of some 3,000 bridges. From these figures, one can see the need for suitable and reliable methods to accurately assess the existing bridges and other railway assets. An accurate and advanced assessment will allow keeping in service many assets that otherwise will be condemned to repair, strengthening and/or replacement. This is of particular interest for infrastructure assets located in Eastern European countries, where low levels of maintenance for many years derived on a huge number of bridges and other infrastructure to need attention. Because interventions in many bridges at the same time is not economically feasible, their advanced assessment could help prioritize the repair/strengthening works in the coming years. The main objective of this deliverable is to present the existing possibilities for an accurate assessment of railway assets condition.

The two specific objectives of this report are:

1. - To describe a set of proposed advanced assessment methods that may be incorporated in the life-cycle management of railway infrastructures

2. - To see how the costs and benefits of the proposed advanced assessment methods may be incorporated within a LCA framework.

The scope of this report is limited to the assessment of relevant railway infrastructure assets. The assessment methods considered will be those applicable to the following asset types, as selected in Deliverable 2.1 of MAINLINE:

- Cuttings
- Metallic Bridges
- Lined Tunnels
- Track (including rails, sleepers, ballast, switches and crossings).

The most appropriate assessment method for every specific infrastructure asset strongly depends on many variables. This deliverable seeks to help the assessing engineer, first showing the available alternatives and, after that, in the decision making for the best method and technique to be used within a Life-Cycle Assessment framework, where the optimization of cost in a wide sense (including environmental costs) is the final objective.

1.2 Conclusions

The deliverable presents the most advanced assessment methods that can be applied nowadays in railway assets described above (according to Deliverable 2.1) in order to get the most accurate estimate of their actual condition and performance. The proposed methods range from the application of reliability-based techniques to the introduction of monitoring results (both dealing with strength and live loading) in the assessment process as well as the use of proof load testing and system behaviour in the case of bridges, to the use of base-line reference curves of the measured intensity of current along time in the case of switches.

The application of reliability-based assessment methods seems mandatory when the standard deterministic or semi-probabilistic methods declare the asset 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.

The importance of considering system behaviour, 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. 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.” Redundancy is defined as the capability of an originally intact structure to continue to carry load after the failure of one element. The concept of disproportionate collapse must always be kept in mind.

Today's freight trains are quite different from assumed design loadings. Over the last 30 years, trailing rail car weights have increased. The basic nature of train compositions has also changed. Heavy unit commodity, relatively fast intermodal, and mixed freight trains are now commonplace. Allowable rail car loads are also expected to be increased by 10%-20% over the next few years. 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.

Up-dating of models for resistance, loading and structural response to get more accurate theoretical models in the analysis is also considered in an advanced assessment. The performance assessment based only on the Structural Health Monitoring (SHM) data, and disregarding the original information, can be restrictive and provide biased results. The Bayesian approach makes it possible to use the additional monitoring information in conjunction with the already available information. 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 use prior knowledge.

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 proof load test in situ. This test is used to verify component and system performance under a known external load and is normally aimed to provide a complementary assessment methodology to the theoretical assessment. The use of such tests, due to the risks of collapse or of damaging essential elements of the structure, must be restricted to bridges that have failed to pass the most advanced theoretical assessment and are therefore condemned to be posted, closed to traffic or demolished. It is also important that the bridge has a high level of redundancy to be a good candidate. Furthermore, some balance has to be found between the risk of failure under the test load and the benefit of an updated reliability of the bridge. 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 placing a greater load than the expected overload in service.

At the present moment, only a few countries include in their codes or guidelines the possibility of using advanced assessment techniques different from the standard assessment normally based on existing codes for the design of new structures. In the table below, there is a summary of how different countries have implemented in their assessing guidelines for bridges the 5 advanced techniques proposed in this deliverable.

Table 1-1 Advanced assessment for bridges in different countries

	System behaviour	**Probability-based assessment**	**Proof loading**	**Up-dating Dynamic effects**	**Up-dating (materials, loads, models)**
Canada	yes	yes	yes	no	yes
USA	yes	yes	yes	no (to some extent)	yes
Denmark	no	yes	no	no	yes
Switzerland	no	yes	no	yes	yes
UK	no	yes	no	no	yes
Germany	yes	yes	yes	no	no

In the framework of the MAINLINE project, advanced assessments should be decided on the basis of a life-cycle assessment. Inspections and structural assessment based solely on experience may be more expensive and less safe than those based on a more rational approach. The optimal policy has to be chosen based on minimum expected total life-cycle cost criterion including its effect on structural reliability and the expected costs associated with failure.

The goal of an optimal management strategy is to minimize the lifetime cost of a given infrastructure while ensuring that maintains an acceptable reliability level throughout its expected service life. Environmental costs of a chosen strategy also have to be included.

The expected total cost C_{ET} is the sum of its components including the initial cost of the structure C_T, the expected cost of routine maintenance C_{PM}, the expected cost of inspection C_{INS} , the cost of repair, C_{REP}, and the expected cost of failure C_F. The cost of failure include

both direct and indirect cost, as, user costs and others. Accordingly, C_{ET} can be expressed as:

$$C_{ET} = C_T + C_{PM} + C_{INS} + C_{REP} + C_F$$

The objective remains to develop a strategy that minimizes C_{ET} while keeping the lifetime reliability of the structure above a minimum allowable value.

Since the advanced assessment allows to better know the state of a structure, it can be seen as a part of an inspection and therefore as part of the costs of such inspection, C_{INS}. Given that the ultimate goal is to know the exact state of the structure, any additional inspection costs resulting from advanced assessment could be rewarded with a considerable decrease of repair costs and, in turn, in a reduction of life-cycle costs.

1.3 Areas for further work

The following areas of further work were identified during the present study:

1.- Calculation of approximate costs of advanced assessment methods to be included in life cycle cost models and other decision supports systems. Although some reference values are given in chapter 7 of this deliverable, more data on cost is necessary for a further development

2.- Development of reliability-based fatigue assessment methods for metallic bridges

3.- Development of assessment methods specific of joints (riveted, welded) in metallic bridges.

2. General remarks

The project 'MAINtenance, renewaL and Improvement of rail transport iNfrastructure to reduce Economic and environmental impacts' (in short MAINLINE) is an integrated project within the EU's 7th Framework Programme. It is partly funded on the basis of the contract n°285121 between the European Community represented by the European Commission and the International Union of Railways (UIC) acting as Coordinator for the project.

The main objectives of the project are to:

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

The present report D1.2 –Assessment methods for elderly rail infrastructure, has been prepared within the work package WP1 of the MAINLINE project, named ‘Life Extension – Application of new technologies to elderly infrastructure', one of the eight work packages (WP1-WP8) dealing with relevant tasks for maintenance, renewal and improvement of rail transport infrastructure to reduce economic and environmental impacts.

An overview of the general organisation of the project is presented below together with the list of all the partners in work package WP1:

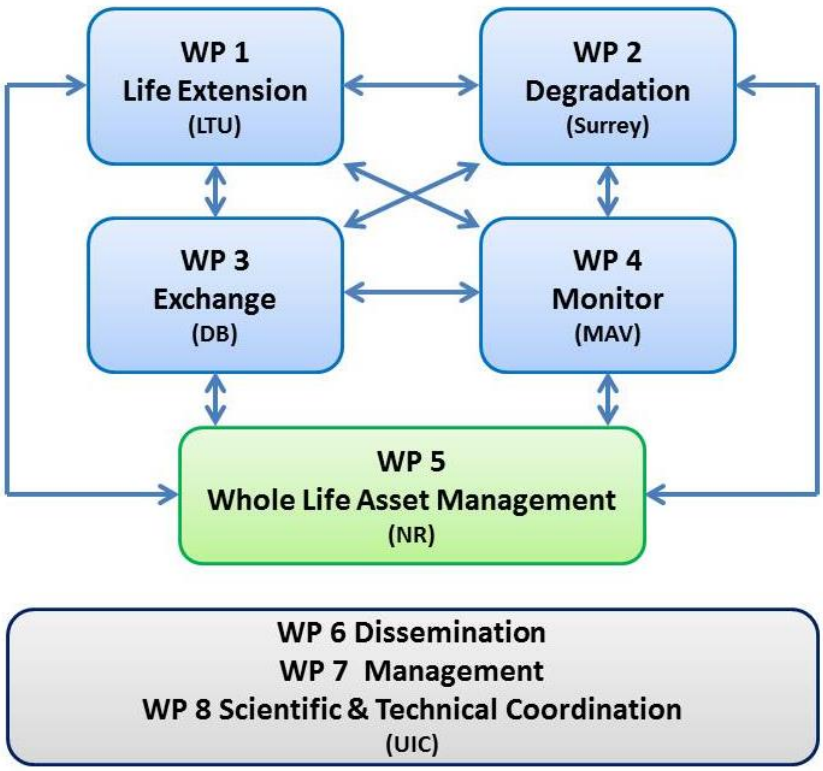


Figure 2-1 General organisation of the project

Part n°	WP1 Partners	Country
1	UNION INTERNATIONALE DES CHEMINS DE FER (UIC)	FR
2	NETWORK RAIL INFRASTRUCTURE LTD (NR)	UK
4	SINCLAIR KNIGHT MERZ (SKM)	UK
7	UNIVERSIDADE DO MINHO (UMinho)	PT
8	LULEA TEKNISKA UNIVERSITET (LTU)	SE
11	UNIVERSITAT POLITECNICA DE CATALUNYA (UPC)	ES
19	SKANSKA AS (SKANSKA)	CZ

The main objective of WP1 is to apply new technologies to extend the life of elderly infrastructure. Further WP1 has the objective to deliver input regarding data to the development of life cycle cost models and other decision supports systems.

WP1 interacts with WP2-6 in the following ways:

Inputs to WP1:

- WP2 will provide information on degradation models.
- WP3 will provide new construction technologies.
- WP4 will provide input on how monitoring can be used to complement assessments.

Outputs from WP1:

- WP2: data for degradation
- WP3: developed new technologies
- WP4: gaps needed to be filled for monitoring to be used for successful assessment
- WP5: data for Whole Life Assessment
- WP6: guidelines to be used by Infrastructure Managers

3. Acknowledgments

This present report has been prepared within work package WP1 of the MAINLINE project by the following team of contractors with LTU as the WP-leader and Universitat Politècnica de Catalunya as the subtask leader responsible for this deliverable:

- Universitat Politècnica de Catalunya (UPC), Spain
- Lulea Tekniska Universitet (LTU), Sweden
- Network Rail (NR), United Kingdom
- Sinclair Knight Merz (SKM), United Kingdom
- Universidade do Minho (UMinho), Portugal
- Trafikverket (TrV), Sweden.

The bridge assessment in Section 6.1 was carried out by Prof. Joan Ramon Casas and Ms. Miriam Soriano at the Technical University of Catalonia in Barcelona, Spain. It was based on a FEM model developed by Prof. Yongming Tu, Southeast University, Nanjing, PR China and Luleå University of Technology, Sweden.

The test to failure was carried out by Luleå University of Technology with Thomas Blanksvärd as project leader on a commission from Trafikverket (Anders Carolin). Financial contributions were also given by LKAB AB, Hjalmar Lundbohm Research Center (HLRC), the the Development Fund of the Swedish Construction Industry, SBUF, and Skanska Spännarmering.

4. Introduction

The assessment of existing infrastructures is becoming essential as their stock 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 information to be obtained.

It is worth saying that when dealing with elderly infrastructure, the recorded information is normally always limited. Therefore, the risk of such limited information should be considered when deciding an advanced assessment. Also, the level of assessment accuracy needs to be commensurate to the level of material/structure accuracy, i.e. there is little point in doing an advanced assessment if material properties etc. are only known to the nearest 10% or similar. There is also the issue of old materials not being to current Eurocode specifications (e.g. steel reinforcement anchorage/bond in concrete – this means that there may be limits to the use of plasticity/rotational capacity). This means that some form of monitoring or intrusive work, and very detailed inspection of the structure, may need to be considered before undertaking advanced assessment methods. This is also an issue considered in this deliverable, where the information of “in situ” or material testing can be introduced in the assessment process by means of Bayesian updating.

The serviceability limit state (SLS) may also require more detailed consideration if advanced assessment methods are used for the ultimate limit state, as degradation/condition is usually mainly dependent on SLS.

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 maximum relevance. Therefore, different levels of assessment should be considered depending on the foreseen results.

4.1 Description of different levels of assessment

In general, the best final assessment approach is the cheapest method which shows the structure to have the required strength (CIRIA, 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 progresses. 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,…". Therefore, the document recognizes that different assessment methods exist and their application should be made in a sequential way.

Assessing the safety of an existing rail infrastructure asset 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. Experience shows that breaking down the assessment of a structure into a maximum of 3 phases is reasonable (Schneider 1997). This is, for instance, the procedure adopted in the case of bridge structures, in the Sustainable Bridges project (SB-7.3, 2008) as shown in Figure 4-1. In this case, 3 levels of assessment are adopted in a progressive way.

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) 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.

In many cases, the use of Intermediate Level checks (level 2) may be sufficient to verify or refute the results of Initial Level assessments. Because of the costs involved and the required expertise needed to execute more advanced inspections and analyses, the decision on whether to collect additional data to perform an Enhanced Level assessment (level 3) must depend on the importance of the bridge, the direct and indirect costs of its closing or replacement including the expected reduction in the life cycle costs that additional data may bring about. Depending on the expected information that would be provided by the new data, it may or may not turn out to be possible to improve the current estimates of a bridge's safety. Therefore, alternative options such as interventions for bridge re-qualification (strengthening and repairs or replacement) must be evaluated and compared based on an analysis of the life cycle costs before the decision to move to an Enhanced Level assessment is made.

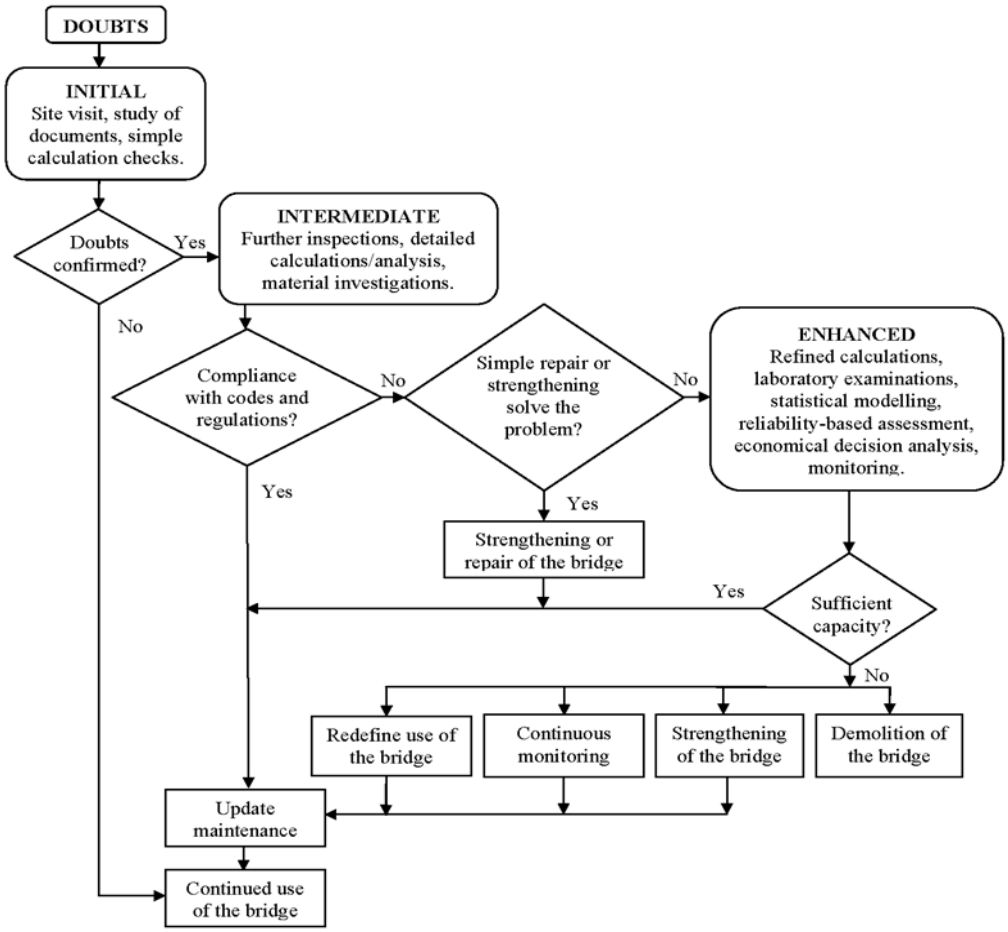


Figure 4-1 Step-level procedure for reassessment of existing bridges (SB-7.3, 2008)

Following the general framework presented in Figure 4-1, in the specific case of the fatigue assessment of existing steel bridges, JRC (2008), proposes the following stepwise procedure where, in this case, 4 progressive steps are presented (see Figure 4-2). However, step 4 is just the implementation of the final decision (demolish, repair,…) and therefore a sequence of 3 main steps is also proposed at the end.

It is worth noting that increasing the level of assessment will require more resources for advanced experimental methods, theoretical analyses,…The decision of going further to the next level should be supported on the saving of resources derived from the final decision. An equilibrium is foreseen between the new expenses derived from the more advanced assessment and the savings because the final decision may result on a lower strengthening or repair need. 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:

$$C_{total} = C_{cons} + C_{insp} + C_{ass} + C_{user} + C_{repair} + C_{failure} \tag{4.1}$$

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 constraint: To guarantee a minimum performance level (safety, service) to the user

$$S > S_0 \tag{4.2}$$

In the presented framework, it makes then sense to look at the feasibility of enhanced or advanced assessment as a mean of reducing the repair and failure costs, and consequently, the total life-cycle cost. Therefore, advanced assessment may be seen as the most effective and cheapest 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.

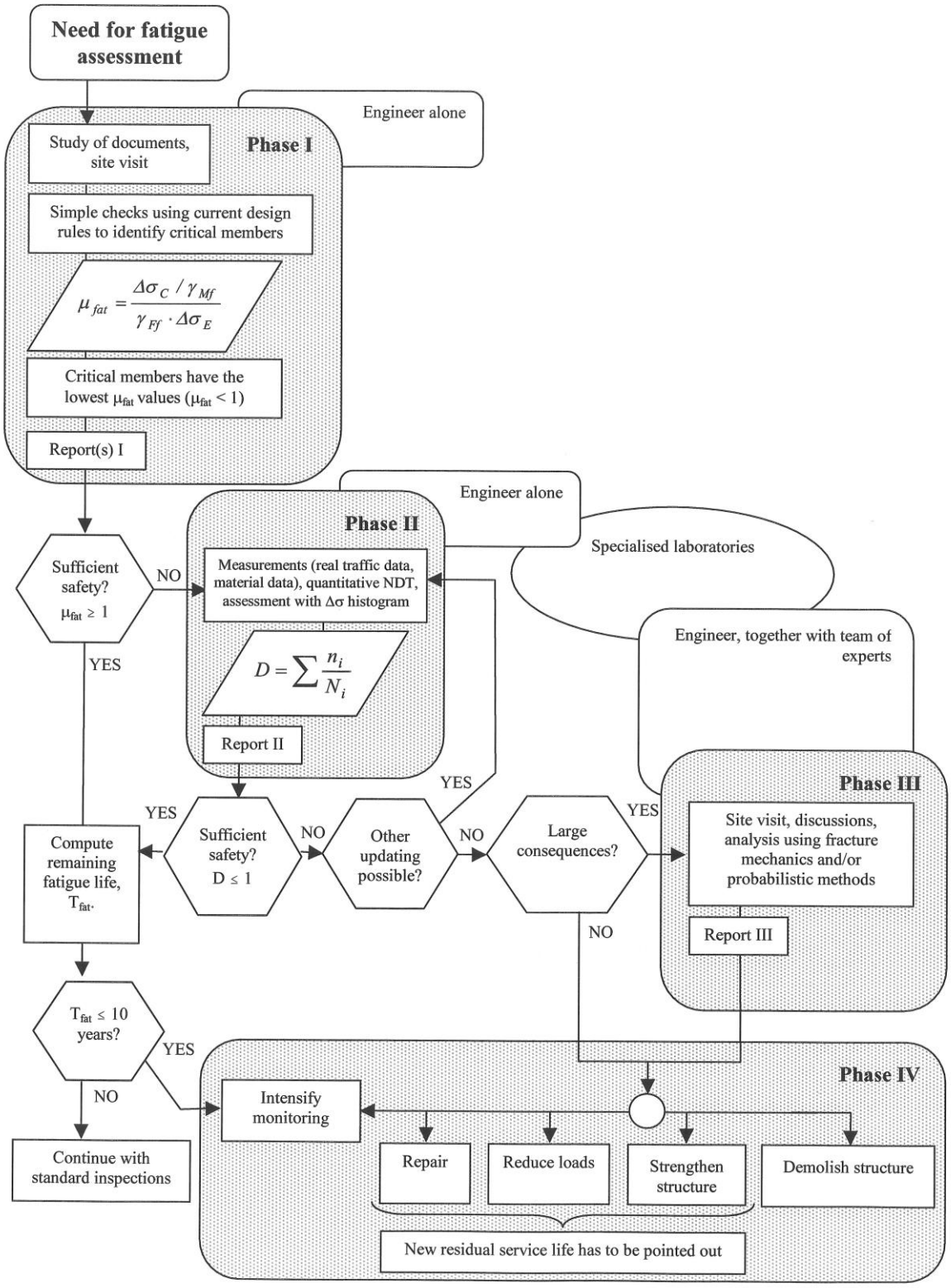


Figure 4-2 Fatigue assessment procedure for existing steel bridge (ECCS, 2008)

4.2 Background and motivation

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 not too much expensive. However, this is not the case when retrofitting structures already in service. Alternative approaches are available which give more realistic results. The high cost of strengthening existing structures makes it more likely to be worth using them in assessment.

4.2.1 Justification for advanced assessment methods

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. This is not the case of non-redundant structures, such as the Minneapolis 35W bridge (USA), NSTB (2008). However, tests in the laboratory as well as experiences from the real world have put in evidence how most bridges present an important reserve of strength when compared to their capacity predicted during the design or even obtained with an 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 were reached in the last years dealing with non-linear behaviour of materials and geometry simulation, a complete accurate modelling is still 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 with the failure of the whole bridge not considering redundancy and robustness

The reserve of strength in existing bridges is clearly shown in the examples presented in Figure 4-3 and Figure 4-4. In the first figure, the case of the bridge substructure (piers) is shown. Although the piers may be considered as failed because of impact loading by a truck or earthquake action, they still have some capability of supporting the bridge deck, which, in fact, is not falling down and consequently could be retrofitted and put into service again at a moderate cost. Figure 4-4 shows the robustness of the bridge shown there. In fact, defining the robustness as the ability of the structure to resist actions that were not taken into account in the design or, alternatively, as the capacity to respond with small global consequences to the apparition of local damage, it is evident that the bridge deck is resisting well (at least versus the permanent loads) the fact that one pier is completely removed from its original position due to a truck impact. This situation is, of course, not considered during the design, as the bridge is designed as a succession of simply-supported structures. The precast girders are not connected over the piers and the only connection between the spans is through the upper reinforced concrete slab. The upper slab was never designed to resist the load combination due to the absence of one pier. However, the membrane (catenary) action of the reinforced concrete element makes possible to resist the permanent loads. A basic assessment of the bridge (not taking into account the non-linear behaviour and membrane action of the concrete slab) will give the result that the bridge should be completely replaced. However, an advanced assessment, either based on a more sophisticated numerical analysis or a load test on the bridge, could discover the possibility of a higher strength and

the possibility to put the bridge in operation again with a minimum retrofitting work. This will influence the terms cost of repair and cost of failure in equation (4.1) of the life-cycle assessment (LCA).

Figure 4-3 Collapse of bridge piers due to vehicle impact (left) and earthquake action (right)

The JRC (Joint Research Centre of EC) - ECCS (European Convention for Constructional Steelwork) document (ECCS, 2008) enhances the use of advanced assessment techniques in the following statement: “For steel bridges including the old riveted ones there are numerous approaches to such assessments, partly standardized by national codes or recommendations. In the light of the development of the European single market for construction works and engineering services there is thus a need to harmonize them and to develop agreed European technical recommendations for the safety and durability assessment of existing structures. These recommendations should follow the principles and application rules in the Eurocodes and provide a scheme with different levels of analysis: a basic level with general methods and further levels with higher sophistication that call for specific expertise.

This document is the background document in support to the implementation, harmonization and further development of the Eurocode 3. The report is part of the so-called background documents on Eurocodes. It has been prepared to provide technical insight on the way existing steel structures could be assessed and the remaining fatigue life could be estimated. It may be used as a main source of support to further harmonize design rules across different materials, and further develop the Eurocodes.

Figure 4-4 Collapse of bridge pier and effect on the deck

4.2.2 Justification for advanced assessment methods in a LCA framework

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, the concluding remarks of the parameter **asset condition** are pointed out**: "**The asset condition is highly relevant to LCAT 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 model. The condition parameters will be subjected to deterioration and will be changed by the action of interventions. The condition parameters relevant to LCAT will be significantly influenced by the deterioration models that are currently being developed by WP2, and these parameters will be input into LCAT development in Task 5.5.

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.

As pointed out in NCHRP (2003), over the life cycle of an asset, a series of repair, maintenance and renewal interventions will normally be undertaken from time to time to return the condition of the asset to a higher level or at least meet the minimum acceptable level and to extend the service life of the asset. This is illustrated in Figure 4-5 from the BLCCA (Bridge Life-cycle cost analysis) report :

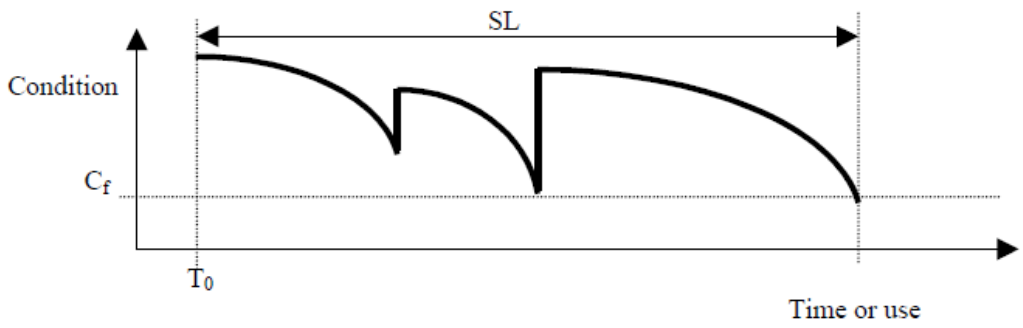


Figure 4-5 Typical model of life cycle condition of an asset with repair and renewal interventions to return the condition of the asset to higher level. (NCHRP, 2003)

Similar approaches are formulated in other specific documents related to life-cycle costing (ISO 15686-5 2008, DB 805 ff 2008).

Figure 4-5 is representative of the case where basic inspection and condition assessment is implemented. However, when a policy of combining advanced assessment with less severe repair/strengthening is applied, we should refer to the following Figure 4-6:

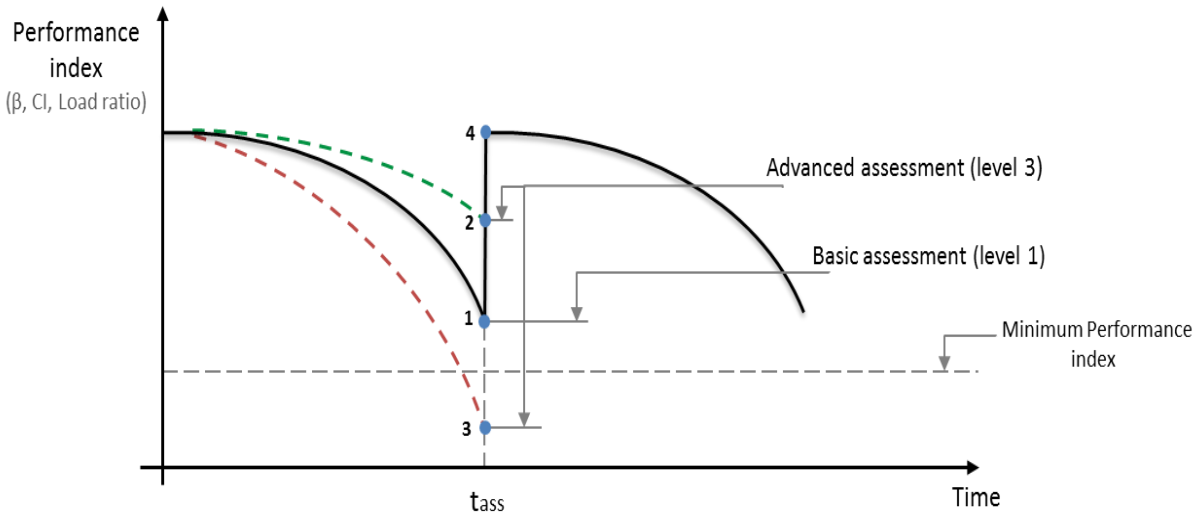


Figure 4-6 Model of life cycle condition of an asset with repair and renewal interventions taking into account the result of an advanced assessment

From a wider perspective, as shown in Figure 4-6, the “advanced assessment” should also be considered in the process of return of the asset condition to the correct performance level (point 4). In the case when 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. In the case that an advanced assessment is decided and implemented in the bridge, 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 with advanced assessment). Within a LCA framework also the time to perform the advanced assessment (t_{ass}) should be considered as one of the variables of the problem. It should also be pointed out that the situation where the performance obtained with the advanced assessment is higher than with the basic one, could not be always the case. In fact, the advanced assessment may discover that the bridge is in a worse condition. Within a LCA framework, the advanced assessment may be viewed as an alternative/complement 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 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 (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.

