SAFERELNET

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SAFERELNET TN Work Package 6

leaded by

Christian Bucher (Bauhaus-University Weimar, Germany)

christian.bucher@uni-weimar.de

State-of-the-Art Report on Assessment and Life Extension of Existing Structures and Industrial Plants

Weimar, March 2006

contributing authors in alphabetic order:

Christian Bucher, Helen Bolt, Maik Brehm, Iolanda Craifaleanu, Matthias Ebert, Milan Holický, Valery Just, Andrzej Kowalski, Oliver Kübler, Andre Lannoy, Michael Macke, Paulo Maia, Jana Marková, Ole Tom Vardal, Ton Vrouwenvelder, John Williams

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PREFACE

As existing facilities / structures are modified, as engineering knowledge advances and as the requirements to extend life increase, industries must demonstrate that operations can continue safely and economically. There is a general recognition across all industrial sectors that this reassessment process is different from the design process. As a minimum, the known conditions and specific functional requirements of existing facilities / structures need to be taken into account (with design 'uncertainty factors' removed where site-specific parameters are available from as-built information and inspections).

Nowadays, many very specific rules and guidelines for special problems are available in certain industrial sectors. Therefore, most of them are neither general enough to use them in another context like a different industrial sector nor do they reflect the entire current state-of-the-art. Due to an increasing number of ageing structures, facilities, systems, and industrial plants, there is a high potential to save a substantial amount of money with a more comprehensive approach.

This report tries to close this gap by presenting the best practice of several industrial sectors and the current theoretical state-of-the-art in one framework for the cost optimal assessment of existing structures consistent with the available information and such that any requirement for the safety of the facility or structure is achieved. It is formulated as general as possible in order to be applicable in each industrial sector for each reassessment problem.

The report is divided into three parts. 'Assessment Methodology' is the first part, that describes the current state-of-the-art of assessment methodologies, reviewing of current codes, inspection techniques and sensor technology, a list of deterioration mechanisms of several materials and exposure on structures, system modelling and generic computational approaches, considerations on decision-making and cost-benefit analysis, and aspects of life time extension. The second part deals with the acceptable safety levels of reassessment. Different approaches to define the target safety levels are explained and a review of the methods used in different industrial areas is given. Some case studies from several industrial sectors are included in the third part of this report.

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Christian Bucher (BUW) Helen Bolt (BOMEL)

This part has been delivered within

SAFERELNET Task 6.1

leaded by

Christian Bucher (Bauhaus-University Weimar, Germany)

christian.bucher@uni-weimar.de

PART I

Assessment Methodology

Document Status

Authors

Name	*Company*
Christian Bucher	Bauhaus University Weimar (BUW)
Matthias Ebert	Bauhaus University Weimar (BUW)
Michael Macke	Bauhaus University Weimar (BUW)
Maik Brehm	Bauhaus University Weimar (BUW)
Ton Vrouwenvelder	Technical University of Delft (TUD)
Ole Tom Vardal	Aker Offshore Partner, Norway (AOP)
Andre Lannoy	Electricité de France, France (EdF)
Valery Just	Electricité de France, France (EdF)
Andrzej Kowalski	Polish Register of Shipping Joint, Poland (PRS)
Iolanda Craifalenau	National Institute for Building Research (INCERC)

Reviewing/Approval of report

Name	*Company*	*Signature*	*Date*

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Table of Contents

EXECUTIVE SUMMARY

The specification of suitable methods related to structural problems in the individual sectors as well as the specification of suitable methods related to non-structural problems in the individual sectors are still in their early stages of development. Consequently, this task aims at establishing a theoretical framework for assessment of existing structures and industrial plants. It attempts to establish "generic assessment problem formulations" which have general applicability for solving assessment problems across the different industrial application areas. This work is focused on structural assessment in several industrial areas. However, no risk assessment procedure will be presented.

According to the objectives, this document reflects the current state-of-the-art in the area of structural assessment. This document contains in detail a description of available codes and its gaps applicable in this area as well as recommendations for improvement, a classification of structural components and materials, a description of demands on structures, and a list of experimental assessment methods and inspection techniques useful for structural assessment. Moreover, an universal cost-benefit analysis as decision making procedure within an assessment is investigated. Finally the assessment methodology can dominantly contribute to an extension of the lifetime of any existing structure, facility, industrial plant, system, process, and product.

1 INTRODUCTION

Today there is no doubt that the assessment of existing structures, maintenance, and the subsequent reconstructions are one of the most important business fields in the construction industry. Approximately 50% of all outputs of the UK's construction industry are from repair and maintenance [44]. Mainly responsible for this is a big construction boom between the late 60s and the early 80s. Hence, many European structures and facilities are now reaching the end of their design lifetimes of 25 to 100 years and need to be assessed to assure their continued safe and economical operation. Figure 1.1 shows the construction time of bridges across Europe. As a result, there is a considerable interest evident in the potential to extend their lifetimes and in developing efficient procedures for the assessment of existing structures and facilities.

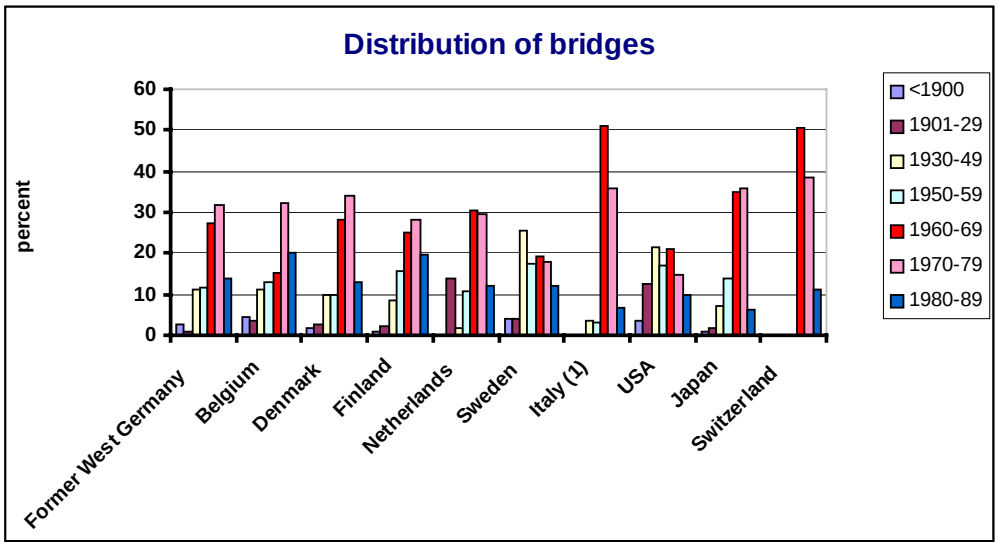


Figure 1.1 Construction completion of road bridges /OECD, 1992/, /Di Mascio et al, 1998/.
Note (1) indicates that data are only for the bridges controlled by Autostrade SPA. [14]

The assessment and life extension of existing structures and industrial plants is nowadays an integrated process which contains a wide range of aspects to be considered. In Figure 1.2 the position of assessment and life extension within an integrated process is described. Risk management ([1]), risk assessment, and maintenance ([8], [9]) are the topics of other work packages of SAFERELNET, which assure not only a detailed view on assessment, but consider its complexity as well.

Obviously, there are some necessities and advantages of rehabilitation and life time extension of a structure or facility in comparison to the alternative of demolishing and reconstructing them: a) historical buildings, like churches, monuments, or bridges, are often listed buildings and have to be maintained, b) some buildings are associated with a social responsibility, c) the saving of natural resources, and d) the possibility to reuse monitored data and assess the current structure and its exposure.

In spite of those advantages, the owners of constructions hesitate to rehabilitate their structures. Very often a new structure is preferred. In average the costs are approximately 10 times higher to increase the reliability level or achieve a specific requirement of an existing structure than to build a new structure with the same reliability level and requirements.

As a consequence, it has to be shown, how the assessment and life extension of existing structures can be more beneficial. Some ideas are, for example: a) the use of additional information about the structure and facilities obtained by experimental assessment, monitoring or other historical data to reduce uncertainties in the models for calculations. b) the introduction of full probabilistic modelling in every design and redesign c) an improvement of inspection and investigation methods, and d) advanced cost-benefit analysis and decision making.

Consequently, a framework at optimal cost is aspired. In this document, the current state-of-the art is reviewed, whereas not only current standards and codes are considered. Innovative processes will be discussed as well. Section 2 reviews the current most frequently used standards and guidelines. In Section 3 the best

aspects are used to develop the recommended framework for assessment. Possible deterioration of material and damage on structural parts is described in Section 4. Section 5 investigates the possible excitation of a structure or facility. Experimental assessment methods including sensors and system modelling are described in Section 6 and Section 7, respectively. Some generic computational approaches are given in Section 8, but a more detailed description can be found in [6]. One of the key tools for an assessment, the decision and cost-benefit analysis, is explained in the ninth section. Some aspects of extending the life time of a structure or facility after the assessment itself are given in Section 10. Conclusions and references are stated in the last two sections.

The important aspects of monitoring are excluded from this review, because the thematic network SAMCO [13] funded by the European Commission, is strongly focused on these issues. Interested readers are therefore referred to the corresponding documents.

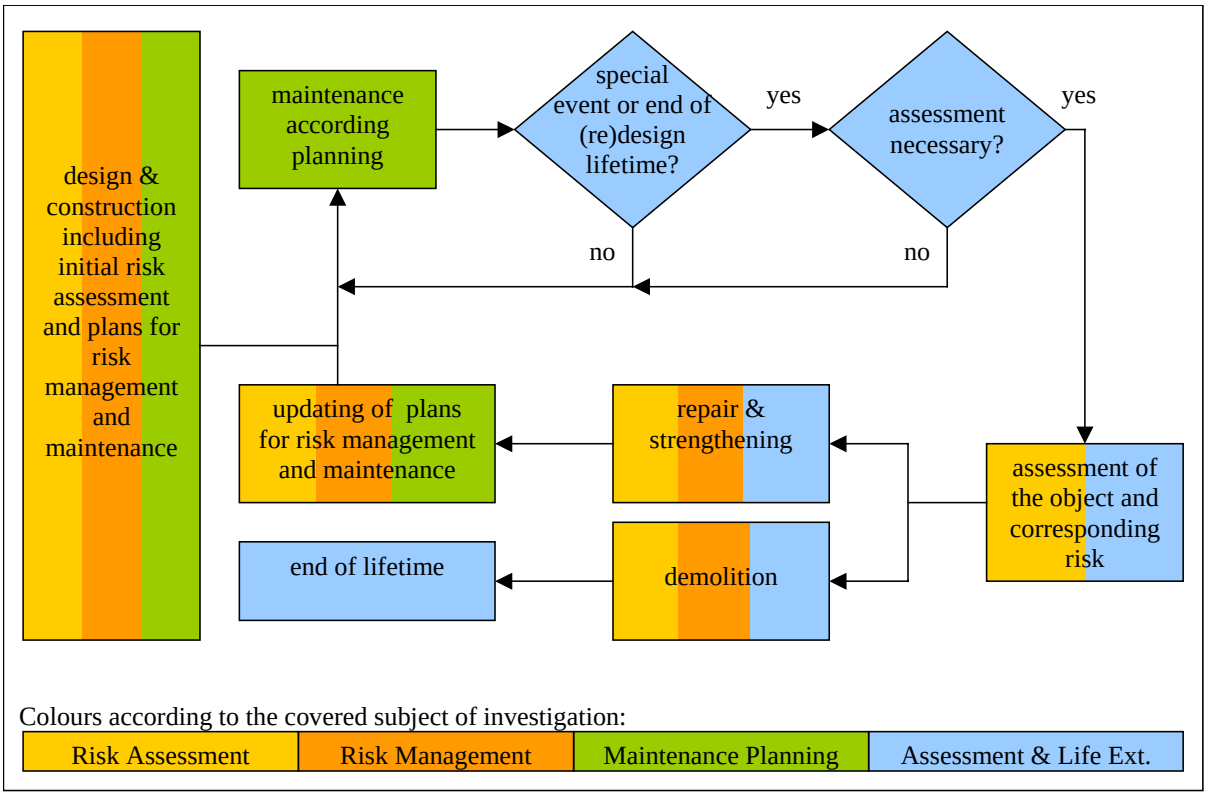


Figure 1.2: Position of the work of work package 6 in an integrated facility management process

2 GUIDELINES, CODES, AND STANDARDS – A REVIEW

This section reviews the state-of-the-art represented in common most developed guidelines, codes, and standards in the area of the assessment of existing structures, industrial plants, products, and systems.

Table 2.1 gives a general overview of the current state-of-the-art of common rules, codes, and standards sorted by industrial sectors and investigation topics. This list is not exhaustive.

Table 2.1: Overview of the state-of-the-art in several industrial sectors
(for more details see WP8 [10])

Industrial sector	Modification of Target safety levels, refinement of uncertainties or partial safety factors	Guidelines for assessment of existing facilities / structures	Cost – Benefit – Aspects
Overall sectors	JCSS guide [43]; ISO 2394 [26]	JCSS guide [43]; ISO 2394 [26]	JCSS guide [43],
Offshore Oil and Gas	NPD [63]; Very crude suggestions by API [61]	ISO 19902 (DRAFT) [16]; ISO 2394 [26]; ISO 13822 [29]	
Marine Transportation	ANSI/ASME B31G; DNV RP-F101		Jonkman 2003 [60]
Buildings	Reduction of partial safety factors [54]	ISO 13822[29]; ISO 2394 [26]; national guidelines	JCSS guide [43]
Motorways and Bridges	- Very crude assumptions by introducing a condition factor (BD21/93 [31], BD44/95 [32]) - Reduction of resistance to consider deterioration (limited to non visible deterioration) [30] - Rodriguez et al. [75]	- bridge management systems PONTIS [80], DANBRO [35] and BRIME - theoretical models ([38],[47]) - further special national guidelines - ISO 2394 [26]	JCSS guide [43]
Process Industry		Special company intern guidelines (e.g. Shell [77])	
Power Plants	Lower target safety level for existing nuclear power plants than for future INSAG-3 [56]/ INSAG-12 [57]	- Special company intern guidelines (e.g. CPPE) - for nuclear installations: Safety Guideline [55] and French rules RCC-M	
Aircraft		European (UK, France, Germany) guidelines ([46], [59])	

A more detailed review considers the following main aspects:

1. Clear definition of **scope** and applicability of the framework. If needed, the kind of material, structure or specific requirements should be defined. Furthermore, the industrial application can be mentioned.

2. Definition of used **terminology** (e.g., structural assessment, reassessment, maintenance, lifetime). This is necessary, because the meaning of some words is not the same in every industrial sector. Otherwise, the guideline, code, or standard can lead to confusion and finally to mistakes.
3. Definition of the **triggers** of an assessment. The guideline, code, and standard should indicate, which requirements have to be fulfilled to start with an assessment. This is important to ensure safety on the one hand and avoid high costs on the other hand.
4. Providing of a general **overview** (e.g., flow chart) with detailed explanations of each step. It is necessary to provide an overview in graphic and text to lead the user and avoid mistakes.
5. Consideration of **economical, environmental, social, and cultural aspects** in each assessment step. An assessment always tries to reach a predefined target safety level with a minimal effort. To ensure long-time profitability, not only economical aspects have to be met, environmental, social, and cultural aspects are important as well.
6. Consideration of **additional information**. The knowledge about an existing object is normally higher than about an object which is going to be designed. Hence, the additional information can be used to refine assumptions (e.g., load, material resistant, mathematical approaches) and reduce uncertainties. Finally, this increase of knowledge will increase the safety and in an optimal case reduce the costs.
7. Regulation of **responsibilities** in each assessment step to evade conflicts in case of defects and failures.
8. **Documentation and reporting** of results. This is very important to give the responsible decision maker all necessary information in a very clearly arranged way. Furthermore, a good documentation is the presupposition of reusing results.
9. **Reuse of results** of the assessment will directly save costs and shall be used as often as possible and shall be recommended in every code.

The following subsections present the result of the review of ISO/DIS 13822 [29], Probabilistic Assessment of Existing Structures (JCSS) [43], and ISO/DIS 19902 [16].

2.1 ISO/DIS 13822

The Draft International Standard ISO/DIS 13822 (16-8-2000) “Bases for design of structures – Assessment of existing structures” is described in this section.

2.1.1 Scope

This draft standard provides general requirements and procedures for the assessment of existing structures, including buildings, bridges, industrial structures, based on the principles of structural reliability and consequences of failure. The recommended basis for practical reliability assessment is ISO2394 (General principles on reliability for structures) [26]. Further recommended codes are ISO2854 (Statistical interpretation of data – Techniques of estimation and tests relating to means and variances) [27] and ISO12491 (Statistical methods for quality control of building materials and components) [28].

2.1.2 Terminology

According to (ISO/DIS 13822):

Maintenance: *Routine intervention to preserve appropriate structural performance.*

Assessment: *The set of activities performed in order to verify the reliability of an existing structure for future use.*

Inspection: *On-site non-destructive examination to establish the present condition of the structure.*

Monitoring: *Frequent or continuous, normally long-term, observation or measurement of structural conditions or actions.*

Lifetime: no remarks found

According to (ISO2394):

Maintenance: *Total set of activities performed during the design working life of a structure to enable it to fulfil the requirements for reliability.*

Assessment: *The set of activities performed in order to find out if the reliability of structure is acceptable or not.*

Inspection: no remarks

Monitoring: no remarks

Lifetime: no remarks

2.1.3 Trigger

Initiators of an assessment are circumstances like,

- *an anticipated change in use or extension of design working life*
- *a reliability check (e.g., for earthquakes, increased traffic actions) as required by authorities, insurance companies, owners)*
- *structural deterioration due to time-dependent actions (e.g., corrosion, fatigue)*
- *structural damage by accidental actions*

others, *like the preservation of the historical appearance of the structure and the preservation of its historical materials.*

2.1.4 General overview

A proposal for the general framework of assessment is explained in Chapter 4 of the standard, which refers to the flow chart in Annex B (Figure 2.1). The easily traceable description includes the most important steps. In the explanations, the use of additional information and cost-benefit aspects is briefly mentioned. It is not foreseen to use the experience or knowledge from similar objects.

2.1.5 Economical, environmental, social, and cultural aspects

At the end of the assessment, cost considerations are taken into account. *The cost and risk associated with each of the interventions should be estimated.* However, recommendations for an advanced cost-benefit analysis are not given and not mentioned in the flow chart.

Socio-economic criteria might influence the target reliability level for existing structures. Furthermore, the standard agree, that economic, social, and sustainability considerations are more important for the assessment existing structures than for the design of new structures. The appearance and materials of historical structures shall be preserved.

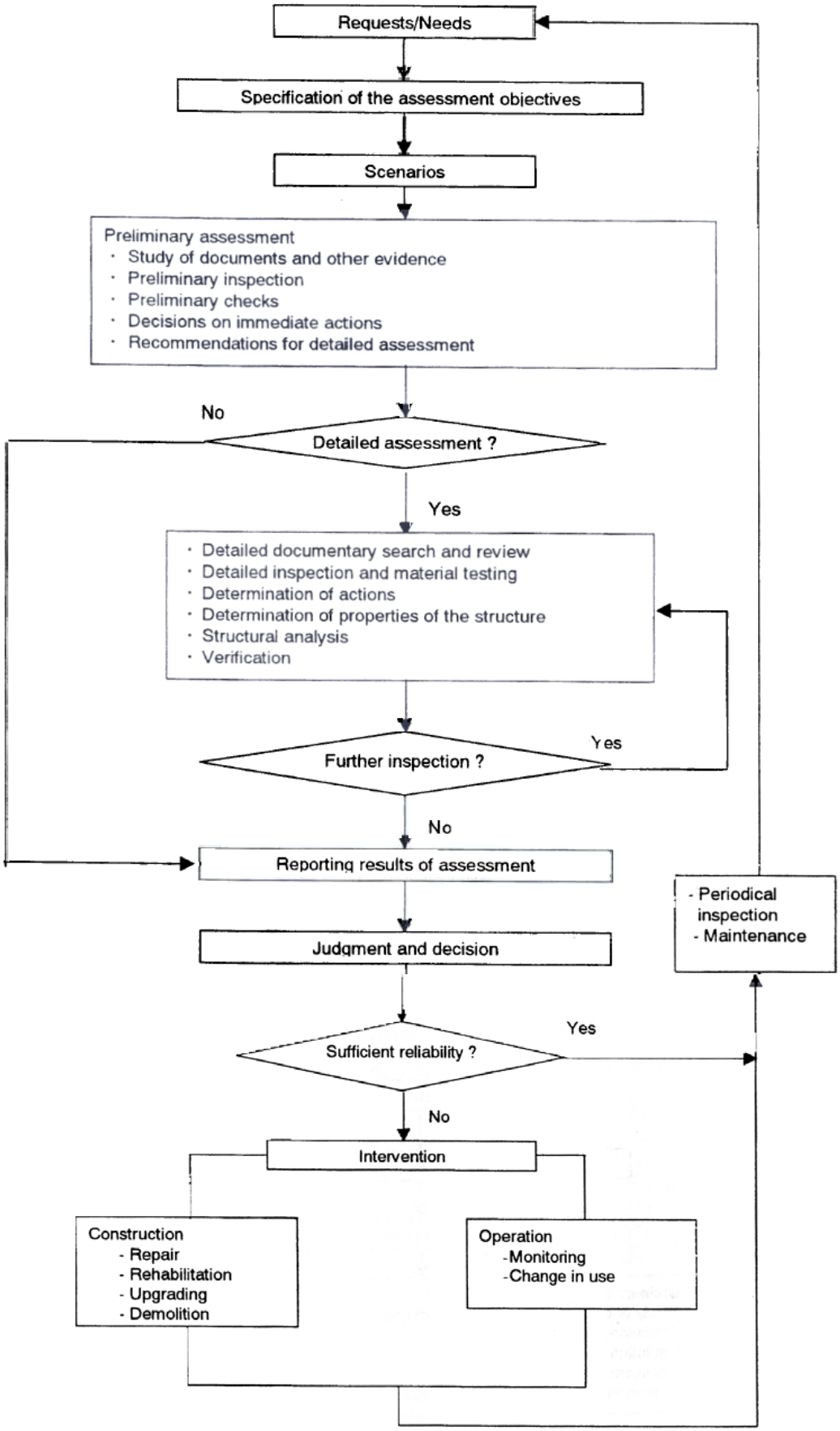


Figure 2.1: General flow chart of an assessment of existing structures, Annex B of [29]

2.1.6 Additional information

The implementation of additional information during an assessment by means of inspections is mentioned. *Partial safety factors given in current codes may be modified to take into account the inspection and test results (concerning e.g., quality of workmanship, conditions of maintenance, strength variation of materials).* In addition, *lower target reliability levels for existing structures may be used if they can be justified on the bases of socio-economic criteria.* There is no reference or additional information to realize this approach in reality.

2.1.7 Responsibility

The responsibilities are distributed to the engineers and the client or owner. In general, the engineers have to indicate the preferred solutions, but the client makes the decision in collaboration with the relevant authority. Only if the decision is not covenant with common societal and safety rules, the engineer is allowed to announce a higher authority. In danger, the engineer is authorized to make fast decisions to guarantee a minimum level of safety.

2.1.8 Documentation and reporting

It is mentioned that reports can be necessary after each assessment step. For a final report a table of contents and a short descriptions to each part is provided by the standard. The main parts are Title page; Name of engineer and firm; Synopsis; Table of contents; Scope of assessment; Description of the structure; Investigation; Analysis; Verification; Discussion of evidence; Review of intervention options; Conclusions and recommendations; Reference documents and literature; and Annexes.

2.1.9 Reuse of results

Main emphasis is spent in the report to the results and the improvement of the inspection and maintenance planning of the actual object. *In all cases, an inspection and maintenance plan during the remaining working life should be specified depending on the results of assessment and the utilization plan, and submitted to the client. The date or conditions for the next assessment should be recommended.* An impact to existing similar or new designs is not mentioned.

2.1.10 Conclusion and remarks

The draft international standard ISO/DIS 13822 describes the principles of the assessment of existing structures very well. Special problems like economical aspects or consideration of new information is mentioned but definitively not applicable in practice only with this or the referred standards. The results of the assessment are used for the assessed object, but not for similar or further new objects.

The Annexes about “Updating of measured quantities“, “Testing for static and dynamic properties of structures”, “Assessment of time-dependent reliability”, “Target reliability level”, and “Design of upgrading” give some useful additional practical background information to perform an assessment. References are provided for further detailed information and examples.

2.2 Probabilistic Assessment of Existing Structures (JCSS)

Even though the guideline ‘Probabilistic Assessment of Existing Structures’ [43] is not legally confirmed, it is one of the most frequently used documents in practice to perform a safety assessment of existing structures. *The nature of the report is educational, it contains practical and operational recommendations and rules for the assessment of existing structures.* The description is given below.

2.2.1 Scope

This guideline is applicable to structures of different types. Unfortunately, no more specific explanations are provided.

2.2.2 Terminology

Maintenance: *is defined as a set of activities that are carried out to retain or restore a structure in an operable state*. The three types are: Corrective Maintenance (*no inspection is carried out and repair is done after failure has occurred*), Preventive Maintenance (*no inspection is carried out, but replacement or maintenance at a time that no failure has occurred*), Condition Based Maintenance (*inspections are planned in advance and when measured parameters no longer meet prescribed criteria repair or replacement must be carried out*).

Assessment: *The assessment of the reliability of an existing structure aims at producing proof that it will function safely over a specified residual service life. The reliability assessment is mainly based on the result of assessing hazards and load effects to be anticipated in the future and of assessing material properties and geometry taking due account of the present state of the structure.*

Reassessment: The guideline uses 'assessment of existing structures' instead.

Inspection: *Is an investigation intended to update the knowledge about the present condition of the structure.* It can be distinguished: qualitative inspection and quantitative inspection

Monitoring: *A continuous observation with the use of technical equipment of important parameters affecting the overall behaviour of the structure such as vibrations, deformations etc.*

Lifetime: In this guideline the term 'Residual Service Life' is used to describe the lifetime of a structure regarding the serviceability limit state. It is defined as the period of time a *structure is intended to serve its purpose*.

2.2.3 Triggers

The presented assessment initiators are:

- *Deviations from the original project description are observed*
- *Adverse results of a periodic investigation of its state*
- *Doubts about the structural safety caused by evidence of damage*
- *Unusual incidents during the use (such as impact of vehicle, avalanches, fire in the building, earthquakes), which could have damaged the structure*
- *A clearly inadequate serviceability*
- *Suspicion of possible impairment of the structural safety related to building materials, to construction methods or to the statical system*
- *The discovery of the design or construction errors*
- *A planned change of the use of the structure*
- *The expiry of a residual service life granted on the basis of an earlier assessment of the structure*
- *Simply because of doubts about the safety of the structure*

2.2.4 General overview

The assessment of a structure can be divided into three phases. Figure 2.2 shows the relation between the phases.