The main objective of WP5 is to create a tool (Life Cycle Assessment Tool - LCAT) that can compare different maintenance/replacement strategies for track and infrastructure based on a life cycle evaluation. The evaluation shall quantify:

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

The chosen methodology for the LCAT is based on recommendations from Task Group T5.3 of MAINLINE, i.e. results from the LCA analyses are converted into monetary units and added to the financial cost (results from LCC analyses). To increase the usability and transparency of the results concerning costs (financial and environmental) it is of vital importance that the user can break-down costs related to different categories such as *agency costs, user costs or environmental costs* into minor units to identify the cost-drivers.

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 to stop the normal traffic operation in the bridge. However, as will be seen later in this deliverable, some advanced assessment techniques requiring the intervention in the bridge, will derive in possible user costs 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.

The aim of WP1 is to provide a catalogue concerning maintenance strategies to extend the service life of predefined existing types of assets. Proper maintenance of existing structures 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 economically effective and environmental friendly. Such strategy can be the result of an advanced assessment which reveals that the asset still has adequate structural capacity to continue its function without any up-grading. Therefore, besides other maintenance/renewal interventions, the final outcome of WP1 will feed into the LCAT considering assessments as a way to extend the service life of existing assets. This will provide a solid basis for decision-makers when and “how much” to repair, or when to replace or renew an existing asset.

4.3 Scope and objectives

The two main objectives of this report are:

1. - To describe a set of proposed advanced assessment methods that may be incorporated in the life-cycle management of railway infrastructures, what information is required for an advanced assessment, and the limitations that may exist.

2. - To see how the costs and benefits of the proposed advanced assessment methods may be incorporated within a LCA framework.

The scope of this report is limited to the assessment of relevant railway infrastructure assets. The assessment methods considered will be those applicable to the following asset types, as selected in Deliverable 2.1:

- Cuttings
- Metallic Bridges
- Lined Tunnels
- Track (including rails, sleepers, ballast, switches and crossings)
- Other civil engineering structures (including culverts, retaining walls and coastal/river defences).

5. Description of latest developments for advanced infrastructure assessment

5.1 Introduction

The objective of this chapter is to introduce the most advanced methods for railway infrastructure assessments with the aim of obtaining the best estimates of the actual condition and try to postpone or reduce future repair or strengthening solutions.

The chapter is divided according to the main infrastructure assets identified within the MAINLINE project as those which should receive more attention among all: bridges (mainly metallic), tunnels, switches and crossings and cuttings. Other earth-works such as embankments are not included in this project as they are the subject of other related projects (SMARTRAIL).

5.2 In bridge engineering

According to Casas et al. (2010) and Wisniewski et al (2012), an enhanced level assessment for bridges would involve one or a combination of the following tools and techniques:

- Direct application of reliability analysis 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)
- Proof load testing

In the following, the assessment procedures for bridges present in several Codes in different countries are presented. Only those Codes and recommendations that in some way deal with the listed advanced assessment methods are mentioned. Also the main background and motivation of the proposed advanced techniques are presented.

There also exist general recommendations and guidelines on the subject. For instance, the Guideline SB-LRA (2008) provides useful information on the use of advanced assessment methods and their practical application. The CIRIA report C664 (CIRIA, 2008) is more specific for the basic and intermediate assessment of iron and steel bridges.

5.2.1 Direct application of reliability-based assessment methods

The application of these methods is mandatory 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. In Melchers 1987 and Nowak 2000, a detailed review of reliability-based analysis methods and applications to bridge engineering are presented. 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.

Denmark

The Danish Guideline Document (Danish Road Directorate, 2004) is a good example of this direct application. The document was prepared with the main objective to outline practical

guidelines and models for use in the reliability-based classification of existing road bridges. The purpose of the guide is to create a uniform basis for the reliability-based classification of the Danish Road Directorate's bridges.

When the structure fails the evaluation check, a direct probabilistic analysis may be performed according to the structural reliability principles and the specified stochastic load and resistance models presented in (Danish Road Directorate, 2004). Also, the Guideline provides background information for the development of fully probabilistic load models for different road categories and expected intensity of traffic. Different models are proposed for ordinary traffic and heavy permit vehicle passage. Furthermore, guidance is provided for modelling the traffic loads on various bridge spans, considering the effects of single heavy axles, groups of axles and the multiple-presence of heavy vehicles. Although very versatile, the proposed traffic load models are difficult to use in practical applications and require advanced knowledge of probability and statistical methods.

The stepwise procedure that is proposed in (Danish Road Directorate, 2004) and that could be considered general enough is as follows:

1) Classification consists of a bridge class for Normal passage and a bridge class for Conditional passage. Classification is an iterative process. The starting point requires selection of a required class (e.g. 100, 125, 150, 175 or 200), for which it is desired to verify that the bridge has the required capacity.

2) The limit state function is determined by a deterministic analysis.

3) The required safety index β_{target} is determined on the basis of the limit state characteristics.

4) Stochastic modelling of dead load and materials inclusive model uncertainty

5) Stochastic modelling of traffic load (see chapter 5.2.3)

6) Safety index β for the given traffic population is determined based on available calculation methods.

7) It is checked whether $\beta < \beta_{target}$ where β_{target} is the minimum acceptable safety index.

The chosen reliability-based analysis method will depend on the limit state function's complexity and the safety requirement.

Germany

In Germany, the Guidelines used by the Railways are also based to some extent on a reliability-based concept (DB 805ff-2008).

Switzerland

The new Swiss code for the assessment of existing structures (SIA-269, 2011) is based on structural reliability principles and also provides sets of safety factors different from design to be used in the assessment. Target values of the reliability index are proposed based on the consequences of structural failure and the efficiency of interventions according to table 5.1. In this case, the efficiency of intervention and consequences of failure are not only measured in subjective terms (minor, low,…..) as in other Codes, but also in quantitative values which provides a more rational way for the assessing engineer to define the correct target value of the safety proposed. Probability based assessment is also allowed for bridges which fail the standard safety checks. In table 5.1, ρ is the ratio of direct cost in the event of failure to the

costs necessary to restore the structure after a failure and EF_M is the ratio between the risk reduction as a result of interventions with respect to safety costs. The individual risk is limited to a failure probability of 10^{-5} per year. For the calculation of the risk reduction, the assumption of 3 to 10 million Swiss francs is adopted for the cost of saving an human being.

The Swiss code provides a set of adjustment factors for the standard roadway traffic load and defines special loads and dynamic amplification factors for railway loading.

Table 5-1 Target value of the reliability index with a reference period of 1 year (SIA-269, 2011)

	Consequences of structural failure		
Efficiency of intervention	Minor: $\rho < 2$	Moderate: $2< \rho < 5$	Serious: $5< \rho < 10$
Low: $EF_M < 0.5$	3.1	3.3	3.7
Medium: $0.5 < EF_M < 2.0$	3.7	4.2	4.4
High: $EF_M > 2.0$	4.2	4.4	4.7

United Kingdom (UK)

UK manuals (Highways Agency, 2006), are based on the same structural reliability principles and also provide sets of different safety factors to be used in the assessment. Probability based assessment is also allowed for bridges which fail the standard safety checks. However, they do not elaborate too much and just suggest that such methods require specialist knowledge.

Canada

The Canadian Highway Bridge Design Code (CAN/CSA-S6-06, 2006) devotes several chapters to the load rating of existing bridges. In CAN/CSA-S6-06, the recommended assessment procedure commences with the identification of the most likely failure modes and the determination of the appropriate target reliability index β corresponding to each mode, considering the system behaviour, element behaviour and the inspection level. The recommended target reliabilities are shown in Table 5.2.

Table 5-2 Target reliability index for normal traffic (CAN/CSA-S6-06, 2006)

System behaviour	Element behaviour	Inspection level		
		INSP1	INSP2	INSP3
S1	E1	4,00	3,75	3,75
	E2	3,75	3,50	3,25
	E3	3,50	3,25	3,00
S2	E1	3,75	3,50	3,50
	E2	3,50	3,25	3,00
	E3	3,25	3,00	2,75
S3	E1	3,50	3,25	3,25
	E2	3,25	3,00	2,75
	E3	3,00	2,75	2,50

Note: S1 - element failure leads to total collapse; S2 - element failure does not cause total collapse; S3 - local failure only; E1 - sudden loss of capacity with no warning; E2 – sudden failure with no warning but with some post-failure capacity; E3 – gradual failure; INSP1 – component not inspectable; INSP2 – inspection records available to the evaluator; INSP3 – inspections of the critical and substandard members directed by the evaluator.

However, instead of recommending a direct reliability check in all cases, the CAN/CSA-S6-06 first recommends the adjustment of the partial safety factors based on the target reliability level. Thus, after selecting the reliability index for each failure mode, the set of corresponding partial safety factors is determined, as presented in Table 5.3.

Table 5-3 Maximum permanent load and traffic load factors for assessment (CAN/CSA-S6-06, 2006)

Load category	Symbol	Target reliability index β						
		2,50	2,75	3,00	3,25	3,50	3,75*	4,00
Permanent loads D1	α_{D1}	1,05	1,06	1,07	1,08	1,09	1,10	1,11
Permanent loads D2	α_{D2}	1,10	1,12	1,14	1,16	1,18	1,20	1,22
Permanent loads D3	α_{D3}	1,25	1,30	1,35	1,40	1,45	1,50	1,55
Traffic loads	α_L	1,35	1,42	1,49	1,56	1,63	1,70	1,77

*Note: * - Target reliability index and the corresponding safety factors used also for the design of bridges.* D1 – factory produced components and cast in place concrete excluding decks; D2 – cast-in-place concrete decks; D3 – bituminous surfacing with assumed standard thickness 90mm.

The evaluation is performed using the format presented in equation (5.1), which defines the load multiplier required to cause failure:

$$F = \frac{U \cdot R_r - \sum \alpha_D \cdot D - \sum \alpha_A \cdot A}{\alpha_L \cdot L \cdot (1+I)}$$

(5.1)

In equation (5.1), R_r is factored resistance, D is a permanent load, A is a secondary variable load, L is a primary load (e.g. traffic load), I is the dynamic amplification factor, U is a resistance adjustment factor and α_i are the corresponding partial safety factors for loads. The resistance adjustment factor U is given in (CAN/CSA-S6-06, 2006) for different structural elements and limit states. It accounts for the variation between the actual resistance of the component, as it would be observed in tests, and the resistance calculated using simplified code methods. The factored resistance is computed using nominal material strength parameters retrieved from drawings or historical records multiplied by the same material factors as those used in design (Table 5.4). According to CAN/CSA-S6-06, when the application of equation (5.1) shows insufficient capacity of the structure, the simplified probabilistic assessment, using Mean Load Method, may be performed following the recommendations presented in (CAN/CSA-S6.1-06, 2006).

Table 5-4 Selected material resistance factors for assessment (CAN/CSA-S6-06, 2006)

Material and critical failure mode	Material resistance factor
Concrete	ΦC=0,75
Reinforcing bars	Φ_S=0,90
Prestressing strands	Φ_P=0,95
Structural steel (flexure, shear and tension)	Φ_S=0,95
Structural steel (compression and torsion)	Φ_S=0,90

USA

The trend of direct calculation of bridge capacity based on the direct application of reliability-based methods is not only envisaged in Europe, but is also afforded in the USA. In fact, in 2010, a scan team from the United States visited several agencies in Europe and identified many practices and technologies related to the topics of interest that should be implemented in the coming years in US practice. Between them, there are (FHWA-AAHSTO, 2010):

1. Promoting and increasing the practicing bridge engineer's use of refined analysis for design and evaluation.

2. Encourage States to use refined analysis for evaluation in combination with reliability analysis to avoid unnecessary posting, rehabilitation, or replacement of bridge structures.

3. Encourage the AASHTO Subcommittee on Bridges and Structures to adopt the concept of annual probability of failure (exceedance) as the quantification of safety in its probability-based design and rating specifications rather than the reliability index for a 75-year design life.

The AASHTO LRFR code (AASHTO LRFR, 2003) for Load and Resistance Factor Rating (LRFR) uses similar reliability based assessment principles and the load rating process to those of CAN/CSA-S6-06. The load rating is based on lower live load factors and a set of "legal trucks" that are less conservative than those used when designing new bridges. The evaluation of the member capacity must take into consideration the reduced strength as estimated by the bridge inspector. The AASHTO manual requires a lower resistance factor depending on the severity of the damage as determined by the bridge inspector. If the bridge is classified as non-redundant, a system factor lower than 1.0 is also specified. The Rating Factor *RF* in the AASHTO bridge evaluation process is equivalent to the live load multiplier *F* in equation (5.1). The AASHTO code was calibrated so that existing bridges that satisfy the rating criteria will meet a unique target reliability index β_{target}=2.5 for a five-year rating period, although the use of the system factor would increase this implicit level for non-redundant configurations. For reference, it should be mentioned that target reliability index β=3.5 for a design life of 75 years is used in the AASHTO LRFR bridge design code. Similarly to the CAN/CSA-S6-06, AASHTO LRFR recommends the application of direct reliability index checks with a member reliability index β_{target}=2.5 for special conditions. However, the actual implementation of direct reliability checks in the U.S. has remained within the subject of research and demonstration projects.

The bridge-rating process described in AASHTO MBE (2008) permits ratings to be determined by allowable stress, load factor, or load and resistance factor methods. These three rating methods may lead to different rated capacities and posting limits for the same bridge, a situation that has serious implications with regard to public safety and the economic wellbeing of communities that may be affected by bridge postings or closures. The work done by Wang et al. (2011) was presented in two papers which summarize a research program to develop improvements to the bridge-rating process by using structural reliability methods. The first paper provided background on the research program and summarized a coordinated program of load testing and analysis to support the reliability assessment leading to the recommended improvements. The second paper presents the reliability basis for the recommended load rating, develops methods that closely couple the rating process to the results of in situ inspection and evaluation, and recommends specific improvements to bridge-rating methods in a format that is consistent with the Load and Resistance Factor Rating (LRFR) option in the AASHTO Manual for Bridge Evaluation. The proposed improvements recognize the uniqueness of each existing bridge and take advantage of accessible in situ information to produce bridge ratings that provide for public safety without undue economic impact on the community served.

Other examples and applications

The document prepared by the TC 6 (Fatigue) of the European Convention for Constructional Steelwork (ECCS, 2008) presents a methodology for the reliability-based fatigue assessment of steel bridges as one of the advanced assessment methods (level III in Figure 4-2). The justification is based on the fact that changes in some of parameters required for calculations based either on the classification method (or the similar geometric (hot-spot) stress method) or on the fracture mechanics method can be shown to have a significant effect on the calculated fatigue life. One way in which this uncertainty may be considered explicitly is through the use of probabilistic methods, which can be employed in conjunction with either the classification method or the fracture mechanics method. Normally, when these methods are used, the various “input parameters” (i.e. the detail category, the initial crack depth, etc.) are assigned deterministic values. When probabilistic analysis is employed, these values are replaced with statistical distributions. ECCS/JRC joint report presents a step-wise procedure from deterministic to reliability-based fatigue assessment. In case deterministic approach is enough, no further effort in terms of cost-intensive testing and assessment is needed. Reliability assessment is made only if all other methods have failed to proof the required safety. Thus, the Fracture Mechanics Approach (serviceability assessment for the safe service interval between a detected existing or assumed theoretical crack length) is needed, only if the deterministic or damage accumulation assessment referring to critical details fails. For the case of reinforced or prestressed concrete bridges, a comprehensive study on fatigue assessment by using a reliability-based methodology is presented in Crespo and Casas (1998).

Probabilistic analyses were used by Estes and Frangopol (1999) to optimise bridge repair strategies. A bridge with nine steel beams, side by side, was first analysed to identify critical failure modes. Safety indices were calculated for 16 different cases, and these were then combined to form a time-variant reliability system. All loads were assumed to be time-invariant but the corrosion of the reinforcement in the concrete slab was included in the time-variant analyses. Based on this system, different repair criteria with related costs were investigated, and a cost-effective repair strategy was developed for the bridge.

In a paper by Stewart, Rosowsky and Val (2001) the use of reliability-based bridge assessment in combination with risk ranking was investigated. Instead of focusing on the precise value of safety for a bridge, expressed as a safety index, the calculated safety index was used for comparison of different bridges and as a decision-making tool for prioritizing the need for repair.

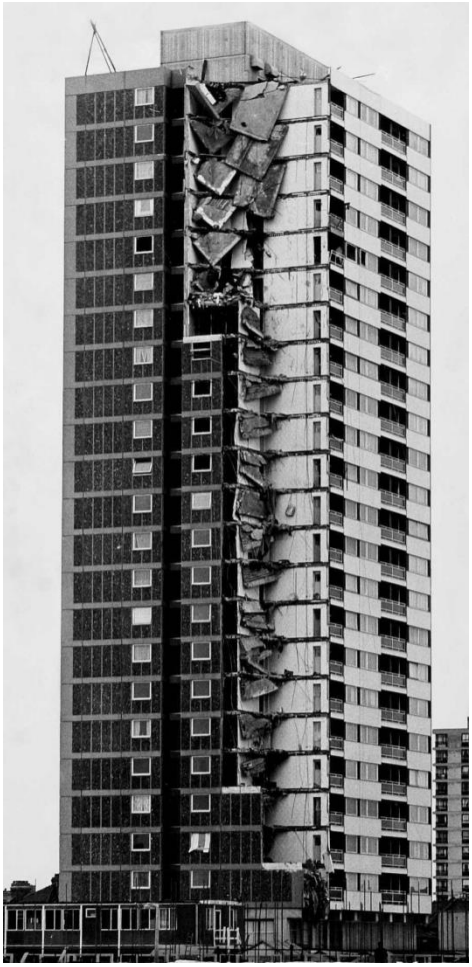
Recently, a methodology for the probabilistic assessment of masonry arches at the serviceability and ultimate limit states was presented by Casas (2011). Firstly, the author explains the definition of the different failure modes and corresponding limit state functions that may occur depending on the type of masonry construction (single-ring and multi-ring). Secondly, according to the fact that the most reported modes of failure of those kinds of bridges in his study proposed a methodology for the fatigue and serviceability assessment. It included an assessment process which includes 3 levels of assessment. Finally, because of the lack of reliable material data (in the statistic sense) or available response models, the author concludes that the practical application to existing bridges using reliability-based methods should be looked as preliminary.

One of the most important railway assets identified in MAINLINE project are metallic bridges. For these bridges, fatigue failure is the dominant failure mode and therefore fatigue assessment is of vital importance. Because parameters affecting the fatigue strength are also random in nature, a probabilistic approach is mandatory. A probabilistic approach for the fatigue assessment of riveted railway bridges was proposed by Imam et al (2006, 2008), with a finite element model of a typical short-span riveted railway bridge being

the focus of investigation (Imam et al 2006, 2008). Further approaches have been developed based on reliability theory in order to incorporate the high degree of uncertainty associated with evaluating bridges in fatigue. Such a methodology is the one developed by Tobias and Foutch (1997) for the fatigue evaluation of riveted girder railway bridges. The approach allows for the inclusion of uncertainties associated with strengths, bridge responses and loadings. These inclusions allow fatigue endurance predictions to be more qualified and give a broader idea of the potential lives of bridges.

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 from SMARTRAIL Project on “Development of a General Rail Transport Infrastructure Safety Framework” (SMARTRAIL 2013).

5.2.2 Consideration of system safety, redundancy and robustness 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. Historically, the partial collapse of the 22-story Ronan Point Apartment Tower in London in 1968 initiated by a gas explosion at the 18th floor was a key event for focusing the attention of the structural engineering community on the important subject of structural redundancy (Figure 5-1). The lack of alternative load paths in the Ronan Point tower did not permit the redistribution of forces after the explosion ripped a load carrying precast panel, led to a chain mechanism that damaged the building all along its height causing progressive collapse of a portion of the structure. Another key event was the collapse of the Alfred P. Murrah Federal Building in Oklahoma City in 1995 which occurred after the detonation of a truck bomb outside the building. In that case, the top floors’ exterior columns were supported by a continuous transfer girder which upon failure, due to the blast, caused the collapse of all the exterior columns and floor areas supported by those columns. Unfortunately, similar collapses were also observed in bridges. In 1980, one pier of the Sunshine Skyway Bridge in Tampa Bay was hit by a cargo ship provoking its collapse and the loss of a large part of the superstructure. Here again, the lack of alternative load paths led to the collapse of the structure.

Figure 5-1 Ronan Point apartment tower 1968

In 1983, the Mianus Bridge in Connecticut lost a whole span after the failure of a “pin and hanger” connection. The bridge deck was supported by two main beams and the spans were made continuous by the “pin and hanger” assemblies. Corrosion of the connection provoked the slip of the pin from its hanger. By knocking out the beam it was supporting, the rest of the structure could not redistribute the load to another part of the structure, which is, in this case, the other main beam. This collapse was thus initiated by the local non-ductile failure of a

connection in the non-redundant two-girder superstructure topology. The importance of the redundancy is very well highlighted in the collapse of the Mississippi River Bridge in Minneapolis in 2007 (Figure 5-2) , in which case, the whole bridge, which has been classified as non-redundant by NTSB (2008), catastrophically collapsed after the failure of a gusset plate connection.

Figure 5-2 Mississippi River Bridge (Minneapolis)

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; 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. The concept of disproportionate collapse must always be kept in mind. A complete study on the robustness of structures is available in COST-TU0601 (2011).

Robustness is qualitatively defined in several codes, including the Eurocodes. However, far from providing a final and definitive answer, the statement in the Code has encouraged debates and research on the topic. Additionally, the EC does not provide a specific approach or criteria to evaluate if a structure is robust or not. On the other hand, guidance is given to limit the extent of damage by requiring specific methods for providing ties and details to connect different structural elements. Structures are divided into classes based on the consequences of failure and no specific prescription is presented for bridges. However, up to now, no code presents methodologies to quantify the robustness of a particular structure. In fact, in the Danish Guideline (Danish Road Directorate, 2004), the robustness of the bridge is considered and indicates that should be also verified. The robustness requirement ensures that the failure of a limited part of the structure does not lead to failure of the whole structure. This can be documented by system safety analysis. Alternatively, satisfactory safety can be verified for key components. However, there is no indication about a practical procedure to quantify or evaluate the robustness degree.

In Germany the DB 805 Guideline presents the stepwise procedure and, in case the S/N-curve concept fails to proof a structure safety to fatigue, a redundancy analysis looks for the weakest elements in terms of low redundancy. Riveted built up sections are more robust and more redundant than a single profile without cover plates. In these details the Fracture Mechanic Concept is applied. A fatigue crack, once it has started to grow, will propagate until its critical remaining cross section fails brittle. The more redundant elements have either

several built up sections requiring to restart the crack propagation or an alternative load path can take over the force.

Although the concept of robustness is also mentioned in the new Swiss Code, methods to quantify robustness or redundancy in existing structures to take them into account in the assessment procedure are not available.

The British Building regulations (British Standards Institution, 2010) propose different classes of structural importance, leading to different levels of refinement in the evaluation depending on the risk to the structure. BSI (2010) also provides several prescriptions on continuity, ductility of the members, and resistance requirements for key elements.

In the Canadian Code, the redundancy of the bridge system and the ductility of the member being evaluated are taken into account by using different target reliability indexes, and the corresponding safety factors, for different bridge components and failure modes (see Tables 5.2 and 5.3). This simple approach takes some advantage of the reserve strength present in redundant structural systems. Furthermore, what is important from the practical point of view, its application does not involve any additional work compared to the standard assessment procedure. However, this methodology is rather conservative, based on judgment calls with the respect to classifying the consequence of member failure without evaluating the actual redundancy of the bridge systems. The approach is based on a notion of risk or a perception of risk. It relates the notion of risk to a target member reliability index rather than directly providing the modification factors. However, the same lack of specificity encountered with the AASHTO approach remains an issue. For example, engineer decisional stage would be helped by a definition of which types of bridge configurations are considered redundant or what are the requirements to classify a member as having sufficient levels of ductility. This lack of specificity is important given that traditionally bridge engineers have used erroneous criteria to classify bridges into the redundant and non-redundant categories based on the number of parallel members or degrees of indeterminacy as will be discussed further below.

In the USA, The AASHTO LRFR provides a set of system factors that may be used to require the members in non-redundant systems to have higher safety factors than those of redundant systems. The objective is to maintain uniform system reliability levels rather than uniform member reliabilities. The system factors in the LRFR provide a more objective alternative to the use of subjectively assigned load modifiers as in the standard AASHTO bridge design specifications. Furthermore, the AASHTO LRFR encourages the use of a direct redundancy analysis check following the approach recommended in (Ghosn & Moses, 1998; Ghosn, 2005). The methodology proposed in AASHTO MBE is a clear example of how measures of redundancy can be incorporated in the rating of existing bridges. Providing system factors for bridge members or specific bridge types that are calibrated via a reliability-based system evaluation approach would ensure that bridge systems will provide a minimum level of safety and robustness.

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.

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.

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 (COST TU-601, 2011):

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

(5.2)

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 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:

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

(5.3)

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 equation (5.3) 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; Janssens, 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.

Probabilistic indicators of robustness were introduced by Frangopol and Curley (1987) and Lind (1995), and are based on the comparison between the safety of the intact and damaged structure, considering structural safety defined by a probabilistic measure, as the probability of failure or the reliability index. This approach was initially introduced for the analysis of a predefined damage state, but extended by Cavaco et al. (2013) to consider a range of damage levels.

Deterministic indicators are the simplest procedure to measure robustness, but will most likely serve as a foundation of code prescriptions that must be calibrated using more advanced methods.

Starossek (2008) provides three deterministic measures of robustness based on the stiffness matrix, damage and the energy of the structure. The first measure is related to the global stiffness matrix of the structural model:

$$R_S = \min_j \frac{\det K_j}{\det K_0}$$

(5.4)

where K_0 is the stiffness matrix of the intact system and K_j the one of the damaged system. The damage scenario is defined as the loss of a main member or connection from the system. The measure is based on the linear elastic behaviour, therefore, as noted by Haberland (2007) it does not adequately measure structural robustness but is more of an indicator of the cross-linkage of the system.

A damage-based measure advanced by Starossek (2007) is given as:

$$R_d = 1 - \frac{p}{p_{\lim}}$$

(5.5)

where p is the maximum spread of damage caused by a given hazard and $p_{\lim}$ is the acceptable damage. Conceptually, the parameter p can be based on structural or economic considerations. For example, p could indicate the percentage of steel that have corroded and p_{lim} could be the maximum corroded percentage that can be accepted. While this can be easily adapted for simple cases, no specific approaches to determine p and p_{lim} have been provided for complex structural systems. Even for simple systems, Eq. (5.5) does not account for the manner by which the damage progresses.

To consider the damage progression process for complex systems, an integral variation of Eq. (5.5) was proposed by Starossek (2008) :

$$R_{d,int} = 1 - 2\int_0^1 \left[d(i) - i\right] di$$

(5.6)

where i is the extent of initial local damage and $d(i)$ is the maximum extent of total damage that follows from the initial damage i. According to Eq. (5.6), a system where the global damage ratio is always equal to the extent of local damage, that is if the damage does not spread with d(i)=i, will be robust with $R_{d,int} = 1$. As shown in Figure 5-3 that gives a schematic

representation of possible relationships between $d(i)$ and i, the damage index in Eq. (5.6) is highly dependent on that relationship. For example, if the relationship between $d(i)$ and i is represented by the curve labelled A, the dashed area between curve A and the diagonal represents its $R_{d,int}$. This index is compared to the index obtained if the relationship between $d(i)$ and i represented by the grey area. For a system that follows curve A, an initial damage equal to half of the total damage represents the most critical scenario while for system B a small initial damage does not spread to other parts of the structure up to a point in which a large initial damage extent has strong consequences on the rest of the structure. Yet, for both cases, Eq. (5.6) gives the same robustness index. This demonstrates that Eq. (5.6) can only be used to compare systems that have similar *d(i)* vs. *i* relationships. The other issue with this measure arises due to the lack of criteria for establishing the necessary relationships between the local and global damage levels.

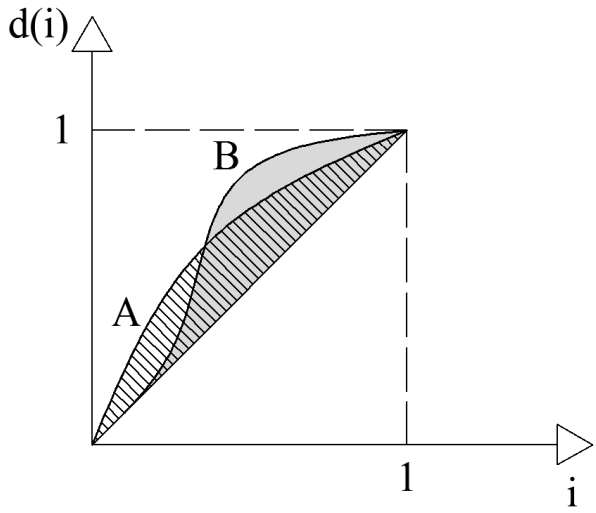


Figure 5-3 Damage based robustness measure (Starossek, 2008)(2008)

The energy based measure also proposed by Starossek (2008) compares the energy released by the initial failure and the energy required to cause collapse. It is formulated as:

$$R_e = 1 - \max_j \frac{E_{r,j}}{E_{s,k}}$$

(5.7)

where $E_{r,j}$ is the energy released by the initial failure of an element j and available for the damage of the next structural element k, in other words the failure of a member is associated with the release of an amount of energy (kinetic, for example in the dynamic process of failure of a suspended bridge hanger) that will dissipate in the rest of the structure ($E_{r,j}$). $E_{s,k}$ is the energy required for the failure of a generic k element different from *j* . If ($E_{r,j}$)is equal or greater than $E_{s,k}$ the failure of element j will lead to the failure of element k and $\mathrm{Re} \leq 0$ indicating that the system is not robust, while values that range between 0 and 1 represent different degrees of robust systems. This method represents an evaluation of the tendency of producing a progressive collapse after an initial damage and can conceptually be applied to evaluate the robustness of complex structural systems. But, like the previously listed measures in Eq. 5.4 to 5.6, the practical implementation of Eq. (5.7) suffers from the difficulty in estimating the required parameters which in this case are the energies.

The three measures advanced by Starossek (2008) give different levels of evaluation of the robustness. But, they do not provide criteria to help determine which structures provide acceptable levels of robustness.

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 (Figure 5-4).

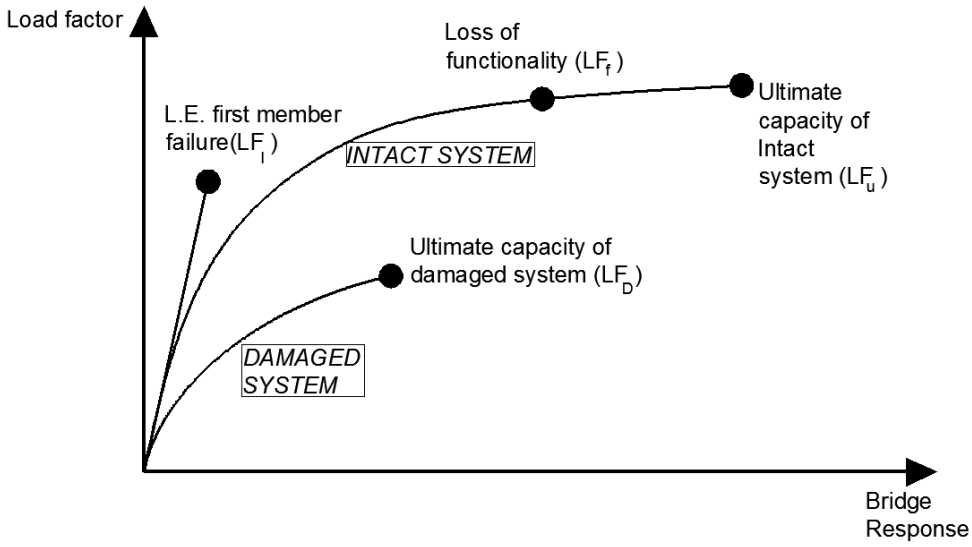


Figure 5-4 Load measures needed to calculate the redundancy of bridge systems

These measures are:

$$R_u = LF_u / LF_1$$
$$R_f = LF_f / LF_1$$
$$R_d = LF_d / LF_1$$
(5.8)

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. In addition to the measures Ghosn and Moses (1998) calibrated criteria for evaluating the redundancy of specific structures based on the reliability analysis of typical configurations. The measures of redundancy advanced by Ghosn are practical and can be implemented in structural codes as demonstrated by the similar measures implemented in the design of offshore structures (ISO 19902, 2007). Based on this approach, a proposed framework to include redundancy in the assessment of existing railway bridges is presented in Wisniewski et al (2006).

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

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

(5.9)

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 corresponds to considering that robustness is defined by the area under the normalized performance 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 5-5), as a structure for which no reduction in performance occurs for any damage level corresponds to full robustness (curve E in Figure 5-5). 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.

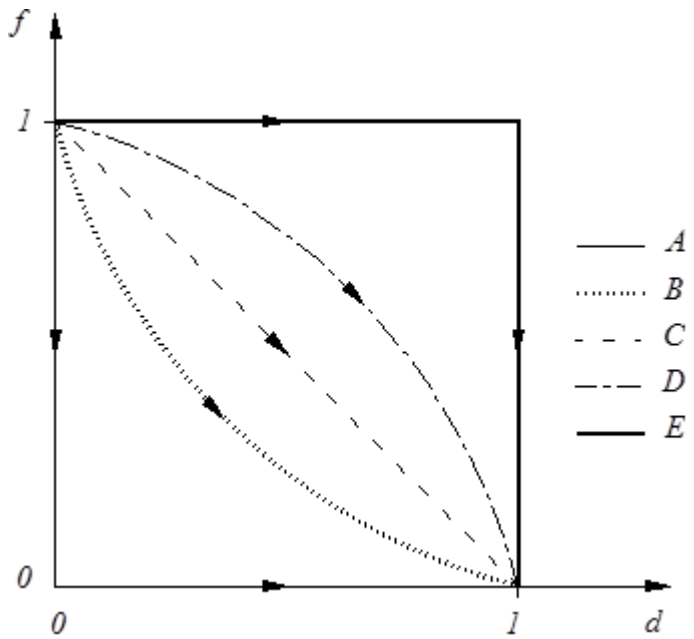


Figure 5-5 Interpretation of robustness according to Cavaco et al. (2010)

This approach for robustness allows considering 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, and can be related to service limit states or to ultimate limit states.

In any case, all the measuring methodologies presented above 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 of 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. (5.2) 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:

- 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.
- 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.
- 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 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 Casas et al. (2013) a comprehensive analysis of the different alternatives proposed to measure structural robustness and redundancy is presented. The different quantification approaches are based upon alternative definitions of robustness. 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 Equation (5.2) 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 behaviour. The prescriptions of detailing, local resistance and other influencing parameters should follow the identification of the structure in the code.

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 behaviour of the materials and be able to describe properly the overall response of the structure.

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.

As shown, 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 and is based on the research works developed in (Ghosn & Moses, 1998; Ghosn, 2005). This was the approach also considered in the SUSTAINABLE BRIDGES project. In the deliverable SB-LRA (2008) the guidelines and the method to follow for the consideration of system performance and redundancy to a specific bridge assessment are presented, based on the concept of the redundancy factor (Deterministic direct analysis method). A reliability-based approach is also presented in this deliverable. 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. Other simplified methods to consider the system behaviour and redundancy of existing bridges in an advanced assessment are also presented in SB-LRA (2008), SB4.4.1 (2007).

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

Today's freight trains are quite different from assumed design loadings. Over the last 30 years, trailing rail car weights have increased. The basic nature of train compositions has also changed. Heavy unit commodity, relatively fast intermodal, and mixed freight trains are now commonplace. Allowable rail car loads are also expected to be increased by 10%-20% over the next few years.

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.

In fact, this has been recognized worldwide and not only from an European perspective. In 2010, a scan team from the United States visited several agencies in Europe and identified many practices and technologies related to the topics of interest. Between them, the team identified (FHWA-AAHSTO, 2010):

1. Conduct research to create the basis to systematically introduce increasing levels of sophistication into analyses and load models with the objective of assessing bridges more accurately.

2. Encourage owners to periodically and routinely reassess traffic highway/railway loading, using recent weigh in-motion data, to ensure that their live load model adequately provides for bridge safety and serviceability for the desired service life and level of safety.

5.2.3.1 Site-specific live loads

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), a method is presented by which one is able to determine site specific characteristic train loads from WIM measurements.

There are two types of weigh-in-motion systems; track-based (WIM) and bridge-based (B-WIM) systems. These are described briefly below but for the interested reader more information is found in Liljencrantz (2007), SB4.3.2 (2007), and appendix D of UIC (2006).

Track based Weigh-in-Motion systems (WIM)

The primary purpose of some track based weigh-in-motion devices is to detect excessive dynamic wheel-rail contact forces to assist with identifying vehicle maintenance requirements and minimise the risk of damage to the track. When using these systems to determine static

axle loads it is important to take into account the accuracy of the system for determining equivalent static axle loads. Bogie or axle peak resolution depends on the location, orientation and span length of the member.

Available techniques are based on strain gauges (strain gauges are welded or glued to the neutral axis of the rail as in Figure 5-6 and Figure 5-7) or fibre optics (a glass fibre sensor is attached to a rail whereby it can be used to measure the vertical deflection of the rail due to a passing axle).

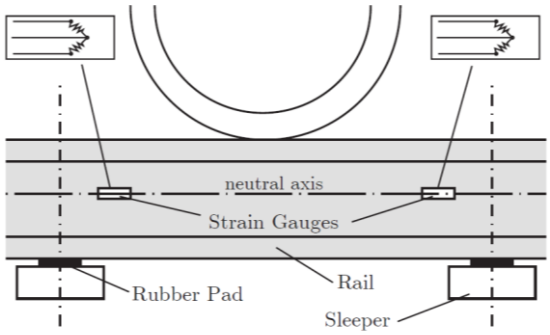


Figure 5-6 Placing and orientation of the strain gauges in a track based WIM (James, 2003)

Each gauge consists of two 350 ohm variable resistors that are positioned with a 45 degree orientation to the rails neutral axis. Between two adjacent sleepers two gauges are welded on each side of a rail and on both rails making a total of eight gauges between adjacent sleepers. The gauges on each rail are welded to form a Wheatstone bridge the total resistance of which is 700 ohms (±5%) per leg. The length of each gauge is 20mm and they are welded to the rail using 100 welding points.

The sampling frequency of the system is 30 kHz and the front end processor has both high and low pass filters. The low filter is designed to produce a clear signal of the load from the axles while the high pass filter is designed to produce a good representation of the peak loading from the dynamic action of wheel irregularities.

The peaks indicate wheel flats, when present, by comparing the peak and average reading of the loading from the axle.

The average reading is thus the most interesting from this studies point of view as it provides information about the loading from the axles hopefully with the dynamics due to the wheel motion and irregularities removed.

Figure 5-7 A typical site showing the wiring and the strain gauges welded onto the rails (James, 2003)

Bridge based Weigh-in-Motion systems (B-WIM)

A bridge, instrumented using strain gauges or strain transducers, is used as a scale for weighing passing trains. This method has the advantage of not disturbing traffic as the measuring instruments are generally fitted to the underside of the bridge. Locomotives of known weight can be used to calibrate the system for varying speeds. The method has been commonly used for the weighing of road vehicles but is less common in the railway industry. A B-WIM system for railways is described in Liljencrantz (2007) and Liljencrantz et al. (2007b).

There are several important differences between B-WIM on railways and B-WIM for regular road traffic. For instance:

- Trains are much longer, and have many more axles than a road vehicle. This means that a single detected vehicle may contain hundreds of axles. In many cases, the acceleration of the train sometimes needs to be taken into account.

- Trains run on tracks. This means that the problems associated with differences in sensor sensitivity based on where in the lane a vehicle is driving do not exist. This can be a large source of inaccuracy in road WIM

- There are no problems with identifying separate trains, since two trains never drive so closely to each other as to be confused with a single entity, and the strain originating from trains on different tracks can usually be separated from each other.

- There are only a small number of locomotive types, and electric locomotives usually do not vary in weight, which opens up the potential for self-calibration.

Of course, as the resolution of the B-WIM algorithm depends on the bridge type and if the track is ballasted or not, for most bridges, the individual axles of a bogie cannot be separated. Figure 5-8 below illustrates signals from the B-WIM system at Årstaberg in Stockholm. A train consisting of 2 locomotives and 6 wagons is passing over the bridge. The two sensors U1 and U4 are installed in the concrete section underneath the same rail but

some distance apart. This instrumentation enables an accurate calculation of not only the bogie weights but also speeds and bogie distances.

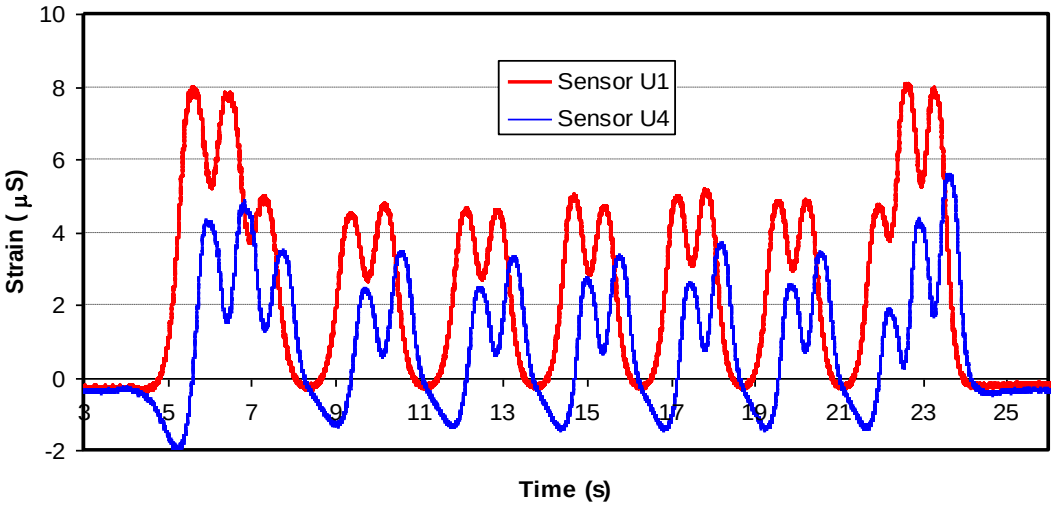


Figure 5-8 Strain measurements recorded from the Årstaberg bridge in Sweden due to the crossing of a train consisting of 2 locomotives and 6 wagons. The two sensors are a known distance apart and from the signals it is possible to calculate the speeds of the train

In González (2011), an algorithm is presented for obtaining axle-loads, spacing and speeds of trains from the data recorded in a BWIM. The algorithm identifies what parts of the signals contain train passages. This is simpler for railway bridges, where each train passage happens with seconds and sometimes even minutes of unloaded signal between passages, than for highway bridges where the traffic cannot be expected to behave so tidily. The algorithm uses the max-to-min difference over a short period of time to identify the sections of the time signal that correspond to passing trains.

An example of 10 minutes of measurement and the identified trains is shown in Figure 5-9. Notice how the track on which the trains are crossing is clearly visible from the strain levels they induce. The trains crossing the instrumented track are boxed in black, while the trains crossing the other track are boxed in grey. As can be noted (see Figure 5-9 at approx. 6 minutes), two trains on different tracks cross the instrumented section at the same time making their axle-load estimation impossible.

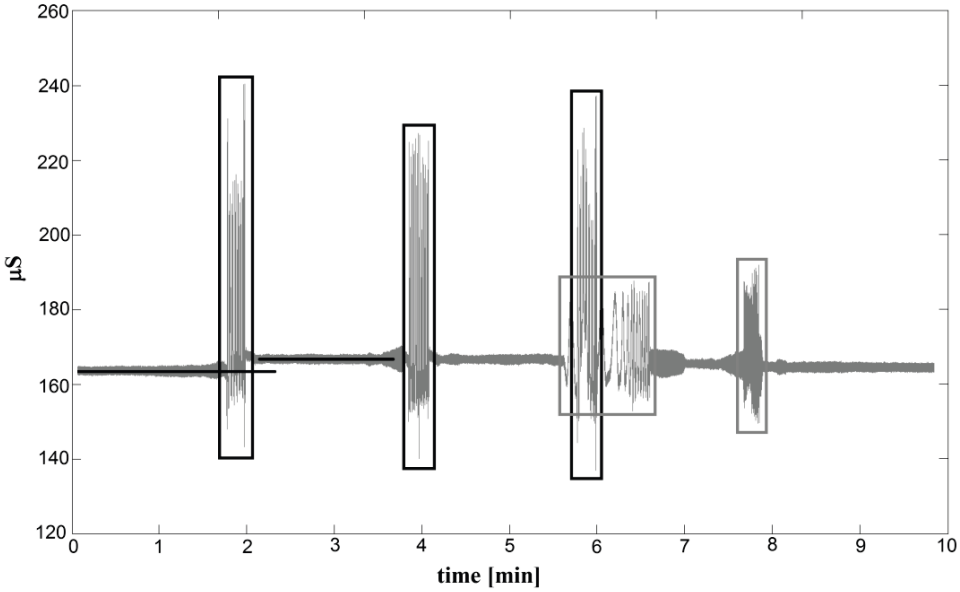


Figure 5-9 Typical 10 min of strain measurement (southern strain gauge). Five detected trains passages are highlighted, as well as the change in the zero-level due to accumulated stress release

For each 10 minutes sampling file, the developed algorithm returns: the effects of temperature and the dead load for each strain gauge and the number of trains and the track they run on. Further, for each train passing on the instrumented track it returns the speed and acceleration of the train, the peak and RMS value of the vertical bridge deck acceleration during the passage, two first estimates (one for each instrumented section) of the axle spacing and axle load that can be refined *a posteriori* using Influence Lines, plus the high frequency content and dynamic amplification factor for each bogie.

5.2.3.2.- Calculation of maximum live-load effects

In many cases, the assessment of bridge at 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. In the case of highway bridges, several initiatives are developed in this sense.

The AASHTO LRFR classifies the bridges based on the truck volumes and assigns different live load factors depending on the Average Daily Truck Traffic (ADTT). Similarly, for checking the safety of the bridge for the crossing of overweight permit trucks, the AASHTO LRFR assigns different live load factors depending on the ADTT of the bridge and the expected number of crossings and the weights of the permit trucks. More interestingly, the AASHTO LRFR allows for the adjustment of the live load factors based on truck weight information collected from WIM systems. A step-by-step procedure explaining how the adjustment process can be executed is provided based on simplified assumptions about the characteristics of the truck weight spectrum. The AASHTO code also allows the reduction of the dynamic amplification factor based on the roughness of the bridge riding surface.

In CAN/CSA-S6-06 different traffic load intensities are used for different road categories. Special load models and different safety factors are used for the assessment of bridges on the corridors, where passage of heavy vehicles is permitted. Furthermore, different categories of permit vehicles are considered, depending principally on the frequency and the condition of the vehicle passage through the corridor. However, the load models are not directly related to the traffic volume and the traffic composition specific for the particular location of the bridge.

Also, (Danish Road Directorate, 2004) provides background information for the development of fully probabilistic load models for different road categories and expected intensity of traffic. Different models are proposed for ordinary traffic and heavy permit vehicle passage. Furthermore, guidance is provided for modelling the traffic loads on various bridge spans, considering the effects of single heavy axles, groups of axles and the multiple-presence of heavy vehicles. Although very versatile, the traffic load models proposed in (Danish Road Directorate, 2004) are difficult to use in practical applications and require advanced knowledge of probability and statistical methods.

As with AASHTO, the UK manual (Highways Agency, 2006) assigns different reduction factors for standard traffic load model to account for ADTT and the surface roughness of the bridges.

A typical output of a WIM system is presented in Figure 5-10, showing a typical histogram of axle load obtained using BWIM in a railway bridge in Sweden.

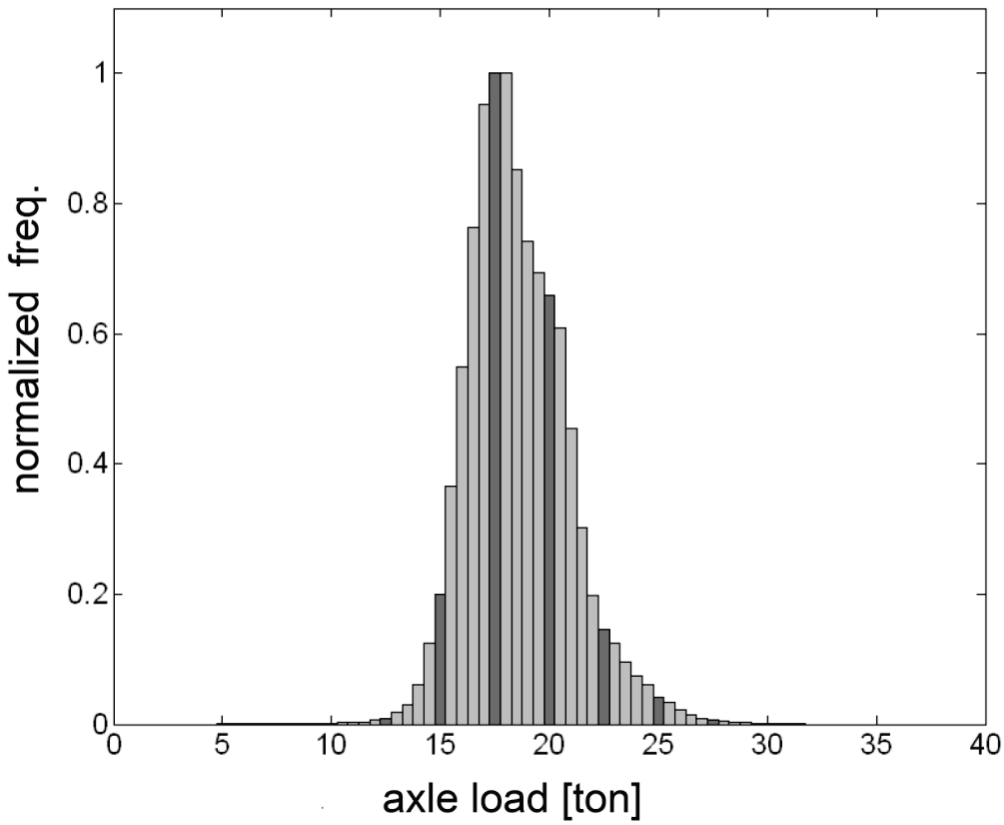


Figure 5-10 Histogram of the axle loads, obtained from over 7500 train passages (Gonzalez 2011)

Based on the site-specific train loads obtained using WIM techniques as in Figure 5-10, a simple but enough accurate method is presented next to derive such maximum effects. The method is based on a methodology developed in Sivakumar et al (2011), Ghosn et al. (2008), for the case of highway bridges but that could be adapted to the case of railway bridges. In the case of railway infrastructure, because the probability of meeting trains in bridges with more than 1 track is very low, when the volume of train runs is high, the one-truck load governs. Therefore, the direct application of the simple method will be as follows:

1. - Obtain the maximum traffic effect (moment or shear) in the bridge under assessment produced by the pass of the train configurations obtained with the WIM system. Using the

WIM data files, the effect of each train in the WIM record is calculated by passing the trains through the proper influence line

2. - Assemble the histogram for train load effect

3. - Fit a Normal distribution to the right tail (values higher than 95 %) of the histogram. In order to be independent of the bin used to build the histogram, it is recommended to perform the fitting in the Cumulative Distribution Function. Obtain the mean and standard deviation of this Normal distribution

4. - Calculate the Cumulative Probability Function (CPF) for the maximum load effect over a defined period of time. To this end, the number of events (N), i.e., the number of trains crossing the bridge during this time period, has to be calculated. If the loading events are independent (which is the case as axle configurations and load schemes of one train do not depend on the configuration of another train) but they are drawn from the same probability distribution, $F_S(S)$, (the Normal distribution obtained in the previous step), then the CPF of the maximum load effects takes the form:

$$F_{s_{max}N}(S) = \left[F_s\ (S)\right]^N$$
(5.10)

The hypothesis of independence between events can be checked using also the WIM records.

If the parent distribution of the initial variable S has a Normal distribution with mean μ_x and standard deviation σ_x, then the maximum value of S after K repetitions approaches asymptotically and Extreme Value Type I (Gumbel) distribution with the following parameters:

Dispersion parameter

$$\alpha_K = \frac{\sqrt{2\ln(K)}}{\sigma_X}$$
(5.11)

and a most probable value u_K given by:

$$u_K = \mu_x + \sigma_x\left(\sqrt{2\ln(K)} - \frac{\log(\log(K)) + \log(4\pi)}{2\sqrt{2\log(K)}}\right)$$
(5.12)

The cumulative Gumbel distribution has the form:

$$F_{s\max,k}(S) = e^{-e^{-\alpha_k(S-u_k)}}$$
(5.13)

where $F_{smax\ k}$ (S) is the cumulative distribution of the maximum of k events, u_k is the most probable value of $S_{max,\ k}$ and α_k is an inverse measure of the dispersion in $s_{max,\ k}$. The corresponding probability density function of the Gumbel distribution is then:

$$f_{s\max,k}(S) = \alpha_k e^{-\alpha_k (S-u_k)} e^{-e^{-\alpha_k (S-u_k)}}$$

(5.14)

The mean value for $s_{max\ k}$ can be calculated as:

$$\mu_{sk} = u_k + \frac{\gamma}{\alpha_k}$$

(5.15)

in which γ is the Euler number γ=0.577216. The standard deviation, $\sigma_{s,k}$ of $s_{max\ k}$ can be calculated from:

$$\sigma_{S,k}^2 = \frac{\pi^2}{6\alpha_k^2}$$

(5.16)

In the deliverable from SMARTRAIL Project on “Development of a General Rail Transport Infrastructure Safety Framework” (SMARTRAIL 2013) other topics related to railway loading and alternative methods to use of WIM data to define maximum loading scenarios are presented.

5.2.3.3.- Bridge-specific 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 measuring data, the load effect, or effects, on a particular bridge, or a range of bridges, can be estimated with confidence. Once the worst static case is known, the final traffic load is commonly calculated through the application of a Dynamic Amplification Factor (DAF) that accounts for the dynamic component contained in the bridge response. The dynamic amplification factors prescribed in design/assessment codes are sometimes based on dynamic load tests of existing bridges and tend to be conservative.

The dynamic amplification factor for railway bridges results from a complex interaction between the properties of the bridge, vehicle and track. The technical report (James, 2003) provides an interesting summary of the key parameters affecting the dynamics of a railway bridge. These are split into four categories:

- Train characteristics,
- Structure characteristics,
- Track irregularities, and
- Others.

The train characteristics that affect the dynamics are:

- Variation in the magnitude of the axle loads,
- Axle-spacings,
- Spacing of regularly occurring loads,

- The number of regularly occurring loads, and
- Train speed.

It is also noted that the characteristics of the vehicle suspension and sprung and unsprung masses play a lesser role.

The structure characteristics are listed in the report as:

- Span or the influence length (for simply supported beams these are equivalent),
- Natural frequency (which is itself a function of span, stiffness, mass and support conditions),
- Damping, and
- Mass per meter of the bridge.