Phase I is called 'Preliminary Evaluation' and aims to remove existing doubts using fairly simple methods such as: (a) visual inspection (qualitative inspection), (b) review of existing documentation, (c) compatibility with new codes, (d) evaluation of possible changes during the passed lifetime (new loads or actions), and (e) simplified assessment of actual condition of the structure (e.g., scaling factors by weighting important parameters). Within Phase I the 'Knowledge-Based Assessment', which needs no structural analysis, and the 'Redesign' as a reanalysis is applied.

In Phase II more detailed assessment is arranged comprising: (a) *Site investigation including quantitative inspections*, (b) *updating of information gained through inspection using statistical procedures*, (c) *detailed*

structural analysis on conventional or advanced tools (limit state analysis, nonlinear material behaviour, redundancy of structure), and (d) *reliability analysis to determine the safety of the structure or the probability of failure of the structure or of its most critical components*. Following kind of assessment can be distinguished inside the Phase II for reliability analysis: 'Degree of redundancy', 'Component reliability', and 'System reliability'.

In Phase III, 'Calling of a team of experts', problems with large consequences in terms of risk or of costs are identified and the proposals obtained in Phase II are carefully investigated.

This is shown in the flow chart below

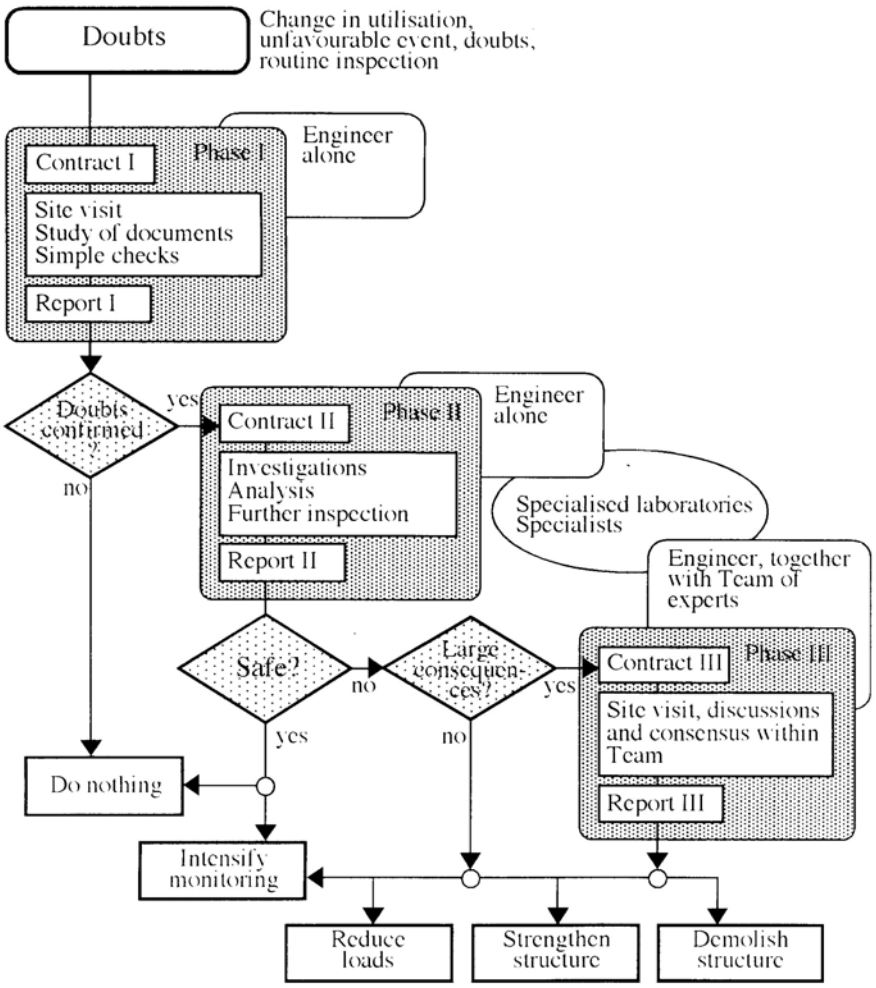


Figure 2.2: Illustration of the three phases approach [43]

2.2.5 Economical, environmental, social, and cultural aspects

The potential consequences in terms of direct and indirect costs are evaluated. Economical considerations normally take into account: (1) expected benefits from residual use of structure (2) associated commitments and costs including costs related to engineering and structural analyses, costs related to repair work, and costs related to planned inspection and maintenance. A special part of this guideline is focused on cost-benefit analysis for simple to medium cases.

Environmental, social, and cultural aspects are briefly mentioned and declared as necessary, but not explained in detail.

2.2.6 Additional information

Updating is based on prior information and collected observations and measurements, It results in posterior information that serves for assessing the structure.

Especially in Phase II the updating of information gained by inspection using statistical procedures is recommended. Analytical methods for updating are described.

Updating of information should be performed based on state-of-the art of reliability analysis. Two different types are distinguished: (1) updating of individual random variables due to measurements, observations related to the individual variable based possibly on Bayesian techniques (2) updating of failure probability by conditioning (i.e., conditional failure probabilities due to measured cracks or due to survival of extreme loads).

Basically, it is agreed that target safety levels for design and assessment are different, but no specific approach is proposed. Finally, it is recommended to use the same target safety level for design and assessment. Target reliability indices are given depending on the consequence of failure and the relative cost of safety measure.

2.2.7 Responsibility

As a rule: Who profits the risks, has to bear the consequences. A 'List of Accepted Risks' shall be established to clarify the responsibilities in detail. This list shall be signed by all involved parties.

The owner is responsible for initiating an assessment. It is recommended to use the experience of a consultant engineer. The engineer formulates a statement on the safety of the structure and proposals for possible solutions and consequences. The owner has to take the final decision.

2.2.8 Documentation and Reporting

After each phase, all results are summarized in a report for the owner. In particular, the report contains all necessary information on the structural safety and conclusions with recommendations for further steps.

2.2.9 Reuse of results

No comments are given.

2.2.10 Conclusion and remarks

This guideline is very useful for researchers and practical engineers. It includes a general description and specific mathematical approaches for a realistic application and recommendations for further reading. Some examples and case studies cover different problems faced in the safety assessment of existing structures. Even though this guideline is strongly focused on structures, practical applications show a wider range. A special part of the guideline focuses on the reliability updating and decision making especially for inspection and test decisions.

All investigated aspects are considered, except the reuse of results. There is no comment about the reuse of information for similar objects. Approaches to adjust the target safety level are missing if new or additional information (e.g., by monitoring or measurements) is available. The proposed cost-benefit analysis is only applicable for simple and medium examples.

2.3 ISO/DIS 19902

2.3.1 Scope

Section 25 of ISO/DIS 19902 *gives procedures for the assessment of existing fixed steel offshore structures to demonstrate their fitness-for-purpose. The aims and procedures are also applicable to topside structures.*

2.3.2 Terminology

Maintenance: *set of activity performed during the working life of the structure in order to enable it to fulfil the requirements for reliability. Activities to restore the structure after an abnormal, accidental, or seismic event are outside the scope of maintenance.*

Assessment: no explicit definition available

Reassessment: no explicit definition available

Inspection: no explicit definition available

Monitoring: no explicit definition available

Lifetime: no explicit definition available

2.3.3 Triggers

An existing structure shall be assessed to demonstrate its fitness-for-purpose if one or more of the following conditions exist:

- *Changes from the original design or previous assessment basis*
    - *Addition of personnel or facilities such that the platform exposure level is changed to a more onerous level*
    - *Modification to the facilities, such that the magnitude or disposition of the permanent, variable or environmental actions on a structure are more onerous*
    - *More onerous environmental conditions and criteria*
    - *More onerous component or foundation resistance data and criteria*
    - *Physical changes to the structure's design basis (e.g. excessive scour or subsidence)*
    - *Inadequate deck height, such that waves associated with previous or new criteria will impact the deck and provided such action was not previously considered*
    - *Extending the original design service life*
- *Damage or deterioration of a primary structural component"*

2.3.4 General overview

An overview of the assessment is given in Figure 2.3.

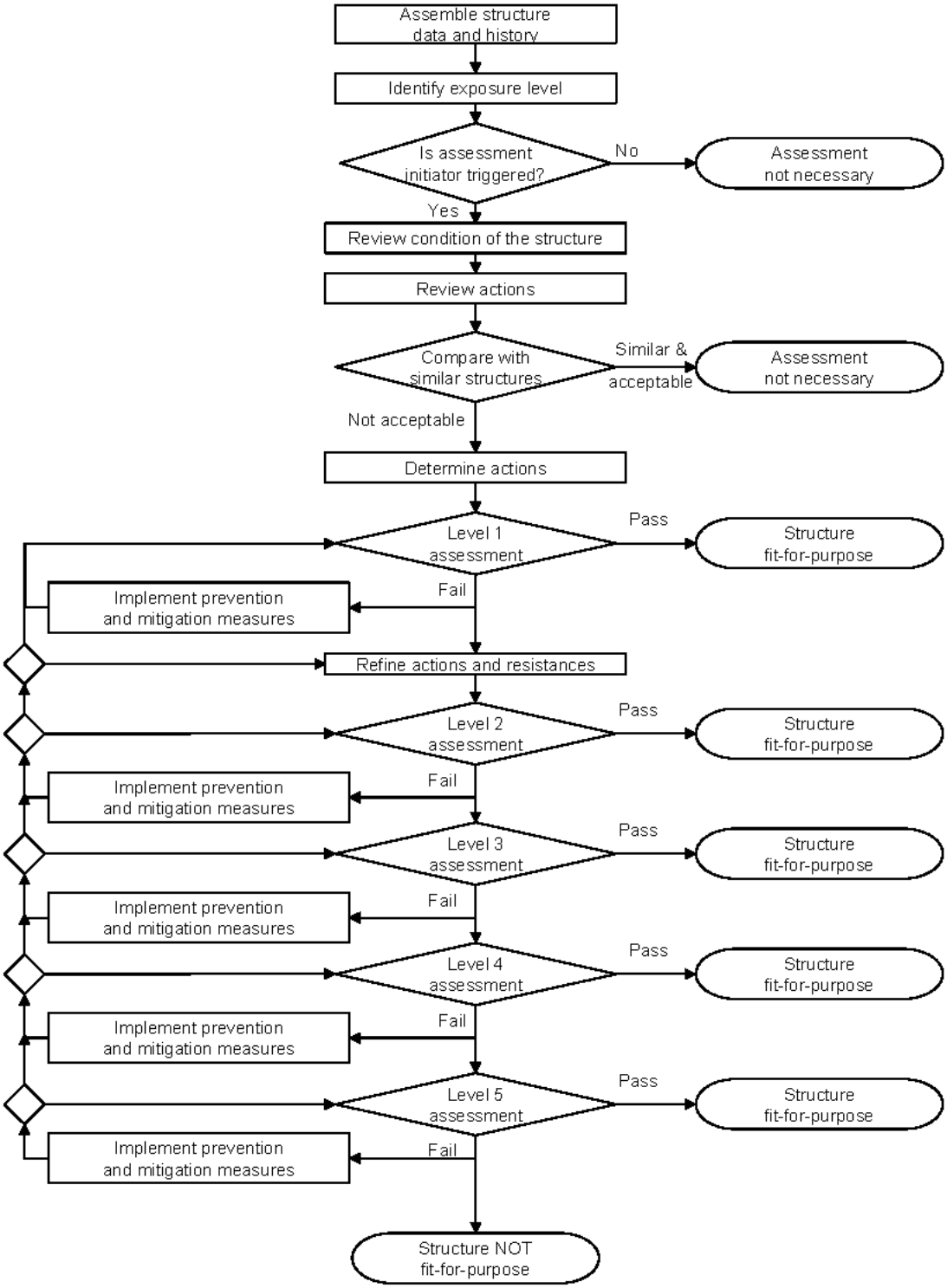


Figure 2.3: General Overview [16]

The main elements in the process of initiating and conducting an assessment of a structure are:

- *Assemble data on structure, its history and exposure level*
- *Determine if assessment initiators is triggered*
- *Review condition of the structure, review actions and compare with similar structures*
- *Level 1 assessment in accordance with ISO 19902*
- *Level 2 assessment, linear elastic redundancy analysis*
- *Level 3 assessment, system strength analysis*

- *Level 4 assessment, structural reliability analysis*
- *Prevention and mitigation*

2.3.5 Economical, environmental, social, and cultural aspects

No comments available

2.3.6 Additional information

It is permissible to accept limited individual component failure, provided that both the reserve against overall system failure and deformations remain acceptable. No analytical approach is recommended.

2.3.7 Responsibility

The owner shall maintain and demonstrate the fitness-for-purpose of the structure for its specific site conditions and operational requirements, based on the principles given in this clause.

2.3.8 Documentation and Reporting

All stages of the assessment shall be fully documented and become part of the structure data.

2.3.9 Reuse of results

In certain circumstances an assessment of one structure may be taken as an assessment of similar structures in similar conditions if specific requirements are met.

By structural integrity management system an updating of the objects data, reuse of data and improvement of the maintenance or inspection plan is insured.

2.3.10 Conclusion and remarks

The draft of the ISO19902 [16] describes excellently the assessment of existing structures and its triggering. As an innovation compared to other codes, it considers the compactness of simple visual inspections, maintenance and assessment in order to extent the lifetime of an object. The aim of this concept called structural integrity management is to figure out iteratively the optimal combination by the concept of the reuse of information and data.

Economical, environmental, social, and cultural aspects are totally neglected.

2.4 P100-92

The Romanian seismic design code, P100-92 [33], includes, starting with 1992, two special chapters, 11 and 12, on the seismic assessment and retrofitting of building structures. These chapters were completed and revised in 1996 [34], a detailed commentary being also added to each chapter.

2.4.1 Scope

Chapter 11 of P100-92 contains provisions and methods for the seismic assessment of existing buildings. The chapter addresses only seismic assessments performed between earthquakes, i.e. does not include requirements for post-earthquake emergency assessments (these are specified in separate regulations).

Chapter 12 of P100-92 provides requirements and specifications for the establishing of seismic retrofitting measures. These are intended for application to existing buildings that exhibited degradation and damage due to past earthquakes and/or have insufficient safety levels at seismic actions determined according to Chapter 11 of P100-92.

2.4.2 Terminology

The concerned chapters do not provide an explicit “Terminology” section.

2.4.3 Trigger

The conditions required for the assessment initiation are specified by other regulations than P100-92.

2.4.4 General overview

The assessment methods used in the evaluation of the seismic safety level of existing buildings are classified as follows:

E_1 = qualitative assessment method;

E_2 = analytical assessment methods; these are of three types:

- E_{2a} = current analytical methods (based on modal response spectrum analysis);
- E_{2b} = nonlinear static analysis methods;
- E_{2c} = nonlinear dynamic analysis methods.

The use of the qualitative assessment method and of the current analytical method is, usually, compulsive. Methods E_{2b} and E_{2c} are recommended for use in the case of buildings for which sufficient information on structural characteristics is available, as specified in the design, when those methods can provide significant additional data as compared to method E_{2a}.

The objective of seismic assessment is the classification of building according to the seismic risk classes defined in Chapter 11 of P100-92. The classification is used for establishing the retrofitting measures and the degree of emergency of retrofitting actions.

In the establishment of the retrofitting strategy, two solutions are usually presented: a minimal solution and a maximal solution. The minimal solution is intended to avoid the collapse of the building, as well as the other phenomena which can generate heavy injuries or human losses. Consequently, the retrofitting solution will specify all the measures intended to avoid the total or partial collapse of the building. The maximal solution shall be designed to satisfy the requirements for the present design of a new building, prescribed in the first part of P100-92.

2.4.5 Economical, environmental, social, and cultural aspects

It is not explicitly mentioned to consider environmental, social, and cultural aspects. Economical aspects shall be considered, but are not described in detail. However, the costs due to the interruption of the use of the building during the works shall be included in the total costs.

2.4.6 Additional information

The assessing expert can decide on performing non-destructive tests for structures for which strength and deformation properties of the materials or of the ground are not known, or for which the identification of the zones with discontinuities, degradations or uncontrolled links is desired.

2.4.7 Responsibility

Depending of the safety level resulted from the assessment, the owner should take, at the proposal of the assessing expert, the appropriate intervention measures for seismic risk reduction, as specified by chapter 12 of P100-92.

The commentary to chapter 11 specifies: *The assessing expert is responsible for the manner in which he performed the safety level and for the intervention measures that he proposes. The owner(s) of the construction are responsible for the application of intervention measures.*

2.4.8 Documentation and Reporting

The seismic assessment of a structure is finalized by an assessment report. The contents of the report is described by specific regulations (other than P100-92).

2.4.9 Reuse of results

The comment to chapter 11 specifies, as additional criteria to be considered for the classification of investigation methods: case studies for similar buildings, history of behaviour to previous earthquakes, history of prior interventions (modification of partition walls, repairs, retrofitting, adding of supplementary stories etc.).

2.4.10 Conclusion and remarks

Some drawbacks of chapters 11 and 12 of P100-92 have been recognized during the years of their application.

A new code for the evaluation and repair of existing buildings, P100-2004, Part 3, is planned to be released in 2006. It will be prepared with the consultancy of the World Bank Project 'Hazard Risk Mitigation and Emergency Preparedness' and is expected to introduce an entirely new approach, harmonized with EN 1998-3. The new code is part of a new set of seismic design prescriptions, which is presently being developed under the indicative P100-2004, and will include most of the new concepts in the European norms.

3 FRAMEWORK FOR COST-OPTIMAL ASSESSMENT OF EXISTING STRUCTURES

This section presents the framework for a cost-optimal assessment of existing structures with the purpose to extend their lifetimes. The framework involves all outstanding aspects of the investigated documents (e.g., ISO13822; JCSS; Draft ISO19902). Moreover, generally confirmed knowledge from contributing researchers is implemented as well. All necessary aspects listed at the beginning of Section 2 are considered and proven regarding the state-of-the-art.

This framework should contribute to an advanced guideline for the assessment of existing structures. Together with further developments it provides a basis for a general standard for the cost-optimal assessment of existing structures.

3.1 Scope

This framework is applicable to every kind of existing structure, facility, process, product, and system of every industrial sector. It is strongly focused on structural assessment aspects, but it also defines the position within an optimal facility and risk management system.

Since the presented framework can only consider principal aspects, specific steps, calculation methods, and requirements of a special type of object and task shall be considered in additional specific guidelines, codes, and standards.

3.2 Terminology

Object: Determines every kind of process, structure, facility, or system or their functional units, which have to be assessed, maintained, or inspected.

Maintenance: Any activity or a set of activities, such as tests, measurements, replacements, replacements, adjustments and repairs, intended to **restore or retain** an object in a specified state in which the object

can perform its required functions. Activities to restore the object after an abnormal, accidental, or seismic event are outside the scope of maintenance.

Assessment: The set of activities and their synergetic interaction performed in order to declare the fitness-for-purpose of an **intended reliable use** of an object over a specified residual or extended lifetime. Activities to restore the object after an abnormal, accidental, or seismic event are within the scope of assessment.

Reassessment: This term can be used instead of assessment if it is not the first assessment.

Inspection: An on-side non-destructive investigation to establish the present condition of an object, especially to determine within a reasonable level of confidence the existence and extent of deterioration, defects and damage.

Monitoring: Frequent or continuous, normally long-term observation or measurement of conditions of the resistance or of the exposure level of an object.

Lifetime: The period during which an object is fit-for-purpose. The fitness of something can be defined by a serviceability limit state or a ultimate limit state of the object itself or of functional units of the object.

3.3 Triggers

An existing object shall be assessed to demonstrated its fitness-for-purpose if one or more of the following conditions exist:

- Present or anticipated change compared to design or previous assessment basis
  - Extension of original lifetime
  - Changes in use or extension
  - Changes of resistant level due to extraordinary events (e.g. earthquake, accidental actions) or time-dependent actions (i.e. corrosion, fatigue)
  - Qualitative or quantitative changes of exposure level (e.g. permit heavy traffic for a bridge, modified chemical attack, more onerous environmental conditions, excessive scour or subsidence)
  - Change of safety requirements due to renewed codes or standards
- Detection of deterioration, damage, or inadequate serviceability which is not expected in a maintenance plan due to
  - Unforeseen use or exposure
  - Error of design or construction
- Doubts about safety, reliability, or serviceability
  - Unusual incidents during use, which could have damaged the object
  - Discovery of design or construction
  - Observation of deviations from design or last assessment
  - Suspicion of possible impairments of the safety related to resistance level, construction methods or system
  - Expiry of residual lifetime
- Others
  - Reliability check as required by authorities, insurance companies, owners
  - Routine assessment included in maintenance strategy
  - Initial assessment after long period of non-service or non-maintenance
  - Assessment to establish a new, renewed, or improved maintenance strategy

3.4 General overview

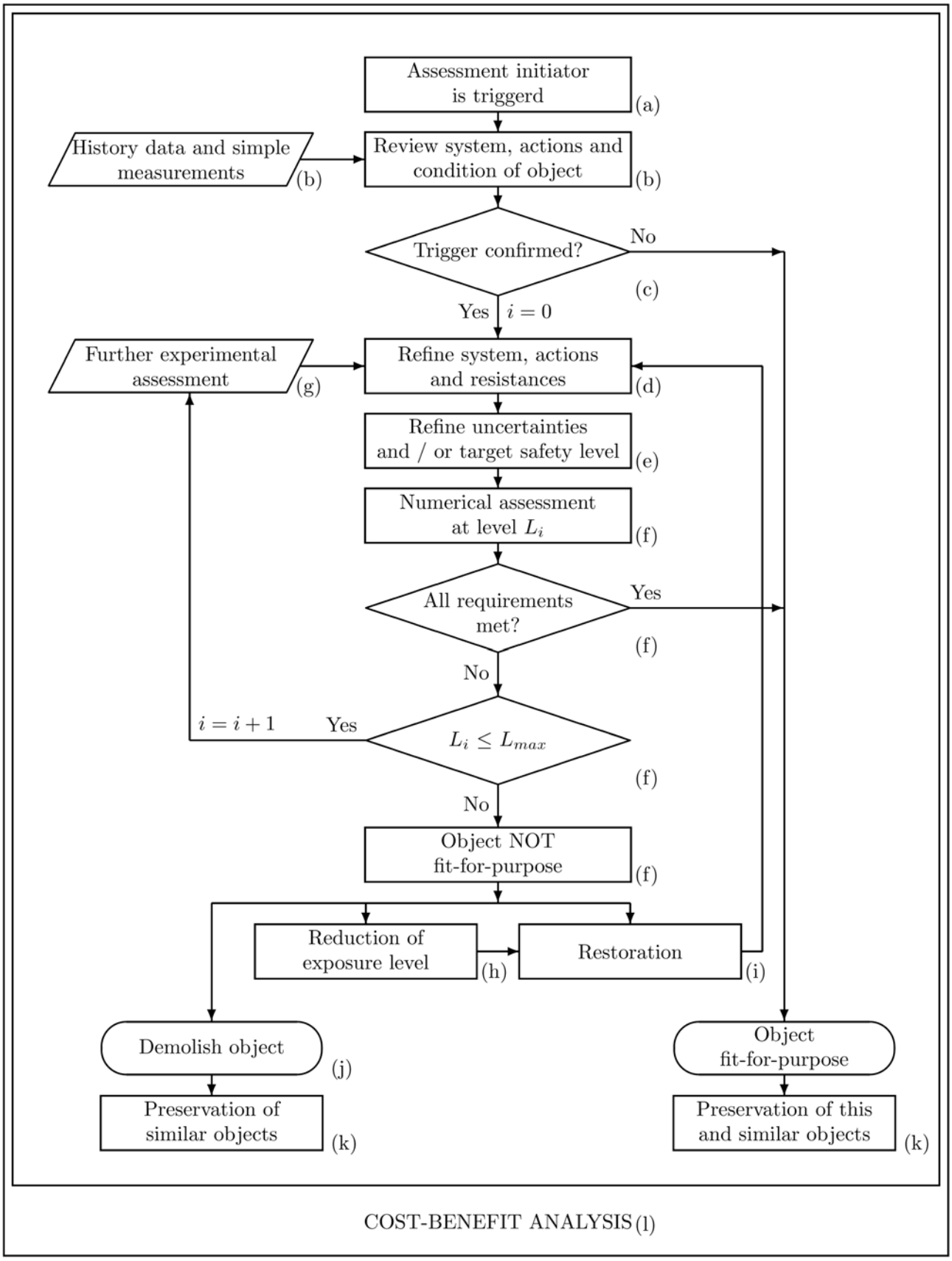


Figure 3.1: Flow chart of an assessment for structures and facilities

Explanation of specific assessment steps in Figure 3.1:

(a) Assessment initiator: An assessment initiator, also called trigger, is the reason to activate the assessment procedure. Fundamentally, the need for an assessment is based on a change in the requirements for the use of the structure or a doubt regarding the validity of the assumptions forming the original basis for the design. The main initiators are described in Section 3.3.

(b) Review system, actions and condition: The current system, actions, and condition have to be compared with the original design performance (and/or previous assessment of this or other similar objects, where applicable). The information is available from original design data, construction data, history data (e.g., monitoring data, special events), analyses and simple visual inspection and measurements (e.g., size of components). Code revisions since the design should be considered. Assessment records and a database of similar assessments should also be examined, where relevant/available.

(c) Trigger confirmed / Assessment required? In the case where the preliminary investigations show that a full assessment is not necessary, the procedure can be stopped, for example, if doubts are not confirmed or the knowledge from a similar object can be used.

(d) Refine system, actions, and resistances: If new information from the first review (b) is available, the assumptions of actions, resistances, model, and system have to be revised. In advanced levels of numerical assessment, it is possible to refine these assumptions by increased knowledge from experimental assessments (e.g., measured thicknesses or yield stress tests).

(e) Refine uncertainties and / or target safety levels: In the first level of calculation, L_0, the latest common codes and standards shall be used to determine the uncertainties of exposure and resistance as well as the target safety level. Very often these codes use a partial safety factor approach. Within more advanced experimental assessments the uncertainties and safety levels can be revised based on increased knowledge or reduced uncertainty concerning the actual composition of an existing object when compared with assumptions necessary in design. The refinement of model and system assumptions has to be considered as well. Section 7 discusses this topic in detail.

(f) Numerical assessment: Numerical assessments of the safety / reliability of the considered object are performed using the refinements in (d) and (e). There are several possible levels; the first (reference) analysis should always be using current recognised codes. Although, based on experience, the engineer may decide to jump from L_0 to a significantly more sophisticated numerical assessment, there are advantages in improving the understanding of the performance of an object in stages, working through the levels of assessment sequentially.