The effects from the track irregularities are governed by the following:

- The profile of the irregularity (shape and size),
- The presence of regularly spaced defects, e.g. poorly compacted ballast under several sleepers or alternatively the presence of regularly spaced stiff components such as cross-beams, and
- The size of the un-sprung axle masses, an increase in un-sprung mass causing an increased effective axle force.

Under the category others, out-of-round wheels and suspension defects are included.

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 vehicle dynamic characteristics. Thus, the Dynamic Amplification Factor (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.

The stresses experienced by members and connections in short-span steel bridges can be considerably affected by dynamic amplification. As it is shown in Imam et al. (2005), using DAF obtained from different structural codes and applied to the same static data results in complete different results concerning cumulative damage as well as remaining fatigue life. In case of connections, the differences can range from 7 to 20 years depending on the DAF used. Therefore, a correct estimate of the actual DAF in the bridge under investigation is of major interest for an accurate assessment.

In a reliability-based assessment of existing bridges it is also important to deal with appropriate statistical distributions for DAF. In this way, the DAF can be considered as a random variable too, which may derive in higher accuracy in the final results. Recent studies have shown that employing a deterministic DAF in a probabilistic assessment is overly conservative, particularly with respect to displacement and less conservative for internal forces. Concerning the most suitable statistical definition of DAF, it has been obtained from experimental results that a Log-normal distribution provides the best fit for both displacement and bending moment. The mean value varies with speed (from 1.0 for speeds lower than 150 km/h to 1.25 for speed of 250 km/h) but is independent of train loading and structural response. The coefficient of variation is independent of both train speed and train loading and the values range from 20 to 25 %.

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 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.

In González (2011) appendix C, an algorithm is presented for extracting the Dynamic Amplification Factor (DAF) and the High Frequency Content (HFC) from a measured dynamic signal. Example of results is shown in Figure 5-11. A filtered version of the signal is calculated to enable axle detection. The approximation, obtained by polynomial fitting, is then used to calculate both the DAF and the HFC. The DAF is here calculated as the ratio between the maxima of the unfiltered (dynamic) and filtered (static) signal. The HFC is evaluated as the Root Mean Square of the difference between the original signal and the filtered one. This quantity was considered useful since it could be used to identify wheel-flats or other abnormalities in a given axle or wheel. In fact, a wheel defect would result in an excessive vibration during one of the axle's passage through both instrumented sections.

In Figure 5-11 it is noted that some trains show unusually high HFC in only one of their axles throughout the different instrumented sections, indicating a possible wheel-flat.

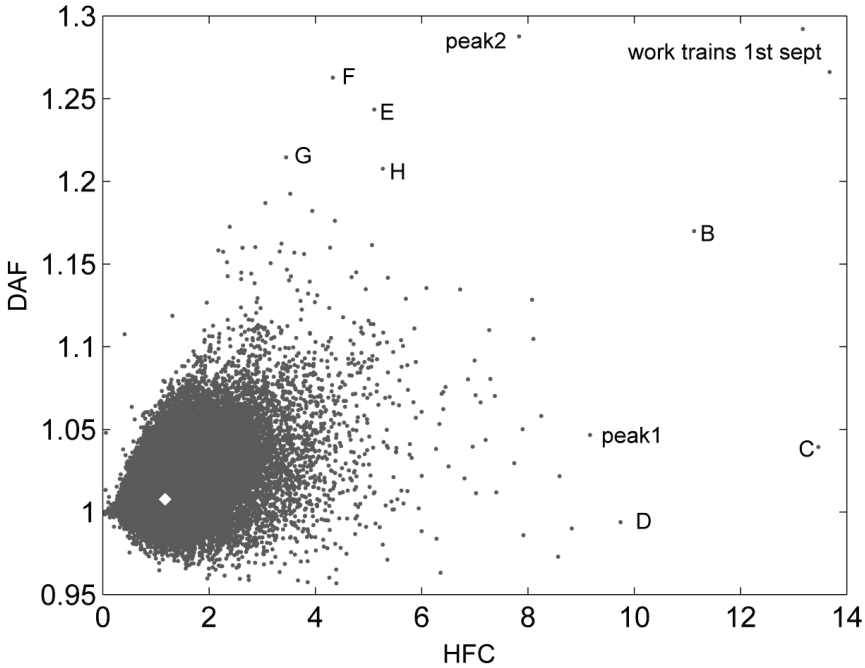


Figure 5-11 DAF vs. HFC for all the bogies detected. The most extreme cases were labelled and studied separately. The white diamond corresponds to the heaviest bogie registered. (Gonzalez 2011)

Also in Figure 5-11 it is shown how the heaviest bogie registered presents a DAF close to 1. This trend of decreasing DAF as axle or bogie load increases has been also observed in highway bridges (ARCHES-D10 (2009)) and is of special interest in the assessment of existing bridges using site-specific data.

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).

According to Brühwiler and Herwig (2008) some authors showed by means of simple dynamic models how the external work due to dynamic action effects (i.e. impact-like events, excitation by road and track irregularities) is dissipated in the structural element before the element fully fails (fractures). In addition, these studies show that:

- Bridge elements will most probably fail in bending after significant plastic deformation if subjected to excessive dynamic traffic action. More brittle failure mechanisms like predominant shear failures are unlikely to occur.

- The most unfavourable scenario for bridge elements is the impact-like excitation of passing vehicles by singular irregularities.

- Marked strain hardening in the structural response increases significantly the dissipation potential.

- Resonance oscillation energy may also be dissipated by plastic deformations of the structural element.

The work of Tobias et al. (1996) studies the fatigue evaluation of riveted steel railway bridges. The thesis presents Monte-Carlo traffic models for evaluating the load effects in riveted railway bridges for North-American conditions. It uses results of weigh-in- motion studies, locomotive and wagon types and train frequencies and configurations. In the work, the dynamic effect is found to closely follow a log-normal distribution. In Figure 5-12, the fitted distributions are shown for the 9.1–18.3m ballasted decks for different speed ranges.

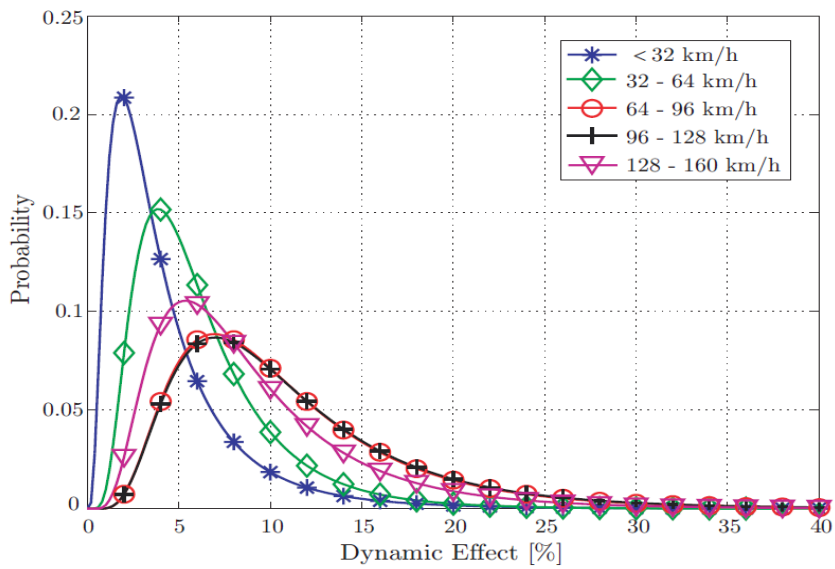


Figure 5-12 Probability vs. dynamic effect for fitted log-normal distribution, for ballasted decks with spans between 9.1 and 18.3 meters. (Tobias, et al., 1996)

5.2.3.4.- Other important site-specific parameters

As previously mentioned, considering site-specific parameters and conditions can be decisive when assessing an existing bridge. There are many other parameters, beside live loads and dynamic amplifications factors, that can be of importance, such as:

- **Load distribution**. Design codes contain often very simplified models for the load distribution. A more sophisticated load distribution taking into account the contribution of the track can increase the load capacity and decrease the deck deflections. For more details on load distribution by the rails, sleepers and ballast, see chapter 3.3 in the report SB4.3.1 (2005) and the report ERRI D214/RP9 (1999).
- **Vertical track stiffness**. Instead of using a nominal distribution of the axle loads, track stiffness can be measured and considered in the FE-model by modelling the track as a beam on spring elements. This is especially important for the assessment of the bridge dynamic performance. There are several methods available for measuring vertical track stiffness, for more information see the UIC document *Guidelines for Railway Bridge Dynamic Measurements and Calculations*, (UIC, 2006).
- **Bridge influence lines**. - Influence lines are frequently used to calculate responses to various load patterns. These can be derived from direct measurements of the load effect in response to a vehicle of known weight. In O'Brien et al (2006) a method is described which enables the evaluation of the bridges actual influence lines from measurements of passing vehicles.
- **Dynamic properties**. It is especially important for the assessment of the bridge dynamic performance to measure actual damping, frequencies and mode shapes. More details are found in UIC (2006).

5.2.4 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 in an advanced assessment (SIA 269, DB 805 ff). CIRIA (2008) provides guidance on the best modelling of trusses and lattice girders and other metallic bridges to take into account correct buckling analysis and behaviour of connections (riveted, bolted or welded) in existing bridges. The design rules assume that the rivet strength can be simply derived from the yield strength of the parent material. However, in reality the rivets are stronger than the parent metal. ECCS 2008 provides a practical example of assessment giving recommendations that includes old materials data/testing and measurements about reassessment of a truss girder bridge.

In the case of metallic bridges, using non-linear modelling to take advantage of possible redistribution should also be considered carefully, because in many cases for a correct result it is necessary to have a non-linear model that cover all the possible buckling modes that may affect redistribution properties.

Model updating can be also carried out via diagnostic load testing. This type of test provides useful information when structural models including grillage or 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), are presented the results of 3 diagnostic load tests in different bridge structures, 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 CAN/CSA-S6-06 provides simple formulas to calculate the nominal strength of concrete and steel based on test results on cores and coupons retrieved from the structure. The nominal strength is obtained as a function of the number of samples and the test results statistics, namely the mean value and the coefficient of variation (COV). The application of this method is required when the drawings and/or data about material strength are not available and the strength of the materials cannot be defined from information on the year of construction and the associated historical material strength data provided in the code.

Danish Code (Danish Road Directorate, 2004) also provides a method to determine the materials strength based on test results for the cases when obtained information about historical material strength is insufficient or cannot be used.

The UK assessment codes also provide a method to determine the materials strength based on test results for the cases when provided information about historical material strength is insufficient or cannot be used.

The importance of updating information on the bridge characteristics from the results of inspection and monitoring has been identified and is nowadays acknowledged. In 2010, a scan team from the United States visited several agencies in Europe and identified many practices and technologies related to the topics of interest. Between them, there is (FHWA-AAHSTO, 2010):

1. - Continue efforts to develop guidelines and training for proper use of non-destructive techniques to detect corrosion and breakage of cables of cable-supported bridges and internal and external tendons of post-tensioned bridges.

5.2.4.1 Structural Health Monitoring

According to Chang (1999), 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. Unlike traditional non-destructive evaluation methods, structural health monitoring techniques use the change in measurements at the same location at two different times to identify the condition of the structure.

According to (Sikorsky, 1999), Structural Health Monitoring (SHM) systems can be classified both in terms of their level of sophistication and by the types of information (and decision making algorithms) which they are capable of providing. These classifications are particularly instructive in understanding the goals of SHM and some of the concepts that are discussed later in this report. The classifications of SHM systems can be summarized as follows:

- **Level I:** At this level, SHM system is capable of detecting damage in a structure, but cannot provide any information on the nature, location, or severity of the damage. It cannot assess the safety of the structure.

- **Level II:** Slightly more sophisticated than Level I. Level II systems can detect the presence of damage and can also provide information on its location.

- **Level III:** A Level III SHM system can detect and pinpoint damage, and quantify the damage to indicate the extent of its severity.

- **Level IV:** This is the most sophisticated SHM system. At this level, the system is capable of providing detailed information on the presence, location, and severity of damage. It is able to use this information to evaluate the safety of the structural system.

Obviously, as the level goes higher, more information could be obtained from the SHM system although the system is becoming more complicated and costly.

Structural health monitoring refers to subjecting the structure to static or dynamic excitations, continuous or periodic monitoring of the structure's response using sensors that are either embedded in or attached to the structure.

New advances in sensor and information technologies and the wide use of Internet is making SHM a promising technology for better management of civil infrastructures. There have been many case studies worldwide in the past decade. While the specific details of each SHM system can vary substantially, SHM basically involves sensor and data acquisition, data transfer and communication, data analysis and interpretation, and data management. Thus a SHM system will typically consist of six common components, namely:

- Sensors and data acquisition networks;
- Communication of data;
- Data processing;
- Storage of processed data;
- Diagnostic and prognostic analysis (i.e. damage detection and modelling algorithms, event identification and interpretation); and
- Retrieval of information as required.

Farrar and Worden (2007) distinguish between (1) SHM as a means of determining the ability of a structure to perform under the aging and damage accumulation that occurs from long-term use in its operational environment and (2) SHM as a tool for rapid screening to provide near real-time information about the performance of a structure during extreme events. Wenzel (2009) has published a comprehensive study on bridge health monitoring that includes chapters on the hardware used in health monitoring, methodologies, applications of these methodologies (materials, methods, systems and functions), decision support systems, damage detection systems and the rating of bridges and methods of risk assessment.

But the systems and methods (sensors, technologies,…) are just a tool for 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 translate into reliable answers on the bridge actual condition and performance in the next or remote 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. 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 known as cluster analysis. They are able to find groups in data relying only in its intrinsic features (Sohn & Kin, 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 (Catbas, et al., 2008; Frangopol & Okasha, 2009; Liu, et al., 2009). However, these studies have mainly focused on information related to the load effects that SHM provides. In fact, based on the long-term monitored strain data induced by heavy vehicle traffic on an existing bridge, Liu et al. present an approach and a practical example to assessing the bridge system performance through a series-parallel system model consisting of bridge component reliabilities. The study provides a solid basis for integrating SHM data into practical assessment of bridge system performance. Furthermore, these studies have primarily used classical inference concepts in which prior information cannot be easily incorporated and, in fact, is neglected.

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 (Frangopol & Okasha, 2009). Figure 5-13 shows a flowchart of the integrated life-cycle management framework with the main features of the framework and the Figure 5-14 shows the framework with extended details. Uncertainty is an integral component in all aspects of this (or any) life-cycle management framework (Okasha & Frangopol, 2012). Therefore, each of the steps specified in this approach is to be conducted with careful treatment of the associated uncertainties.

As shown in Figure 5-13, for a given bridge structure and following this framework, 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. However, 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. Frangopol and Okasha (2009) have used the finite element updating to update the parameters of the structure that are in turn used in updating the lifetime reliability of the structure. Recently, Brühwiler (2013) has shown how the monitored data in a riveted steel railway bridge in Switzerland allowed for accurate determination of fatigue relevant stresses and updated action effects, concluding that the bridge can extend its life for more than 50 years, even after 115 years of service. The standard fatigue assessment without the monitoring data will conclude that the bridge could not follow in normal operation because of the fatigue of one of its main members.

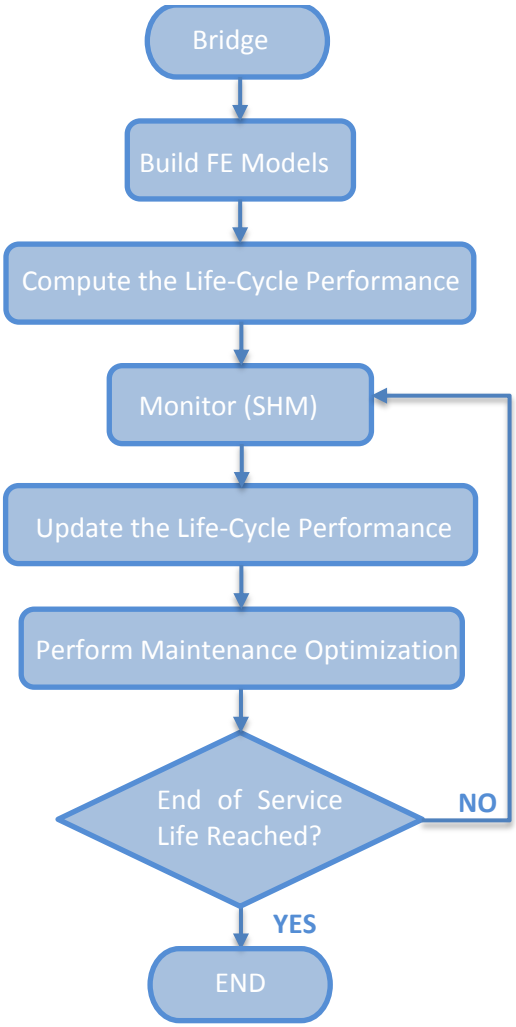


Figure 5-13 Flowchart of framework for bridge management (Okasha & Frangopol, 2012)

Indicators that can be used in representing the behaviour of the structure under uncertainty are numerous. A major factor that is present when deciding the choice of a performance indicator is its compatibility with the framework used. For instance, it is important that the performance indicator provides the ability to be accurately updated by SHM data. Also, depending on the maintenance optimisation method, the performance indicator should be computationally affordable. Performance indicators for both the components of the structure and its overall system are necessary for the optimisation of maintenance. It is necessary to know the effect of the maintenance actions to a component on its performance. One of the possible indicators is the life-cycle performance in the form of lifetime functions.

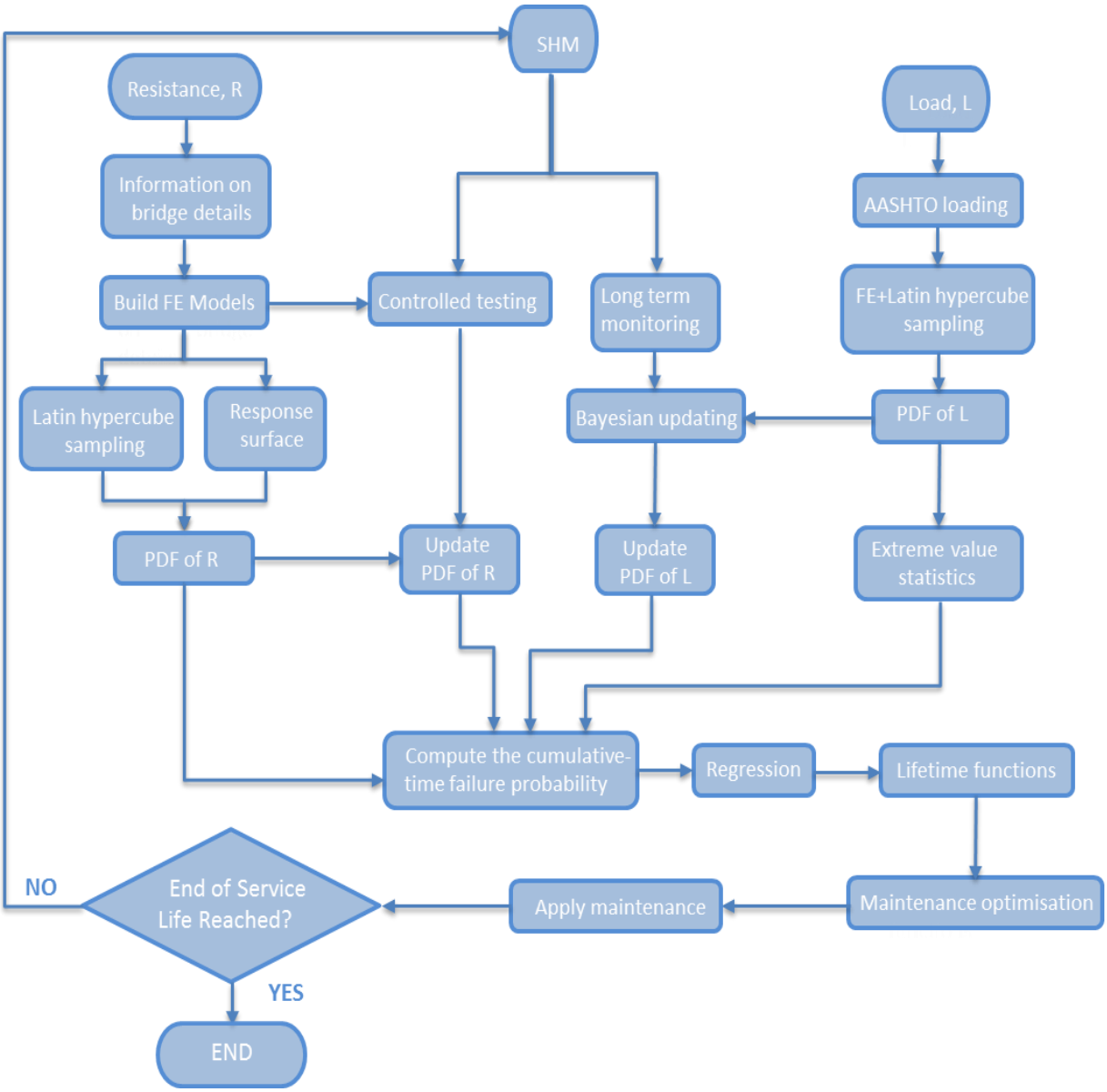


Figure 5-14 Flowchart of framework for bridge management with extended details (Okasha & Frangopol, 2012)

Finally, structural resistance and structural response are updated with SHM. As mentioned above, monitoring of structural health can be beneficial for bridge management in various aspects, and many response quantities can be measured by a large number of devices. Cost and accuracy is a trade-off in the choice of the SHM system and monitored quantities. This is further discussed in chapter 7.

5.2.4.2 Bayesian updating

The performance assessment based only on the SHM data, and disregarding the original information, can be restrictive and provide biased results. The Bayesian approach makes it possible to use the additional monitoring information in conjunction with the already available information. 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 does not make it 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 use 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.

Due to the cost of non-destructive or destructive tests, these are only performed after all available data was collected and analysed. As a consequence, before performing such tests or carry out SHM, the bridge engineer is already capable of estimating the most significant parameters necessary for the analysis of the structure. This estimative can have different levels of confidence, depending on the data available, experience with similar structures, and previous analysis of the structure. This initial knowledge will be denoted as *prior*, since it is acquired before any test is carried out.

Due to the nature of this information, it is usually associated with large uncertainty. The main objective of performing destructive or non-destructive tests or implementing a SHM in the structure is to reduce this uncertainty, and adjust the values obtained.

Let's define the property under analysis (e.g., concrete compressive strength) by θ. Using existing codes or previous experience, it is possible to define a probability density function for this variable as:

$$\theta \sim f_\theta(\theta) \tag{5.17}$$

If no information exists on the property, what is called a non-informative *prior* can be used. This is simply a function that is constant for all possible values of the property.

Let's consider a case where the results of a test are described by a variable *x*. Knowing the nature of the test, we can calculate the probability of the observation being made, for each value of the parameter. For example, if cores from several concrete elements using a concrete of class C30 are tested, it is possible to measure the probability of each result of the core compressive strength. If this work is repeated for several cores, the distribution for values of the unknown parameter θ can be found. It must be noted that this information can be found in the literature for the most common tests.

The posterior distribution of the unknown variable θ can be found through expression:

$$f_\theta^{'}(\theta) = \frac{f_\theta(\theta) \times \prod_{i=1}^{n} f_{x|\theta}(x_i \mid \theta)}{\int_{-\infty}^{+\infty} f_\theta(\theta) \times \prod_{i=1}^{n} f_{x|\theta}(x_i \mid \theta)} \tag{5.18}$$

The denominator of the above expression serves only as a normalization parameter, so that the area below the function is 1. It is, therefore, not important at this stage. As a result, the expression (5.18) can be replaced by:

$$f_{\theta}^{'}(\theta) \propto f_{\theta}(\theta) \times \prod_{i=1}^{n} f_{x|\theta}(x_i \mid \theta) \tag{5.19}$$

The first term of expression (5.19) refers to the *prior* distribution probability. The second term describes the likelihood of a certain value of *x* being output from the test, for a certain value of the parameter θ. For example, $f_{x/\theta}(x/\theta)$ represents the probability of a concrete with compressive strength θ yielding a core resistance *x*, when tested. Direct use of this equation can be made for relatively simple cases. For more complex situations simulation can be employed. In fact, in most of the cases the closed-form solution for equation 5.18 is not achievable due to the difficulty in the computation of the denominator. Therefore, numerical based techniques such as Markov chain Monte Carlo (Coles and Powell 1996), Metropolis-Hastings algorithm (Robert 2007) and slice sampling algorithm (Neal 2003) may be used.

An example of typical results obtained using this methodology is presented in Figure 5-15. The prior distribution represents knowledge before carrying out the test. Considering that it is based on limited information, it is associated with significant dispersion. The likelihood distribution is associated with a smaller dispersion, meaning that it refers to a test with high accuracy.

The posterior distribution represented shows a dispersion between the two previous curves, meaning that execution of the test implies a significant reduction in the dispersion of the parameter (e.g., concrete compressive strength) under analysis. Moreover, since the test yielded a result different from the mean of the prior distribution, there is a small shift in the mean of the parameter.

Let's consider the most common case in which a structural parameter with Gaussian distribution is investigated. Characteristic values of this parameter, using the results of tests, can be found using:

$$x_k = m - t_{vd} \cdot s\sqrt{\left(1 + \frac{1}{n}\right)} \tag{5.20}$$

where *n* is the number of samples or tests, *m* is the mean value obtained from tests and prior knowledge, *s* is the standard deviation obtained from tests and prior knowledge and t_{vd} is the coefficient of the Student distribution dependent on the sample size, prior standard deviation, and probability of occurrence.

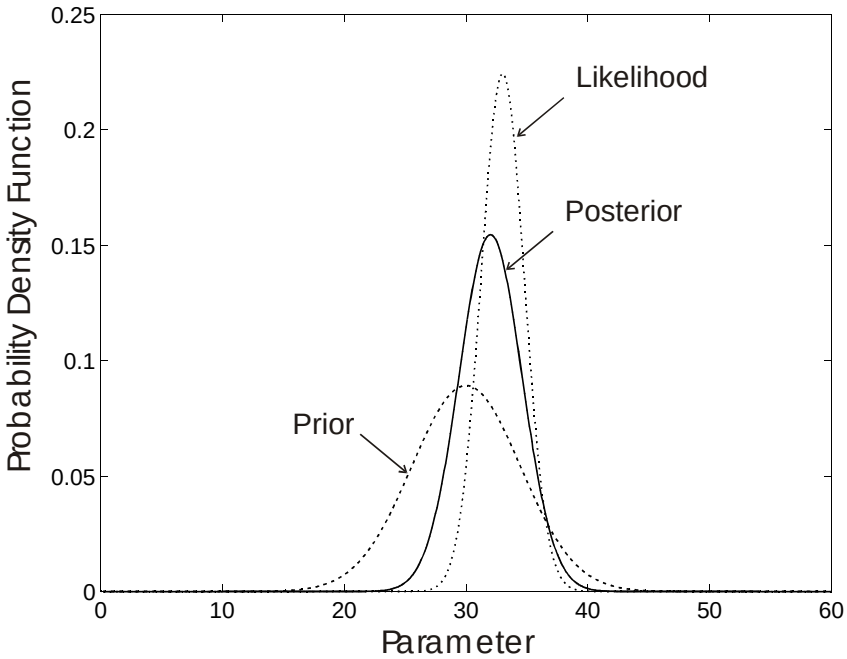


Figure 5-15 Comparison of Prior, likelihood and posterior distributions

If the standard deviation is known from past experience *s* should be replaced by the known standard deviation.

If no prior information exists on the problem, the standard deviation and the mean are given by the sample mean and standard deviation as:

$$m = \frac{\sum x_i}{n} \tag{5.21}$$

$$s = \sqrt{\frac{1}{n}\sum (x_i - m)^2} \tag{5.22}$$

When prior information exists, the updated mean and standard deviation must conjugate the two sources of information.

Conjugate and non-conjugate priors

A prior that is conjugate of the underlying random variable is a prior distribution when, combined with the likelihood function, a posterior function in the same mathematical form of the prior is obtained. For example, if the underlying random variable X is Gaussian (with unknown mean μ and known standard deviation σ), and the prior of its parameter μ is also Gaussian (with hyper-parameters μ' and σ'), the posterior of μ is obtained as Gaussian with hyper parameters:

$$\mu'' = \frac{\bar{x}(\sigma')^2 + \mu'(\sigma^2/n)}{(\sigma')^2 + (\sigma^2/n)}$$

$$\sigma'' = \sqrt{\frac{(\sigma')^2(\sigma^2/n)}{(\sigma')^2 + (\sigma^2/n)}}$$

(5.23)

Where $\bar{x}$ is the sample mean and n is the number of samples.

Conjugate prior distributions can be constructed if the data set of an arbitrary size can be completely characterised by a fixed number of summaries with respect to the likelihood function, i.e. it depends on the existence of sufficient statistics of fixed dimension for the given likelihood function. For example, the sufficient statistics in the case of equation 5.23 are $\bar{x}$ and n. Hence, all the information that is needed from the data set in order to perform Bayesian updating is contained in the number of points in that data set and its mean.

The basic justification for the use of conjugate prior distributions is that it is easy to understand the results, which can often be put in analytic form, and they simplify computations (Gelman, et al., 2003). Unfortunately, it is mostly the case currently that conjugate priors are not used and thus special techniques are used for Bayesian computations. Except in simple cases, explicit evaluation of the integrals in Bayesian analysis is rarely possible. In fact, the recent popularity of the Bayesian approach to statistical applications is mainly due to advances in statistical computing (Ghosh, et al., 2006).

The treatment of SHM data is usually associated with monitoring of extreme events (i.e. load effects of very heavy trucks). 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 to 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 (Beck & Au, 2002).

Metropolis-Hastings algorithm and Slice sampling algorithm

The Metropolis-Hastings algorithm is a simple procedure to simulate samples according to an arbitrary PDF where the target PDF need only be known up to a scaling constant. The basic idea of the Metropolis-Hastings (MH) algorithm, whose development led to very considerable progress in Bayesian analysis, is not to directly simulate from the target density (which may be computationally very difficult) at all, but to simulate an easy Markov chain that has this density as its stationary distribution (Ghosh, et al., 2006; Bocchini, et al., 2013). Instead of finding the closed-form solution of a posterior PDF, which may not be available, samples from this PDF are generated. The power of Markov chain Monte Carlo (McMC) methods, particularly the MH algorithm is their ability to generate the samples without resorting to computing the integration required in the normalising factor (see equation 5.18). In the MH method, samples are simulated as the states of a special Markov chain whose limiting stationary distribution is equal to the target PDF. In other words, the PDF of the Markov chain sample simulated at the jth Markov step tends to the target PDF as j tends to

infinite. The Markov chain samples, which are dependent in general, can be used for statistical averaging as if they were independent, although with some reductions of efficiency in the estimator. Once the samples are generated, the PDF of the posterior can be approximated. The idea is to iteratively generate samples of a Markov chain which asymptotically behaves as the probability density function (PDF) that has to be sampled. The MH algorithm (Hastings 1970, Nicholas et al. 1953) is commonly used for generating such Markov chains.

As in any McMC method, the draws are regarded as a sample from the target density only after the chain has passed the transient stage and the effect of the fixed starting value has become so small that it can be ignored.

To produce quality samples efficiently with MH algorithm, it is crucial to select a good proposal distribution. If it is difficult to find an efficient proposal distribution, the slice sampling algorithm without explicitly specifying a proposal distribution can be used. Slice sampling originates with the observation that to sample from a univariate distribution, points can be sampled uniformly from the region under the curve of its PDF and then only the horizontal coordinates of the sample points are looked at. The following is a brief description of the algorithm based on (MathWorks, 2009):

(a) Assume an initial value x(t) within the domain of f(x).

(b) Draw a real value y uniformly from (0, f(x(t))), thereby defining a horizontal ‘slice’ as S = {x: y < f(x)}.

(c) Find an interval I = (L, R) around x(t) that contains all, or much of the ‘slice’ S.

(d) Draw the new point x (t+1) within this interval.

(e) Increment t to t+1 and repeat steps b through d until the desired number of samples are obtained.

Both methods, the MH and slice sampling algorithms, provide good approximate solutions to the closed-form and for this reason, Okasha and Frangopol (2012) recommend the use of the slice sampling algorithm for the main computation and the MH algorithm (with a normal proposal distribution whose parameters are chosen based on the problem to solve) for checking some of the slice sampler results.

Updating with large SHM samples

In long-term SHM, very large amounts of data are usually generated. The treatment of data sets of this size in Bayesian analysis requires special considerations. First, in the absence of conjugate priors, which is mostly the case in SHM, equation (5.18) has to be computed directly. Construction of the likelihood function requires multiplying all the $f_X(x_i \mid \theta)$ functions at all SHM data points. When hundreds of SHM data points are at hand, construction of such equation is impractical. Alternatively, the likelihood function at a given parameter value can be computed sequentially in algorithmic form as follows:

- Initiate L(θ) = 1

- Repeat for i = 1, ..., N, where N is the number of SHM points.

- L(θ) = L(θ) $f_X(x_i \mid \theta)$

Second, $f_X(x_i \mid \theta)$ is a PDF whose value is most likely less than 1.0. In this case, as N increases, L(θ) is decreasing, creating numerical difficulties in the Bayesian computations. Clearly, a large SHM data set associated with extreme events cannot be accommodated at once in Bayesian analysis. Luckily, a key aspect of Bayesian analysis is the ease with which sequential analysis can be performed (Gelman, et al., 2003). In sequential Bayesian analysis, each posterior acts as the prior in the subsequent analysis. Thus, the whole SHM set can be divided into sets and the analysis is conducted sequentially. In fact, this sequential approach also allows the updating in real-time with the SHM data as they are obtained, where the analysis is conducted with one SHM data point at a time, or in near real-time, where a set of points obtained at regular/irregular-time intervals are used.

Updating of uncertain relations

In civil engineering, it is quite common that a measured parameter depends of a large number of uncertain basic variables. For example, the deflection at mid-span of a bridge can be measured with certain accuracy, although it depends on a large set of parameters including the load applied, the material properties, the geometry of the structure and the resistance of critical cross sections.

The probability of failure can be updated considering a measured property *I* as:

$$P(F \mid I) = \frac{P(F \cap I)}{P(I)} \tag{5.24}$$

where *F* represent the failure event, and *I* the observed event or the additional information available from inspection/monitoring.

The analysis of this type of problems can become extremely complex due to the possible existence of correlation between the different parameters. However, an analysis performed using Monte-Carlo simulation is relatively simple. In fact, let us consider a structural problem where the probability of failure is computed using simulation. This analysis will yield the probabilistic distribution of a set of results, including displacements and stresses, for different levels of loading. Some of these results can be measured in the real structure, although these measurements will always include some degree of error. Let denote the measured quantity by θ, then:

$$p_f = \frac{\sum I(g_i \le 0)\, p(x_i \mid \theta)}{\sum p(x_i \mid \theta)} \tag{5.25}$$

where $I(\cdot)$ is the identity function, and $p(x_i \mid \theta)$ is the probability of displacement associated with the sample x_i occurring.

However, a relatively simple analysis can be performed assuming an inspection is done and the result is an inequality. For example, the crack size is smaller than a certain threshold. Let's define a function *h*, given as:

$$h(x) = x - x_{measurable} \tag{5.26}$$

where x is the measured property, and $x_{measurable}$ is the smallest detectable value of x. The updated probability of failure can then be given as:

$$P(F \mid I) = \frac{P(F \cap h(x) < 0)}{P(h(x) < 0)} \tag{5.27}$$

As an example, Rafiq et al. (2004) have presented a methodology for performance updating of deteriorating concrete bridges fitted with a proactive health monitoring. The results show how sensor information can be accounted for by developing a Bayesian framework.

5.2.5 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 (Gómez, 2010).

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 (Ryall, 2001).

This test is used to verify component and system performance under a known external load and is normally aimed to provide a complementary assessment methodology to the theoretical assessment. The use of such tests, due to the risks of collapse or of damaging essential elements of the structure, must be restricted to bridges that have failed to pass the most advanced theoretical assessment and are therefore condemned to be posted, closed to traffic or demolished. It is also important that the bridge has a high level of redundancy to be a good candidate. Furthermore, some balance has to be found between the risk of failure under the test load and the benefit of an updated reliability of the bridge.

In some cases when the bridge is in poor condition due to lack of maintenance or because of an extreme loading event, such as flooding or impact, the actual resistance of the remaining bridge is difficult to assess without a detailed inspection, but it can still carry a high percentage of design load. Even in the case that this information is provided thanks to an inspection process, the theoretical models to obtain bridge resistance can hardly be adopted. In such cases, a proof load can also be of interest to define the allowable load in the bridge. Of course, to follow up the loading sequence and the bridge response accurately and not produce more damage in the bridge or even its failure, it is mandatory to use an accurate monitoring system. The use of this type of test may also be recommended in the case of bridges with high redundancy level and where an accurate theoretical model of behaviour or an accurate definition of geometry and material properties is not possible due to lack of information (no drawings). This can be, for instance, the case of old masonry arch bridges. If the failure could be sudden, without warning, proof testing should not be used.

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 placing a greater load than the expected overload in service. In Gómez (2010), Gómez and Casas (2010) and Casas and Gómez (2013), a complete description and guidance on the appropriate target proof load to

be used based in very simple parameters of the bridge as span-length and percentage of heavy traffic can be found. The method is based on the reliability-based calibration of a target proof load factor that multiplies the nominal traffic load described in the design code.

During the execution of a load test, the structure very often reveals reserves of resistance that had not shown in the structural analysis, especially in the case of concrete or masonry structures. In many cases, the cost derived from the 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. Therefore, load test on bridges are usually a great benefit to engineers, as they help the understanding of the behaviour of bridges in service and in the later stages of loading under the action of the service loads.

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.

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, the load test assays 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 poof 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 (ARCHES-D16, 2009). Indeed, more than 250 bridges were subjected to proof load test in Ontario, Canada, without showing any signs of damage. (Bakht & Jaeger, 1990).

5.2.5.1 Monitoring and instrumentation

The first step when planning the test is to visit the site and have a close look at the structure. The access to the structure should be checked together with the access to the parts of the structure where it is proposed to attach instrumentation. It may be necessary to consider providing temporary access to allow the instrumentation and loading equipment to be installed.

The next important issue is the position of the loads: The desired positions for the loads to be applied will have been determined as part of the feasibility study. For instance, it will be known whether only global or local loading or a combination of both is to be applied. It will

also be known where the loads are to be applied both along and across the structure. These requirements, together with the available possession time, will help to determine the method of loading.

An important part of the test planning and site inspection is to decide on the instrumentation to be used and its location.

The instrumentation may include:

- Applied load:

    - load cells

- Deflection:

    - inductive mechanic gauges,

    - dial gauges,

    - inclinometer (by calculating deflection curve on the base of measured angles in several sections),

    - laser technique

- Strain:

    - vibrating wire strain gauges,

    - resistance gauges,

    - optical fibres

- Temperature:

    - Thermocouples,

    - optical fibres

Monitoring should ensure that the bridge is not behaving beyond the elastic limit of the materials. The basic instrumentation consists of load cells, strain sensors, displacement transducers and inclinometers. Additional instrumentation may consist of vibration measuring equipment, thermometer and anemometer and equipment for acoustic emission (AE) analysis to observe the development of micro-cracks and material degradation in concrete and masonry bridges.

Prior to conducting a load test, the engineer must determine the goals of the test and the types and magnitude of the measurements to be made. Preliminary calculations or alternatively the execution of a soft or diagnostic load test may be needed to estimate the range of the measurements as well as the best locations for the instrumentation.

Strains

Strain data may be needed at several locations consistent with the needs of a proof load test. Strain sensors are usually attached on critical members to monitor response. Also, locations are selected so that the analytical model can be validated if such a model is available. This is done by placing sensors on several main load-carrying members and monitoring

simultaneously. Subsequently, the measured responses can be compared to the predicted values from the model. Finally, attachment details can be studied by placing strain sensors so as to obtain stress concentrations.

Direct measurement of strain is of vital importance as strain can be directly related to the limit of assumed elastic behaviour of materials, mainly for concrete and steel.

Data should be monitored in the field to ensure proper operation of equipment and to prevent damage to the structure. For a typical installation, data will be taken with each increment of loading as well as at every new position of load application.

Displacements

Displacement measurements are often an important part of the load testing program, especially for proof loading. These help to determine linear behaviour while the test loads are being incremented and also to determine whether the displacements are recoverable when the test loads are removed. Typically, only a few locations need to be monitored during the test. Vertical deflections are usually required only at mid-span of the structure.

The measurement of relative vertical displacements between the top and bottom flanges of a girder can establish the integrity of the section, particularly if extensive deterioration is present. In some cases, such as at bearings, the measurements of horizontal displacements may be helpful in determining whether a bearing is functioning as designed.

Rotations and Other Measurements

The measurements of end rotations can establish the extent of end restraint which exists at bearings. The elastic curve for a bending member can be developed by measuring rotations along the length of the member.

Depending on the test objectives, other data may be useful, such as temperature and wind speed. The position of the test vehicle, both transversely and longitudinally should be recorded. Other measurements may be needed such as for crack openings, slippage, and rigid body motion.

Acoustic Emission

The Acoustic Emission (AE) system can be used for long or short term monitoring (evaluation) of structures. It is usually used on the second type of usage in the context of bridge load testing and bridge evaluation.

To date, no standards have been set for field monitoring using acoustic emission techniques on structures like bridges, but a series of RILEM Recommendations is in preparation by the RILEM TC ACD(ARCHES-D08). Recent test results have shown that the calculated b-value may be used as an effective index to assess the damage in reinforced concrete elements. The b-value is based on the Gutenberg-Richter formula and is obtained from the frequency-amplitude plots of acoustic emission. 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 micros cracks coalesce and start forming major and visible cracks (Sagar et al. 2012). A complete description of the application of AE to an existing real bridge is presented in ARCHES-D16. 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.

One of the most important is that carried out by the German government through a cooperative research Project called EXTRA II (BELFA (2007), whose main objective is the

design, construction and commissioning of a special truck, fast and flexible, that allows the performance, in a rational way, of proof load tests in situ.

The successful experience of the vehicle for road bridges led to adopt this technology to apply it in railway bridges constructed in concrete and masonry. Thus, they have developed a prototype vehicle for load testing in railways bridges called BELFA-DB, which can be used in bridges with spans up to 15 m (Figure 5-16 and Figure 5-17). This includes 80% of the railway bridges of concrete and masonry in Germany.

Figure 5-16 Vehicle BELFA-DB over the Renha Bridge

Figure 5-17 Vehicle BELFA-DB over a masonry arch bridge

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 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 5.18). For a service-proven bridge, the magnitude of the maximum load carried is unknown, however, it can be determined statistically by using weigh-in-motion data as presented in Wang et al. 2011.

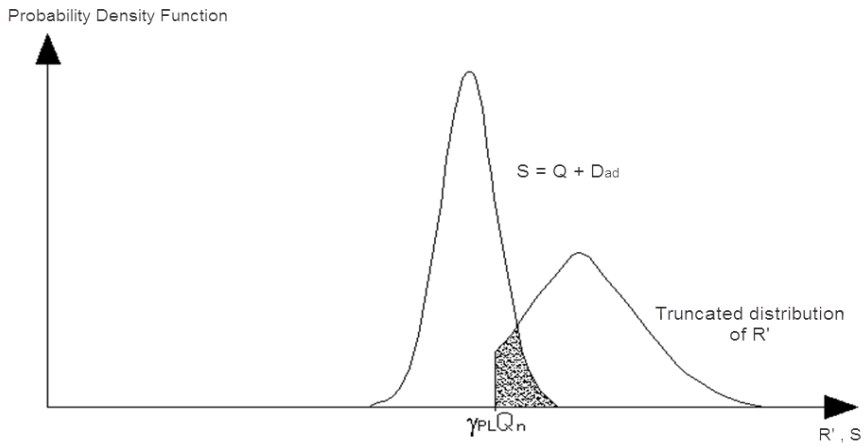


Figure 5-18 Truncation of PDF of Resistance as a result of proof load test

5.3 In tunnel engineering

The European tunnel guideline (DIRECTIVE 2004) gives information on minimum safety requirements for tunnels in the Trans-European Road Network.

For the assessment of wall thickness of tunnel linings a special technique developed in Germany makes use of an ultrasonic reflection technique for the determination of the wall thickness. Due to the inhomogeneous structure of the material, concrete is difficult to test for all non-destructive methods. If acoustic methods are applied to concrete, this inhomogeneity causes scattering at aggregates which complicate the interpretation of the measurement results. In order to minimise the disturbing influence of scattering the method performs measurements along scan paths that include the points defined by the German Guideline RI-ZFP-TU. By means of spatial over-sampling spatial averaging which improves the signal quality dramatically may be introduced.

5.3.1 Introduction to the Tunnel Condition Monitoring Index (TCMI) - Network Rail

TCMI provides a quantifiable, auditable and repeatable measure of lined tunnel condition. It is a high-level asset management tool which can be used to measure and demonstrate the change in condition of tunnels over time.

The TCMI grades the structure on a scale of 0 to 100, where 100 indicate a tunnel with no detectable defects. The TCMI score is derived as a result of non-judgemental recording of defects during detailed examinations. It is not an indication of structural adequacy, fitness for purpose or safety. The operation of TCMI depends upon consistent labelling of elements, and application of the objective and unambiguous severity and extent defect codes.

A report format and examination process are in use that are compatible with TCMI scoring, making TCMI an integral part of the examination process. An algorithm incorporated within the detailed report calculates the condition score and the examiner is not conscious of TCMI being a separate or ‘bolt-on’ process.

Examination Report

The examination report for tunnels is produced using a Microsoft Excel spread sheet. All tunnel specific report workbooks are generated from a master template that employs macros and add-ins to produce the appropriate number of defect recording pages required for each specific tunnel report. This is based on information input at the report set-up stage.

A series of standard alpha-numeric defect codes are used to describe the range of defects commonly found in tunnel linings and associated structural elements. These codes describe both the extent and severity of each defect type. Depending on the type of defect, the extent is either the square area, the number of, or the length of the defect. Defect severity is determined by category using pre-defined category codes specific to each defect type.

Scoring

The TCMI scoring process is automated and requires no additional input other than recording defect details using the standard defect codes. Every defect that is recorded using standard codes is scored in TCMI. Algorithms are used to interpret the defect codes into TCMI scores. The Minor Element TCMI scores are calculated as follows:

Minor Element TCMI = 100 – S_f

where S_f is the severity or extent factor.

Defect scores are aggregated within each minor element to give an individual minor element score which is displayed on the defect recording page in the report and also in the TCMI output summary page.

Different defects carry different TCMI ratings, according to their relative significance to the condition of the tunnel. To incorporate multiple defects per minor element, it is not possible to simply add all the severity/extent factors together and deduct them from 100, since the score could result in a negative value.

To avoid the potential for negative TCMI scores, the impacts of the second and subsequent defects are calculated after the impacts of the previous defects have been taken into consideration. For example, if the severity and extent factors of three defects on the same minor element are 50, 35 and 20 respectively, the following procedure is adopted:

Impact of defect type 1 is 50% of 100 = 50
Minor element score = 100 - 50 = 50
Impact of defect type 2 is 35% of 50 = 17.5
Minor element score = 50 – 17.5 = 32.5
Impact of defect type 3 is 20% of 32.5 = 6.5
Minor element score = 32.5 – 6.5 = 26

Thus the severity/extent factor (S_f) that is input into the algorithm is an aggregation of the severity and extent factors of all the defects within the minor element.

Note that if two identical defects of the same severity are recorded in the same minor element, the extents are combined to form one defect. The algorithm does not treat them as multiple defects.

Element weighting is applied to give greater significance to defects in the crown compared to defects in the haunch and to defects in the haunch compared to defects in the sidewalls. Tunnel Bore minor element scores are aggregated to give Tunnel Section (Major Element) scores as follows:

Major Element TCMI = $100 - \frac{\sum Ef \; x \; Sf}{\sum Ef}$

where:

E_f = Minor element factor;
S_f = Severity/extent factor.

The overall TCMI score for the Tunnel Bore is calculated to be the average of the Major Element TCMI scores within it (i.e. Tunnel Sections, Shaft Eyes and Side Voussoirs).

Additional weightings are applied to specific circumstances, e.g. circumferential cracks in the Tunnel Bore that are within 1 section length of the Portal are weighted higher than the same defect occurring further into the tunnel.

When calculating the Tunnel Bore score, non-standard section lengths are weighted according to their length. For non-standard section lengths (e.g. short end sections) the Bore score is calculated as follows:

$$\text{Tunnel Bore TCMI} = \frac{\Sigma\ \textit{TS score} \times \textit{TS length}}{\Sigma\ \textit{TS lengths}}$$

Each Portal receives a separate overall TCMI score, as do Cross Passages and Adits. This is because these major elements may be subject to different examination schedules to the Bore.

5.3.2 Reliability-based methods and risk and robustness assessments

As in the case commented for bridges, also in the case of railway tunnels, the most recent and advanced assessment methods introduce the concepts of probabilistic-based methods for safety evaluation and the consideration of robustness and risk in the global failure aspects. Risk and reliability acceptance criteria are also presented (Diamantidis, et al., 2000; Diamantidis & Holicky, 2011). The most appropriate maintenance and up-grading and strengthening policies can be derived based on the application of these advanced methods with the objective of minimizing costs and from a life-cycle perspective. The safety target considered when a preventive maintenance or a reinforcing scheme is decided, will be derived by analysing the recent risk history of the railways in terms of the frequency of occurrence of accidents and the extent of their consequences. In this case, the expected consequences of the feasible accidents must be evaluated.

The concepts of robustness and progressive failure are important especially for tunnels since accidental actions as for example fire should not lead to a collapse of the tunnel. Risk analysis can be used as a decision tool for the selection of the optimal repair and strengthening measures. The European tunnel guideline (DIRECTIVE 2004) also deals with the risk analysis to be carried out in the case of highways.

5.4 In track system: Rail, switches and crossings

In general riding quality for tracks is characterized by standard deviations. Indeed, the vertical standard deviation or a mix of vertical and horizontal standard deviations is used to describe the roughness of a track and thus assessing riding quality and riding comfort. Therefore the intervention level is a comfort driven value, not a safety limit. In Uzarski and Mahoney (1997), the so-called Track Structure Condition Index (TSCI) is presented. This condition index is computed from the ties condition index (TCI), rail, fastenings and joints condition index (RJCI) and the ballast and subgrade condition index (BSCI). Utilizing a continuous index scale ranging from 0 to 100, these indexes are derived from the presence, severity and density of defect based track distresses.

However, it is obvious that the quality demand for riding comfort depends on the train speed. For low speed levels a specific standard deviation leads to a better riding comfort than for high speed levels. This influence is not reflected within a standard deviation. Speed dependent threshold values are therefore necessary. Therefore a more advanced assessment tool has been developed by the Austrian railways and is based on another riding quality index, so called MDZ (for Mechanized Tamping Train in German). This index describes the roughness of a track or the riding quality based on accelerations in the car body. More precisely, within this quality data the differences of the accelerations in a short distance are summed up and referred to a point as a gliding mean value for 100 m of track length. The ideal track can be calculated for a zero MDZ index value.

$$\mathrm{MDZ} = V^{0.65}\,\frac{1}{L}\sum_{0}^{L}\mathrm{Diff}\left(\sqrt{v^2 + \left(h + \Delta^2 u\right)^2}\right) \tag{5.28}$$

with

- MDZ: riding quality
- V: speed [m/s]
- L: gliding influence length [m]
- v: vertical track failure [mm]
- h: horizontal track failure [mm]
- Δu: super-elevation failure [mm]

This value allows to take into account all track irregularities (vertical and horizontal ones). Failures cause accelerations in the train; recalculating the riding quality for different train speeds is thus possible. To make the results of this Austrian research comparable with international approaches, all the evaluations have been based on vertical standard deviation (mainly used in Europe) and MDZ. The results do not differ significantly, but some details can be more easily identified with the MDZ quality index. More information on this track condition index is available in deliverable D2.2 of MAINLINE.

Currently, most inspections are carried out at the switch site by experienced maintainers, using mechanical tools. The potential for reducing time spent by maintainers on these tasks depends on the technology available, that is, what variables can be remotely monitored, and what can be inferred from them about the state of the switch. This leads to a potential reduction in the number of inspections required. By using modern remote access technology, the cost of the time spent is also reduced (since on-site inspection, with its safety arrangements and transport, is much more expensive than off-site supervision of monitoring).

According to INNOTRACK project, around 50% of the LCC for a S&C system comes from routine maintenance, which is generally organized according to fixed periods which are determined using a conservative estimate of the amount of time it is safe to leave parts of the S&C unmaintained. This means that more money is spent than necessary, in periodic maintenance. Technical staff is also exposed to line side hazards more than is necessary, because of the extra time they spend on site. Additionally, these maintenance and inspection periods are not always sufficient to mitigate the risk of right side failures. Inspection by humans often has only a superficial insight into the operation of a switch. A more efficient approach to maintenance would be to have accurate automatic condition monitoring systems which can direct maintenance activities more efficiently by only specifying and scheduling tasks which are needed.

It was identified in the INNOTRACK project that the critical parameters to the operation of a S&C system are the force, displacement, throw time and (for electric actuators) motor current. It was demonstrated within the project that it is possible, with relatively cheap sensors, to measure significant variation in key parameters during the onset of common faults. 39 % of faults reported on an actuator type showed variation in the key parameters when simulated. Deliverable D3.3.4 of INNOTRACK contains a general introduction to the field of advanced condition monitoring, followed by a detailed examination of the more established methods. Some faults can be detected perfectly well with these simple methods.

It is important not to over-complicate monitoring systems by using complex algorithms where simple ones are adequate. Following from this review, a new algorithm was developed to address the problem of incipient faults at an early stage of development. This algorithm uses qualitative and quantitative trend analysis to pick out changes to the shape of waveforms from measurements of key parameters. The results were very promising, showing a strong ability to detect early trends towards faulty conditions.

In INNOTRACK project the actual inspection and assessment tools used by DB and Banverket are summarized. In the case of the German railways, the inspection includes the whole area of the S&C from or until the joints of the normal rail. The following subsystems are checked:

1. Track components: Track, Rail, Sleeper, Fasteners, Joints, Ballast, Switch blade, Frog, Check rail

2. Signalling components: Actuation system, Locking device, Point machine, Sliding chairs / blade rollers

3. Heating components: Heating device, Track components.

In the case of Banverket, the inspection includes the whole area of the S&C from or until the joints of the normal rail on all three tracks involved (in a simple S&C). The following subsystems are checked: Track position, Rail, Sleeper, Fasteners, Joints, Ballast, Switch blade, Frog, Check rail, Snow cover, Actuator, Switch blade position detector, Locking device, Heating device.

Improved condition monitoring capabilities can reduce the life-cycle costs of S&C in several different ways. To begin with, it is important to define which costs are to be attributed to the S&C. The list below points out some obvious and some subtle areas of cost where improved condition monitoring may have a positive impact.

1. Parts and labour costs for maintenance & inspection tasks carried out on S&C

2. Cost of delays caused by S&C failures

3. Cost of delays caused by S&C maintenance at inconvenient times

4. Costs arising from the increased risk to staff maintaining/inspecting S&C on running railways

5. Installation cost of S&C where it is linked to the length of the life-cycle.

An advanced assessment method has recently been implemented in the inspection of switches. It is based on the base-line reference curve of the measured intensity of current along time in the switch operating system (figure 5.19). The current intensity can be related to the required force applied to the switch moving system during the operation time. With the switch in correct performance, the green curve is obtained in figure 5.19). Each movement of the switch is compared with the reference curve to detect possible malfunctions (red curve in figure 5.19).