If the object does not pass the code requirements, a more advanced numerical assessment can be applied taking account of real aspects of performance, often not included explicitly in codes. Some possibilities are: considering non-linear as opposed to linear approaches, or investigating system performance of a whole structure and not just individual components. The most advanced method is the system reliability analysis based on a generic approach. Obviously, it is difficult to define a firm hierarchy of such methods. The range of more advanced levels depends on the object and differs from industrial sector to industrial sector. Risk analysis and cost-benefit aspects shall be considered throughout. If a numerical assessment demonstrates all requirements are met, the object may be deemed fit for purpose.

Additional experimental assessments can become necessary (see (g)) to increase the knowledge of the exposure level, resistance level, and model and system assumptions of the investigated object, which finally leads to a loop with different numerical assessment levels, L_i, with increased accuracy. The highest level L_{max} depends on the available numerical and experimental assessment methods and cost-benefit relations. Assuming all tools are used and the object does not pass the numerical assessment, the loop ends and the object is declared as NOT fit-for-purpose.

A detailed description of advanced numerical assessment methods is given, for example, in WP5 [6].

(g) Experimental assessment: Experimental assessment includes decisions about what should be measured, which method has to be used and the subsequent interpretation of the results. Obviously, it depends on

the next numerical assessment step and the cost-benefit relation (and/or may be needed to overcome particular uncertainties about the construction or condition of the object). A detailed description of various methods is available in Section 6.

(h) Reduction of exposure level: If the object is not fit-for-purpose, an option to keep it operating in some form is to limit the exposure level. To adopt this approach, it has to be guaranteed that no higher exposure level is possible. This assurance may be achieved operationally (e.g., by de-manning) or additional modification of the object may be necessary with restoration or mitigation measures (i).

(i) Restoration: Restoration (sometimes known as mitigation) is defined as an essential or minimum set of retrofit steps such that the service or ultimate life can be extended for a specified time period. The main restoration methods are the increase of resistance or reduction of the exposure level. After restoration / mitigation, an additional general assessment is necessary to verify the performance of the new object for the new conditions.

(j) Demolish object: The investigated object will be decommissioned and destroyed if it cannot be shown, or made to be, fit-for-purpose. Therefore, aspects of environment-friendly recycling or reuse of parts of the object have to be considered, and the health and safety issues in demolition process are of particular importance. Any reused parts need an additional intensive experimental assessment and an advanced maintenance programme (WP7 [8]). The data of the assessment should be used to verify similar existing or further objects.

(k) Preservation: Preservation defines all activities, which allow keeping the system in a state such that a continuous safe and reliable operation is guaranteed during the entire service life. This is of paramount importance for systems which are subjected to deterioration with usage and age, as well as for a further extension of lifetime. Preservation encompasses different activities: information updating (WP5 [6]); re-assessment (Section 10); and, most notably, maintenance (WP7 [8]) ensuring objects remain in the condition assumed in the fitness for purpose assessment. If an assessment has shown an object not to be fit-for-purpose, preservation of other similar objects is important to ensure they do not deteriorate to the extent that they too would fail an assessment on the basis of their changed condition.

(l) Cost-benefit analysis: The results of a cost-benefit analysis can be used as input to the decision processes. Such decisions have to be made during the whole assessment process (e.g., should the object be demolished or restored, or what kind of experimental assessments are worthwhile performing). A cost-benefit analysis is a rational decision tool where the optimal decision maximizes the total expected benefits minus costs in the design or remaining lifetime. All benefits and costs have to be expressed in monetary units and are discounted to, for example, the time of decision. In the decision process all information and aims should be considered by weighting social, economic, and environmental aspects. The description of modelling a cost-benefit-function is described in Section 9.

3.5 Economical, environmental, social, and cultural aspects

By following the assessment procedure, many decisions have to be made in order to take the next assessment step and to declare a solution how to proceed with the assessed object. Lots of those decisions have to be made by people who are no specialists in every field the decision will cover. This leads in many cases to arbitrary decisions which are not optimal by considering a global view. As an input to the decision process the results of a rational cost-benefit analysis have be used.

A detailed description to perform a cost-benefit analysis is presented in Section 9.

3.6 Additional information

Updating of information should be performed based on state-of-the art of reliability analysis. Two different types are distinguished [43]

- *updating of individual random variables due to measurements, observations related to the individual variable based possibly on BAYESIAN techniques*
- *updating of failure probability* or target safety level *by conditioning (i.e., conditional failure probabilities due to measured cracks or due to survival of extreme loads)*

A detailed description BAYESIAN techniques and its application is given in Task5.4 [7]. The updating of target safety levels is explained in Task 6.2 (Part II of this document).

3.7 Responsibility

The owner is responsible for the object and has to initiate the assessment. It is recommended that the expertise and experience of engineers are engaged for certain tasks. Hence, the responsibilities are distributed between the engineers and the client or owner. In general, the engineers have to advise and explain the preferred solutions, but the client makes the decision in collaboration with the relevant authority. Only if the decision is not in accord with common societal and safety rules, is the engineer allowed to assume higher authority. Rapid decisions by the engineer to ensure safety aspects are not compromised may also be justifiable.

3.8 Documentation and Reporting

At each stage when a decision has been made, results have to be summarized in a report for the owner. In particular, the report contains all necessary information on the safety and conclusions with recommendations for the next decision-making step. Additionally, a final report summarises the main results of the assessment. The results should be stored in a database to inform other projects. The sub-step reports and final report should include the main parts:

- Title page
- Name of engineer and company
- Synopsis and keywords
- Table of contents
- Scope of assessment
- Description of the structure/object
- Investigation
- Analysis
- Verification
- Discussion of evidence
- Review of intervention options (incl. cost-benefit)
- Conclusions and recommendations
- Reference documents and literature
- Annexes

3.9 Reuse of results

Since for many objects each design is re-design, the reuse of results can avoid double work and consequently save money. There is considerable advantage if many similar kinds of objects having similar environmental conditions and exposure levels (e.g., offshore structures, standardized bridges, pipelines) are present. The benefits can almost be multiplied by the number of objects, while the costs for the assessment are lower for further objects. Worldwide, Europe-wide, or company-wide databases should be used to save and recall the data. The level of access should be restricted due to the presence of sensitive data. Furthermore, the results of an assessment of an object should not be restricted to the assessment itself. The results are also useful to update the maintenance plan of the current, similar, or new objects.

4 STRUCTURE AND MATERIALS

4.1 Structure

The structural system of an object has to be analysed in preparation of experimental and numerical assessment procedures.

This structure can be divided into (1) structural components (substructure, superstructure), (2) structural parts (beam structure, surface structure, body structure), and (3) structural members (single beam, single slab) with connection elements (welds, screws, dowel).

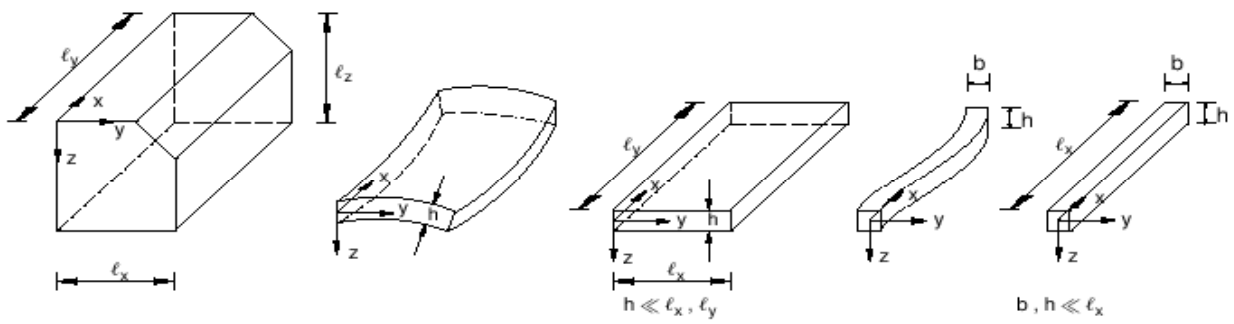
The definition of structural components is very different for the several structural types. In this report the possible components of structural types are not described. However, the following questions help to characterize the structural system.

- Of what kind are the main load bearing systems?
- Do the constructed load bearing systems have the designed properties?
- Do already all structural members exist?
- What are the connection elements?

4.1.1 Structural Parts and Members

All structural parts and members with local or global load bearing functions have to be assessed. In Table 5.1 structural parts / members are classified by the geometrical shape and the kind of stress. (i.e., compression (C), tension (T), bending (B), complex stresses (CO), torsion (TO)).

Table 4.1: Types of structural parts / members and their stresses ([78], [65], [73])

compact body structure	thin surface structure		thin beam structure	
„thick“ bodies: foundation, dames, thick containment	curved: membrane (T) shell (C, T)	plane: plate (B, TO) shear panel (like wall: C) folded plate (CO, TO)	curved: ring (C, T) arch (C) cable (T)	plane: beams (B, TO) tension bar (T) column (C truss (C, T) frames (CO, TO) grid (CO, TO)
				

This classification is based on the single elements of a structural system or of a structural component. Considering more complicated systems or structures, combined and additional effects are possible. For example, an open-web girder made up of many trusses acts like a plate.

The majority of non-reinforced concrete structural members are low stressed foundations. Reinforced members are beams, surface structures, and shells. Plane or space bar structures are commonly made of steel or timber. Masonry is mainly used as walls or arches.

4.1.2 Connection Elements

The type of connection and its connection elements are depending on the material.

Concrete: In concrete structures joints in cause of the production process are necessary. The connections between beams, columns, walls, and plates are often connected monolithically. In most cases a pass through reinforcement or a connection of the reinforcement between the structural members is necessary. For pre-cast elements the same technique may be applied.

Steel: The force fitted connection for beam-like steel elements and plates are mainly the welding connection. These connections are very stiff. Due to the complicated process of welding under construction site conditions, montage joints are often made by screws. The connections of historical steel structures are often made of bolts. Adhesive joints in steel constructions are restricted to serial production and light-weight structural members, which are often used in aerospace industry.

Timber: In timber constructions adhesive joints and a lot of mechanical connection elements are used. Traditional elements are anchors, screws, nails, dowels, and clamps, whereas the modern connections are realised by special dowels, like claws dowel, ring dowel, or from steel plates formed components (nail plates, beam shoe).

In addition several connection elements are present to connect different materials. Typical elements are tie bars and screwed plates.

4.1.3 Classification of Damages

The term ‘error’ is defined as human action. It is the difference between results of actual human action and the aims of the action. An ‘error’ can result in one ore more ’defects’.

The term ‘defect’ is to see in the relation with state of the structure. A ‘defect’ is the negative difference between an aimed value and the reached value if the differences increase prescribed tolerance limits. A ‘defect’ can result in one or more ‘damages’.

A ‘damage’ is a change on a structure affecting its appearance, serviceability, durability, functionality, and load bearing capacity. Reasons are either a ‘defect’ on the site of resistance or an overstress.

Damage in engineering systems is defined as intentional or unintentional changes to the material or geometric properties of these systems, including changes to the boundary conditions and system connectivity which adversely affect the current or future performance of that system. All damage begins at the material level and progresses under appropriate loading scenarios to component and system level failure at various rates.

Damage can occur in different ways:

- Gradually, (e.g., fatigue, creep, corrosion)
- Suddenly and predictably, (e.g., aircraft landings, planned explosions in confinement vessels)
- Suddenly and unpredictably (e.g., foreign-object-impact on turbine blades, earthquake-induced damage in civilian infrastructure, or battle damage to military equipment).

Table 4.2 gives an overview of damage in concrete structures. These are damages in cause of defects in the material, non-sufficient experience, and on mistakes during the time of construction and defects in workmanship.

Table 4.2: Classification of defects and damages (based on [47])

Concrete	• Poor workmanship (surface porosity, honeycomb, stratification) • Mechanical damages (abrasion, delamination and spalling due to impact load; delamination and spalling due to excessive load from temperature effects, differential movements of closely spaced elements, post-tensioning; small distance of the bearings from the free edge) • Cracks and open joints between the segments (structural and non-structural; with or without presence of water, exudation, efflorescence) • Deterioration (peeling, spalling and delamination of concrete cover due to freezing and thawing, corrosion of the reinforcement, ASR, leakage through concrete, constant presence of dirt, moss and other vegetation,) • Wetting (wet exposed concrete surface in the tidal zone and in the splashing zone of waves and driving cars, exposure to the marine environment, leakage of water through concrete element due to the damaged or missing waterproofing membrane, damaged joint sealings on the sidewalk or edge beams) • Concrete cover depth defects • Carbonation front in the concrete cover • Chloride penetration depth
Reinforcement	• Corrosion of the stirrups and main reinforcement • Corrosion of the pre-stressed and post-tensioned tendons • Corrosion of the ducts for post-tensioned tendons • Corrosion of the anchorage elements and deviators for external pre-stressing and for stay cables (including the deterioration of the protective paint • Damage of the protective cover of external tendons and stay-cables • Ungrouted ducts
Bearings (steel, reinforced concrete, reinforced elastomeric, pot bearings)	• Excessive deformations (displacements, rotations) • Deterioration (reinforced concrete, rubber) • Corrosion and deterioration of the protective coatings (steel bearings, steel elements of the pot bearings, reinforced elastomeric bearings, fixing bolts) • Mechanical damage (broken bolts, broken or deformed steel elements)
Expansion joints	• Corrosion and deterioration of the protective coatings of steel elements • Mechanical damage (broken fixing bolts, broken or deformed steel elements, cracked welds due to impact load, damaged rubber surface, malfunction of the expansion joints due to specific causes) • Construction defects (missing fixing bolts, thin welds, welding imperfections, missing waterproofing membrane, missing or malfunction of expansion joint drainage system) • Deterioration (e.g., rubber membrane–leakage, sealant along the expansion joint, surface rubber, expansion joint sealant)

Safety barriers (steel and concrete)	• Deterioration of concrete elements (due to freezing and thawing, presence of de-icing salts) • Deformations (transverse or vertical displacements) • Corrosion of steel elements • Mechanical damages due to impact loads

4.2 Materials – Resistance and Deterioration Processes

As mentioned in the previous section, all damage begins at the material level. Hence, the knowledge of the kind of damage depending on the material is essential to perform an assessment. Table 4.3 gives such a short overview, which will be described in detail in the following sections.

Table 4.3: Materials and deterioration processes

Structural Steel	Pre-stressed and RC	Masonry	Timber	Composite Materials/ Plastics
• Fracture • Fatigue • Corrosion • Creep	• Fracture • Cracking • Corrosion of rebars • De-bonding • Creep and Shrinkage • Carbonation • Chloride ingress • Alkali reactions • Frost induced damage	• Fracture • Cracking • Mortar deterioration • Brick masonry unit deterioration like o Spalling o Dusting o Flaking	• Fracture • Biological Damage due to o Fungi o Insects o Marine borers o Bacteria • Physiochemical Damage due to: o Weathering o Chemicals o Mechanical forces o Heat	• Delamination • Chemical Damage • Aging

4.2.1 Structural Steel

The main damages on the material steel are fracture, fatigue, corrosion, and creep. Due to a high interest in this topic, the Thematic Network FITNET [15] investigates these damages in detail. However, a short description is given below.

4.2.1.1 Fracture

Fracture destroys the cohesion by creating surface or volume discontinuities within the material. Hence fracture occurs at the larger scale of crystals. Brittle fracture by cleavage and ductile fracture resulting from large localized deformation are the two main basic mechanisms of local fracture. [63]

Brittle fracture is characterized by rapid crack propagation with low energy release and without significant plastic deformation. The fracture may have a bright granular appearance. The fractures are generally of the flat type and chevron patterns may be present. Ductile fracture is characterized by tearing of metal and significant plastic deformation. The ductile fracture may have a grey, fibrous appearance. Ductile fractures are associated with overload of the structure or large discontinuities. [109]

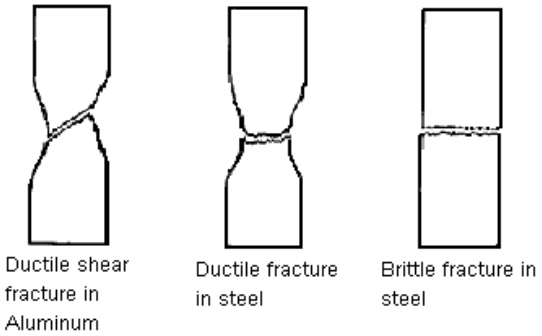


Figure 4.1: Main mechanisms of local fracture [109]

Figure 4.2 shows a typical experimental function with the most important limit parameters. The material stiffness significant decreases after the exceeding of yield strength σ_y until fracture happens.

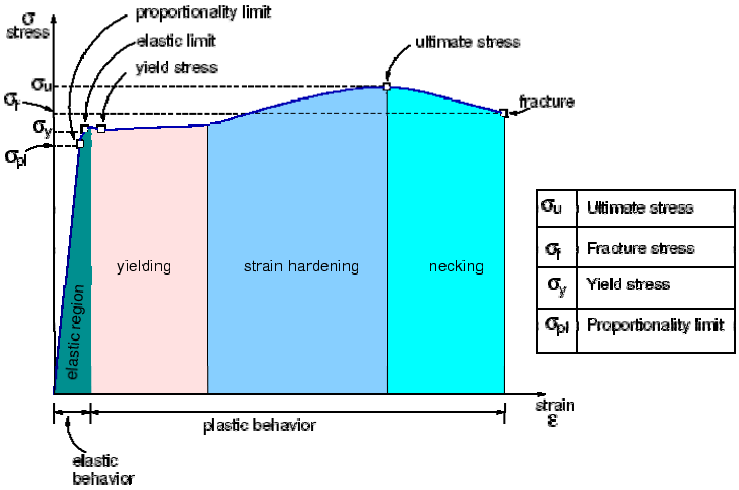


Figure 4.2: Stress-Strain Curve of a ductile material [103]

4.2.1.2 Fatigue

Fatigue is a phenomenon that results in the sudden fracture of a component after a period of cyclic loading in the elastic regime. Failure is the end result of a process involving the initiation and growth of a crack, usually at the site of a stress concentration on the surface. [83]

Occasionally, a crack may initiate at a fault just below the surface. Eventually, the area of the cross section is reduced in such a way that the component ruptures under a normal service load, but one at a level which has been satisfactorily withstood on many previous occasions before the crack propagated. The final fracture may occur in a ductile or brittle mode depending on the characteristics of the material. Fatigue fractures have a characteristic appearance that reflects the initiation site and the progressive development of the crack front, culminating in an area of final overload fracture. [83]

Figure 4.3: High tensile steel bolt failed, [83]

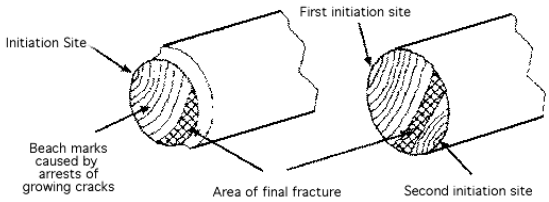


Figure 4.4: Fatigue failure in a circular shaft with one or two initiation points, [83]

Figure 4.3 and Figure 4.4 illustrate fatigue failure in a circular shaft. The initiation site is shown and the shell-like markings, often referred to as beach markings because of their resemblance to the ridges left in the sand by retreating waves, are caused by arrests in the crack front as it propagates through the section. The hatched region on the opposite side to the initiation site is the final region of ductile fracture. Sometimes there may be more than one initiation point and two or more cracks propagate. This produces features as in Figure 4.4 with the final area of ductile fracture being a band across the middle. This type of fracture is typical of double bending where a component is cyclically strained in one plane or where a second fatigue crack initiates at the opposite side to a developing crack in a component subject to reverse bending. Some stress-induced fatigue failures may show multiple initiation sites from which separate cracks spread towards a common meeting point within the section. [83]

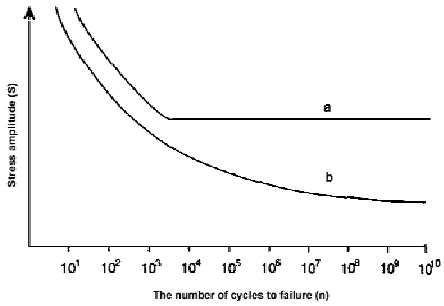


Figure 4.5: Stress-number of cycles to failure (S-N) curve [83]

Fatigue strength is determined by applying different levels of cyclic stress to individual test specimens and measuring the number of cycles to failure. Standard laboratory tests use various methods for applying the cyclic load (e.g., rotating bend, cantilever bend, axial push-pull, torsion). The data are plotted in the form of a stress-number of cycles to failure (S-N) curve, see Figure 4.5. [83]

Owing to the statistical nature of the failure, several specimens have to be tested at each stress level. Some materials, notably low-carbon steels, exhibit a flattening off at a particular stress level as at (a) in Figure 4.5 which is referred to as the fatigue limit. As a rough guide, the fatigue limit is usually about 40% of the tensile strength. In principle, components designed in such a way that the applied stresses do not exceed this level and should not fail in service. The difficulty is a localized stress concentration may be present or introduced during service that leads to initiation, despite the design stress being normally below the ‘safe’ limit. Most materials, however, exhibit a continually falling curve as at (b) and the usual indicator of fatigue strength is to quote the stress below which failure will not be expected in less than a given number of cycles which is referred to as the endurance limit. [83]

Summarising, fatigue break mainly depends on these factors: [83]

- Number of load cycles (load changes)
- Stress differences of each load cycle
- Mean value of stress in each cycle
- Existence of local stress concentration

4.2.1.3 Corrosion

Acting of oxygen generates chemical compounds called rust, chloride compounds and sulfur compounds in presence of water on the steel surface. The process corrosion has good condition in the water, in soil, and inside the concrete. Corrosion will be accelerated by chemical loads (e.g., from industry), condense water, and high temperatures over 60°C.

Corrosion in a wider meaning is a phenomenon which results from chemical degradation of metals and alloys due to reaction with agents in the service environment, e.g. the rusting of steel in moist air. May take the form of uniform attack over the whole surface or as highly localised pitting. [83]

When metal atoms are in a chemical environment that allows or causes them to release electrons, they become positively charged ions that take part in chemical reactions, provided an electrical circuit can be com-

pleted. The net effect is that the metal component corrodes away where the electrons are released and the useful cross-sectional area is reduced. This effect can be concentrated locally to form a pit or, sometimes, a crack if a high level of tensile stress is acting, or it can extend across a wide area to produce general wastage. The load carrying capacity is therefore reduced and an eventual failure may occur simply because a load in the upper part of the normal spectrum exceeds the residual strength of the component. [83]

Localised corrosion that leads to pitting may provide sites for fatigue initiation and, additionally, corrosive agents like seawater may lead to greatly enhanced growth of the fatigue crack. Pitting corrosion also occurs much faster in areas where microstructural changes have occurred due to welding operations. [83]

Some metals display an inherently greater corrosion resistance than others, but even those most resistant to normal atmospheric conditions are vulnerable to some reagent. Stainless steels, for example, are almost never attacked under oxidizing conditions because they have an inbuilt chromium content (at least 12 per cent), which forms an electrically insulating and self-healing protective oxide film preventing most corrosive agents being able to set up an electrical circuit. If the oxide film is mechanically damaged it immediately reforms and keeps the reactive agent away from the metal. However, if the environmental conditions are reducing or exclude the oxygen necessary to re-form the chromium oxide film, stainless steels will corrode away almost as fast as non-stainless varieties. [83]

For certain metallic alloys the effects of stress and corrosion combine to produce the troublesome phenomenon of stress corrosion cracking. It requires all of the following ingredients to occur simultaneously. [83]

- a susceptible microstructure;
- an applied or residual tensile stress - residual stresses contain balanced regions of tension and compression; usually resulting from forming operations
- a mildly aggressive chemical environment, but seldom one producing any visible corrosion product on the surface of the component.

Figure 4.6: Stress corrosion cracking of a pipe [83]

4.2.1.4 Creep

Creep is the slow plastic deformation of metals under a constant stress, which becomes important in [94]:

- The soft metals used at about room temperature, such as lead pipes and white metal bearings.
- Steam and chemical plant operating at 450-550°C.
- Gas turbines working at high temperatures.

Creep can take place and lead to fracture at static stresses much smaller than those which will break the specimen when loaded quickly in the temperature range 0.5 to 0.7 of the melting point T_m. The Variation with time of the extension of a metal under different stresses is shown in Figure 4.7a. Three conditions can be recognized: [94]

- The primary stage, when relatively rapid extension takes place but at a decreasing rate. This is of interest to a designer since it forms part of the total extension reached in a given time, and may affect clearances.
- The secondary period during which creep occurs at a more or less constant rate, sometimes referred to as the minimum creep rate. This is the important part of the curve for most applications.
- The tertiary creep stage when the rate of extension accelerates and finally leads to rupture. The use of alloys in this stage should be avoided; but the change from the secondary to the tertiary stage is not always easy to determine from creep curves for some materials.

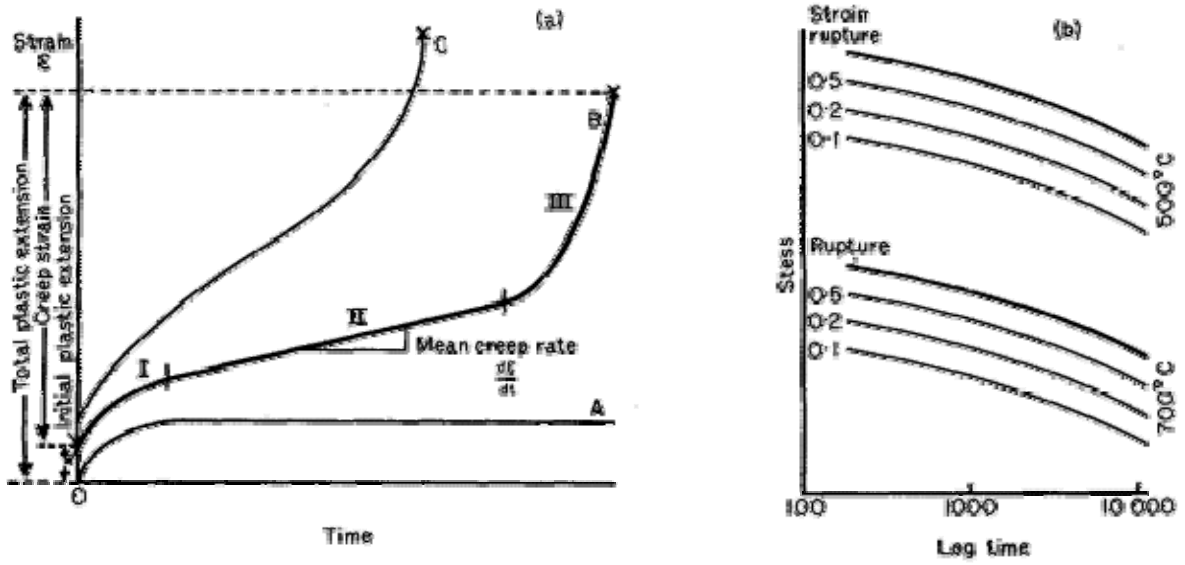


Figure 4.7: (a) Family of creep curves at stresses increasing from A to C;
(b) Stress-time curves at different creep strain and rupture [94]

The limited nature of the information available from the creep curve is clearer when a family of curves is considered covering a range of operating stresses. [94]

As the applied stress decreases the primary stage decreases and the secondary stage is extended and the extension during the tertiary stage tends to decrease. Modifying the temperature of the test has a somewhat similar effect on the shape of the curves. [94]

Design data are usually given as series of curves for constant creep strain (0,01-0,03%, etc.), relating stress and time for a given temperature. It is important to know whether the data used are for the secondary stage only or whether it also includes the primary stage (Figure 4.7b). In designing plants that work at temperatures well above atmospheric temperatures, the designer must consider carefully what possible maximum strains he can allow and what the final life of the plant is likely to be. The permissible amounts of creep depend largely on the article and service conditions. [94]

4.2.2 Reinforced / Pre-stressed Concrete

4.2.2.1 Corrosion, Carbonation, and Cracking and of reinforcement

Corrosion of reinforcing bars induces an early and very important deterioration of concrete structures and reduces their service life. In particular the increased use of de-icing salts and the attack from sea salt in coastal areas have exacerbated corrosion of concrete reinforcing bars. It causes the reduction of reinforcing bar section. The cracking of concrete cover is produced by the expansion of corrosion products. Hence, the loss of composite interaction between steel and concrete due to bond deterioration happens. Additionally the same material damages are possible as for the structural steel described in Section 4.2.1.