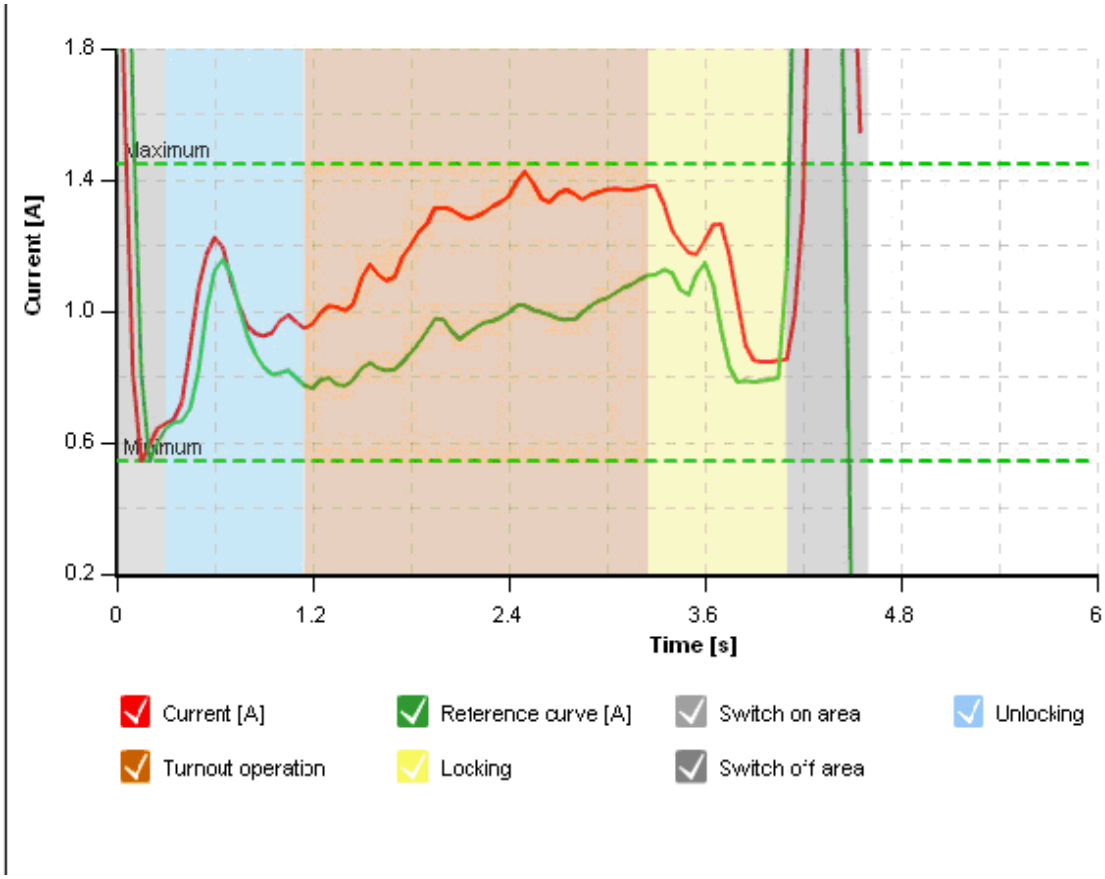


Figure 5-19 Assessment of switches based on measured intensity of current

The curve is divided in different parts according to different operations during the movement (switch on, unlocking, turnout operation, locking and switch off). According to that, different criteria are defined to conclude that the performance is incorrect. In general, the points to be considered are:

- Power consumption: related to an excess of force to start the movement or to follow moving the element once the movement has started
- Time: Comparing if the required time in each phase (unlocking, turnout operation, locking) is much higher than in the reference curve.

Only one warning of a malfunction (one measurement out of the defined threshold) does not mean a defect. At least three consecutive points should be higher than the threshold to be identified as a defect in the switch. The differences between the actual and reference curves are normally associated to typical defects (lack of oil in the hydraulic systems, etc…).

5.5 In earth works

The document CIRIA C591 (2003) provides some indications about the condition appraisal of cuttings.

For steep and high cuttings, foot access may not be possible, and specialist surveying techniques such as photogrammetry may be required. Inspection of rock cuttings is less affected by the presence of vegetation than on soil embankments and cuttings.

The use of Stereo Oblique Aerial Photography (SOAP) has been shown to be a viable method for the rapid collection of condition data. This method takes oblique photographs from a helicopter and allows geotechnical interpretation to take place as the photographs are being taken, as well as in more detail when studying stereographic pairs. The method is particularly useful for rapid assessment of slopes with clearly observable failures.

The objectives of a stability study, which is part of the assessment, and the generation of the business case for future work for embankments are covered in CIRIA C592. The approach to soil cutting stability is also similar and the proposed methods for embankments (see also SMARTRAIL Project) can also be of application. The main difference is in the study of rock slope stability as follows.

As part of the assessment stage, a geotechnical model of the cutting slope should be produced. This should include an interpretation of the stratigraphy, the discontinuity sets within the rock mass gained from measurements during the inspection, the groundwater conditions and the effect of seasonal variations on the stability of the slope. This model can then be used as the basis for stability analyses carried out on the cutting slope being considered.

Once the geometry of the slope has been ascertained, the likely mode of failure deduced, and strength and water pressure parameters determined, slope stability analyses are undertaken.

As has been previously discussed, the stability of a rock cutting slope is almost always determined by the presence of discontinuities within it. The simplest method that can be used to determine the stability of a rock slope relies on the use of stereographic analysis to compare the orientation and strength of discontinuities to the geometry of the rock slope. Such techniques are known as kinematic admissibility analyses, and they are discussed in detail in Priest (1985). Use of kinematic analysis gives a very useful indication of the potential for planar, wedge or toppling failure within a rock cutting slope, which may not be obvious from visual inspection alone.

More rigorous analysis of the stability of a rock slope can be undertaken using limit equilibrium analysis techniques, described in detail in Hoek and Bray (1981). Such analyses can be completed for planer, wedge and toppling failure modes, but they are based on the assumption that the discontinuities within a rock slope are infinitely persistent, which may not be the case in the field. However, the techniques are still useful for the determination of factors of safety against slope failure, which can be used to prioritise the various sites under consideration.

A further level of detail in the analysis of a rock slope can be achieved using computer programs. Several programs are available, ranging from those that simply apply the kinematic admissibility and limit equilibrium methods described above, to those that carry out a full numerical analysis of a discontinuum, either in two or three dimensions. Assessment for a business case is, as with infrastructure embankments, different to design assessments. It involves the use of sources other than those obtained from a ground investigation, including records of past failures, performance of other cuttings in similar materials and the strategic value of the route.

The reporting requirements and methods of prioritisation for cuttings are identical to embankments and CIRIA report C592 should be consulted.

According to CIRIA C591 there is clearly a lack of data on the costs associated specifically with earthwork repair and maintenance. Only by clearly understanding these costs can more accurate budgets be prepared to assist in forward planning and effectiveness of maintenance operations.

In the UK, the Soil Slope Hazard Index (SSHI) and the Rock Slope Hazard Index (RSHI) were adopted by Network Rail for the condition assessment of soil and rock slopes (Network Rail, 2008).

In 2003, Network Rail commissioned Babtie Group (Babtie Group 2003) to develop a system to facilitate quick and repeatable inspection of railway earthworks (both soil cuttings and embankments) to enable impartial assessment of their condition. This resulted in the development of the Soil Slope Hazard Index and associated algorithm. Babtie Group produced a report on the principles and requirements of the SSHI algorithm and how they were achieved [Babtie Group 2003]. The key points of this report are fully explained in Deliverable D2.2 of MAINLINE.

The SSHI and associated algorithm have been developed to facilitate the rapid and repeatable inspection of rail earthworks together with an impartial assessment of their condition. It takes information from specially designed pro-formas, completed in the field by geotechnically qualified inspectors, which summarise the pertinent information on the condition of an earthwork. This includes information on the slope geometry, such as height and angle, information on the slope composition and drainage, and information on any failure mechanisms that are imminent or active. Because the inspections are carried out by walking over an earthwork it means that the level of detail obtained from the inspections is very good compared to alternative remote monitoring techniques. The semi-quantitative data from the inspection and associated desk study is input into a specially designed algorithm, which converts the raw data into three factors:

• Actual Failure Factor – built up from evidence of past, on-going or imminent failures, such as tension cracks.

• Potential Failure Factor – takes account of features that could make future failures more likely, even if no failure has yet occurred, such as the soil type, poor earthwork drainage or the slope angle.

• Height Factor – takes into account the scale of an earthwork.

The algorithm weights these factors and combines them to produce a Soil Slope Hazard Index, based on which the earthwork is categorised as Poor, Marginal or Serviceable. The algorithm is designed so that the results are transparent and can easily be traced from the source data. This is carried out for five possible failure modes, namely rotational, translational, earthflow, washout and burrowing. The algorithm is not an analytical process, rather a semi-quantitative indication of general condition. It is based upon simple manipulation of empirical observations obtained from a combination of desk studies and site inspections. In conclusion, the SSHI and its associated algorithm are considered to be a cost-effective and satisfactory way of carrying out rapid, repeatable inspections to produce an impartial assessment of an earthwork's condition. Further information on the application algorithm used to calculate the SSHI can be found in Network Rail (2004).

Various options have been considered during the development of the algorithm, but its development has been guided by the following principles:

The Nature of the Algorithm

• The system is not an analytical process, rather a semi-quantitative indication of general condition.

• The system is based upon simple manipulation of empirical observations.

• The scoring and algorithm should be as transparent as possible.

• It should be possible to monitor changes in the condition for each failure mode between cyclical inspections.

• The Earthwork Condition should be able to differentiate between Poor, Marginal and Serviceable conditions.

The Relationships between Failure Modes, SSHI and Earthworks Condition

• The SSHI will be derived for each mode of failure.

• The Earthworks Condition will be based on the failure mechanism producing the highest SSHI.

• The system should give an indication of the geotechnical risk posed by an earthwork. Network Rail will use this value as the basis for determining the overall risk posed by taking into account appropriate consequence factors.

• The Earthwork Condition for an earthwork should not be the aggregate of failure mechanisms but should reflect the most critical failure mechanism.

Permitted Results from the Algorithm

• A Poor condition should be possible if just one of the five major failure mechanisms is present.

• A Poor condition should not be generated because of an aggregate of relatively minor features, but should reflect a poor condition with respect to one or more of the failure mechanisms.

• A Poor condition should be possible even with small earthworks if features observed on the site warrant it.

• A Poor condition should not be generated purely on the basis of the size and geometry of an earthwork.

Characteristics and behaviour of rock cuttings are very different to that of soil cuttings. For this reason, different condition scoring systems are used for rock cuttings. The RSHI (Rock Slope Hazard Index) system was developed by TRL to aid in identifying potentially hazardous slopes. It was developed for use in the assessment of highway rock slopes and is based around quick and standardised field data collection containing estimates of geotechnical, geometric and remedial work factors (TRL 2000). Further explanation about the derivation of the index is available in Deliverable D2.2 of MAINLINE.

In Mcmillan and Nettleton (2011) a Rock Slope Hazard Index is derived based on two stages. The first stage derives a RSHI from rapid, standardized field data collection. The results are used to classify rock slopes into four action categories. This is based on standard forms, one

for geotechnical parameters and the other for geometric parameters. The forms present a number of options for each parameter. Selection of the relevant options is based on visual assessment of the parameter in the field. The second stage derives a Rock Slope Hazard Rating (RSHR) from semi-probabilistic analysis of data recovered from a detailed filed survey. The RSHI is intended to act as a coarse sift, identifying potentially hazardous slopes. The RSHR is intended to act as a fine sift, identifying the level of hazard at each rock slope and allowing prioritization of maintenance. This hazard rating takes into account the probability of failures of the rock slope reaching the road and the probability of vehicle incidents because of the rock failure. The main advantages and disadvantages of the RSHI are fully described in deliverable D4.1 of MAINLINE.

The paper by Maerz (2000) describes some of the techniques currently used for stability assessment for rock cuttings.

The Rock fall Hazard Rating system (RHR) was designed to proactively address the issue of rock falls for road cuts and rail lines (Pierson & Van Vickle, 1993). The system goes much further than other classification systems, in addition to looking at material properties, taking into account such diverse aspects as rock fall history, volumes of material that might fail, and the capacity of existing mitigation measures to contain that volume of rock, in a quasi-probabilistic manner. In addition, the system is useful as a screening technique, allowing high-risk slopes to be quickly identified by a preliminary rating. The preliminary rating consists of an assessment of the slope into one of three categories, A, B, C (high, moderate, low) for two considerations: 1) Estimated potential for rock fall on roadways, and; 2) Historical rock fall activity. Typically only slopes with preliminary ratings of A are given detailed ratings.

The detailed rating system uses 10 categories with 4 nominal rating criteria and scores, although interpolations of scores between criteria are allowed. The scoring is a power progression, with score $y=3^x$ where x is a rating between 1 and 4 and allows scores of 3, 9, 27, and 81 respectively.

The following are the categories:

1. Slope height (25, 50, 75, or 100 feet),

2. Ditch effectiveness (good, moderate, limited, or no catchment),

3. Average vehicle risk (vehicle present in rock fall section 25, 50, 75, or 100% of the time),

4. Sight distance (100, 80, 60, or 40% of stopping distance when viewing a 6. object),

5. Roadway width (44, 36, 28, or 20 feet including shoulders),

6. Structural condition *discontinuous rock* (discontinuous joints-favourable orientation, discontinuous joints-random orientation, discontinuous joints-adverse orientation, or continuous joint-adverse orientation),

7. Rock friction (rough-irregular, undulating, planar, clay infilling or slicken sided), or

6. Structural conditions *eroded rock* (few differential erosion features, occasional erosion features, many erosion features, or major erosion features),

7. Difference in erosion rates (small, moderate, large-favourable structure or large-unfavourable structure),

8. Block size/volume of rock fall event (1/3, 2/6, 3/9, or 4/12 ft/cubic yards),

9. Climate and presence of water on slope (low to moderate precipitation; no freezing periods; no water on slope, moderate precipitation or short freezing periods or intermittent water on slope, high precipitation or freezing periods or continual water on slope, or high precipitation and long freezing periods or continual water on slope and long freezing periods,

10. Rock fall history (few, occasional, many, or constant falls).

All the categories are then added up, and the highest scores are deemed to have the highest priority in terms of remediation.

The rock fall hazard rating system, Ontario, (RHRON) is a modified version of the RHR system. It attempts to address the overemphasis on high slopes and large volumes that occur as a result of the power relationship between rating and score. In Ontario, with its relatively lower slopes than locations like Oregon, the RHR system was just not sensitive enough. In addition, five new parameters were added, and several parameters were re-defined.

The basic formulation for RHRON is:

$$RHRON\ (\%) = (F1 + F2 + F3 + F4) * \frac{100}{36}$$

where:

F1 = Magnitude: “How much rock is unstable?”

F2 = Instability: “How soon or often is it likely to come down?”

F3 = Reach: “What are the chances of this rock reaching the highway?”

F4 = Consequence: “How serious are the consequences of the blockage?”

For the *preliminary screening*, each of these F factors is directly rated on a scale of 0 to 9. For the *detailed rating*, each of these F factors is calculated from a number of individual ratings also on a scale of 0-9:

$$F1 = \frac{(R2 + R3 + R4)}{3}$$

$$F2\ ravelling = \frac{(R1 + R9 + R11 + R4 + R5 + R6)}{6}$$

$$F2\ sliding = \frac{(R1 + R9 + R11 + R5 + R6 + R8)}{6}$$

$$F2\ erosion = \frac{(R1 + R9 + R11 + R4 + R7 + R10)}{6}$$

$$F3 = \frac{(R4 + R13 + R16 + R19)}{4}$$

$$F4 = \frac{(R17 + R18 + R19)}{3}$$

where:

R1=History of rock falls
R2=Volume of the largest potential rock fall
R3=Volume of total potential rock fall
R4=Face irregularity
R5=Face looseness
R6=Joint orientation/persistence
R7=Rock intact strength
R8=Rock joint shear strength
R9=Block size
R10=Slake durability
R11=Water table height
R12= Slope height
R13=Crest angle
R14=Ditch and shoulder width
R15=Ditch capacity
R16=Overspill potential
R17=Average vehicle risk
R18=Decision sight distance
R19=Available paved width
R20=Remediation cost

5.5.1 Generalised soil slope condition assessment algorithm

Following review of the SSHI and RSHI algorithms, SKM have identified some possible limitations to their application within the deterioration models to be developed for the MAINLINE LCAT.

- They have been developed based on UK experience alone and may not be suited to the assessment of earthworks in other European regions.
- There is the possibility of some double accounting within the SSHI algorithm, for example the additive effects of poor drainage and observed surface water, which may limit accuracy of condition scoring. In reality there are complex cause and effect relationships between the base values and observed condition parameters, as well as between the observed values themselves. If these interdependencies are ignored when calculating condition then there is a danger of double counting the contribution to condition score. However the complexity of the interdependencies means that it is not practical to attempt to model them all. The challenge is to account for the major interdependencies without making the model too complex.

- The specific nature of some of the input parameter values may not be compatible with examination data available in other European countries.
- Simultaneous monitoring of SSHI scores for the 5 different failure modes, and counting only the most onerous as the overall condition may not be the most appropriate way to deal with the inter relationships between degradation mechanisms. The SSHI model treats failure modes separately but in reality there is clearly interaction between them.

SKM have therefore looked for the means and methods to suggest a more generalised algorithm that can be readily adapted to suit different regions and authorities within the European Union. Due to the time and resource restraints, efforts have been focused on developing a prototype condition assessment algorithm for cohesive soil cuttings only. Based on the methodology proposed, the prototype algorithm could be further developed and extended to cover other soil types and rock cuttings. A complete description of the new proposed algorithm can be found in Deliverable D2.2 of MAINLINE.

6. Examples of advanced assessment

In the following example it is shown the use of reliability-based assessment techniques on a bridge that has been selected in WP1 as a candidate to carry out a test up to failure to see the failure mechanisms developed in this bridge type, very popular among railway bridges.

6.1. Aby bridge (Sweden): Direct reliability-based assessment analysis

Some of the advanced methods presented in this deliverable will be applied to a particular bridge: the Aby bridge, a steel truss railway bridge that was located in Northern Sweden (figure 6.1). 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 6.2).

Figure 6-1 Aby bridge in its original location

Figure 6-2 Aby bridge removed from the location for load testing

6.1.2 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.

In appendix A the original drawings of the bridge are displayed, which were made available by TRAFIKVERKET, through LTU. More details about the bridge geometry and bar connections can be found there.

6.1.3 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 6.3 is presented a perspective of the FEM model. The general mesh is presented in figure 6.4 and a detail of one of the joints is shown in figure 6.5.

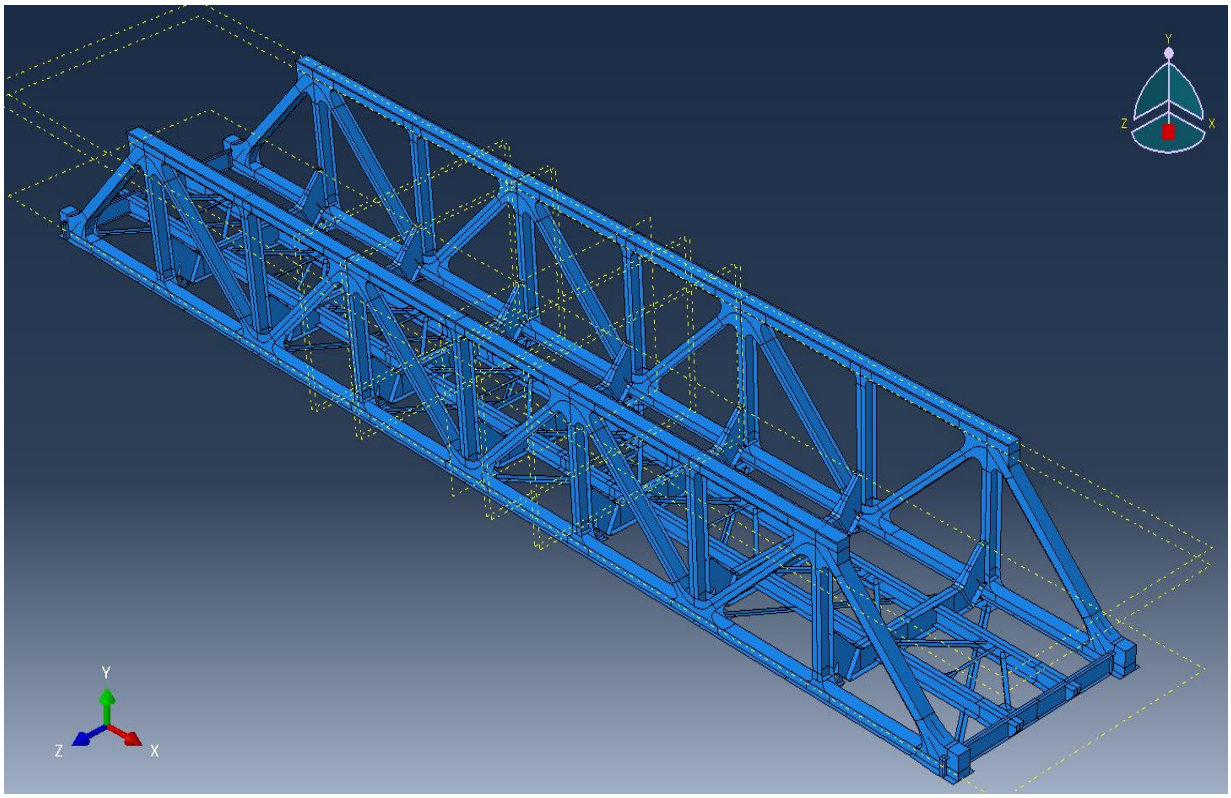


Figure 6-3 General view of the model of Aby bridge

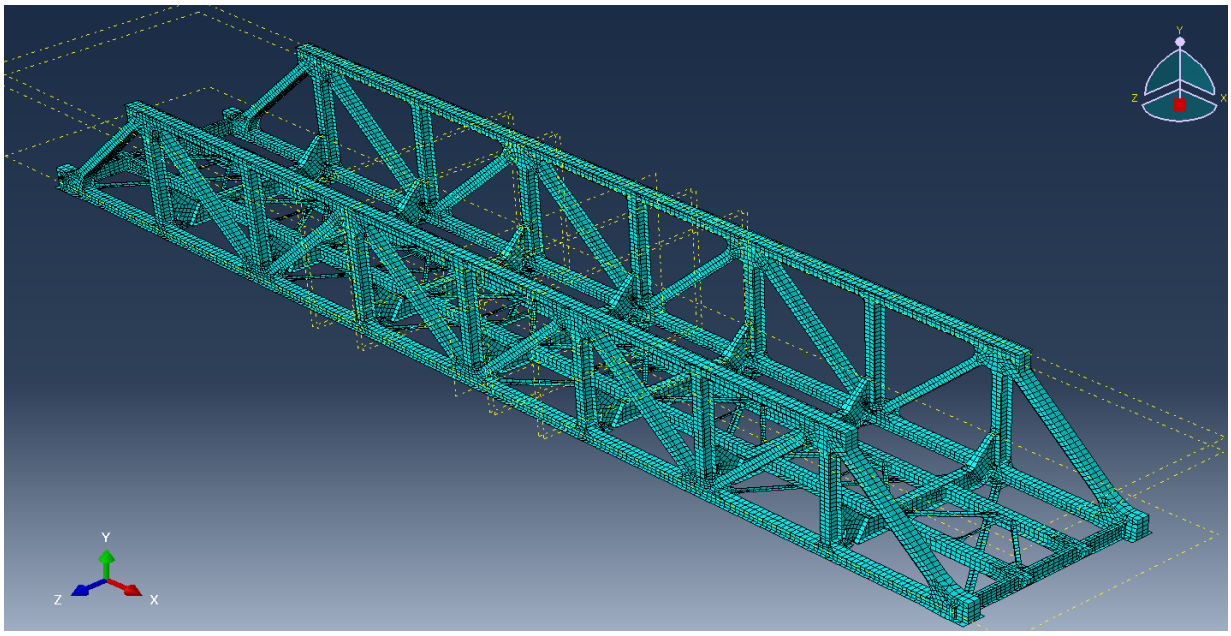


Figure 6-4 View of the FEM mesh

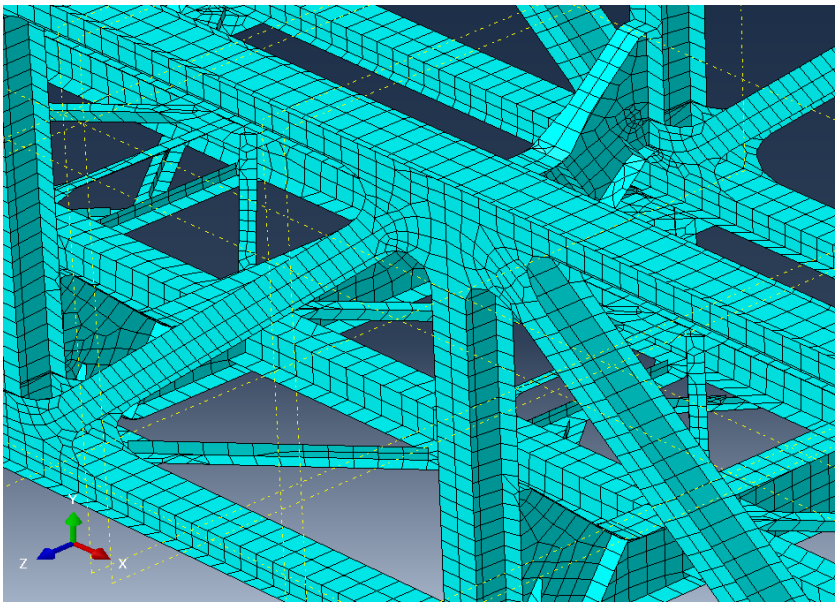


Figure 6-5 Detail of mesh at the joints

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).

6.1.4 Variability of parameters

In order to take into account the complex non-linear behaviour of this bridge, the proposed FE model is also very complex and needs an important 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 considering 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 6.1, 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 6-1 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.

6.1.5 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 Latin Hypercube sampling method according to the parameters presented in table 6.1. The number of simulations was decided as 100, small because the FEM model is very complete 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 6.6 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 6.7.

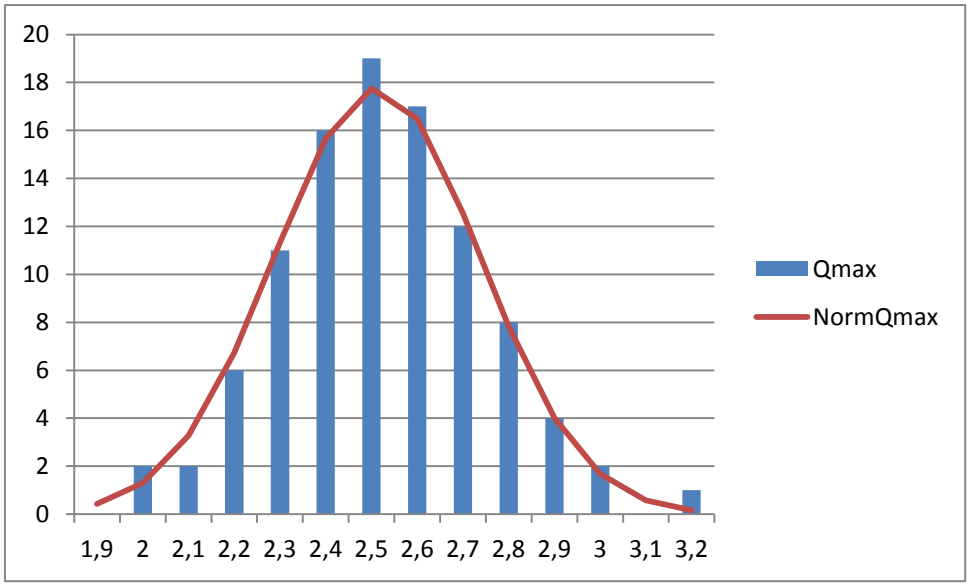
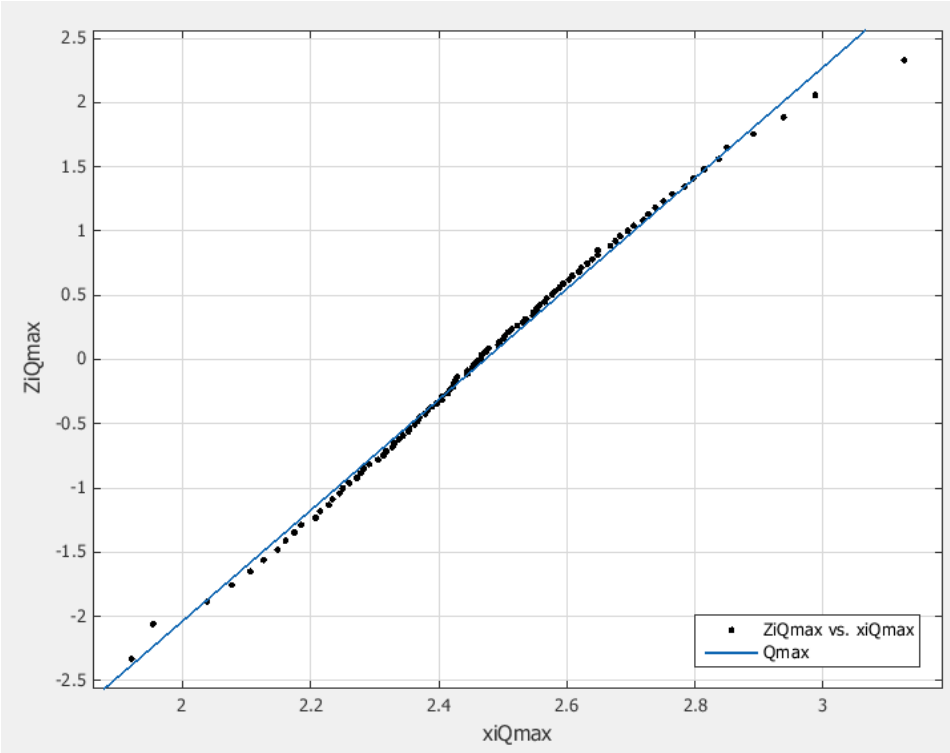


Figure 6-6 Histogram of Maximum Load Factor and Normal fitting



Linear fit Matlab	
m	4,457
n	-11,2
Rsquare	0,9942
μ	2,512901055
σ	0,224366166

Figure 6-7 Simulation 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 6.8 and 6.9. In the first one, the compression and tension along the longitudinal axis are shown. Figure 6.9 shows the general Von Mises stresses.