Passive threshold of steel: Normally, the steel embedded in concrete is protected by the alkalinity (pH-value = 12,5 to 13,5) of the water pores. Only if the pH-value decreases under 9 a corrosion of steel is possi-

ble. Corrosion mechanisms for steel in concrete structures are induced by different chemical reactions. There are four main mechanisms possible:

- Corrosion induced by oxygen
- Corrosion induced by carbonation of the concrete
- Corrosion induced by chloride-ions
- Corrosion susceptibility by stress cracking

The first three occur on normal and pre-stressed steel, whereas the last one only occurs on pre-stressed steel bars. With the presence of catalysts (e.g., sulphates (SO_4^{-2}), de-icing salts), the deterioration of iron will be accelerated independent from the mechanism.

Oxygen: This kind of corrosion of steel is induced by oxygen of the air, which will diffuse through the concrete cover. For a relevant reaction a relative humidity (RH) of 80% is necessary. Consequences are: section reduction of the steel bars, crack generation in the concrete, spalling, or the failure of structural members.

Carbonation: The diffusion of carbon dioxide in the cement paste causes a reduction of the alkalinity of the concrete. The fastest reaction velocity occurs with a relative humidity of 50 to 70%. Further, the rate of carbonation depends on porosity and moisture of the concrete.

$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O$ (presence of water is necessary)

Due to the bigger volume of the steel corrosion products, the concrete cover spalls over large areas. By the presence of discoloured zones in the surface of the concrete, the recognition is possible. The colour can vary from light grey to intense orange. By means of an optical microscope carbonation is indicated by the presence of calcite crystals and the absence of calcium hydroxide, ettringite, and unhydrated cement grain.

In non-reinforced concrete the carbonation is an advantage, due to the decrease of the porosity, which effects a stronger cement paste.

Chloride Ions: Chlorides are negative ions of salts, which can reach the concrete by application of de-icing salts, from seawater, or the air (PVC-fire, sea air). The chloride ions are innocuous for the concrete itself. Only if the chloride reaches the steel bars, corrosion will occur.

For this corrosion mechanism only a small relative humidity necessary. The very local concentrated deterioration of steel is called pitting corrosion and can be very deep. In contrast to the oxygen-induced corrosion, these corrosion products will not expand (i.e., the concrete cover does not spall). Due to a difficult detection, this kind of corrosion is very dangerous.

A high chloride content of the concrete in most of the cases is caused by the use of de-icing salts, sea salt spray, direct sea water wetting, the exposure of concrete to chemicals (structures used for salt storage, brine tanks) or by fire of PVC. Other possibilities to have chloride ions in the concrete is to add chloride set accelerators, to use sea water in the mixture of concrete or to use contaminated aggregate. Figure 4.8 shows an example for a chloride attack on a concrete column in sea water.

Figure 4.8: Chloride attack [65]

Cracking: In addition to the first three corrosion mechanisms, the corrosion susceptibility by stress cracking and hydrogen embrittlement only occurs to pre-stressed concrete structures. Under very high and perpetually existing stresses in grain boundaries the steel cracks slowly, which results in an undeformed fracture. In certain conditions the corrosion of steel causes the synthesis of hydrogen that diffuse in the non-deteriorated steel. The occurring high pressures can generate an undeformed brittle fracture, which is very dangerous due to its sudden occurrence. The only method to avoid this deterioration is to limit the concrete cover to 5cm to the cladding tube and reduce the crack width on the concrete surface under 0.2mm.

4.2.2.2 Debonding

The bond mechanism is the interaction between reinforcement und concrete that offers the possibility to transfer stresses to combine the tensile strength of the steel and the compressive capacity of the concrete. Debonding describes the loss of bonding between reinforcement and concrete, which reduces the loading capacity of the structure enormously. [66]

Three mechanisms are responsible for bonding: chemical adhesion, friction, and mechanical interlocking between the ribs of the reinforcement bars and the concrete, see Figure 4.9. The effect of chemical adhesion is very small and is lost immediately when slipping between the reinforcement and concrete starts. The activated forces of the bearing action make it possible to guarantee bonding, but induces stresses in the concrete as well. [66]

If the concrete around the reinforcement bar is penetrated by resulting longitudinal splitting cracks, and there is no transverse reinforcement that can continue to carry the forces a splitting failure occurs. However, a small negligible contribution to the bond mechanism is still given by friction. When the concrete around the reinforcement bar is well confined, videlicet it can withstand the normal splitting stresses, and the reinforcement does not start yielding, a pull-out failure is obtained. [66]

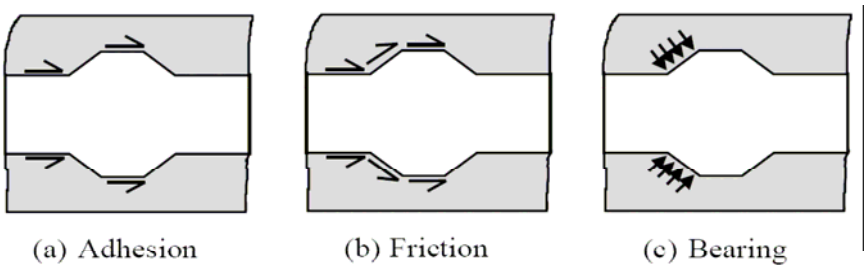


Figure 4.9: Idealised transfer mechanisms [66]

A sufficient concrete cover, ribbed ductile reinforcement, and sufficient transverse reinforcement can avoid debonding.

4.2.2.3 Creep and Shrinkage

Creep is a time dependent deformation behaviour due to constant stresses and, consequently, the increase of the immediately appearing deformation. If the compression stress does not exceed the fatigue strength, the creeping process decays. The deformation reaches a limit value. Figure 4.10 shows the process for loading and unloading.

The movement and rearrangement of water in the cement stone basically describe the creeping process. A lot of conditions and parameters influence the creeping, for example, the size and duration of load, the environmental conditions, the water-cement ratio, and the dimension of structure.

A describing parameter for creep deformations is the creep value C or the creep number φ. These values depend on the age of the concrete, on the time of loading and on the intensity of the creep generating stress.

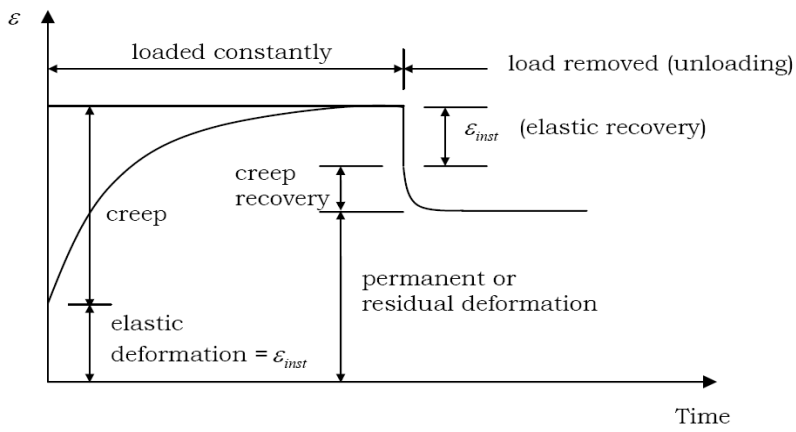


Figure 4.10: Concrete under constant axial compressive stress [40]

Shrinkage is the time dependent contraction of the unloaded concrete and starts with the time of solidification of the concrete. The further process depends on the water content of the concrete, the dimensions of structural component, and the humidity of environmental air. For ordinary thickness of structural components it is assumed that the predominant part of shrinkage is finished after three years.

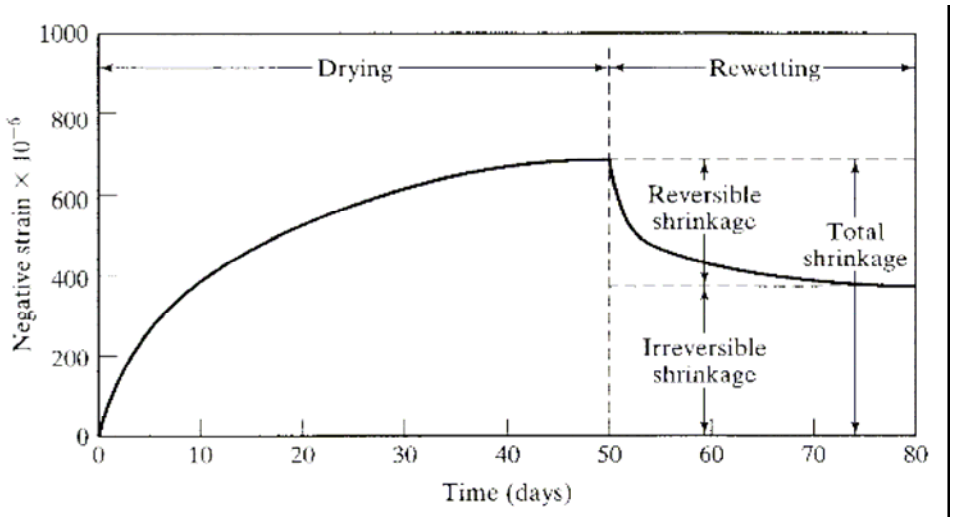


Figure 4.11: Shrinkage during drying and rewetting process [62]

4.2.3 Masonry

The deterioration of masonry can be classified into in cracking, spalling, bulging, leaning, and mortar or brick masonry unit deterioration. A clearly arranged overview is given in [52] and [81]. If the mortar or brick is based on cement, additionally, the same deterioration can appear as for concrete (see Section 4.2.2).

4.2.3.1 Cracking

Masonry is able to deform elastically over long periods of time to accommodate small amounts of movement. Large movements normally cause cracking. The cracks may appear along the mortar joints or through the masonry units, depending on the ductility of both materials.

The large deformation of masonry inducing the cracking can result from:

- differential settlements of foundation or different movements between buildings,
- drying shrinkage or creep,
- improper supports over openings,
- corrosion of steel and steel wall reinforcement,

- the effect of freeze-thaw cycles,
- bulging and leaning of walls,
- expansion of salts, and
- expansion and contraction due to ambient thermal and moisture variations (see Figure 4.12).

Further detailed information is given in [81].

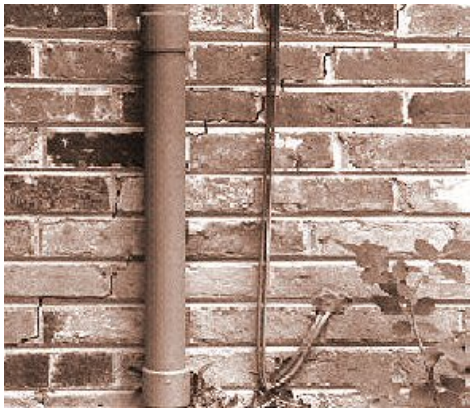


Figure 4.12: A common masonry wall crack probably caused by thermal or moisture expansion. [81]

Figure 4.13: An extreme case of structural failure in a masonry wall due to foundation settlement. [81]

4.2.3.2 Deterioration of Mortar

The two most frequently deteriorations of mortar are random cracking and out washing of the mortar by flowing water.

4.2.3.3 Deterioration of Brick Masonry Units

Deterioration of the bricks itself can occur in form of spalling, dusting or flaking due to mechanical or chemical damage. Spalling, a mechanical damage, results from moisture entering the brick and subsequent freezing. Also very rapid temperature changes caused by fire exposure can generate spalling. An example is presented in Figure 4.14. Chemical damage due to the leaching of chemicals from the ground or elsewhere into the brick results in internal deterioration. Dusting and flaking of the brick are the external signs of such damages.

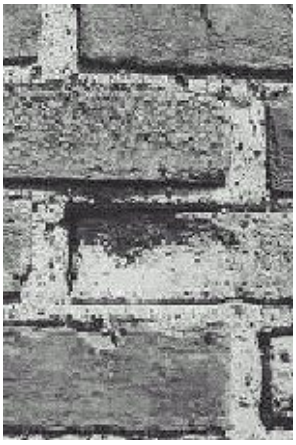


Figure 4.14: The brick shown here is highly spalled from the effects of excessive moisture penetration and subsequent freezing. [81]

4.2.4 Timber

A very dominant role water plays in the deterioration of timber. The properties of wood, such as strength, dimensional change, and combustibility, are directly affected by the presence of water. Depending on the moisture level of the wood, the conditions are more or less favourable for attacks by biological agents (fungi, insects, marine borers, and bacteria) and exposure to physicochemical agents (weathering, chemicals, mechanical forces, and heat). In general, the sapwood is more endangered than the heartwood.

Heckroodt [52] gives a very quick overview about the different agents, whereas Ridout [74] describes the deterioration processes in more detail. Furthermore, both authors suggest remedial and preservation actions.

4.2.4.1 Fungi

The development of fungi highly depends on the wood moisture and the temperature. Optimal conditions for the development are a moisture level between 30%-50% and temperatures between 20°C and 40°C. To avoid fungi a wood moisture level below 22% or over 60% and a temperature below 10°C or over 50°C are recommended. Moreover, adequate oxygen, nutrients, and the absence of naturally occurring wood extractives or chemical preservatives are necessary conditions for fungi. The most common wood-destroying fungi and their damage symptoms are listed in Table 4.4, whereas those of wood-disfiguring fungi are tabled in Table 4.5.

Table 4.4: Wood-destroying fungi [52]

Fungi	Appearance and character
Dry rots	Typically found in timber in direct contact with wet concrete or brickwork. Feed mainly on cellulose. Attacked timber typically has deep cracks along and across the grain, in soft and low on strength. Softwood is mainly attacked. Mycelium appears as silky white sheets or cotton-like white cushions.
Wet rots	Feed on cellulose and lignin. There are several white rot and brown rot species with different deterioration symptoms. Infected wood is very often discoloured and breaks into cubic pieces.
Soft rots	Regarded as superficial forms of wet rot and develops in very wet conditions. Make longitudinal cell wall cavities, leaving the surface layer soft. All types of timber can be attacked.

Table 4.5: Wood-disfiguring fungi [52]

Fungi	Appearance and character
Staining fungi (blue stain in service)	Attack is visible as dark-blue streaks and patches in the timber or underneath transparent or semi-transparent surface coatings on timber. Feed on the cell contents, not on the cell wall. Penetrate sapwood.
Moulds	Mostly attack sapwood. Fungi produce powdery or woolly mycelium and masses of spores at the surface. Colours range from black, brown and green to orange. Can also grow on timber treated with preservatives.

4.2.4.2 Insects

Beetles or borers and termites are the two main kinds of wood-destroying insects. Table 4.6 gives an overview about the most important beetles. Beetles are pass four distinct stages in the life cycle: egg, larva, pups, and adult beetle. The larva does the most damage. Termites live in large colonies, with a wide distribution in the tropics and certain temperature regions. Several species are a significant risk to any wood product. By leaving the surface intact, the interior will be completely destroyed. Keep in mind there are lots of other insects like wasps or carpenter bees, which can destroy the wood.

Table 4.6: Wood-destroying beetles [52]

Borer	Infestation	Appearance
Lyctid borers	Infest only hardwood with large pores and enough starch (e.g. oak).	Small, round flight holes (1.6mm) and fine, flour-like frass.
Anobiid borers	Attack hardwoods and softwoods, especially old furniture. Need damp, humid conditions, with poor ventilation.	Round flight holes (2mm) and coarse granular powder of digested wood.
Cerambycid borers	Attack mainly sapwood of softwood. Very serious threat to construction timber, but restricted to particular geographical regions.	Normally invisible until oval-shaped flight holes (3-10mm) appear. Frass marks on irregular surfaces

Table 4.7: Wood-destroying termites [52]

Termite type	Character	Occurrence
Subteranean termites	Soft bodied, cannot survive in the open, depend on a source of moisture, thus live underground in nests and forage in enclosed galleries. Eat the interior of timber, leaving the surface intact Entry holes and breakthroughs are plugged with soil	Attack wood in contact with soil. Short connecting galleries may be constructed.
Dampwood termites	Dependent on a source of moisture Narrower tunnels in dry than in damp wood	Initially infest damp, decayed timber and later dry, undecided
Drywood termites	Do not require a source of moisture; live in wood out of ground contact Greatly influenced by the air temperature and relative humidity Faecal pellets are poppy-seed shaped. Tunnel in all directions	Infest dry sound timber (e.g. inside buildings located in coastal regions)

4.2.4.3 Marine borers

Marine borers are animals that can be found along all coastlines. They are most active in warm waters and estuaries, whereas all types of timbers are susceptible to attack. The most important marine borers are contrasted in [52].

Table 4.8: Marine borers [52]

Borer	Infestation	Appearance
Molluscs (shipworm)	Infestation and destruction of submerged timber can be very fast Holes of up to 10mm in diameter opening to the exterior	Very long round tunnels with smooth shell like surfaces that riddle the infested timber

Crustaceans	Occurs in temperate waters Less prevalent and virulent than ship-worm, may take years to destroy sub-merged timber	Attack is from the surface, with pits and holes of 2mm or less in diameter appearing only after many months or years Damage is similar to that done by wood beetles, in wooden piles mostly between the high and low-water, arks (hourglass-shaped piles)

4.2.4.4 Bacteria

When wood is exposed to soil and aquatic environments or to out of ground conditions where wood is subjected to periodic wetting and drying, bacteria will attack the cell walls by cavitation, erosion, and tunnelling, causing structural damage. The damage is rather crucial, but they provide the basis for the attack of other biological agents.

4.2.4.5 Weathering

The combined effect of sunlight, heat and water deteriorate exposed timber. The sunlight's UV component breaks down lignin (a structural component of wood). Results are bleaching and, later on, a grey appearance of the timber. The process is very slow and mainly effects the aesthetic appearance of the timber. Dust-laden wind abrades the surface layer. Preferentially the soft early wood is eroded, resulting in a characteristic roughened surface appearance. Cyclic wetting, like rain, dew, and other forms of precipitation and subsequent drying causes swelling and shrinkage. Continued repetition leads to surface deformation. Warping, cupping, and cracking occur when weathering acts seriously.

The deterioration is rather a contribution to structural failure than being unattractive. The crucial aspect is the deterioration of the surface, which paves the way for penetration of fungi.

4.2.4.6 Chemicals

Since wood is naturally mildly acidic (pH 2 to 5,5), it is resistant to weak acids, but will be damaged by strong acids and alkalis. Bundles of soft cotton-wool-like fibres are typical damage symptoms.

A common cause of chemical attack is the rusting of steel in or on wet timber. The highly alkaline conditions, as a result of the corrosion process, break down the structural components of the wood. According to the deterioration of steel, this effect is accelerated by the existence of salty conditions (e.g., from salt-type preservatives, salt-laden sea air). In addition, the reaction of steel with wood extractives (e.g., tannins) induces dark blue stains.

4.2.4.7 Heat

The immense heat in cause of fire is responsible for the fire damage to timber in constructions. To start burning, large wooden members require a constant source of heat. The charring rate for structural timber ranges from 15 to 20mm in 30 min.

However, wood is a bad conductor of heat and the charred surfaces may serve to insulate the interior of the timber. Therefore, structural members with a huge cross sectional area protect themselves against further deterioration.

4.2.4.8 Mechanical Forces

Unbalanced tensional, compressive, bending, and torsional forces can cause permanent deformation by creep or even structural damage (e.g., splits, cracks and breaks along and across the grain). This kind of failure is normally prevented by design and building practices. Further, localized pedestrian or vehicle traffic leads to abrasive damage of wooden surfaces. Additionally, changes in moisture content cause swelling and shrinkage, which can result in mechanical damage.

4.2.5 Plastics and elastomers

According to their properties and production process, plastics are divided into two main groups: thermoplastic (thermoplasts) and thermosetting (thermosets). The damage of plastics can have physical and chemical causes.

Physical attack are due to loss or migration of plasticisers or other additives, with absorption of liquids or vapours, with crazing due to stress or fatigue, with mechanical damage including wear and tear, or with excessive heat or cold. The main effects of physical deterioration are: [115]

- crazing due to stress, often in combination with a particular environment such as solvent vapour or moisture – so called environmental stress crazing
- changes in flexibility due to plasticiser leaching or mitigation
- an oily bloom on the surface of the material, occasionally a solid bloom – due to plasticiser or other additive migrating to the surface, and
- distortion due to uneven loss of plasticiser.

Chemical attacks are associated with chemical reactions occurring to the polymer, or occasionally to additives. They are nearly always ongoing and irreversible. The responsible factors for chemical attacks are: light, heat, oxygen, moisture, ozone or other atmospheric contaminants, contact with chemical agents in use, cleaning, repair or by accident. Signs of chemical degradation may be: discolouration, embrittlement, severe crazing, crumbling (especially of foams), blistering, bloom, weeping odours, and surface acidity. [115]

Further information on the properties and deterioration of plastics and elastomers can be found in [39].

4.2.6 Composite Materials

Composites are formed from two or more types of materials. Hence, two main deterioration processes can occur: deterioration of the different materials themselves and deterioration of the connection or the connection elements. The material deterioration is described in the previous subsections separated for each material. The connection deterioration is more serious because the essential part of the composite is the connection. Depending on the used materials different connection deteriorations can occur, for example, debonding, delamination, and disruption.

Typical composites are fibre reinforced concrete and plastic, metal matrix composites, ceramic matrix composites, plastic-impregnated or laminated paper or textiles.

5 ACTIONS

Load actions can be considered to be permanent, variable, or exceptional.

Permanent loads have small and slow temporal variations around their mean value (e.g., self weight, earth pressure) or are loads which monotonically have a limiting value (e.g., pre-stressing, temperature, shrinkage, creep, settlements, eccentricities)

Variable actions are loads whose variations in time are frequent and large. These are all actions caused by the use of the structure (e.g., means of transportation or human), also called live loads and by most of the external actions (e.g., wind, snow, waves, temperature).

Exceptional actions have considerable magnitudes, but the probability of occurrence for a given structure is small related to the anticipated time of use. The duration of this load is short. Those can be accidental loads (e.g., impact loads, explosions, fire) and natural hazards (e.g., avalanches, landslides, floods, earthquake). [58]

Table 5.1: Classification of actions

Permanent	Variable	Exceptional
Dead Load (structural/ non structural members)	Physical Loads Live Loads Wind Loads Hydrodynamic Load Snow Load Earthquake load Fluid and Earth Pressure Loads Thermal Loads Erosive Loads Mechanical Attack Chemical Attack Biological Attack	Accidental Loads Impact Explosion Heat / Fire Natural Hazards Avalanches Landslides Floods Earthquake

The EN 1991 [16] is the current European regulation for the determination of loads.

5.1 Dead Loads

Dead loads are to be considered to act permanently. They are “dead,” stationary, and unable to be removed. The probability of occurrence at a point-in-time is close to one and the variability is negligible. In comparison with other kinds of loads, the uncertainties of the magnitude are small.

The self-weight of the structural components normally provides the largest portion of the dead load of a structure. Non-structural elements that are unlikely to vary during construction and use (parapets, kerbs, etc.) have to be included in the calculation of the total dead load. Superimposed dead loads include the weight of non-structural elements that are likely to vary during construction and use (road surfacing, pipes and utility services, etc.). The weight density and the dimensions of a structural part are assumed to have Gaussian distribution. Values of total variability and spatial correlations can be found in [58].

5.2 Physical Loads

5.2.1 Live Loads

Live loads are not permanent and can change in magnitude and occurrence. They include loads due to the intended use of the structure (fixtures) and loads due to environmental effects caused by the sun, earth weather and water. In most cases they are considered as static or pseudo-static, but they are often dynamic.

Traffic loads arise by any kind of vehicles and pedestrians. The load due to car traffic is in most cases the dominating load in assessment of existing bridges and therefore plays an important role in the probabilistic-based assessment. The live loads of humans or equipment in building constructions are also important. Important dynamic loads caused by engines or machines, the movement of traffic on the bridge, centrifugal forces on curved bridges and braking forces have to be considered.

The magnitudes of live loads are difficult to determine with the same degree of accuracy, as it is possible for dead loads. Traffic data can be collected from weigh-in-motion systems and a probability density function can be constructed to describe the actual measured loads. This probabilistic, site specific loading model is then used in place of the deterministic, general loading model provided in codes of practice, and gives a more accurate load rating for that specific structure.

5.2.2 Wind Loads

The total wind load is the sum of acting outside and inside wind pressure that depend on the height and shape of the structure, roughness of the surrounding terrain and structures, topology, and basis wind velocity. Moreover, ice and snow have to be considered if the behaviour or the shape of the structure can be influenced.

A mean wind velocity (expressed as dynamic pressure), surface drag coefficient, aerodynamic shape factors and projection areas of structures estimate the wind load used for the static design. These assumptions are only valid for structures which are not vulnerable to vibrations. That means, for instance, that the deformations by consideration of the dynamic wind forces are not higher than 10% in comparison with static wind load. The mean wind velocity is the 10 minutes average wind velocity at an elevation of 10m above ground in horizontal open terrain exposure and is Weibull distributed. Logarithmic or power laws describe the variation of the mean wind velocity with height over horizontal terrain of homogenous roughness. The terrain roughness of the ground surface is aerodynamically described by size and spacing of obstacles. Various terrain categories are classified in [58].

Dynamic wind loads have to consider the vulnerability of vibrations in structures; for example, structures over 100m height, structures with special ratios between height and width, structures with small damping values, high rise buildings, skyscrapers, television towers, chimneys, suspension bridges, cable stayed bridge, and bridges with spans more than 150m. Different dynamic wind loads are distinguished: gust excited vibrations, vortex excited vibrations, flutter, galloping, buffeting, and ovalling. Table 5.2 gives some properties of the different wind types.

Table 5.2: Types and properties of dynamic wind loads

Gust excited vibrations	- gust of wind causes fluctuations of wind load - by strong wind load in wind direction, in high elevation negligible
Vortex excited vibrations (Karman vibrations)	- turbulence stream away from structure, normal to wind direction - especially by circular ground plan
Flutter	- interaction of air stream and elastic bridge movement - coupling of bending and torsion vibrations or only torsion - normal to wind direction
Galloping	- very slender, not circular structure
Buffeting	- two or more structures in direct neighbourhood - structures are situated in the stream away area of other structures, possible gust

	excitation or turbulence excitation
Ovalling	- cross- section deformed vibrations in the cause of periodic stream away of turbulence - thin walled, not stiffened, circular cylindrical shell structures

5.2.3 Hydrodynamic Loads

Hydrodynamic loads are resulting from wave loads or loads of non-steady flows.
Wave loads are depending on the extent of water surface or fetch, the wind velocity and duration. For (re)design the determination of definitive extreme waves, determination of sea condition spectra and the determination of the orbital velocities are important. Wave loads have a big influence on ships, offshore structures, and dams. Also moving liquids in vessels or tanks can result in respectable wave loads.
Another kind of hydrodynamic loads are fluid flow loads. They occur on the piles of offshore structures and rudders of ships, for example. Furthermore, pressure changes or fluid velocity changes in closed pipes (e.g., pipe break) or turbines can cause high fluid loads. Vortices from the fluid flow past a structure initiate vibrations. Vortex-Induced Vibration depends on shedding frequencies and their interactions, added masses and damping, Reynolds number, lift coefficient, and correlation of force components. [42]

5.2.4 Snow Loads

Snow loads are loads which are caused by snow and ice as static loads as well as by dynamical loads in consequences of drifts. This kind of load is mainly acting on the root of a structure. The size depends on various parameters like geographical location, roof exposure to wind, thermal building conditions, roof slope, and building configuration. Furthermore, the local snow load is influenced by snow drifts, sliding snow, unbalanced snow loads, ponding instability, and other building modifications.

In same cases, additional wind loads which could result from changes in shape or size of the construction works due to the presence of snow or the accretion of ice have to be considered. Snow may also influence the thermal loads of a structure.

The Eurocode 1 [16] offers an European snow map of snow loads. However, additional detailed national guidelines are available for every country.

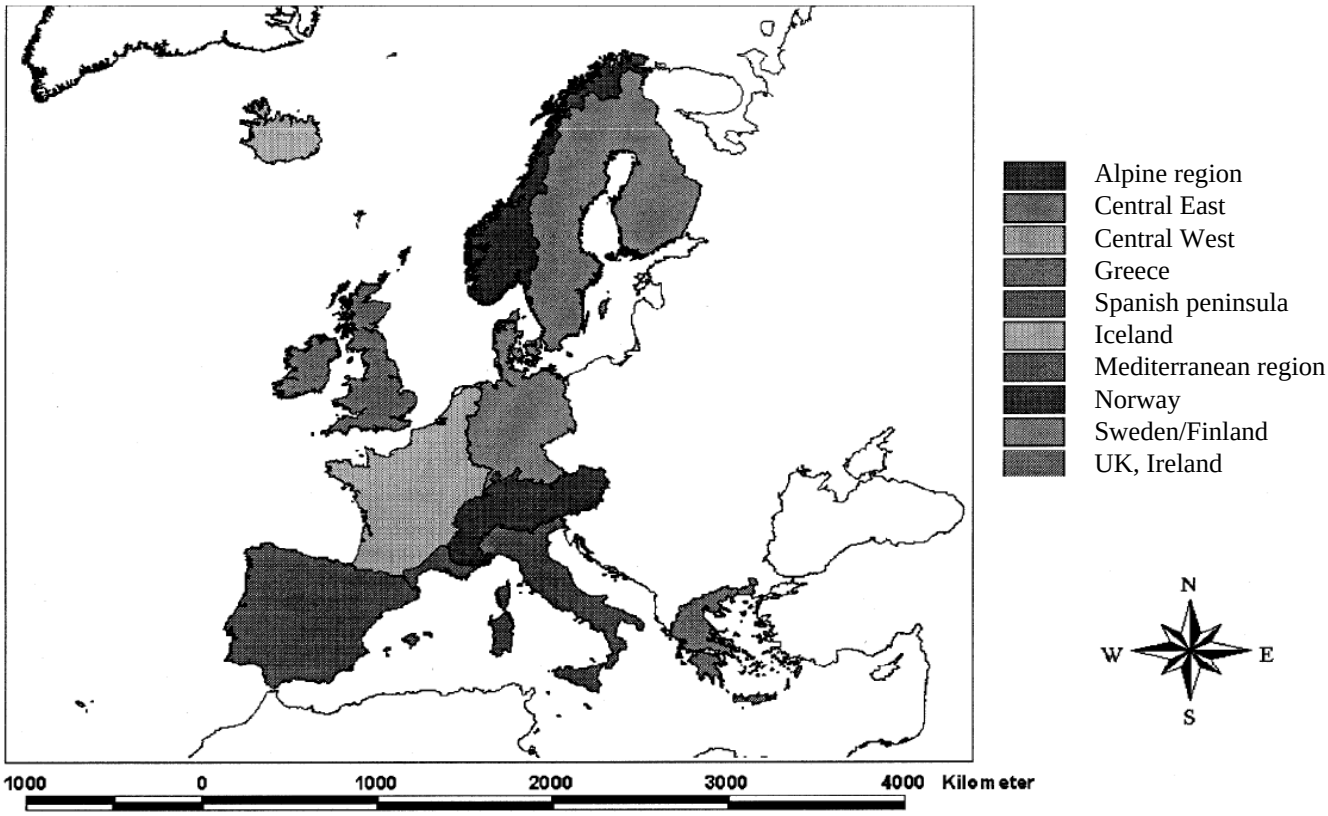


Figure 5.1: European regions of climate [16]

5.2.5 Earthquake load

Earth quake loads are dynamical loads considered to act mainly in horizontal direction. They are characterised by acceleration, velocity, and corresponding magnitudes. Seismic zone maps specify the probability of occurrence of an earthquake in a special region and its magnitudes. The damage due to an earthquake depends on relation between the eigenfrequencies of the quake, the building, and the soil. Further, magnitude, duration, and the distance to the epicentre are crucial., The most common method in earthquake design is the modal response spectrum analysis, implemented in design codes and supported by various software. A push-over analysis can be performed only if member capacities are known, and that can be obtained by a prior dimensioning based on the results of a modal response spectrum analysis.

For example, very stiff, compact structures with a small number of stories vibrate very fast and the displacements are small. They react on the quake as stiff, nearly undeformed body. Very soft, slender buildings have a low frequency and strong deformations. Their vibrations fall behind the vibration of soil, and consequently they avoid the soil movements because of their deformation. The disastrous effects of the 1985 Mexico City earthquake are a notorious counter-example of the above affirmation. In Mexico City, very tall and slender buildings were the most affected by the low predominant frequency of the earthquake motion, which was of about 0.5 Hz. The "Mexico City effect" (short predominant frequencies of ground motion) also produced considerable damages in Bucharest, during the March 4, 1977 earthquake. Few stiff buildings can be excited to own vibrations, which are in resonance with the quake. Thus, earthquake loads to a structure can occur at different sizes.

5.2.6 Fluid and Earth Pressure Loads

Liquids can produce horizontal loads and vertical loads. The vertical loads are the simple dead load of the fluid acting on the structure or the uplift forces (e.g., for a ship). Horizontal pressure forces of a liquid increase linearly with depth and are proportional to the density of the liquid.

Earth pressure loads are also acting as vertical dead loads and horizontal pressure loads. Normally, they produce no uplift forces. The horizontal pressure is not linearly distributed over the depth.

5.3 Thermal Loads

Loads in cause of temperatures or temperature differences can be important alone or in combination with different loads. Thermal loads are distinguished in loads effected by the design of the structure (e.g., orientation, condition of the external surfaces, HVAC (heating, ventilation, air conditioning) systems), by construction (e.g., hydration of massive structural parts), by service or use of the structure (e.g., vessels with cold or hot fluids), and by conditional on climate fluctuation of temperature (daily and annually). The extreme influence of temperature is given for a fire load case.

Changes in temperature result in elongation, deflections, movement of supports, residual stresses, and changes in the properties of the material. Low temperatures can cause embrittlement, increased stiffness, and decreased deformability; high temperatures can effect decreasing of strength and stiffness.

Especially, statically indeterminated structures (e.g., frames, bridges), line-like structures (e.g., railways, pipes, power supply lines, cables, airplanes), and structures with huge dimensions (e.g., halls, bridges) are affected by temperature changes.

5.4 Erosive Loads

Erosive loads depend on the materials behaviour of a structure. They are acting directly on the material and can cause local failures that can cause structural failures. It is possible to distinguish mechanical attacks, chemical attacks, and biological attacks.

5.4.1 Mechanical Attack

Mechanical attack includes every kind of mechanical abrasion on the surface of a material. Abrasion can be caused by the environmental influences, like water, wind, sand, or ice and by normal use of the material.

5.4.2 Chemical Attack

The result of every successful chemical attack is corrosion of the material. Such an attack can be initiated and accelerated by the sun, ultraviolet radiation, air, water, and chemical substances, like chloride of sulphate. Every kind of material can be affected by chemical attacks. For more details, see Section 4.2.

5.4.3 Biological Attack

Also biological agencies, like micro organisms, insects, fungi, or lichen can attack a material. They can destroy the material mechanically (e.g., eating of insects) or chemically (e.g., secreting of acids from fungi). Especially wood is affected by biological attacks.

5.5 Accidental Loads

Typically every natural disaster can be considered as an accidental load. Floods, landslides, and avalanches are exhaustively presented in Task 2.5 of the SAFERELNET [5].

5.5.1 Impact Loads

Impact loads are loads from the impact of vehicle, a ship, or an airplane against a structure. They can also occur by falling objects or by wind or an explosion moved objects. Often these events are induced by human or mechanical failure. These loads are often extraordinary. Thereby, most of designs do not consider such events. This depends on the occurrence probability of the event and the reliability of the structure. The crash of a car with a column in a garage or the crash of a forklift inside a hall is normal and shall be considered. The collision of cars with columns of bridges or of ships with bridges and offshore structures has also a higher probability. The crash of an airplane is considered for constructions with high safety levels (nuclear power plant). All these actions have a short duration and have to be only combined with dead load.

The probability of a structure being hit by an airplane is very small. Since the events in the USA in 2001 this assessment has been not true, because of the planned impact with structures by humans. Figure 5.2 shows the time-force histories of an impact by the airplane Boeing 707. The effect of fire is assumed neglected heres.

Boeing 707-320
m = 90 ton
v = 100 m/s
A = 36 m^2

0	0
30	20
150	20
200	90
230	90
250	20
320	10
330	0

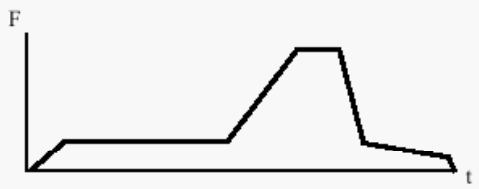


Figure 5.2: Impact characteristics for the airplane Boeing (t in [ms] and F in [MN]) [58]

5.5.2 Explosion and Weapon Action

Compression waves are the main action of chemical explosions or weapon action. In most cases these load cases are neglected.

5.5.3 Heat and Fire

The extreme influence of temperature is given for accidental fire load case. Further description is given in Section 5.3.

6 EXPERIMENTAL ASSESSMENT AND INSPECTION STRATEGIES

An assessment is a detailed investigation of a structure, especially after the analysis of the experimental results. Simple experiments can be an assessment as well. However, there are inspections or repairs which are not directly related to assessment, for example, during maintenance activities.

In general two types of inspection can be distinguished:

- The ‘qualitative inspection’, also called ‘visual inspection’, is performed to observe parameters of an object. Those parameters are, for example, surface characteristics, visible deformations, cracks, spalling, and corrosion. The description of damage of the object is given in qualitative terms, like no damage, minor damage, moderate damage, and severe damage. These categories have to be specified.
- The ‘quantitative inspection’, also called ‘in-depth inspection’, is a type of information resulting in a set of values of parameters that characterize the condition of structural elements. Crack depth and length, corrosion area and depth, displacements, residual stresses, damping, and eccentricities are examples of that information.

With regard to the method, experimental assessment methods can be categorised as:

- Inspections (i.e., simple visual inspections and measurements, sometimes this step is done within a maintenance process)
- Destructive techniques (DT)
- Non-destructive techniques (NDT)

Another classification relates to the object of assessment. It is possible to assess the:

- Resistance level (i.e., materials)
- Exposure level (i.e., determine the wind or waves by monitoring)
- Relation between resistance and exposure level (i.e., maximal loads for ultimate or serviceability limit states)

Important parameters of the structure (vibrations, deformations) can be continuously observed. This observation method is called monitoring.

Table 6.1 gives an overview about possible experimental assessment methods, which are described in detail in this section.

Due to cost and time aspects, excellent coordination between numerical and experimental assessment methods is required. Consequently, careful planning is recommended and the related uncertainties such as the probability of detecting some damage and/or the accuracy of the result should be specified and taken into account. Before starting with an experiment, the following questions should be answered (see also Eurocode 0, Annex D [24] for similar):

- What is expected from an experimental assessment?
- Are the expected results useful for the specific task?
- What is needed for the next numerical assessment step?
- Which kind of experimental assessment is the optimal for this aim?
- Do the expected results justify the expected costs?

Table 6.1: Overview of experimental assessment techniques

Method	**Resistance Level / Deterioration**	**Exposure Level**	**Resistance-Exposure-Relation**
Inspection (visual)	Visual assessment Dimensional measurements	Visual assessment Dimensional measurements	Visual assessment
DT	Removal of samples for testing Destruction of test specimens or replica objects	Not common	Static Load Testing (failure / ultimate limit state)
NDT	Dye Penetrant (DP) Eddy Current (EC) Electromagnetic Methods Magnetic Particle Inspection (MPI) Radar Methods Radiographic Examination Ultrasonic Methods Ultrasonic Creeping Wave Acoustic Emission Monitoring Infrared Thermography Alternating Current Potential Drop (ACPD) Alternating Current Field Measurement (ACFM) Flooded Member Detection (FMD) Metallographic Replication Hardness Measurements	Forces: ‘Weigh in Motion’ to measure loads applied on infrastructures Environmental condition monitoring Wind velocities – cup anemometer Wave heights – radar Motions: Sensors or accelerometers to measure motions Temperature: Thermocouples Resistance Temperature Detectors Fibre Optic Temperature Sensors Corrosive Agents: PH-value indicators Ion (Chloride, Sulphate, Nitrate) content measurements Acid content measurements	Static Load Testing (serviceability limit state) Proof Load Testing Diagnostic Load Testing Dynamic Load Testing

6.1 History of the assessment object

The first step of any assessment is to review the history of the assessment object. Possible available information about the structure is:

- Review of existing design documents like description of the structure, drawings, calculations
- Determination of the applied codes
- Information about the complete lifetime (accidents, extreme loads and other special events)
- Information about previous inspection and the consequences
- Information about possible, suffered modification of the structure

6.2 Visual assessment

It should be performed before every kind of in-depth experimental assessment.

To ensure an optimum collection of information an assessment concept is needed with the following requirements:

- Access equipment needed (e.g., scaffold, tower wagon), necessary downtimes (e.g., lane closure)
- Possible removal of everything that prevents good visual access
- Competencies and responsibilities
- Inspection equipment
    - Cleaning tools including wire brushes, screwdrivers, brushes, scrapers, etc.
    - Inspection tools including pocketknife, ice pick, hand brace, bit and increment borer for boring timber elements, chipping hammer, etc.
    - Visual aid tools including binoculars, flashlight, magnifying glass, dye penetrant, mirror, etc.
    - Basic measuring equipment including thermometer, centre punch, simple surveying equipment
    - Recording materials such as appropriate forms, field books, recorders, cameras, etc.
    - Safety equipment including rigging, harnesses, scaffolds, ladders, bosun chairs, first-aid kit, etc.
    - Miscellaneous equipment should include C-clamps, penetrating oil, insect repellent, wasp and hornet killer, stakes, flagging, markers, etc.

During a visual assessment the attention should be paid on:

- Inspection equipment
- Verification of information gathered during the planning of the assessment
- Old coatings, impregnation or protections
- Appearance of the surface
- Differences of the colour of surfaces
- Presence of cracks, their appearance and pattern
- Superficial deterioration of the material skin
- Deterioration of the material itself
- Exposed structural members, e.g., rebars in concrete structures

- Deformations of the structure
- Presence of water, humidity water, leakage, etc.

The findings and results have to be described in detail and summarized in a report that is the basis for any subsequent assessment.

6.3 In- Depth Assessment

In-depth assessment usually performed as a follow-up investigation to a visual inspection to better identify any deficiencies found. Important methods and techniques for this kind of assessment are introduced in the following sections.

6.4 Assessment of Members of Selected Materials

Common concrete member defects include cracking, scaling, delaminating, spalling, efflorescence, popouts, wear or abrasion, collision damage, scour, and overload. The inspection of concrete should include both visual and physical examination.

6.4.1 Assessment of Concrete Members

6.4.1.1 Detection of Cracks

Two of the primary deterioration effects noted by visual inspections are cracks and rust stains. Cracking in concrete is usually large enough to be seen with the bare eye, but it is recommended to use a crack gage to measure and classify the cracks. Cracks are classified as hairline, medium, or wide cracks.

Hairline cracks cannot be measured by simple means such as pocket ruler, but simple means can be used for the medium and wide cracks. Hairline cracks are usually insignificant to the capacity of the structure, but it is advisable to document them. Medium and wide cracks are significant to the structural capacity and should be recorded and monitored in the inspection reports.

Cracks can also be grouped into two types: structural cracks and non-structural cracks. The dead and live load stresses cause structural cracks. Structural cracks need immediate attention, since they affect the safety of the structure. Thermal expansion and shrinkage usually cause non-structural cracks in the concrete. Those cracks are insignificant to the capacity, but they may lead to serious maintenance problems. For example, thermal cracks in a deck surface may allow water to enter the deck concrete and corrode the reinforcing steel. The length, direction, and extent of the cracks and rust strains should be measured and reported in the inspection notes.

6.4.1.2 Detection of Delaminations

Delamination occurs when layers of concrete separate at or near the level of the top or outermost layer of reinforcing steel. The major cause of delamination is the expansion or the corrosion of reinforcing steel due to the intrusion of chlorides or salts. Hammer sounding is used to detect areas of unsound concrete and usually used to detect delaminations. Tapping the surfaces of a concrete member with a hammer produces a resonant sound that can be used to indicate concrete integrity.

Areas of delamination can be determined by listening for hollow sounds. The hammer sounding method is impractical for the evaluation of larger surface areas. For larger surface areas, chain drag can be used to evaluate the integrity of the concrete with reasonable accuracy. Chain drag surveys of decks are not totally accurate, but they are quick and inexpensive.

6.4.2 Inspection of Steel and Iron Members

Common steel and iron member defects include corrosion, crack, fatigue and overstress.

The most recognizable type of steel deterioration is corrosion. The cause, location, and extent of the corrosion must be recorded. This information can be used for rating analysis of the member and for taking preven-

tive measures, to minimize further deterioration. Section loss due to corrosion can be reported as a percentage of the original cross section of a component. The depth of the defect can be measured using a straight-edge ruler or calliper.

Cracks usually initiate at the connection detail, at the termination end of a weld or at a corroded location of a member. Cracks then propagate across the section until the member fractures. Since all of the cracks may lead to failure, structure inspectors need to look carefully at each and every one of those potential crack locations.

One of the most important types of damage in steel members is fatigue cracking. Fatigue cracks develop in structures due to repeated loadings. Since this type of cracking can lead to sudden and catastrophic failure, the inspector should identify fatigue-prone details and should perform a thorough inspection of these details. For painted structures, breaks in the paint accompanied by rust staining indicate the possible existence of a fatigue crack. If a crack is suspected, the area should be cleaned and given a close-up visual inspection. Additionally, further testing such as dye penetrant (see Section 6.5) can be done to identify the crack and to determine its extent. If fatigue cracks are discovered, inspection of all similar fatigue details is recommended.

Symptoms of damage due to overstress are inelastic elongation (yielding) or decrease in cross section in tension members, and buckling in compression members. The causes of the overstress should be investigated. The overstress of a member could be the result of several factors such as loss of composite action, loss of bracing, loss of proper load-carrying path, and failure or settlement of bearing details.

Similar to concrete members, there are advanced destructive and non-destructive techniques available for steel inspection. Some of the non-destructive techniques are described in the Sections to 6.5 to 6.7. Visual inspection is not confined to the surface, but may also include examination of bearings, expansion of joints, drainage channels, post-tensioning ducts and similar features of a structure.

6.5 Non-Destructive Techniques for Inspection (NDT Inspections)

Non-destructive testing is a descriptive term used for the examination of materials and components in such a way that allows materials to be examined without changing or destroying their usefulness. The types of detectable defects are cracks, porosity, voids, inclusions, and so forth. Visual Inspection as NDT was already described in Section 6.2. An overview of important methods is given in Table 6.1.

6.5.1 Electromagnetic Methods

Several electromagnetic effects have been exploited for detecting and localizing steel reinforcement in concrete, see Table 6.2.

Table 6.2: Electro magnetic scanning of a concrete surface

Physical effect	What can be determined?	Explanation
Permanent magnetism	location, concrete cover	attractive power between reinforcement and a permanent magnet on the concrete surface is measured
Electro- magnetic induction	location, concrete cover, diameter	magnetic flux is influenced by magnetic material in the electro-magnetic field
Scattering of a magnetic field	location, concrete cover, diameter	first the reinforcement is magnetized by a permanent magnet; then magnetic field is measured by using a hall probe; the steel reinforcement causes a scattering of the field

The effect of magnetic induction is the one that is predominantly used in commercial devices. State-of-the-art products allow an easy scanning of the concrete surface and generate the results in an image format.

The advantages of the electromagnetic methods are:

- The concrete cover can be determined reliably, whereas the determination of the bar diameters in practical cases sometimes causes problems.
- Results are obtained immediately; no time-consuming post-processing is necessary.
- State-of-the art devices are cost-effective.

There are some limitations of the electromagnetic methods:

- The methods work reliably only up to a concrete cover of about 100 mm.
- For high reinforcement ratios the resolution of the method might be not sufficient. So it can be happen that bars located close to each other are detected as one bar.

For most of the practical cases, however, the electromagnetic methods and the corresponding commercial devices are the most effective tools.

Figure 6.1 gives an example of the function of a covermeter. A measurement is performed by rolling the covermeter across the surface of the concrete. A concrete specimen with rebars and the corresponding diagram of a measurement is shown. The horizontal axis correlates with the measured distance, while the vertical axis correlates with the measured cover. The highest points of the curve show the positions of rebars. These peaks correspond to the minimal distance between the coil and the rebars and are equal to the amount of concrete cover.

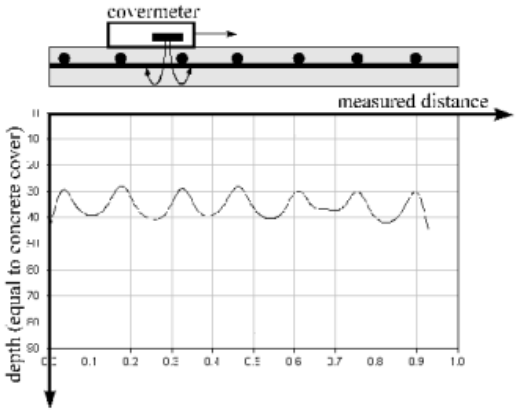


Figure 6.1: Diagram of a covermeter measurement, [71]

6.5.2 Ultrasonic Methods

Ultrasonic inspection uses sound waves of short wavelength and high frequency (above 18kHz) to detect flaws or measure material thickness. Usually pulsed beams of high frequency ultrasound are used via a hand-held transducer that is placed on the specimen. Any sound from that pulse that returns to the transducer like an echo is shown on a screen, which gives the amplitude of the pulse and the time taken to return to the transducer. Defects anywhere through the specimen thickness reflect the sound back to the transducer. Flaw size, distance and reflectivity can be interpreted. Because of its complexity considerable technician training and skill is required.

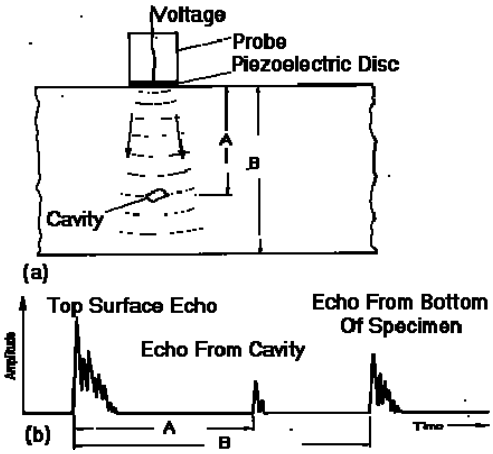


Figure 6.2 Principle of ultrasonic testing [106]

6.5.3 Magnetic particle inspection

Magnetic particle inspection is used to find surface and near surface flaws in ferromagnetic materials such as steel and iron. It has an advantage over liquid penetrant since it requires less surface cleaning.

A magnetic field is established in a component made from ferromagnetic material. The magnetic lines of force or flux travel through the material and exit and renter the material at the poles. Defects such as cracks or voids are filled with air that cannot support as much flux, and force some of the flux outside of the part. Magnetic particles distributed over the component will be attracted to areas of flux leakage and produce a visible indication.

There are variations in the way the magnetic field is applied. But they are all dependent on the above principle. The iron particles can be applied dry or wet suspended in a liquid, coloured or fluorescent.

While magnetic particle inspection is primarily used to find surface breaking flaws, it can also be used to locate sub-surface flaws. But its effectiveness quickly diminishes depending on the flaw depth and type. Surface irregularities and scratches can give misleading indications. Therefore it is necessary to ensure careful preparation of the surface before magnetic particle testing is undertaken.