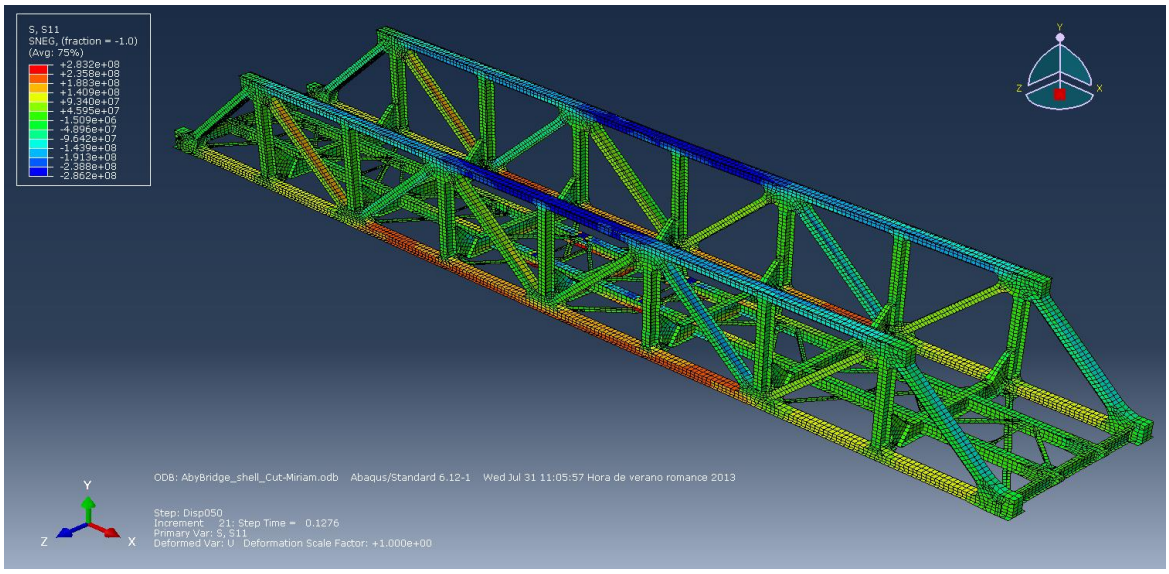


Figure 6-8 Stresses in the longitudinal direction corresponding to maximum load

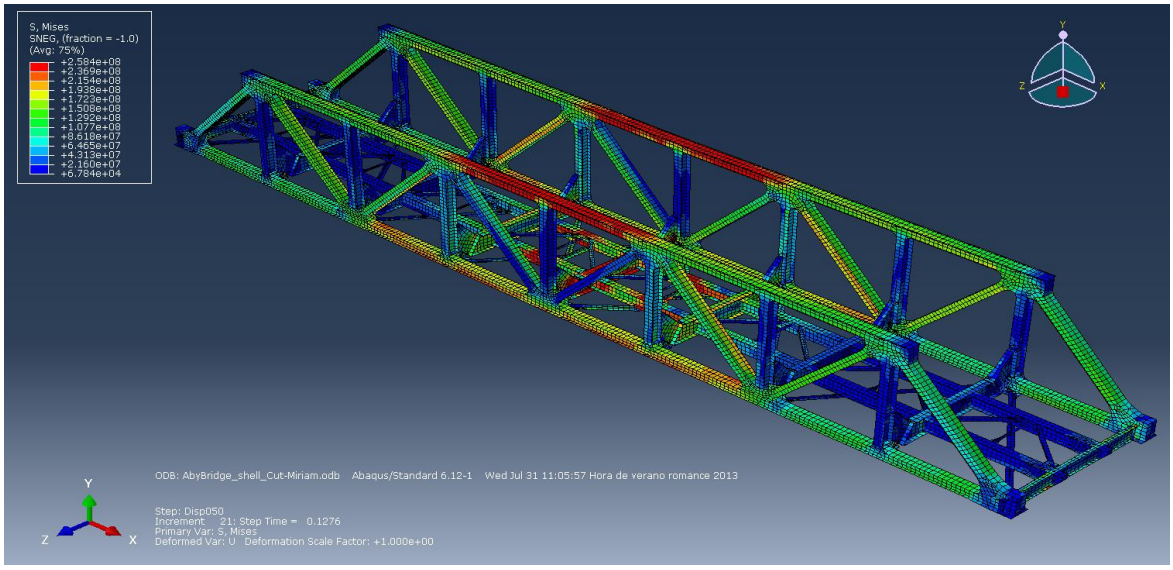


Figure 6-9 Von Mises stresses corresponding to maximum load

The deflections in the bridge corresponding to the maximum applied load are shown in figure 6.10. The load applied versus deflection at mid-span can be seen in figure 6.11.

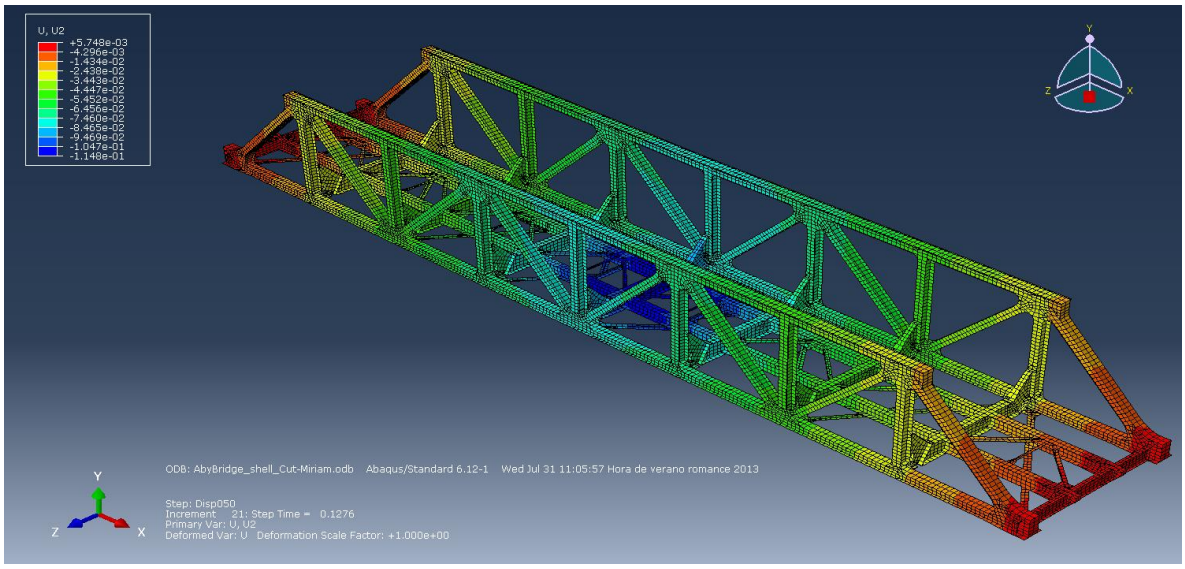


Figure 6-10 Deflection corresponding to maximum load

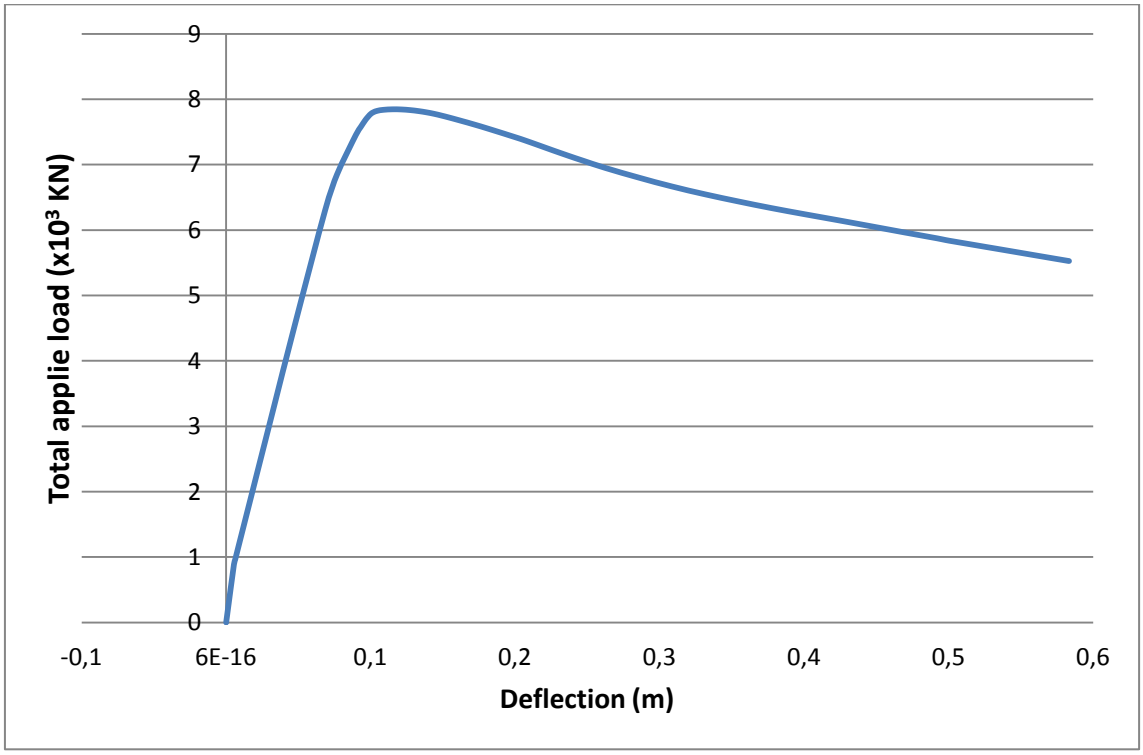


Figure 6-11 Load- deflection result for simulation N. 95

After reaching the maximum load, the bridge presents a post-peak behaviour as shown in figure 6.11, arriving to the ultimate load with corresponding stress state as presented in figure 6.12. The figure also shows the buckling on several members leading to the final failure of the bridge.

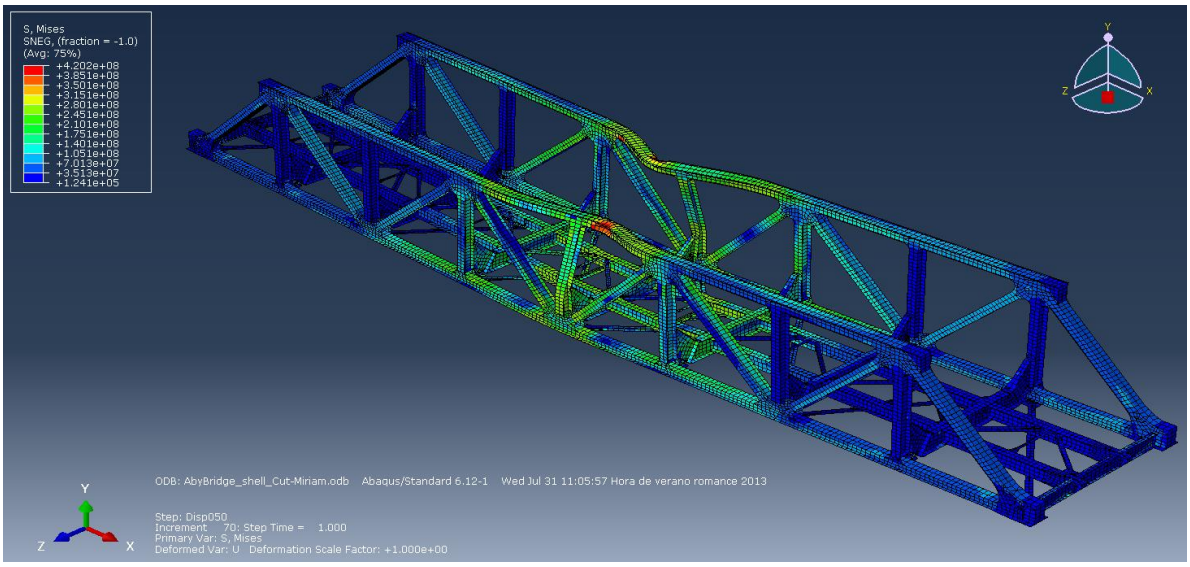


Figure 6-12 Stress (Von Mises) corresponding to ultimate load for simulation N. 95

A different failure mode is shown in figure 6.13 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.

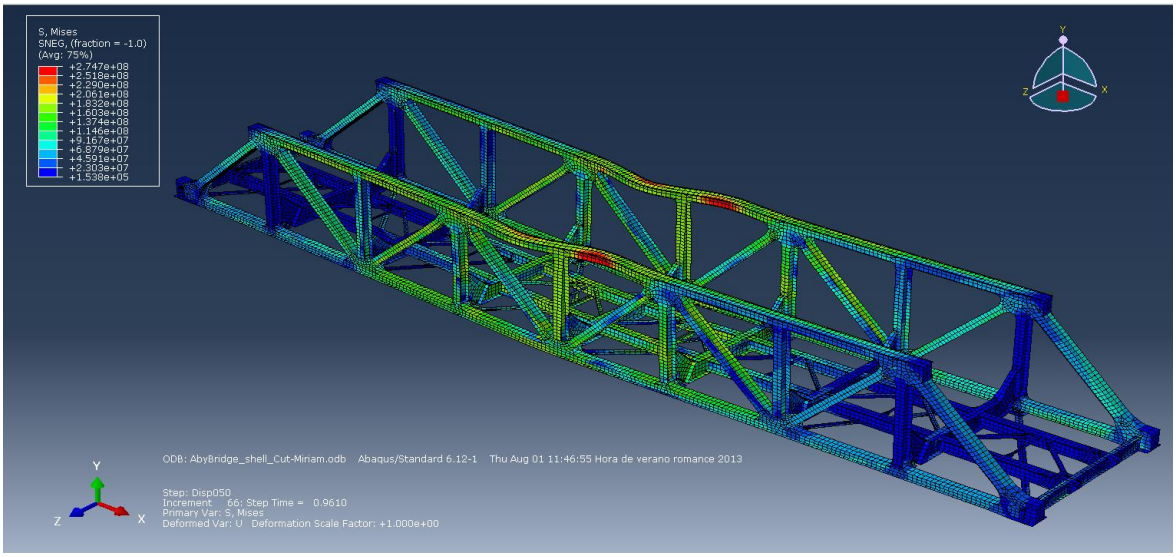


Figure 6-13 Stress (Von Mises) corresponding to ultimate load for simulation N. 1

The stress states and load versus deflection curve for this simulation N. 1 are shown in figures 6.14 to 6.16.

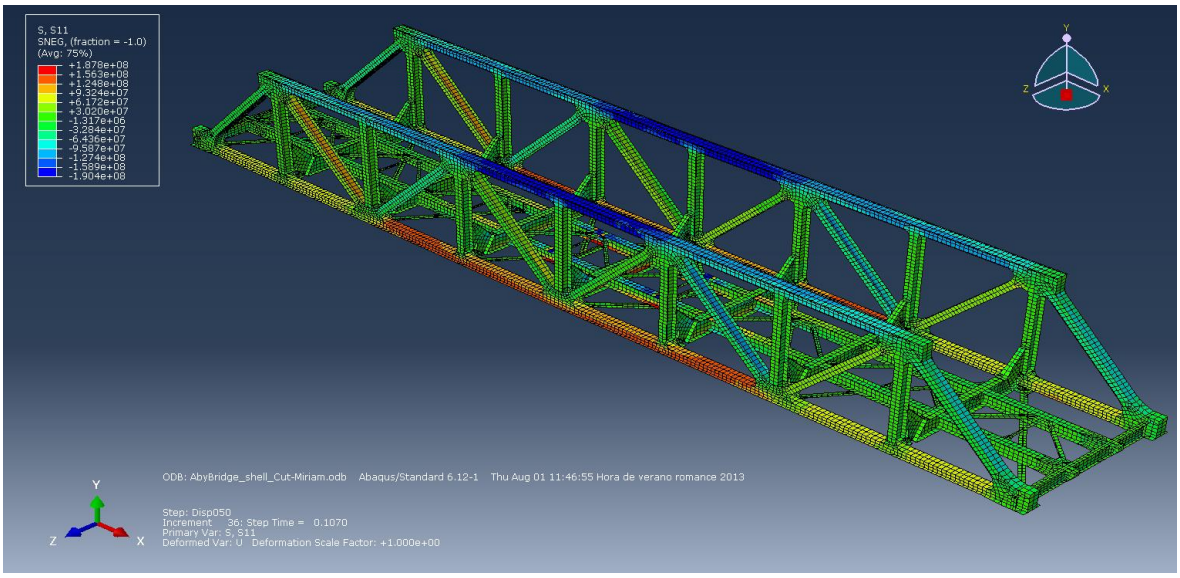


Figure 6-14 Stresses in the longitudinal direction corresponding to maximum load (simulation N.1)

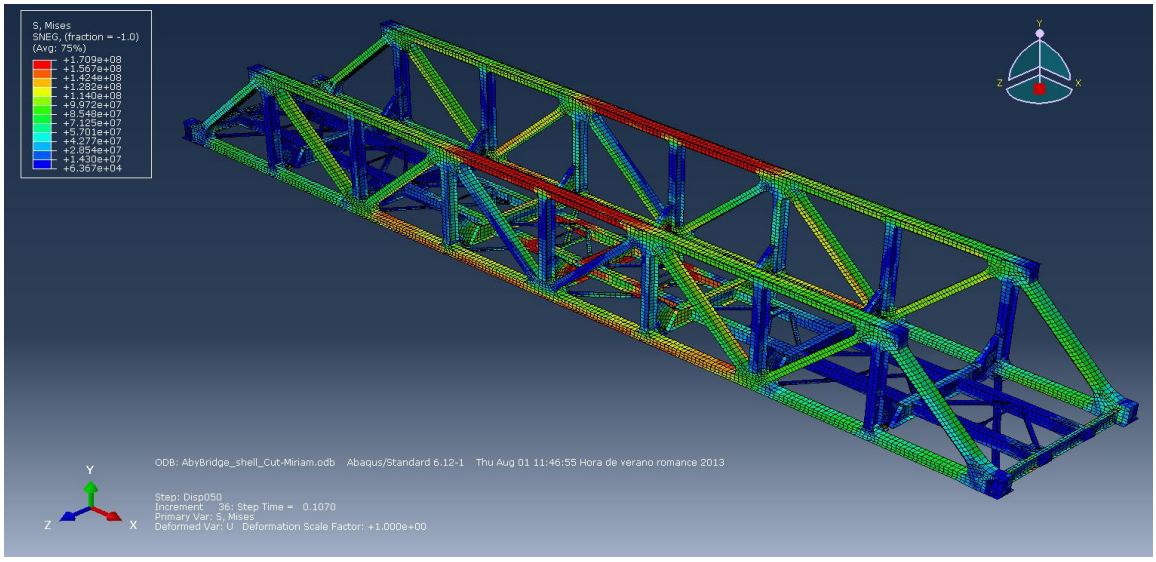


Figure 6-15 Von Mises stresses corresponding to maximum load (simulation N.1)

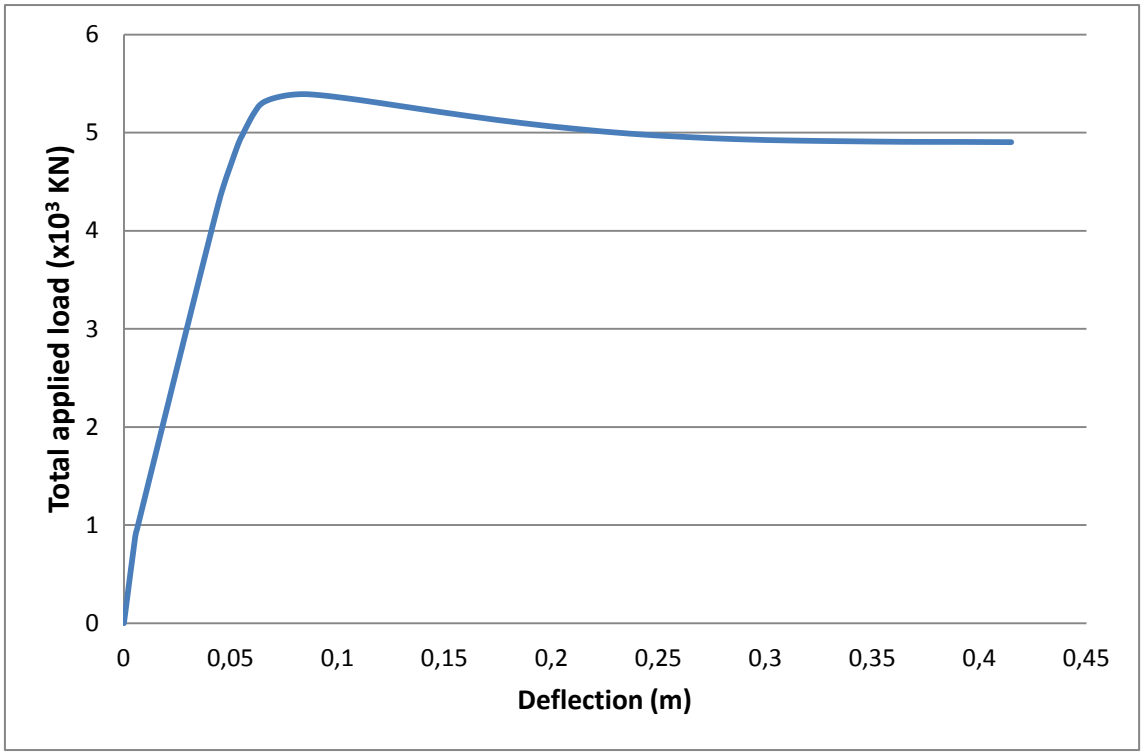


Figure 6-16 Load vs. Deflection for simulation N. 1

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 6.6) is the generalized resistance. 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. 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, 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.

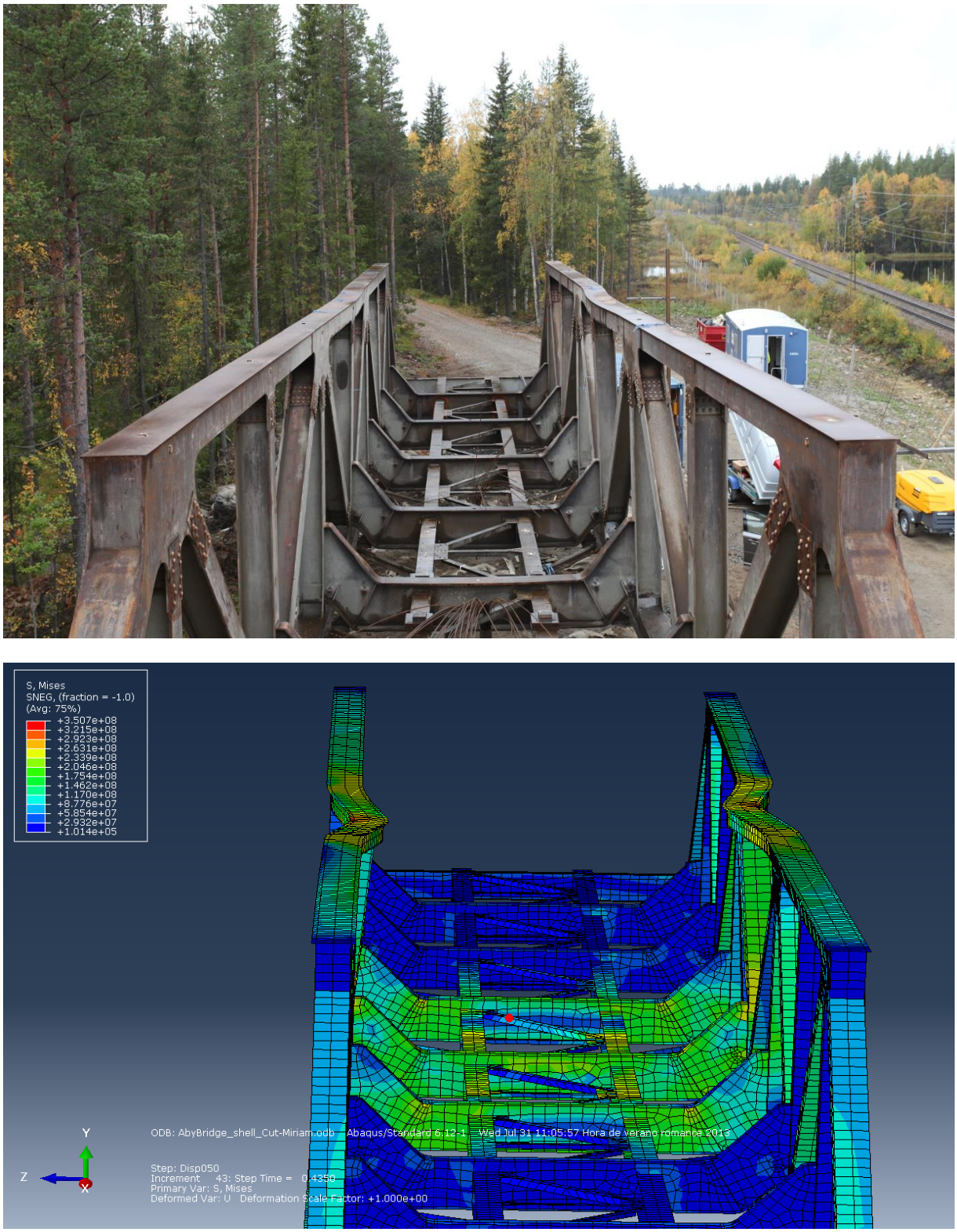


Figure 6-17 Test to failure and numerical model of the Aby bridge (Sweden). The figure shows a comparison between actual and predicted deformations for a steel truss bridge which was loaded to failure in September 2013. The failure was initiated by instability of the two top girders. This was predicted by the reliability based assessment method using a non-linear finite element model

7. Advanced assessment within a LCA framework

Inspections and structural assessment based solely on experience may be more expensive and less safe than those based on a more rational approach. The optimal policy has to be chosen based on minimum expected total life-cycle cost criterion including its effect on structural reliability and the expected costs associated with failure.

The goal of an optimal management strategy is to minimize the lifetime cost of a given infrastructure while ensuring that maintains an acceptable reliability level throughout its expected service life.

The expected total cost C_{ET} is the sum of its components including the initial cost of the structure C_T, the expected cost of routine maintenance C_{PM}, the expected cost of inspection C_{INS} , the cost of repair C_{REP}, and the expected cost of failure C_F. The cost of failure includes both direct and indirect cost, such as user costs and others. Accordingly, C_{ET} can be expressed as (Frangopol, et al., 1997):

$$C_{ET} = C_T + C_{PM} + C_{INS} + C_{REP} + C_F$$

(7.1)

According to Frangopol et al. (1997) the objective remains to develop a strategy that minimizes C_{ET} while keeping the lifetime reliability of the structure above a minimum allowable value.

Since the advanced assessment allows to better know the state of a structure, it can be seen as a part of an inspection and therefore as part of the costs of such inspection, C_{INS}. Given that the ultimate goal is to know the exact state of the structure, any additional inspection costs resulting from advanced assessment could be rewarded with a considerable decrease of repair costs and, in turn, in a reduction of life-cycle costs.

To implement an optimum lifetime strategy, the following problem must be solved: minimize C_{ET} subject to guarantee a minimum performance level of the infrastructure.

The performance level may be measured in terms of the condition index, the remaining load capacity, etc. To take into account the uncertainties inherent in the minimization problem, an appropriate measure of the performance can be the probability of failure. Then the boundary condition becomes:

$$\boldsymbol{P}_{f,life} \leq \boldsymbol{P}^*_{f,life}$$

(7.2)

where $P^*_{f,life}$ = maximum acceptable lifetime failure probability (also called lifetime target failure probability). Alternatively, considering the reliability index:

$$\beta = \Phi^{-1}(1 - P_f)$$

(7.3)

Where Φ is the standard normal distribution function. The optimum lifetime strategy is defined as the solution of the minimization problem subject to the condition:

$$\beta_{f,life} \geq \beta^{*}_{f,life} \tag{7.4}$$

where $\beta_{f,life}$ and $\beta^{*}_{f,life}$ are the lifetime reliability index and the lifetime target reliability index, respectively.

According to equation (7.1), the decision about the use of advanced assessment methods can be incorporated in the LCA optimization framework through the term C_{insp}. It becomes clear from figure 4.6 that the realization of and advanced assessment has clear consequences both in the actual performance index and in the time to reach the minimum required performance. Therefore, the decision on carrying out an advanced assessment does not only affect the term C_{insp} in equation (7.1), but also the terms C_{rep} and C_F. The last one through the up-dated probability of failure obtained from the assessment. In this case, more than a cost, it may become a benefit, if the outcome of the assessment derives in a lower cost of repair. However, this cannot be always the case, as the advanced assessment may discover a lower condition rating than the one predicted by the basic assessment. The same can be mentioned from the C_F as the actual probability of failure will be up-dated. A lower probability of failure will give a lower expected cost of failure.

The main objective of LCA is to spend the minimum possible amount of financial and natural resources, keeping the structures safe and serviceable along their whole life. Therefore, the objective of the optimization process is to determine optimal maintenance and repairing (M&R) strategies. Moreover, advanced assessment methods update structural knowledge of the structure and should affect the M&R planning. One great interest is then to compare optimal M&R strategies without and with inclusion of advanced assessment results. When no advanced assessment is performed, the optimal solution is searched by minimizing both the cumulative cost for the owner at the end of service life including the maintenance cost and the failure cost at the end of the service life. On the other hand, when advanced assessment is performed, the optimal solution is the same but in this case the cumulative owner cost at the end of service life includes the assessment cost and the maintenance cost.

The existing correlation between the costs of the alternative assessment methods should be taken into account. In fact, the cost of performing a SHM can also affect the cost of a proof loading, for instance, as part of the sensors deployed to carry out the monitoring can also be used in the load test. Therefore, some decisions may affect several costs at the same time.

In the following, the cost of some advanced assessment methods is discussed.

7.1 Structural Health Monitoring

The cost of monitoring depends on monitoring times and durations. The monitoring cost is composed of fixed cost due to the preparation and analysis of monitoring, and of variable cost that depends on the duration of monitoring.

Orcesi and Frangopol (2011) have developed an optimization of maintenance strategies based on SHM information in order to minimize the costs related to it. According to them, the expression of the cumulative monitoring cost would be:

$$C_{mon\_system} = \sum_{j=1}^{m^{mon}} \left(\sum_{k=1}^{n_j^{mon}(t)} \frac{\left(C^{j}_{mon,ref\frac{d_k}{d_0}} + C^{fj}_{mon} \right)}{(1+r)^{t_k}} \right) \tag{7.5}$$

Where $C^{j}_{mon,ref}$ reference cost of monitoring component j during d_0 days, d_k = length of monitoring (in days), C^{fj}_{mon} =fixed cost of monitoring component j, m^{mon} =number of components that are monitored, r = yearly discount rate of money, n_j^{mon} (t) is the number of monitoring times for the component j until t, and t_k = time of kth monitoring of component j.

The reference cost of monitoring of a component will be a function of the cost of the sensors and the number and performance of the sensors deployed. Normally as the cost increases, the number of sensors decrease. The optimum solution in terms of cost can be obtained via a multi-objective optimization procedure (Marsh and Frangopol 2007).

For instance, in the case of monitoring of corrosion in reinforced concrete elements, the most extensively used method to determine is the linear polarization resistance (LPR) measurement (Qian 2005). The fixed cost of this type of sensors embedded in the concrete is in the range of 50 to 300 $ (Marsh and Frangopol 2007).

7.2. Proof load testing

Recently, Alampalli and Ettouney (Alampalli and Etttouney, (2010), Ettouney and Alampalli, 2012) have made an attempt to develop a generalized model to evaluate the value of load testing and the effect of load testing on bridge life by considering load testing costs and both qualitative and quantitative benefits.

In most cases, the estimation of cost and benefits in order to determine the value of load testing is done qualitatively by the bridge owner through studies of structural condition, resources required, expected sources of reserve strength where analytical models are not appropriately modelled, and the probability of the test providing a better load rating. But if the method can be automated and made a part of a LCAT, then this tool may be able to advise owners on when load testing may be appropriate and suggest load testing as an option among other recommendations.

As they are, in general, associated with a specific structure with defined objectives, load testing costs can be estimated with relatively high confidence once the structure is decided. The main sources of costs associated with load testing include:

- Traffic control costs (C_T). The normal traffic should have to be controlled during the loading operation and the costs of traffic control depend on the size and traffic volume of the bridge, length of time required to perform the test, and work zone control guidelines effective at the bridge location.

- Sensors and other instruments (C_S). Depending on the test objectives the instrumentation and sensors quantity can be variable but are one of the most important sources of costs.

According to the most appropriate instrumentation as explained in the previous chapters, some indicative costs are provided:

- Strain gauges: the cost of 1 measuring grid is among 80 to 130 € for package of 10 gauges. For 2 measuring grids, a cost of 100 to 150 € for package of 5 gauges can be estimated. The cost of a Kit for strain gauge deployment is around 1000 €.
- Inductive mechanic gauges: 500 €/unit
- Displacement transducers and inclinometers: 1000 €/unit
- Load cells: 3000 €/unit
- For field tests, a compact data acquisition system is also advisable, with an approximate cost of 2000 €
- Data analysis costs (C_D). The characteristic of the bridge and the documentation available, will affect the complexity of data analysis, modelling and calculation. These costs of their modelling, calculation and analysis of data obtained in the load test will give the costs of analysis.
- Labour costs (C_L). The labour cost will be directly related with the length of time required to perform the test and the quantity of operators needed. This quantity will depend basically on the type of bridge and its characteristics and, also, the type of load test to execute.
- Loading costs (C_W). These costs will depend basically on the material used to load the bridge, trucks or other elements (explained above), cost of drivers, and, also, the location of the bridge, equipment relocation costs, etc.
- Other miscellaneous costs (C_{MISC}). Remaining costs required to execute the test not mentioned above.

Therefore, the total cost of a bridge load test can be estimated as (Alampalli & Ettouney, 2010):

$$C_{LOAD\_TEST} = C_T + C_S + C_D + C_L + C_W + C_{MISC} \tag{7.6}$$

For a short to medium bridge (total length between 10 to 60 meters) an estimate of the total cost is around 50,000 €.

7.3 WIM measurements

Approximate cost of WIM measurements for railway loads may be around 40,000 €.

8. Summary and conclusions

As reported in Wisniewski et al. 2012, several years of research at National, European and International levels, including several European Projects, as well as practical implementations of these concepts on specific projects have demonstrated the benefits of incorporating advanced assessment and load rating in bridge assessment codes. However, the proposed advanced assessment methods for **bridges** as presented in the previous chapters are not yet included in most of the codes and recommendations or national or international regulations, where a standard basic assessment is normally applied. However, several countries have already included in their codes the possibility of using to some extent the proposed methods. In table 8.1 there is a summary of the advanced methods by country. As can be seen there, at the present moment, none is fully implementing all possible choices. It also becomes evident that USA and Canada are the countries where more of the proposed advanced methods are considered.

Table 8-1 Advanced assessment for bridges in different countries

	System behaviour	**Probability-based assessment**	**Proof loading**	**Up-dating Dynamic effects**	**Up-dating (materials, loads, models)**
Canada	yes	yes	yes	no	yes
USA	yes	yes	yes	no (to some extent)	yes
Denmark	no	yes	no	no	yes
Switzerland	no	yes	no	yes	yes
UK	no	yes	no	no	yes
Germany	yes	yes	yes	no	no

The generalised soil slope condition assessment algorithm developed by SKM within MAINLINE looks as the most advanced algorithm nowadays for the assessment of soil cuttings. In its present form it is just suitable for cohesive soil cuttings only. However, based on the methodology proposed, the prototype algorithm could be further developed and extended to cover other soil types and also rock cuttings. A complete description of the algorithm can be found in Deliverable D2.2 of MAINLINE.

The decision on using or not the proposed assessment methods presented in this deliverable will depend on the minimization of the total expected cost C_{ET}:

$$C_{ET} = C_T + C_{PM} + C_{INS} + C_{REP} + C_F$$

where C_T is the construction cost, C_{PM}, the expected cost of routine maintenance, C_{INS} the expected cost of inspection, C_{REP} the cost of repair, and $C_{F,}$ the expected cost of failure. The cost of failure includes both direct and indirect cost, such as user costs and others.

According to figure 8.1 (same as figure 4.6), advanced assessment allows to better know the state of a structure in the time where a decision should be taken, and, therefore, it can be

seen as a part of an inspection and therefore as part of the costs of such inspection, C_{INS}. Given that the ultimate goal is to know the exact state of the structure, any additional inspection costs resulting from advanced assessment could be rewarded with a considerable decrease of repair costs and, in turn, in a reduction of life-cycle costs.

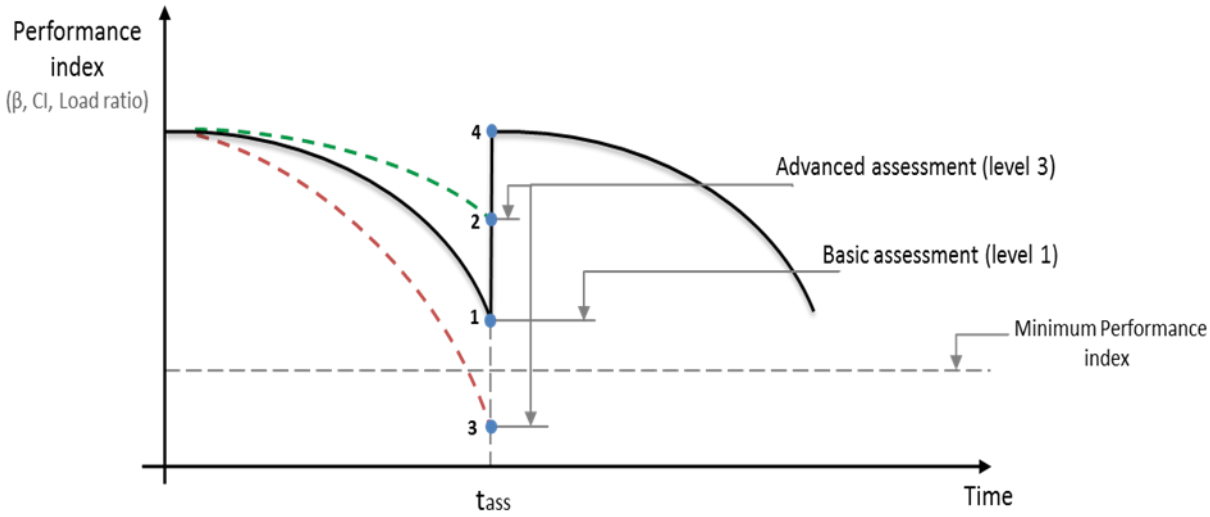


Figure 8-1 Model of life cycle condition of an asset with repair and renewal interventions taking into account the result of an advanced assessment. More comments are given in Section 4.2.1

Therefore, in the framework of a LCA, advanced assessment has to be considered as the first alternative, being the feasibility of its application decided within the same analysis where degradation and repair/strengthening have to be considered.

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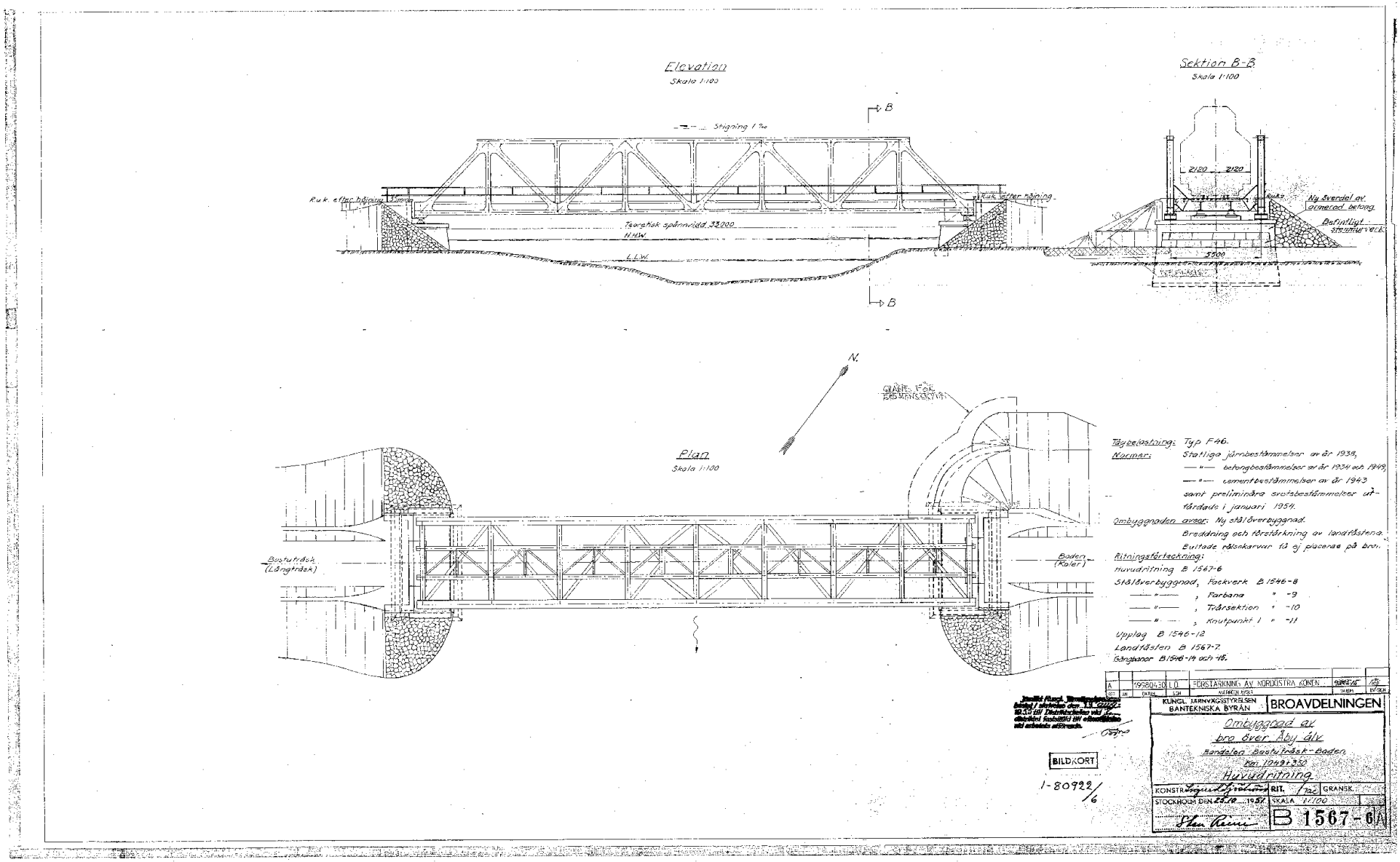
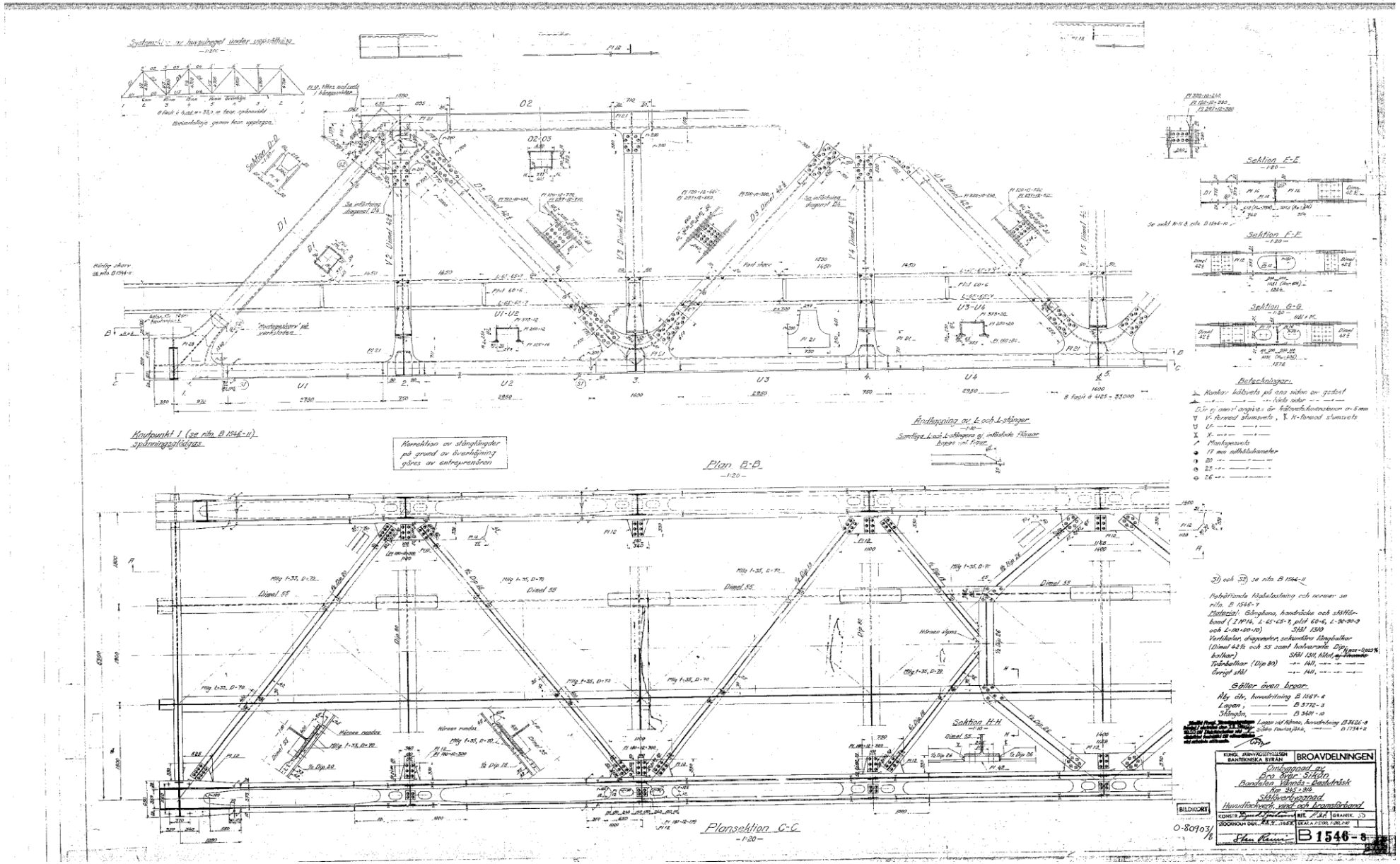
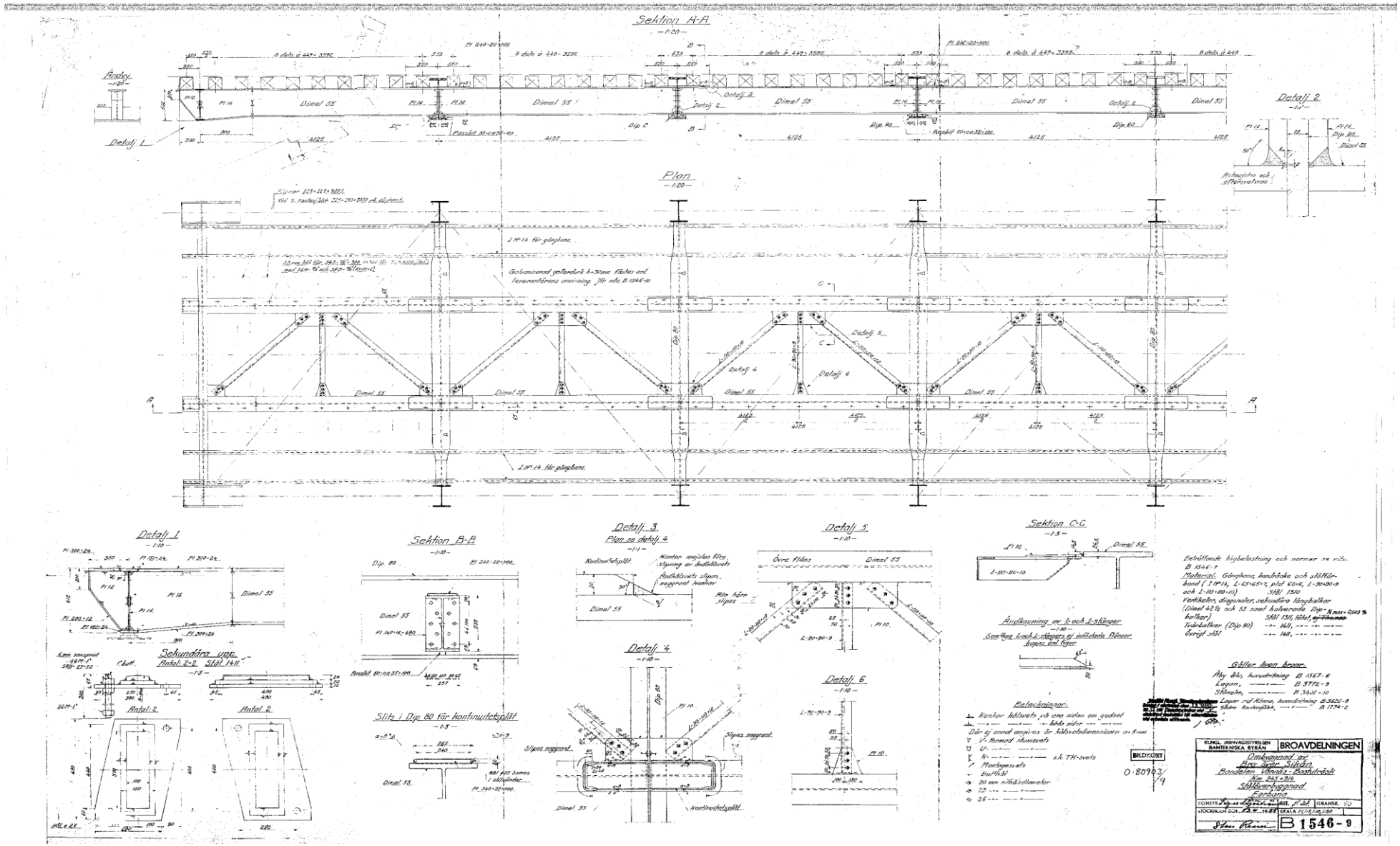
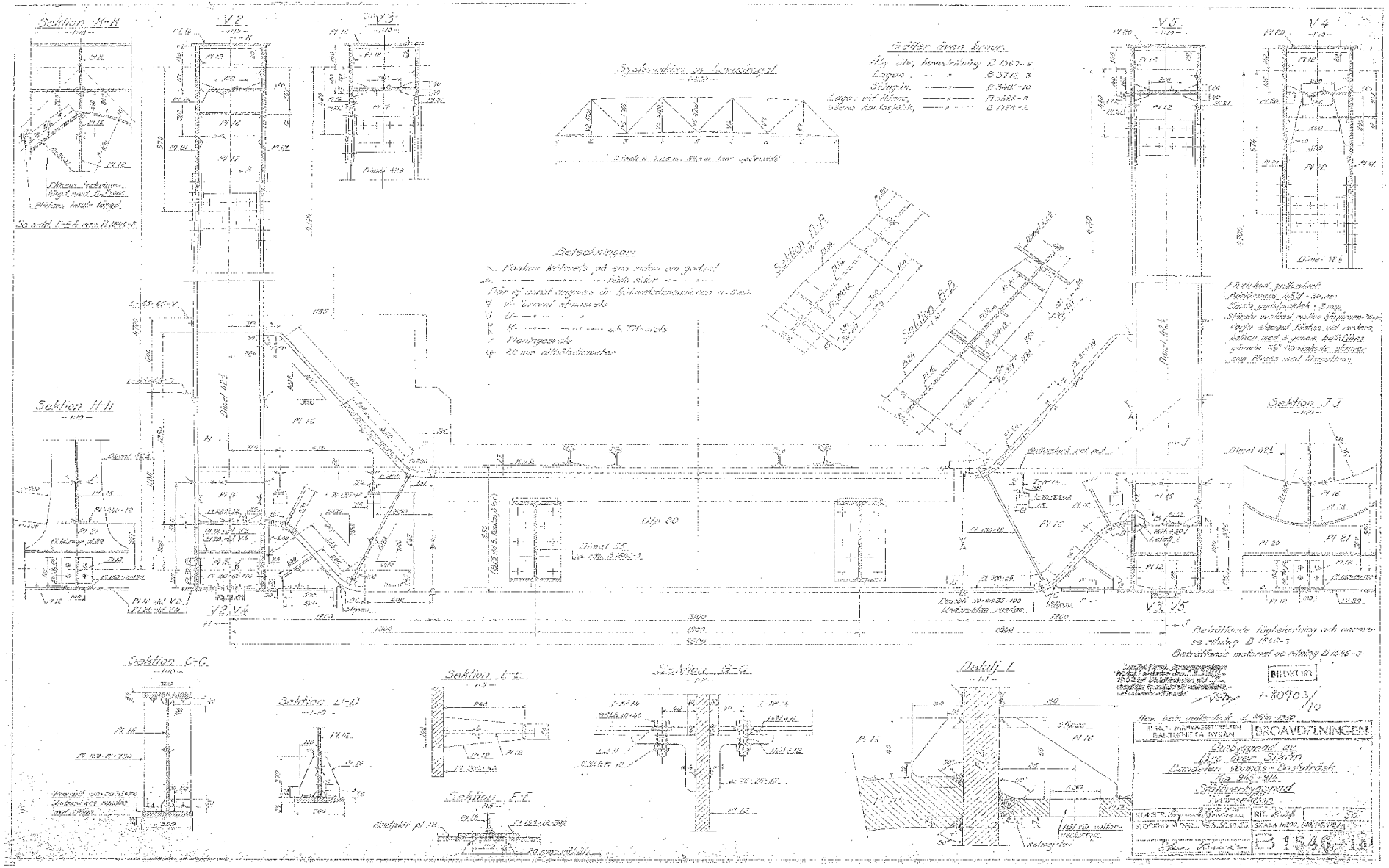
10. APPENDIX A – DRAWINGS OF ABY BRIDGE









Material: Gångbana, handräcke och stöttförband (I Nº 14, L-65×65×7, plåt 60×6, L-90×90×9 och L-110×110×10) Stål 1310
Vertikaler, diagonaler, sekundära långbalkar (Dimel 42½ och 55 samt halverade Dip-balkar) Stål 1311, tätat, ~~ej Thomas~~ N max = 0,009 %
Tvärbalkar (Dip 80) —»— 1411, —»— —»— —»—
Övrigt stål —»— 1411, —»— —»— —»—

Gäller även broar:

Åby älv, huvudritning B 1567-6
Lagan, —»— B 3772-3
Stångån, —»— B 5401-10
Lagan vid Kånna, huvudritning B 3626-8
Södra Routasjökk, —»— B 1734-2

Kungl. Järnvägsstyrelsen beslöt den 13 okt. 19 Distriktschefen vid ... fastställd till efterrättelse vid utförande.

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BROAVDELNINGEN

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Farbana

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STOCKHOLM DEN 23.4. 1955 SKALA 1:1, 1:5, 1:10, 1:20
Sten Reini
B 1546 - 9