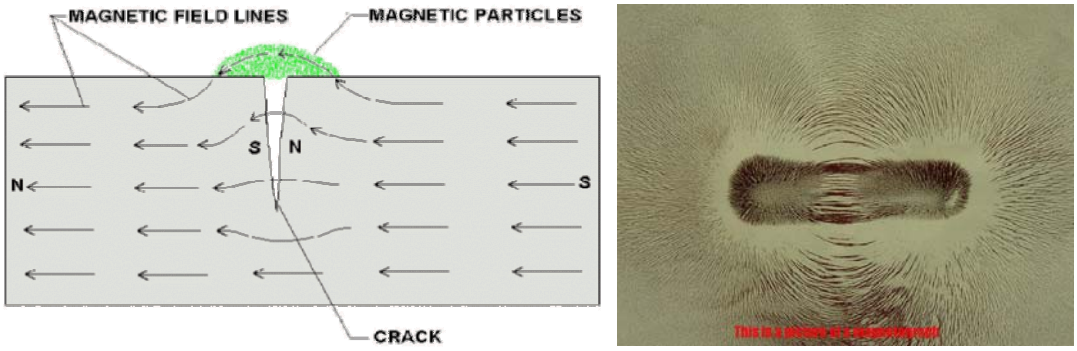


Figure 6.3: Principle of Magnetic particle inspection (left); Magnetograph (right) [96]

6.5.4 Radar Methods

The radar technology bases on the impulse-echo principle. The advantage of the Radar principle is the maximum inspection depth of about 50 cm, that means larger than for electro-magnetic methods. But, the interpretation of the data obtained appears to be difficult and reinforcement close to the concrete surface cannot be identified clearly. The Radar method, therefore, is beneficial in the case of large reinforcement diameters and high concrete covers. A useful application is the localization of prestressing cables. In this case the insensitivity against near surface reinforcement appears to be an advantage of the method. Usually prestressed cables are located deeper inside the concrete members than the untensioned reinforcement.

6.5.5 Acoustic Emission Monitoring

Acoustic Emission (AE) refers to the generation of transient elastic waves during the rapid release of the deformation energy from localized sources within a material. The source of these emissions in metals is closely associated with the dislocation movement accompanying plastic deformation. In metals and concrete the initiation and extension of cracks in a structure under stress calls acoustic emission. In reinforced concrete corrosion products formed on a corroding rebar push out on the concrete surrounding. Other sources of Acoustic Emission are melting, phase transformation, thermal stresses, cool down cracking and stress build up. Slow crack growth in ductile materials produces few Acoustic Emission (AE) events, whereas rapid crack growth in brittle materials produces large quantities of high amplitude

The Acoustic Emission technique is based on the detection and conversion of these high frequency elastic waves to electrical signals. This is accomplished by directly coupling piezoelectric transducers on the surface of the structure under test and loading the structure. The output of each piezoelectric sensor is amplified through a low-noise preamplifier, filtered to remove any extraneous noise and furthered processed by suitable electronic equipment.

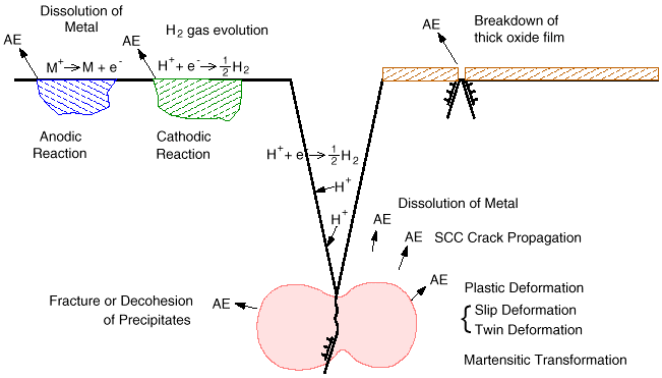


Figure 6.4: Schematic Acoustic Emission sources during corrosion, stress-corrosion cracking and corrosion-fatigue [90]

Acoustic Emission (AE) is mainly used for metals. Quite new is the application of the acoustic emission analysis to concrete. Concrete differs from the above-mentioned materials by its inhomogeneity, its high attenuation, and the large dimensions of the structures created with it.

Acoustic Emission analysis on concrete is still far away from a routine application, but it has proved to be a valuable tool for laboratory tests and scientific investigations. The progressive automation of the method will increase considerably its applicability and acceptance.

Acoustic emission is a cost-effective and sensitive method for monitoring the integrity of load-bearing structures and pressurized systems. It monitors large areas and, if necessary, difficult-to-access areas. Once defect indications are observed, acoustic mission is backed up by the other non-destructive testing methods.

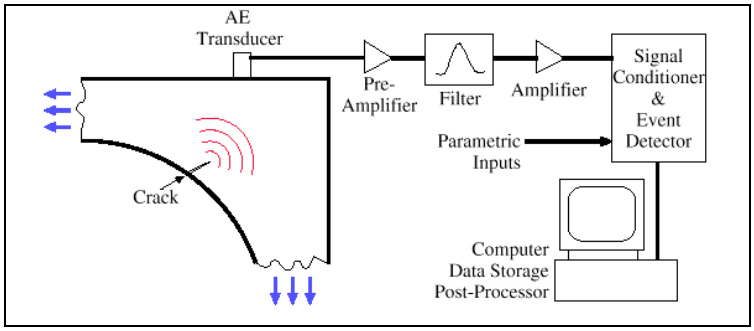


Figure 6.5 Acoustic Emission testing system setup, [53]

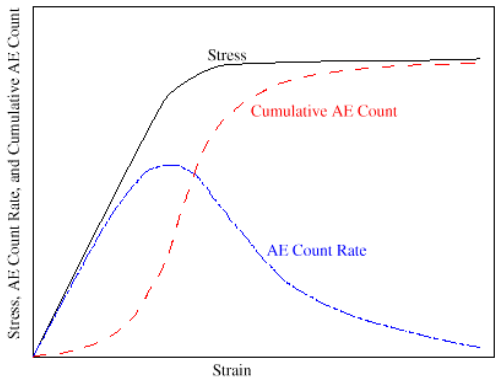


Figure 6.6: Tensile stress- strain curve and Acoustic Emission signals, [53]

6.5.6 Infrared Thermography (IT)

Infrared thermography (IT) is a non-contact optical method, which utilizes differences in heat transfer through a structure to reveal the locations of hidden defects. Heat is applied evenly to the surface of a component. As it cools, the component emits energy of various wavelengths. Variations in the cooling of different regions of the component are detected and measured with an infrared camera. The camera is connected to a computer, which converts the reading into a thermal image. Defects in the component can be detected from the variations shown on the thermograph.

Typical types of defects that can be located using IT include voids in the grout of masonry walls, delaminations in concrete slabs, and excessive moisture in wall and roof insulation. Infrared thermography is used for the evaluation of bridge decks including the detection and quantification of delamination. However, infrared thermography is limited by environmental conditions and has difficulty by evaluating decks with asphalt overlays. The dual-band infrared thermography using two different infrared wavelengths simultaneously overcomes some of the operational problems (primarily surface emissivity variations) encountered with standard infrared thermography.

One of the methods selected for the bridge inspection is an active or transient thermography. This method is dissimilar to the conventional thermographic methods in the utilization of time-dependent heating or cooling of the structure. Depending on the type of defect and thermal characteristics of a target, an external heating or cooling is applied in the form of short energy pulses. The created thermal perturbation is then followed by a differential time-resolved infrared image analysis.

Coating defects, such as blistering and sub-surface corrosion spots, or excessive corrosion of the steel members can be detected in infrared images as a result of the differences in the thermal diffusivity of the defective and non-defective areas. The temperature rise of the heated surface is governed by the amount of energy deposited and the speed of application, combined with the thermal properties of the surface material. As regards to the detection of defects, the amount of contrast observed at either surface is a function of the defect's dimensions and depth from the observed surface, the initial temperature rise and the material's thermal properties.

The physical phenomena behind active infrared inspection can be visualized by following propagation and detection of an induced thermal perturbation. An induced thermal 'wavefront' can be imagined to flow from the exposed surface into the material. For a defect-free, homogenous material, the 'wavefront' of heat passes through uniformly. However, where there are defects, such as delaminations or cracks (filled with air or an oxide), these create a higher thermal impedance to the passage of the 'wavefront'. Physically, when the defects are near to the surface, they restrict the cooling rate due to an insulation blocking effect, and thereby produce 'hot spots'. When this surface is viewed by a thermal imager, temperature differences arising from the defect's presence become clearly visible shortly after the deposition of the heat pulse.

The equipment required to perform active thermography falls into two separate areas: the heating source and the thermal imaging/analysis system. The typical heating sources utilized are pulsed quartz lamps. Thermographic analysis can be also performed by cooling the target instead of heating.

The thermal/imaging analysis system typically includes infrared thermographer integrated with PC-based image acquisition and processing hardware.

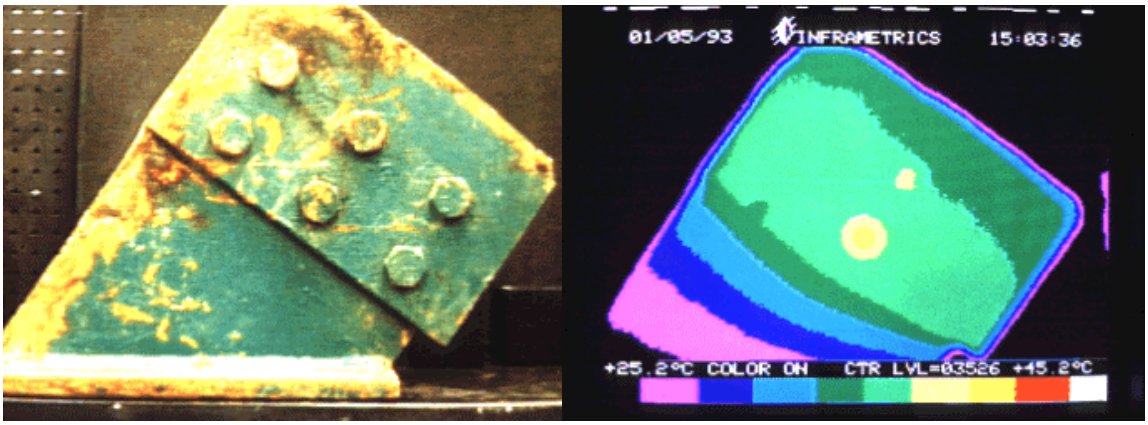


Figure 6.7: Bolted connection plat with two loose bolts, investigated by thermography [93]

6.5.7 Penetrant Testing

Penetrant testing is one of the most frequently used methods of NDT. This method is used to locate and identify surface defects or flaws (cracks or pores) in nonporous materials (metals, ceramics, plastics). It is easy to perform, reliable, rapid and inexpensive. Applications are:

- Detection of cracking and porosity in welded joints
- Detection of surface defects in castings
- Detection of fatigue cracking in stressed materials

The surface of the part under evaluation is coated with a penetrant in which a visible or fluorescent dye is dissolved or suspended. The penetrant is pulled into surface defects by capillary action.

After a waiting period to insure the dye has penetrated into the narrowest cracks, the excess penetrant is cleaned from the surface of the sample. A white powder, called developer, is then sprayed or dusted over the part. The developer lifts the penetrant out of the defect, and the dye stains the developer. Then by visual inspection under white or ultraviolet light, the visible or fluorescent dye indications, respectively, are identified and located, thereby defining the defect.

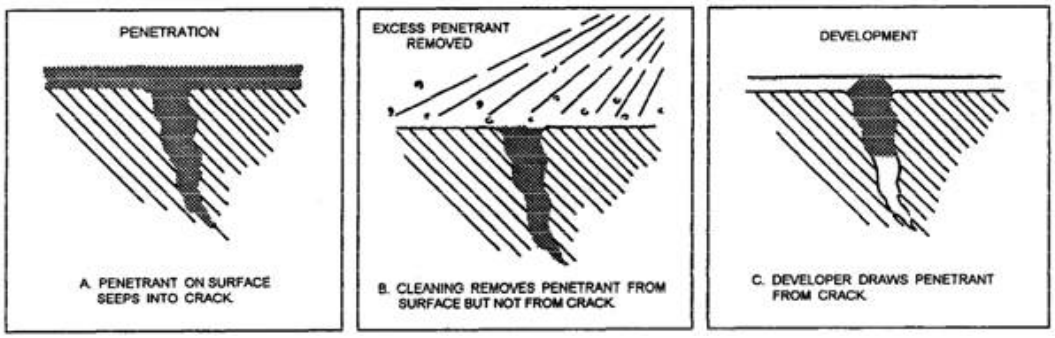


Figure 6.8: Principle of Liquid Penetrating Testing [109]

6.5.8 Eddy Current

This method is used primarily for detecting surface flaws and can only be used for conductive materials.

Electrical current is passed through a coil of wire producing a magnetic field. When the coil is placed near a conductive material, the changing magnetic field induces current flow in the material. These currents travel in closed loops and are called Eddy Currents. Eddy Currents produce their own magnetic field. The presence of a defect will affect this magnetic field and consequently the distribution or the density of the current in the coil. This can be measured and used to find flaws and characterize conductivity, permeability, and dimensional features.

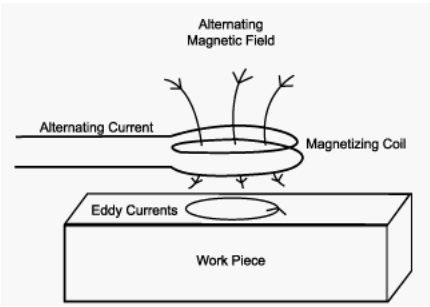


Figure 6.9: Principle of Eddy Current [88]

6.5.9 Radiographic Examination

Radiography is a method of detecting internal flaws by passing an ionising radiation through an object and recording the results of radiation on radiographic film. An X-ray image of the flaws is produced.

It detects defects that are not open to any surface, but the procedure requires access to opposite sides of the specimen to be inspected. The test object is placed between the radiation source and the detector. The thickness and the density of the material that X-rays must penetrate affect the amount of radiation reaching the detector. This variation in radiation produces an image on the detector that shows the internal features of the test object.

The radiation is generated by X-ray machine or by an isotope such as Iridium or Cobalt. This method is frequently used for examination of welds in shipyards.

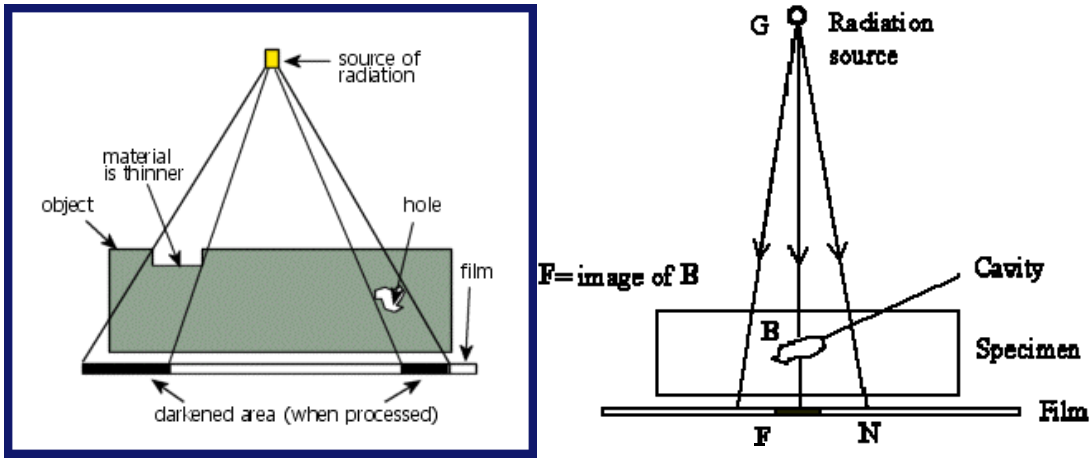


Figure 6.10: Principle of Radiography [89]

6.6 Load Capacity Assessment

This Section is based on [47].

6.6.1 Static Load Tests

Analytical or predictive approaches used to determine load ratings may be overly conservative. For example, the actual load carrying capacity of a bridge is often higher than the predicted capacity. This may be due to system effects, load redistribution, and so on. Thus a proof, but also a diagnostic or load test may be more appropriate if analytical analyses produces an unsatisfactory load rating or an analytical analysis is difficult to conduct due to deterioration or lack of documentation

A diagnostic test may be used to verify or refine analytical or predictive structural models, whilst a proof load test is used to assess the actual load carrying capacity of a structure.

Diagnostic testing has the benefit of explaining why the structure is performing differently than assumed. The disadvantage to this method, as opposed to proof load testing, is that the results are determined for service loads and need to be extrapolated to ultimate load levels.

6.6.1.1 Proof Load Testing

A proof load test involves the process of loading and observation of the related reactions of an existing structure or a part of it. The purpose is to assess its load bearing safety and serviceability. It is characterized by the fact that the testing load is increased following a fixed regime of loading and unloading cycles until the ultimate testing load is reached. The ultimate testing load is defined as the limit value of an acting load during the loading test, at which just no such damages occur that would affect the bearing capacity and serviceability for the future lifetime of the structure. Although a load test of a full-scale structural element or of a complete structure is a costly and time-consuming operation, it generally yields valuable results. A single loading case may not be able to provide the range of information required and it may be necessary to perform a series of tests to satisfy the technical requirements.

Loads may be applied using dead weight or by mechanical means (Figure 6.11, left) and consideration need to be given to any effect the loading method may have on the observed behaviour. Materials, which can be used, include building materials, water (Figure 6.11, right) cast-iron weights and loaded vehicles. Water is

fairly easy to handle by pumping but it has the disadvantage in terms of its low density compared with other materials.

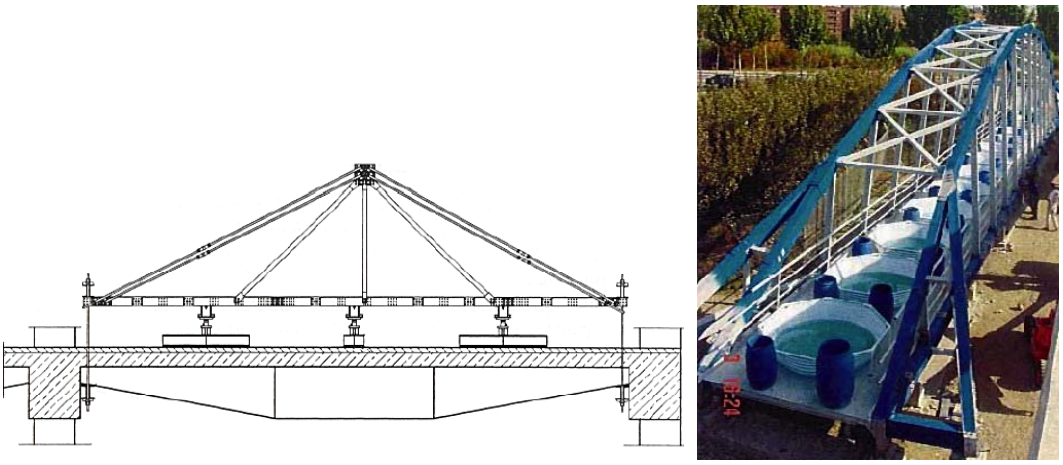


Figure 6.11: Static load test with water tanks and truss frame [79]

Conventional methods of proof loading tests are time-consuming and lead to long-term road closings for bridges. Furthermore, they can result in a perforation of the bridge sealing, which protects the structure against water penetration. Another possibility to apply loads to the structure is by placing trucks or locomotives or even military tanks of known weights at various points of the structure. These tests can also be done with incremental loads. Testing may also be carried out by passing the test vehicles over the bridge at incremental speeds starting from static position.

In order to provide a more efficient method for the in-situ loading tests some partners in Germany developed a special loading vehicle BELFA [85]. The layout of the operation mode of the loading vehicle is shown in Figure . The application of BELFA is limited to road bridges up to a span (total or individual) of 18 m.

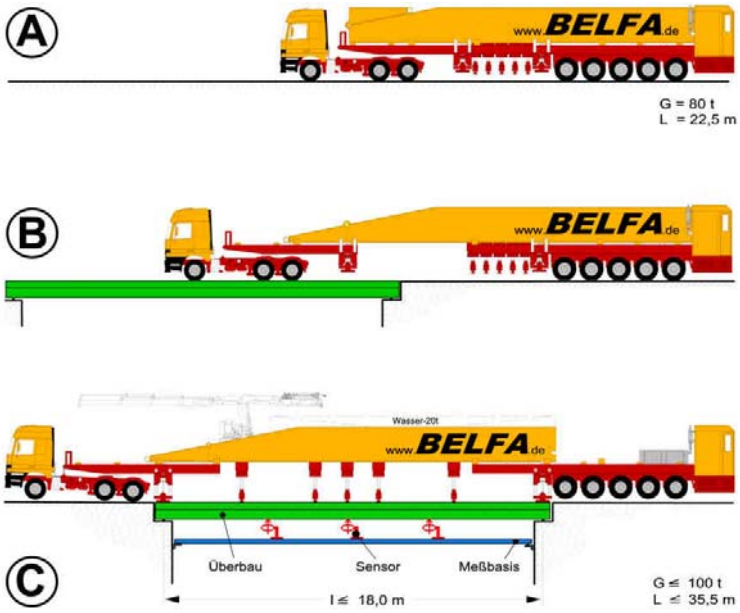


Figure 6.12: Operation modes of Loading Vehicle BELFA [85]

6.6.1.2 Diagnostic Load Testing

Diagnostic load testing involves driving pre-weighed trucks or other vehicles across a structure along various transverse paths at both a crawl speed (pseudo-static test) and at full speed (dynamic test). The weight of the vehicle is chosen to not exceed the current rating level of a bridge. Before testing starts, strain transducers and other instruments such as displacement gages are set up at predetermined locations of the structure. Measurements are recorded as the test vehicle is driven across structure. From the data collected during the diagnostic test, a number of significant properties that affect actual load-carrying capacity can be determined. These properties, which are typically estimated in order to perform a traditional load rating, include load

distribution, support restraint, flexural resistance of the superstructure elements (cross-sectional properties including the state of composite action), and effects of impact. Furthermore, the recorded strain can also help indicate the level of other, more difficult to quantify, sources of strength. By gathering enough response data, a more accurate structural model of the bridge can be created and used in the final bridge rating.

6.6.2 Dynamic Load Tests

When the structural damage is small or it is in the interior of the system, its detection cannot be carried out visually. A useful more non-destructive assessment tool is dynamic testing. It relies on the fact that the occurrence of damage or loss of integrity in a structural system leads to changes in the dynamic properties of the structure, which are natural frequencies, mode shapes and damping. For example, the degradation of stiffness due to the cracking of the reinforced concrete changes the modal properties. Dynamic measurements can give information on the position and severity of the damage that has occurred. Generally, the eigenfrequencies decrease, the damping increases and the mode shape changes slightly due to such damage of concrete.

Forced vibration testing incorporates the methods where the vibration is artificially induced. Amplitude and frequency of the applied input excitation are in most cases under control by the use of properly designed excitation systems.

Forced vibration tests have the advantage of suppressing effects of extraneous noise in the measured structural response. Tests using both ambient and forced vibration methods have shown that damping and frequencies can be measured more accurately with forced vibration and that the higher modes can only be excited to measurable levels by forced excitation.

The physical means through which the excitation is realized may be termed a vibrator, exciter, or shaker. It is a device used for transmitting a vibratory force into the structure. The excitation device can be either of the contacting type, which means that the exciter stays in contact with the test structure throughout the testing procedure, or of the non-contacting type such as impactors. Physically mounted devices like vibrators are used for full-scale testing of large structures. Appropriate contacting vibrators are usually of the eccentric rotating mass or electrohydraulic type.

Figure 6.13: Dynamic testing exciter: eccentric mass vibrator; drop weight;
servo- hydraulic shaker (in combination with BELFA [85])

6.7 Measurement of physical parameter

This section describes the methods for the measurement of important parameters like deformations (displacements, strains), forces, motions, and temperatures. An extensive collection of the working principles of various sensors is given by [100].

6.7.1 Deformations

6.7.1.1 Global Deformation Measurement

The most important sensors for deformation measurements are linear variable differential transducers (LVDT), magnetostrictive transducer, draw wire transducers, capacitive position sensors, and rotary capacitive displacement transducers that are explained in this section.

Linear variable differential transducers are the forefront of displacement measurements after over 90 years. They are very robust and can be applied according to the requirement, like resistance to water, pressure, and radioactive radiation. The resolution of a LVDT is infinite and guarantees a high accuracy and repeatability. Its principle is based on magnetic transfer, between one primary and two secondary coils with an iron core in the center. Hence, no physical contact across the sensing element is present.

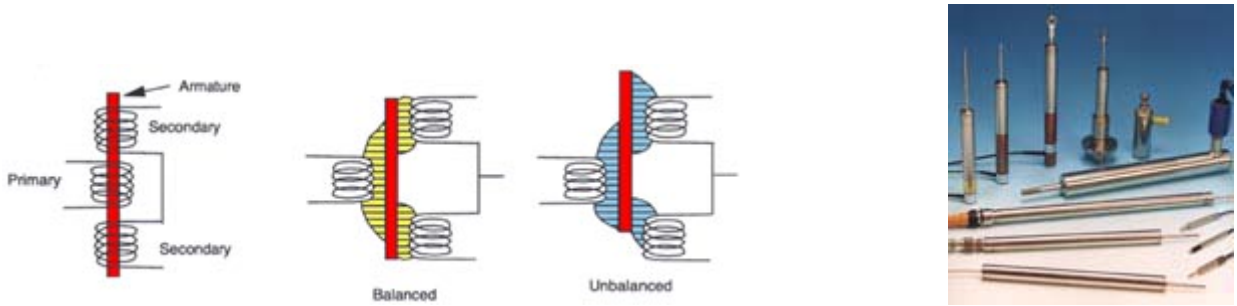


Figure 6.14: Principle (left) and examples (right) of LVDT's [100]

Magnetostrictive transducers are robust sensors and ideal for continuous non-contact displacement measurements with a clearance between 0.5m and 7.7m. The principle is based on various magneto-mechanical effects. The interaction between a current pulse, sent out from the electronics module at one end of the transducer, and a magnetic field generates an ultrasonic wave, which travels along a wave-guide. The resulting acoustic pulse is detected. By the measured time interval between the output and input pulses, the distance to the position of the magnet can be determined. There is no wear, and no effect from any non-ferrous dirt, debris or liquids in the space between transducer and magnet. [97]

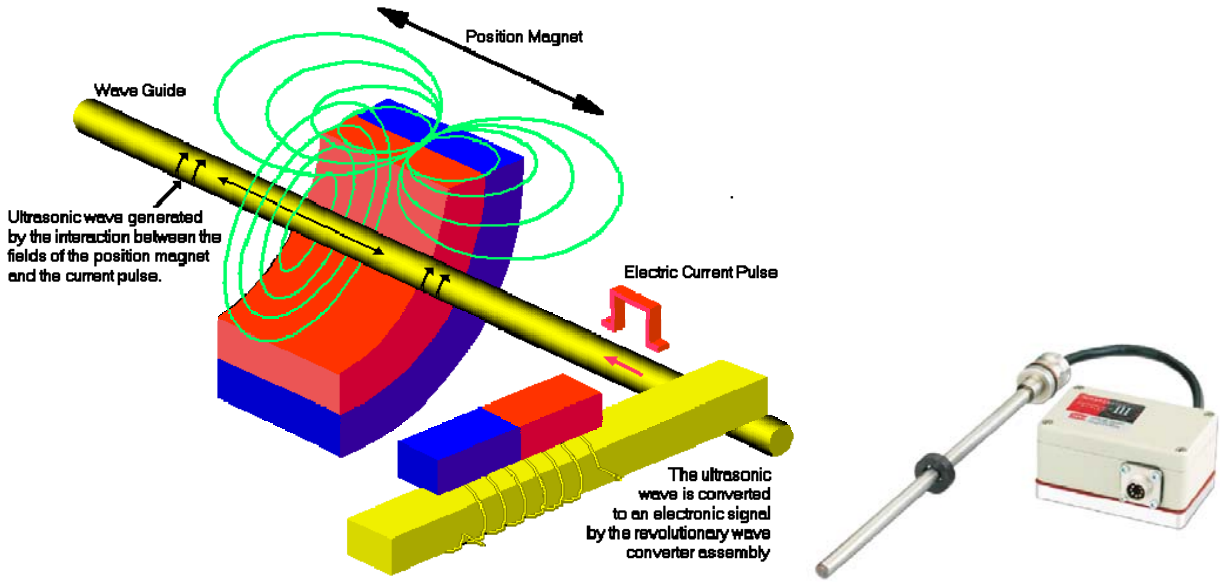


Figure 6.15: Principle (left [97]) and
example (right [112]) of magnetostrictive transducer

Draw wire transducers also known as string pots are effectively electronic tape measures. They give an output proportional to the length of a spring-loaded cable, which is pulled out of the unit. The transducer can be easily installed by bolting down the housing and attaching the draw wire to the part, which is to be meas-

ured. In most cases, the cable is a plastic coated steel wire and is tensioned by an internal spring pack. Several outputs are possible, for example, in form of resistance, Voltage, current, or a display. [97]

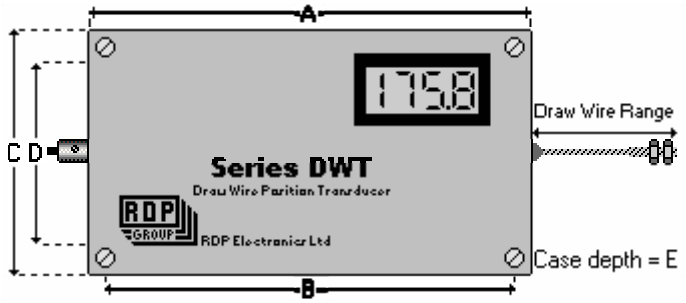


Figure 6.16: Draw wire transducer [97]

Capacitive position sensors are non-contact sensors that measure the distance between their front face and the target. They are suitable for the measurement of machine parts on a production line for example. Anything which contains water or carbon in a reasonable level is a suitable target material, especially, metals, many types of plastic, and glasses. A good target has a high relative permittivity and low resistivity. The capacitive position sensor is not applicable in combination with splashing water. [97]

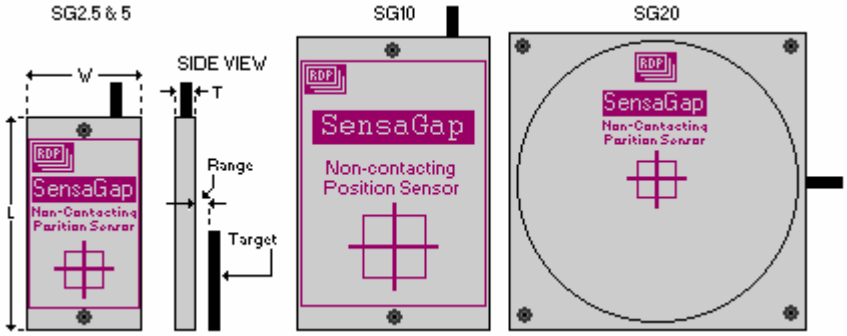


Figure 6.17: Three types of capacitive position sensors [97]

Rotary capacitive displacement transducers measure the rotary position of its shaft, with respect to its body. No physical contact exists across the sensing element. The quality of the data is ensured, as the sensing element does not contribute to any stiffness. The application field of such sensors is wide. It is useful for angular position, as well as for incline or tilt measurements.

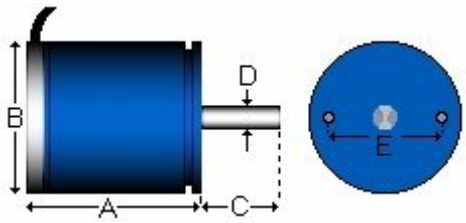


Figure 6.18: Rotary capacitive displacement transducer [97]

6.7.1.2 Local Deformation Measurement

Short-term strains and long-term strains are distinguished. Short-term strains can be measured using electrical resistance strain gauges. Although these gauges can be attached directly to reinforcing steel in the field, this method requires attachment under field conditions and is very difficult. Consequently, the gauges should be attached to separate lengths of reinforcement. This allows the gauges to be attached to the bars under laboratory conditions and permits proper attachment of leads and waterproofing. The gauged bars can be tied directly to the reinforcement cage. Electrical resistance strain gauges can be applied to hardened concrete surfaces. Depending on the character of a material surface, special techniques are needed to prepare the surface prior to application of the gauges. Weldable electrical resistance strain gauges are available for use on steel structures and large size.

Measuring long-term strains should be done with gauges designed specifically for this purpose. While electrical resistance gauges may not be recommended for this purpose, there exist other gauge technologies (fiber

optical sensors, Carlson strain meters, vibrating wire gauges, etc.) that are designed to have long-term stability, are robust for installation on site, and are provided with leads already attached. The gauges can be either installed directly in the structure or by casting them in concrete blocks. Then the blocks are cast in the concrete structure. The latter method is discussed because of doubts about the effect of differential creep and shrinkage between the block and the concrete.

Long-term strain measurements are generally used to determine prestress losses. For this purpose, when positioning the gauges, it is necessary to consider distribution of the prestressing forces. Gauges should be placed at the centre of gravity of the prestressing force close to midspan and parallel to the strand.

Measurements of long-term strains should begin as soon as the concrete is placed although readings in fresh concrete may not be appropriate for the initial readings. Initial readings should be taken before and after every significant event that affects the stress in the structure. After completion of the structure, it may only be necessary to obtain data every few months.

Sufficient readings should be obtained so those trends in the data are clearly discernable.

- Mechanical Strain Gauges: both short-term and long-term surface strains can be measured using mechanical strain gauges. In this method, the distance between two points on the concrete surface is compared with the length of a standard invar reference bar.
- Vibrating Wire Strain Gauges: The advantage of the vibrating wire strain gauge over more conventional electrical resistance (or semi-conductor) types lies mainly in the use of frequency, rather than a voltage, as the output signal from the strain gauge. Frequencies may be transmitted over long cable lengths without appreciable degradation caused by variations in cable resistance, contact resistance, or leakage to ground. Vibrating wire gauges also have excellent long-term zero stability.

Figure 6.19: Vibrating Wire Strain Gauges, [107]

Recent advances in fiber optic technologies led fiber optic strain sensors to become an alternative to classical resistance gauges. These sensors are widely immune against rough environmental conditions and show good long-term performance. Typical fiber optic strain sensors are fiber Bragg gratings.

6.7.2 Forces

Load cells have the function, to measure the applied load. Load cells can be distinguished according to the type of the output signal (pneumatic, hydraulic, and electric like strain-gage load cells).

Weigh in motion of road vehicles provides useful tools to collect data about the weights and dimensions of road vehicles, and the loads applied to the infrastructures. The main applications of WIM are traffic and road monitoring, pavement engineering, bridge engineering, and weight limits enforcement.

6.7.3 Motions

The dynamic response of a structure concerning ambient vibrations (state of the art technology) can be assessed by use of different methods, which could be measurements of motion, deformation or forces along the points of interest.

Concerning measurement of motion the most important methods are acquisition of displacements, velocities or accelerations. Basically it does not play any role which of these parameters is measured, because integration or differentiation can transform any of these signals to each other. The current practice is using accelerometers for dynamic testing, because very accurate results can be obtained with this type of transducer.

In principal accelerometers are devices constituted by mass-spring-damper-systems that produce signals proportional to the acceleration in a frequency band below their own resonance. Different systems and working principles can be distinguished by the construction of an accelerometer; the main groups are piezoelectric sensors.

Piezoelectric sensors are the most frequently used accelerometers. The mostly cylindrical shapes have dimensions of about 2.5cm for height and 1.3cm in diameter, for typical unit rated 10000 g. A wide temperature range (-195 to 260°C) is possible. Most units have a frequency range of at least 5 kHz, with typical resonance frequencies of over 30 kHz and a minimum response in the range of 0.01 to 5 Hz. Consequently, they do not respond to constant accelerations. For accurate measurements, the accelerometer and cable must be calibrated as a system. If the cable length is changed, another calibration is required. By measuring the charge generated by an acceleration, whereby a charge amplifier is required, the cable effects vanish. The charge amplifier is expensive compared to other amplifiers. High insulation resistance and low noise in the cabling between the accelerometer and the charge amplifier is required. Otherwise, measurement errors will be introduced into the system. Thus, the cable must be of special low-noise type and must be fastened in place to reduce vibration-induced electrical noise. Piezoelectric accelerometers with built-in amplifiers are also available, which overcome the cabling and amplifier problem, but need a separate power supply. [101]

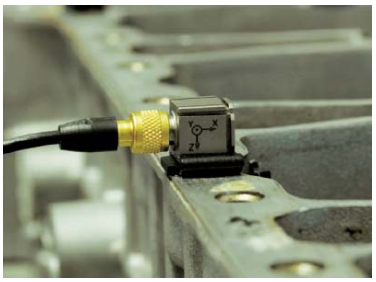


Figure 6.20: Piezoelectric accelerometer (left) [86], force-balance accelerometer (right) [84]

The electromechanical **force-balance accelerometer** is another commonly used sensor, also called servo accelerometer. They use a controlled-loop system to measure the accelerations. They consist of a permanent magnet and a sensing mass attached to a moving coil. The internal circuitry consists of a position sensor, error amplifier, and a voltage-to-current (*V*/*I*) converter. When the mass begins to move away from the null position because of an external acceleration, the position error generates a coil current, returning the mass to the null position. A voltage signal representing coil current is used as the accelerometer output signal. Forced-balance accelerometer require supply voltage of ±15 Vdc. Output signals of ±5 V are typical. The maximum frequency is limited to about 50-100 Hz. [101]

Capacitive sensors sense a change in electrical capacitance between two internal plates with respect to acceleration, to vary the output of an energized circuit. They are very stable and accurate devices, which are insensitive to base strain and transverse acceleration effects. Low-amplitude, low-frequency requirements can be satisfied.

Figure 6.21: Triaxial capacitive accelerometer [114]

6.7.4 Temperature

There are three units to measure temperature: Kelvin, Celsius, and Fahrenheit.

Temperature sensors can be distinguished in contact and non-contact temperature sensors. Contact temperature sensors are thermocouples, thermistors, liquid in glass thermometers, resistant temperature detectors, bimetallic thermometers, filled system thermometers, and semiconductor thermometers. Non-contact temperature sensors are radiation thermometers, thermal imagers, ratio thermometers, optical pyrometers, and

fibre optic temperature sensors. A recommended information source where a good overview with many references is given is [108]. In the following the mentioned methods will be described.

Table 6.3: Comparison of various temperature sensors.

Sensor Type	Output	Range °C	Accuracy ±°C	Robustness	Cost
Thermocouples	40-50 µV/°C	-270 to 2300	1.5	high	low
Thermistor	5% / °C	-50 to 200	0.2	high	medium
RTD	0.4% / °C	-200 to 600	0.2 to 0.3	medium	low to medium
Semiconductor	10mV/C or 1µA/°C	-40 to 125	1.5	medium	low
Bimetallic	displacement	-100 to 300	2	high	low
Fibre optic	various	-100 to 200	1	medium	very high

Thermocouples are the simplest sensors to measure a temperature. The physical phenomenon of the Seebeck-effect is used, where a thermal electromotive force is generated when the junctions of a pair of electrical conductors (metals) have different temperatures. Any two wires of different materials can be used as a thermocouple if connected together as in Figure 6.22. When the junction temperature, T_{Hot}, is different from the reference temperature, T_{Ref}, a low-level DC voltage will be available at the +/- terminals. The value of E depends on the materials, on the reference temperature, and on the junction temperature. The three most common TC are Iron-Constantan (Type J), Cooper-Constantan (Type T), and Chromel-Alumed (Type K). The accuracy of the measurement increase with decreasing wire diameters. The smallest practical usable diameter is 75µm. The ASTM standards ([16], [21], etc.) give instructions for use and application.

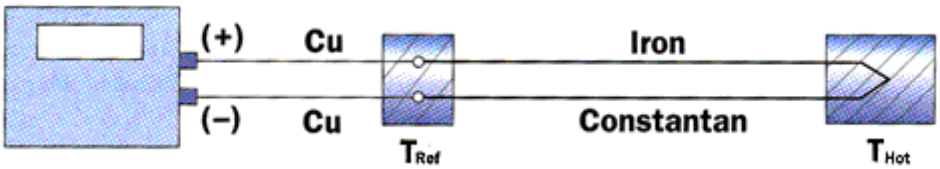


Figure 6.22: Simple thermocouple, using Iron-Constantan wire. [69]

Figure 6.23: Thermocouple devices [98]

A **thermistor** or thermoresistor is a thermally sensitive resistor that exhibits a change in electrical resistance with a change in its temperature. The resistance is measured by passing a small, measured direct current (dc) through it and measuring the voltage drop produced [108]. Negative Temperature Coefficient (NTC) thermistors exhibit a decrease in electrical resistance when subjected to an increase in body temperature and Positive Temperature Coefficient (PTC) thermistors exhibit an increase in electrical resistance when subjected to an increase in body temperature [117]. Thermistors are frequently used in every day life, for example, in mobile phones, in cars, or to measure body temperature. For use and application the standard [22] is recommended.

Figure 6.24: Epoxy coated thermistors [117]

Liquid in glass thermometer is a glass cylinder with a bulb at one end, a capillary hole down the axis connected to the reservoir in the bulb filled with a coloured fluid, and an engraved temperature scale. The physical phenomenon of increasing volume of liquids when the temperature increases is used. Recommended ASTM standards are [16] and [20].

Figure 6.25: Liquid in glass thermometer

Resistant temperature detectors (RTD) are wire wound and thin film devices that measure positive temperature coefficients of electrical resistance of metals. The electrical resistance increases with increased temperature. The sensors are very precise with resolutions of ±0.1K and return stable output for long periods. Compared to thermocouples they have a smaller temperature range (-200°C to 650°C), higher initial costs, and less robust against vibrations. Platinum, Copper, and Nickel are the most common materials used for resistant temperature detectors. Due to the nearly linear temperature coefficient, platinum is used for high accuracy sensors. [17] is the recommended standard for platinum resistance thermometers.

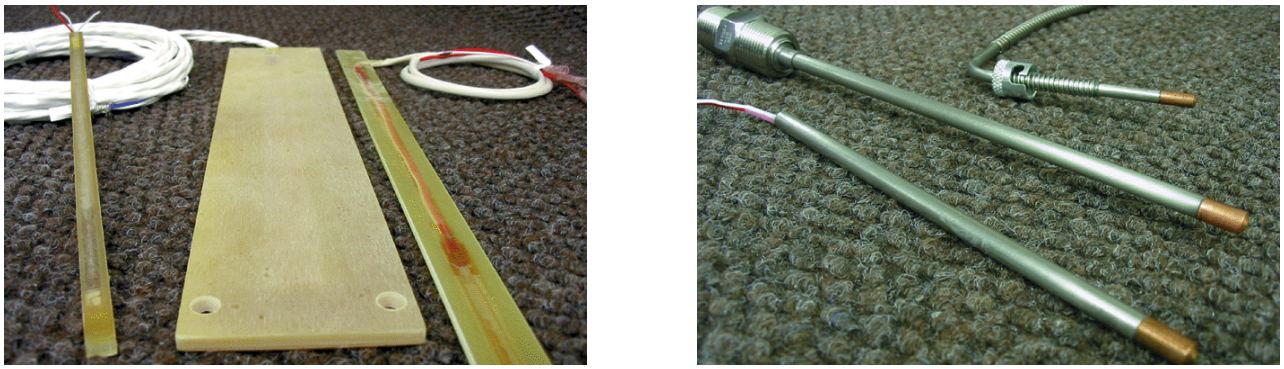


Figure 6.26: Stator RTD (left) and bearing RTD (right) [98]

Filled system thermometers are gas filled closed systems, whereas the gas contracts due to change in its temperature. This type of sensor can operate without any energy source and is widely used in industry and commerce.

Figure 6.27: Filled system thermometer [99]

Bimetallic thermometers are based on the principle that two metals expand at different rates as a function of temperature. Often bimetallic stripes are coiled in spirals to get sensitive sensors for small temperatures. They are independent of any external energy source.

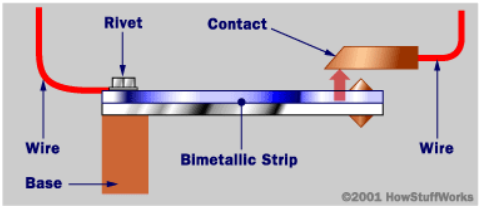


Figure 6.28: Bimetallic strip thermometer to control maximal temperature [92] (left), bimetallic thermometers for industry [99] (right)

Semiconductor temperature sensors are electronic devices fabricated in a similar way to other modern electronic semiconductor components such as microprocessors. The semiconductor diodes have voltage-current characteristics that are temperature sensitive. The characteristics are linear outputs, relatively small size, limited temperature range (-40 to +120°C typical), low cost, good accuracy (around ± 0.5°C) if calibrated but also poor interchangeability. The sensors are grouped in: voltage output, current output, resistance output, digital output, and simple diode types.

Figure 6.29: Semiconductor temperature sensor [102]

Radiation thermometers sometimes called Pyrometers measure temperature from the amount of thermal electromagnetic radiation received from a spot on the object of measurement. They are used widely in manufacturing process of metals, glass, cement, ceramics, semiconductors, plastics, paper, textiles, and coatings. The principle is quit simple. Every object above absolute zero (-273.15°C) emits thermal radiation, much in the infrared portion of the electromagnetic spectrum. The different radiation appears due to differences in the wave speed when a wave is travelling through a material. In vacuum the radiation travels with the speed of light. Their physical interaction with materials can be described mathematically. Many standards are available in this sector. One of the British standard is [18].

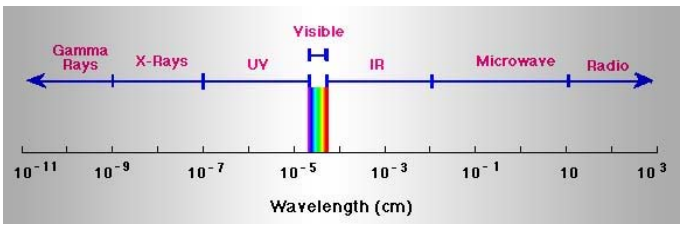


Figure 6.30. Example of a radiation thermometer (left) [116]; Electromagnetic spectrum [110]

Thermal Imagers or infrared imaging cameras are used for infrared thermography, thermology, thermal wave imaging, thermovision, and thermal infrared night vision. By application of such devices many cost

savings and cost avoidances in commerce and industry are possible. Preventive and predictive maintenance, as well as non-destructive testing are some field of application. The principle is similar to the radiation thermometers. Instead of measuring one point, many points will be collected and visualized in a 2d plot. A specific colour schema indicates the lower and higher temperatures. Commercial cameras operate in a wavelength range between 0.7 10^{-4} to 1.4 10^{-3} cm. [108]

Figure 6.31: IR camera (left),
thermal image of the space shuttle in false colours (left) [108]

Ratio thermometers are also called ratio pyrometer, two colour pyrometers, or dual wavelength thermometer. The function is similar to the radiation thermometer. In contrast, they measure at two separate waveband and create the ratio of the signs. The ratio technique eliminates or reduces errors in temperature measurement caused by changes in emissivity, surface finish, and energy absorbing materials, such as water vapour, between the thermometer and the target. Hence, these types are more accurate compared to one-band measurements (radiation thermometers).

The origin of **optical pyrometers** goes more than 100 years back to the past. The filters of the optical devices restrict the spectrum to a narrow wavelength band of 6.5 10^{-5} to 6.6 10^{-5} cm which is the visible red light band. Thus, only hot specimens with a temperature more than 700 °C can be measured. An optical pyrometer allows the operator to compare the intensity of light radiated from a target wavelength to the known brightness of an internal calibrated lamp. A measurement accuracy of ± 0.5% of the temperature being observed can be achieved.

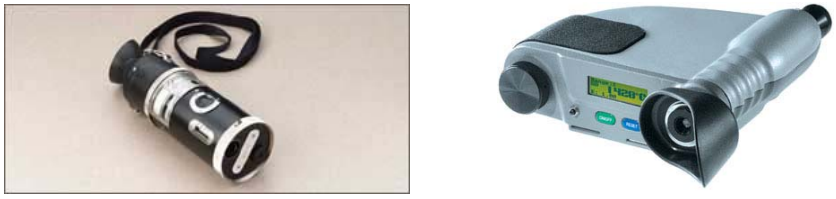


Figure 6.32: Two versions of optical pyrometers, (left [116], right [105])

Fibre optic temperature sensors and their associated measuring devices are expensive and hence applied only when they have a compelling advantage for specialist applications. They use a flexible transparent fibre, to direct radiation to the detector. Hence, they are useful when it is difficult or impossible to obtain a clear sighting path to the target, as in a pressure chamber. [113]

6.8 Processing of Measurement Data

6.8.1 Data Acquisition Hardware

Data acquisition systems need to be selected based on the types and quantity of sensors. Output from the sensing elements can generally be measured using manual-read-out boxes or automated data-acquisition systems (ADAS). ADAS requires an investment in equipment and installation time but greatly facilitates data acquisition particularly where a lot of readings are required in a short time. Manual readings are labour intensive to obtain and still require input into a computer for data reduction and analysis.

In the past, automatic data loggers were the accepted form of automation. Data loggers include strip-chart recorders, printers, and tape or disk recorders.

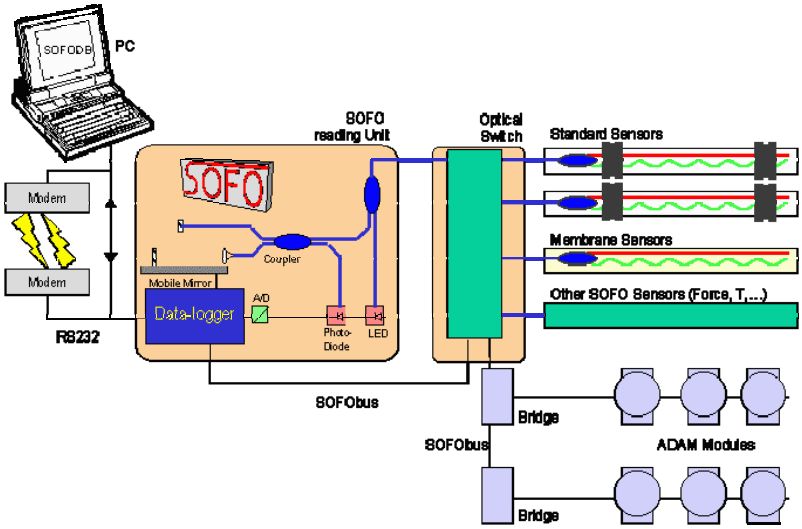


Figure 6.33: Fiber Optic Monitoring System (SOFO), [104]

6.8.2 Data Acquisition Organization

After installing and wiring the sensing elements with the DAQ-hardware, the taking of the readings and their processing must be carried out in a systematic, organized way. The first issue that comes up is the scheduling of the data acquisition, a process that is strongly related to the nature of the physical phenomena and the capabilities of the acquisition system. This is usually done by defining the acquisition speed and setting the time schedule for the collection of data from each sensor.

However, beside this synchronous type of data acquisition there exists another method of collecting data, the event-driven acquisition. Depending on the nature of the monitored system it can often be of interest to measure a series of quantities only when one critical measurand has gained a certain threshold value. In this context phenomena of diverse nature may be measured at different speeds with different time schedules. For the same reason a combination of various data acquisition subsystems may be used to measure these phenomena.

As these subsystems can significantly differ with respect to data formats and functionality, the readings of all components have to be synchronized and integrated in a global repository for raw data. Therefore it is essential to perform clock synchronization for all subsystems and to complete the measurements with a time stamp. An accurate structure and hierarchical organization of this repository is indispensable for the further use of the raw data. Equally important is an intuitive nomenclature for both the channel and file descriptions. It is usually not recommended to integrate the raw data of all measurements directly into one database. On the one hand, different phenomena need different post-processing and analysis methods. On the other hand, the volume of the acquired data can usually be drastically reduced by appropriate post-processing methods without any significant loss of information (e.g. statistical methods, mean values, variation, maximum and minimum values, etc.). For the same reason it is recommended to archive raw data in a compressed format. We can, e.g., imagine, that a digital sample of a time varying signal can be analysed in the frequency domain to obtain some modal data. The description of these modal parameters requires disproportional less space than archiving the whole sample of data in the time domain. Once a certain revision of data has been performed, it makes sense to integrate this reduced stock of data in one or more central databases with a uniform and well-specified data format. The technical bases for this data organization are relational database concepts. This leads us to the next requisition.

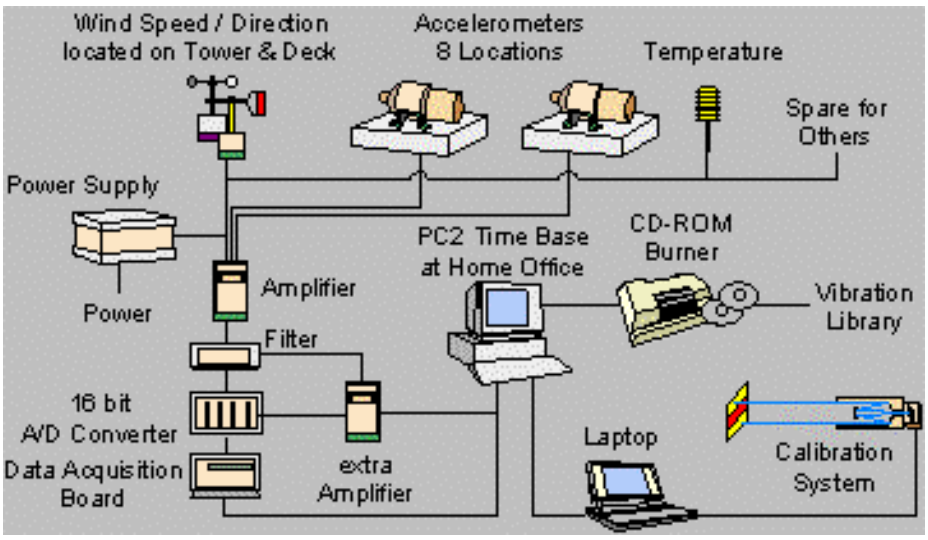


Figure 6.34: Data acquisition for BRIMOS, [118]

The monitored data usually is of interest for a series of different addresses each of them requiring dedicated means of representation and different levels of detail. Depending on how critical the yielded data proves to be for the operational system, the outcome of the data evaluation may address different recipients, following e.g. a hierarchical flow chart classified by the hazardousness, urgency, responsibility, and other criteria. This situation demands different means of data representation and user interfaces. Client-Server database systems like Microsoft SQL Server or Oracle offer the advantage of central data administration by preventing inconsistency and redundancy of the archived data. Different client applications can communicate with the relational database over the network by being offered different rights of access, views and manipulation features. Beside the access to the data from different machines in the Intranet, it might be of interest to have an additional interface for the monitored data on the Internet. Web technologies with graphical browsers and high multimedia capabilities have become more and more powerful recently. The technical basis for such an implementation is currently the Microsoft Information Server, Active Server Page technology and ActiveX Data Objects. In this context a variety of data presentation techniques may be implemented, starting form simple data tables, enhanced reports, and ending up with sophisticated animations.

Beside this type of networking and remote data access, it is usually important to have also remote access to the computer that actually performs the data acquisition. Civil infrastructures are usually difficult to access and not located in the immediate neighbourhood of the supervising engineers. Therefore the DAQ-computer should be controllable remotely and has therefore to be approachable via wireless or traditional line access. A prerequisite is therefore the presence of a modem and a network protocol for controlling the data acquisition and for the file transfer. There exist several products on the market, which provide the functionality for these tasks, starting from classical FTP and ending up with sophisticated programs for remote control such as pc. Anywhere from Symantec, a program that provides you complete control of the remote machine as if you were on site.

Moreover a PC-based implementation offers the platform for the integration of further functionality beyond the narrower range for measurements of deformations, loads, and material deterioration. Considering the case of highway infrastructures, the installation of visual traffic surveillance through a network of video cameras could be one example for such extensions. Other candidates would be road weather stations for visibility measurements, detection of precipitation, fog and black ice, and an according control of variable message and traffic signs on the highway. Therefore, whenever a monitoring system is planned to match the actual requirements, it should anyhow be designed for easy expandability and wide flexibility to extensions. [47]

6.9 Evaluation and Interpretation of Measurement Data

6.9.1 Model Assumptions

Modelling is the idealization of the physical reality of a structure in a mind model, to simulate the real mechanical behaviour. This is done in two steps with two different models. At first the mechanical model is created and based on this the calculating model, which can be an analytical or numerical model, see Figure 6.35.

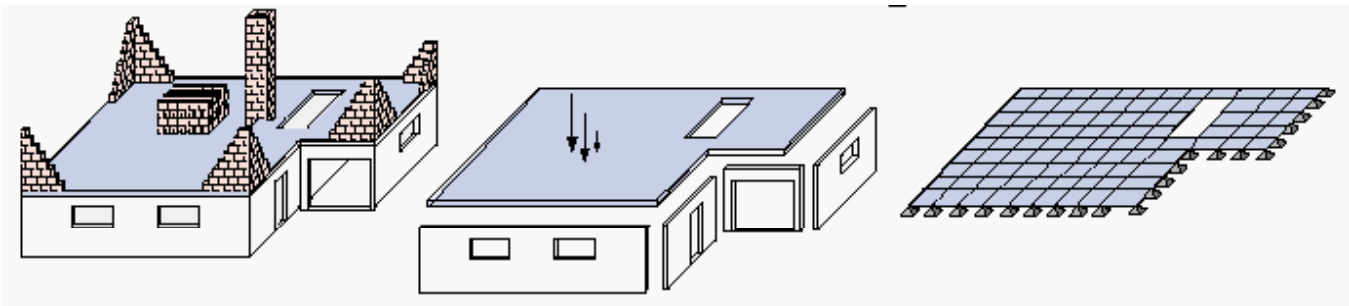


Figure 6.35: Structure- Mechanical Model- Analysis Model ([73])

The first step, the transformation of the real structure in a mechanical model, needs assumptions about the

1. Mechanical system of the structure or parts of the structure
2. Boundary conditions and transition conditions
3. Material
4. Expected behaviour
5. External action and load cases

Table 6.3 introduces parameters and necessary decisions for the mechanical model. Most of these parameters have a stochastic character, which should be considered.

Table 6.3: Parameters and decisions for a mechanical model

Assumptions	Parameters, Decisions
mechanical system	- beam, plate, shell, ...see 4.1.1 - geometrical data
boundary conditions and transition conditions	- simply support, rigid connection, support by spring - spring constant, bedding factor
material	- Young's modulus, Poisson ratio, mass density, - modal damping values, localized damping behaviour - orthotropic or isotropic - homogeneous or heterogeneous - real or smeared properties
expected material behaviour	linear elastic, non-linear elastic, elastic- plastic, elastic- plastic hysteretic etc.
external action and load cases	see chapter 2.2 Actions

Then an analysis model with a chosen theory is created for the mechanical model. The analysis method can be exact or can be an approximation. It can be analytical or numerical.

The method of analysis depends of the type of experimental parameters, which are obtained from the sensing elements.

6.9.2 System Identification

In some cases the answer to specific questions can be directly deduced from measured data as e.g. in static proof load tests, where the observed system behaviour gives the information whether specific requirements are satisfied or not. However, if predictions about the system's prospective behaviour and the corresponding actions are requested, all investigations will be based on certain models.

These models, that describe the considered system on the one hand and the actions on the other, are mainly governed by parameters. Some of these parameters might be directly obtained from measurements as for example the temperature of a surrounding medium or the system's geometry. Other parameters have to be identified based on the measured data. An example for such a parameter is the stiffness of a mechanical model's element. The process of model parameter estimation is often referred to model updating. Several techniques have been developed that range from the trial and error approach to very sophisticated methods. While some of these approaches use the measured data directly in the form as they were observed, other techniques utilize auxiliary quantities as input quantities that are derived first from the measured data.

Furthermore, the quality of a model is influenced by additional expert knowledge. This means, assumptions have to be made by the analyst. These assumptions range from the fundamental theory, the respective model is based on, as far as parameters that are considered to be known. Such parameters can be for example assumed mass densities or the chemical composition of certain media.

An evaluation of all considered models can be made by assessing the correspondence of these models, i.e. the response of the system's model that was predicted due to an action that is described by another model. The whole process becomes even more complex if relations between actions of different nature, the system and the resulting response have to be described. An example could be that corrosion due to the action of an aggressive medium leads to a deterioration and consequently to a mechanical damage of a structural system. As a consequence, the structural response due to a mechanical loading will be different to that of the intact system.

From this comparatively simple example one can derive questions such as

- Which actions have to be taken into account?
- Does there exist an interaction between several actions that can be described by sub-models?
- How do the various actions affect the system's properties? Are there relations between parameters of the actions´ and the system's models?
- Do different approaches lead to different predictions or even to inconsistent results?
- Which quantities have to be measured if certain model parameters shall be identified? Are there correlations between measured data and specific system parameters?

The relations between the components of such investigations are illustrated in Figure 6.36.

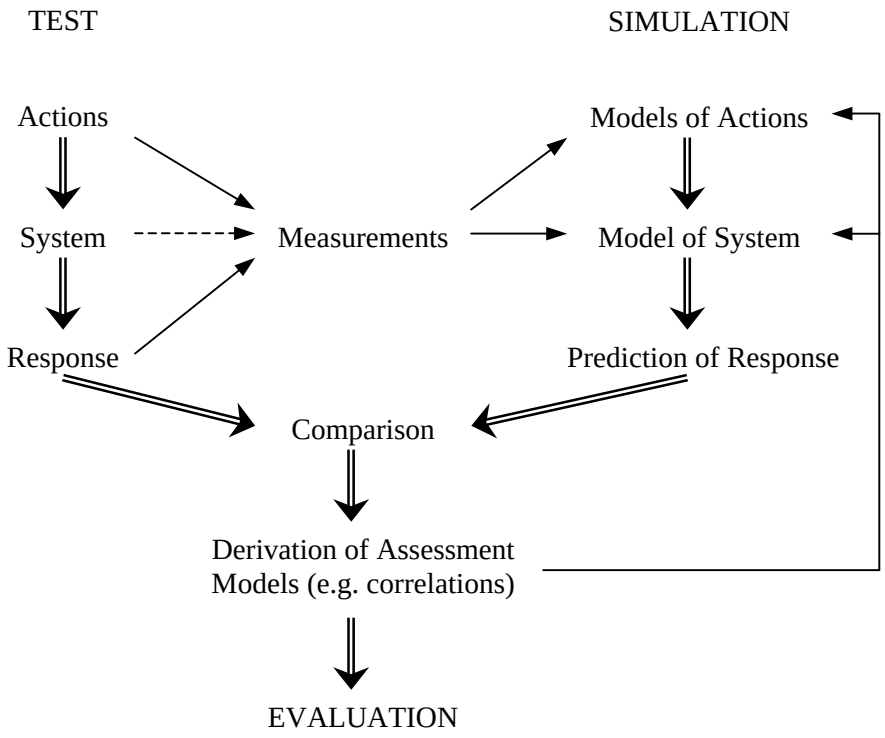


Figure 6.36: Relations between Test, Simulation and Evaluation

All observations, models and predictions are related to uncertainties that are to be evaluated in order to obtain a validation of the final assessment's reliability.

7 STRUCTURAL SYSTEM MODELING

7.1 Linear vs. Nonlinear Structural Analysis

The visual and non-destructive investigations can be only a requirement to do a numerical and reliability assessment. Interpretation and identification of the observed and measured results is very important, namely the comparison of measured and calculated data to validate the model assumptions. A numerical model of the assessed structure has to be built for this purpose.

A static or dynamic analysis with this model can be based either on a linear or non- linear approach. A lot of non-linearities are possible – large deformations, plasticity, creep, stress stiffening, contact (gap) elements, hyper elastic elements, and so on. Principal categories can be distinguished: material non-linearities, geometric non-linearities, and changing of the status.

7.1.1 Material Non-linearities

Nonlinear stress-strain relationships are a common cause of nonlinear structural material behaviour. Some structural material non-linearities are elasticity, creep, nonlinear elasticity, hyperelasticity, and viscoelasticity. The relationship is also path dependent (except for the case of nonlinear elasticity and hyperelasticity), so that the stress depends on the strain history as well as the strain itself.

Many factors can influence stress-strain properties, including the mentioned load history (as in elasto-plastic response), environmental conditions (such as temperature), and the amount of time that a load is applied (as in creep response).

7.1.2 Geometric Non-linearities

In case of geometric non-linearities the equilibrium conditions will be formulated for the deformed structure. Due to the dependency of the stiffness on the deformation an iterative calculation is necessary. Four types of geometric non-linearities are distinguished:

- Large strain assumes that the strains are no longer infinitesimal (they are finite). Shape changes (e.g., area, thickness) are also accounted for. Deflections and rotations may be arbitrarily large.
- Large rotation assumes that the rotations are large but the mechanical strains (those that cause stresses) are evaluated using linearised expressions. The elements of this class refer to the original configuration.
- Stress stiffening assumes that both strains and rotations are small. A first order approximation to the rotations is used to capture some nonlinear rotation effects.
- Spin softening also assumes that both strains and rotations are small. This option accounts for the radial motion of a body's structural mass as it is subjected to an angular velocity. Hence, it is a type of large deflection but small rotation approximation.

7.1.3 Changing Status

Many common structural features exhibit nonlinear behaviour that is status-dependent. For example, a tension-only cable is either slack or taut; a roller support is either in contact or not in contact. Status changes might be directly related to the load (as in the case of the cable), or they might be determined by some external cause. Situations in which contact occurs are common to many different nonlinear applications. Contact forms a distinctive and important subset to the category of the non-linearities induced by status changings.

7.2 Soil-Structure Interaction (SSI)

The interaction of soil and structure can have a great influence on the dynamic behaviour of the structure. In principle, the soil and the structure have to be investigated together. But due to the small effects of interaction, the soil and the structure will be investigated separately in most cases.

Accelerations of the soil effect inertia forces of the structure. These forces cause deformations of the structure and the soil. This effect is called inertia interaction. The frequency decreases and the damping often increases in cause of this action that is important for very slender and very massive structures.

Two basic methods are used for the computation of SSI. The direct method models the structure and the soil in one finite-element-model. Only a restricted area of the soil can be considered. In the case of static load a twice until threefold width of a fundament is considered.

The previous SSI analysis assumes that the free-field motion at the base of the structure is uniform. For large structures such as bridges and arch dams the free-field motion, at all points where the structure is in contact with the foundation, is not uniform**.**

7.3 Fluid-Structure Interaction (FSI)

Fluid-structure interaction (FSI), the coupling of unsteady fluid flow and structure motion, is an important field of computational mechanics. The accelerated masses of fluids, which are surrounded by stiff or flexible walls, create a hydrodynamic pressure against the wall of the container. This pressure is superposed with the hydrostatic load. The hydrodynamic compression consists of connective and impulsive pressure. The vibration of the fluid surface (swash, sloshing) is called connective pressure. Impulsive pressure is called by the inertia forces of the fluid against the movement of the container. There are different assumptions for the fluid. It can be compressible or incompressible and with or without friction to the structure wall.

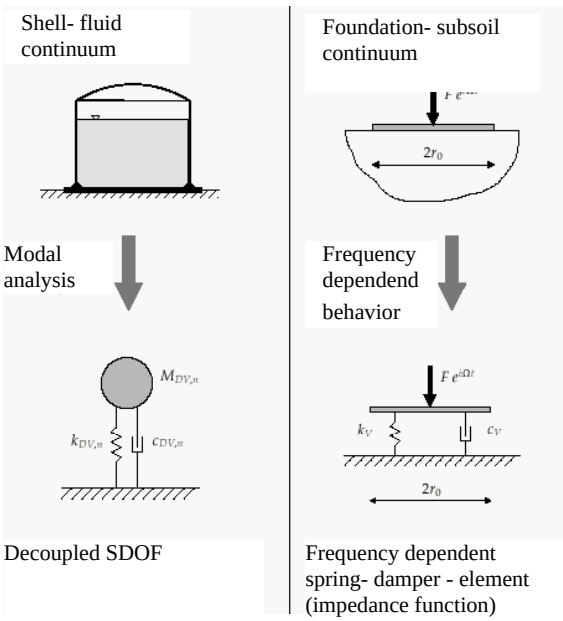


Figure 7.1: FSI and SSI, [50]

7.4 Damage/ Deterioration Models

Deterioration models describe the slow degradation and change in strength of a material. Material deterioration leads to reduced member performance, reduced structural performance and ultimately reduced reliability of a structural system. The models relate the causes of deterioration to its mechanisms and outcomes, such as change in cross-sectional area member strength and structural load rating.

Deterioration models are used to predict the change in structural parameters due to the intended structural loading, environmental conditions, maintenance practices, and historical data. The degradation at both the system and component level is based on time-dependent deterioration models that relate the structural system to the environment. Thus, deterioration models are fundamental considerations in the development of prediction models for bridge performance or condition. Practically, deterioration models can be used to correlate service environment and service life to optimal maintenance scheduling and life cycle analysis.

8 GENERIC COMPUTATIONAL APPROACHES

8.1 Basic Variables, Limit States and Fragility

The **basic random variables** $\mathbf{X}$ represent any kind of uncertainty present in the engineering system under investigation. The uncertainties may range from physical and statistical uncertainties to model uncertainties. Typical examples of **physical uncertainties** are loading conditions, environmental exposure, geometric properties or material parameters. **Statistical uncertainties** arise mainly from incomplete statistical information, e.g., due to a small number of performed tests. Finally, **model uncertainties** describe the uncertainties associated with the idealized mathematical descriptions used to approximate the actual physical behaviour of the system.

When evaluating the system performance, we have to identify **limit states**, i.e., conditions in which the system ceases to perform its intended functions in some way. In structural engineering, such limit states for components or systems may be strength or deformation related. Nevertheless, limit states can be also viewed in a broader socio-economic context, where they are related to repair or reconstruction costs that are in excess of a desired amount, opportunity losses, or morbidity/mortality. Thus, there will be no loss of generality, when we restrict ourselves in the following to structural limit states.

Generally, failure (i.e., an undesired or unsafe state of the structure) is defined in terms of a **limit state function** $g(\cdot)$. Having given a vector of n random variables $\mathbf{X} = [X_1, X_2, \ldots, X_n]$, the limit state function $g(\mathbf{x})$ divides the random variable space in a **safe domain**

$$S = \{\mathbf{x} : g(\mathbf{x}) > 0\} \tag{8.1}$$

and a failure domain

$$F = \{\mathbf{x} : g(\mathbf{x}) \le 0\} \tag{8.2}$$

The definition of the limit state function is not unique. However, the **probability of failure**, i.e., the probability that failure will occur, which is defined as

$$P(F) = P(g(\mathbf{x}) \le 0) = \int_{g(\mathbf{x}) \le 0} f(\mathbf{x}) d\mathbf{x} \tag{8.3}$$

is a unique quantity. In other words, the probability of failure does not depend on the particular choice of the limit state function.

The above limit state probability (probability of failure) can also be written in a slightly different form as

$$P(F) = P(F|L)P(L) \tag{8.4}$$

in which the L is a random quantity describing the intensity of the demand on the system, $P(F|L)$ is the conditional limit state probability given L, and $P(L)$ is the probability that L occurs. The conditional probability of a component or system to attain a (performance) limit state, which may range from loss of function to collapse, is called **fragility**. The fragility provides a probabilistic measure of the safety margin with respect to a design basis or other events specified. It can be used to evaluate system weaknesses or deficiencies identified during a condition assessment and can provide a means to assess if the observed weakness or deficiencies might be expected to have a significant impact on system risk. Nevertheless, since the fragility is nothing else then a conditional probability, it is subject to the same generic computational approaches as the probability of failure.

8.2 First Order Reliability Method (FORM)

The concept of the **first order reliability method (FORM)** is based on a description of the reliability problem in standard normal space. Hence transformations from dependent non-normal variables to independent normal variables with zero mean and unit variance are required. In the following we assume that we are working in standard normal space. (A possible transformation from non-normal to normal space can be done by utilizing the Rosenblatt transformation.)

Let us assume again that the limit state function $g(\cdot)$ is given as

$$g(\mathbf{U}) = g(U_1, U_2, \ldots, U_n) \tag{8.5}$$

where U_i (with $i = 1,2,\ldots,n$) are standard normal random variates. Frequently, $Z = g(\mathbf{u})$ is called **safety margin**. We want now to linearize the limit state function around a point $\mathbf{u}^*$. In first order reliability method the **expansion** or **design point** $\mathbf{u}^*$ is chosen such as to maximize the probability density function of $\mathbf{U}$ in the failure domain (**point of maximum likelihood**). Geometrically, this coincides for the standard normal space with the point in the failure domain having the minimum distance β from the origin, as shown in Figure 8.1. (Note that this geometrical formulation is in general not valid in other random variables than the standard normal space.)

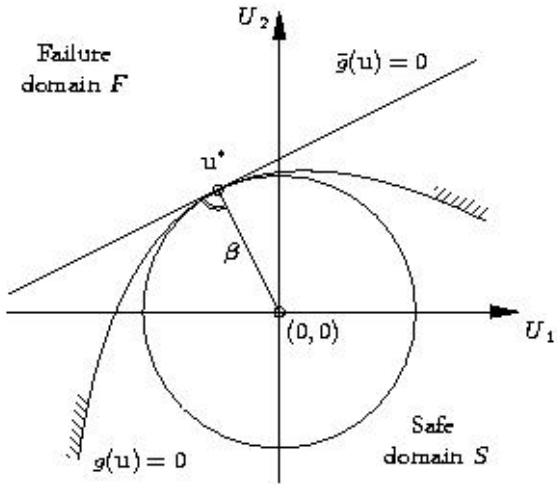


Figure 8.1: Expansion point $\mathbf{u}^*$ and reliability index β in standard normal space.

From the geometrical interpretation of the expansion point $\mathbf{u}^*$ in standard normal space it becomes quite clear that the calculation of the design point and, consequently, the **reliability index** β can be reduced to an optimization problem of the form

$$\beta = \min\left[\sqrt{\mathbf{u}^{\mathrm{T}}\mathbf{u}} \mid g(\mathbf{u}) = 0\right],\ \beta > 0,\ g(0) \in S \tag{8.6}$$

This leads to the Lagrange function

$$L = \mathbf{u}^{\mathrm{T}}\mathbf{u} + \lambda g(\mathbf{u}) \rightarrow \min! \tag{8.7}$$

Standard optimisation procedures can be utilized to solve for the location of $\mathbf{u}^*$.

For determining the failure probability, the exact limit state function $g(\mathbf{u})$ is replaced by a linear approximation $\bar{g}(\mathbf{u})$ as shown in Figure 8.1. Expanding the limit state function in $\mathbf{u}^*$ in a Taylor series (with only the linear terms retained) gives

$$\bar{g}(\mathbf{u}) = g(\mathbf{u}^*) + \sum_i (u_i - u_i^*) \left.\frac{\partial g(\mathbf{u})}{\partial u_i}\right|_{\mathbf{u}=\mathbf{u}^*} \tag{8.8}$$

or simply

$$\overline{g}(\mathbf{u}) = \boldsymbol{\alpha}^{\mathrm{T}}\mathbf{u} + \beta \tag{8.9}$$

In the above equation, the $\boldsymbol{\alpha}$ -values are so called **sensitivity factors**.

As can be seen, the linearised limit state function is a linear combination of normally distributed random variables. I.e., the limit state function is also a normal random variable with mean

$$\mathrm{E}\left[\overline{g}(\mathbf{u})\right] = \beta \tag{8.10}$$

and variance

$$\mathrm{Var}\left[\overline{g}(\mathbf{u})\right] = 1 \tag{8.11}$$

Since failure is defined as $\overline{g}(\mathbf{u}) \le 0$, the failure probability utilizing the linearized limit state function is

$$P(F) = \Phi(-\beta) \tag{8.12}$$

This result is exact if $g(\mathbf{u})$ is actually linear. The above result can be improved by utilizing a second order approximation of the limit-state function. This is called **second order reliability method (SORM)**.

8.3 Monte Carlo Methods

8.3.1 Plain Monte Carlo Sampling

In reliability analysis we are interested in determining functionals of the form

$$\int_D w(\mathbf{x}) f(\mathbf{x}) d\mathbf{x} = \mathrm{E}[w(\mathbf{x})] \tag{8.13}$$

where D denotes the integration domain and $f(\mathbf{x})$ is a joint probability density function. The most prominent example of such a functional certainly is the probability of failure

$$P(F) = \int_{g(\mathbf{x}) \le 0} f(\mathbf{x}) d\mathbf{x} \tag{8.14}$$

Introducing an indicator function $\mathrm{I}(\cdot)$ that equals one if its argument is true, and zero otherwise, we can write the probability of failure as

$$P(F) = \int_D \mathrm{I}(g(\mathbf{x}) \le 0) f(\mathbf{x}) d\mathbf{x} = \mathrm{E}[g(\mathbf{x}) \le 0] \tag{8.15}$$

In order to determine the failure probability, in principle, all available statistical methods for estimating expected values are applicable. If m independent samples $\mathbf{x}^{(k)}$ (with $k = 1,2,\ldots,m$) of the n-dimensional random vector $\mathbf{X}$ are available, then the estimator

$$v = \sum_{k=1}^{m} \mathrm{I}(g(\mathbf{x}^{(k)}) \le 0) \tag{8.16}$$

yields a consistent and unbiased estimate of the failure probability. The variance of this estimator is given as

$$\mathrm{Var}[v] = \frac{1}{m}\mathrm{Var}[\mathrm{I}(g(\mathbf{x}^{(k)}) \le 0)] = \frac{1}{m}(P(F) - P^2(F)) \tag{8.17}$$

Therefore, for small values of $P(F)$ the variance of its estimator is approximately

$$\mathrm{Var}[v] \approx \frac{1}{m} P(F) \tag{8.18}$$

The required number m of samples to achieve a certain—most preferably small—value of the variance of the estimate is independent of the dimension n of the problem. However, for small values of the failure probability and small number of samples the confidence of the estimate is very low as can be seen from the above equation.

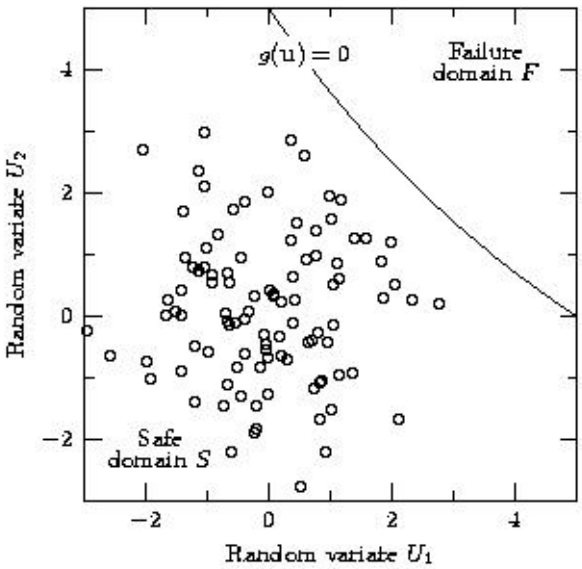


Figure 8.2: Plain Monte Carlo simulation in standard normal space.

As can be seen from Figure 8.2, if the estimated failure probability is quite small, we quite seldom have a hit in the failure domain. (For the shown $m = 100$ samples there is no hit in the failure domain F at all.) In other words, only a small fraction of the samples will contribute to the estimator of the failure probability and therefore the estimate will have a large variance. For a sample size of $m = 10^4$ we get an estimate of the probability of failure of $\hat{P}(F) = 4.0 \cdot 10^{-4}$ with a sampling error of $e = 50\%$, which is defined as

$$e = \frac{(Var[v])^{1/2}}{v} \tag{8.19}$$

8.3.2 Importance Sampling

In order to reduce the standard deviation of the estimator to the order of magnitude of the failure probability itself, m must be in the range of $m = 1/P(F)$. For values of failure probability in the range of $1 \cdot 10^{-6}$ this can not be achieved with reasonable computational effort. Alternatively, strategies have top be employed which increase the "hit-rate" by artificially producing more samples in the failure domain than should occur according to the distribution function. One way to approach this solution is the introduction of a positive weighting function $h(\cdot)$ such that

$$P(F) = \int_D \mathrm{I}(g(\mathbf{x}) \le 0) f(\mathbf{x}) d\mathbf{x} = \int_D \mathrm{I}(g(\mathbf{x}) \le 0) \frac{f(\mathbf{x})}{h(\mathbf{x})} h(\mathbf{x}) d\mathbf{x} \tag{8.20}$$

This positive weighting function can be interpreted as the density function $h(\bar{\mathbf{x}})$ of a random vector $\bar{\mathbf{X}}$. Therewith the failure probability is

$$P(F) = \mathrm{E}\left[\mathrm{I}(g(\bar{\mathbf{x}}) \le 0) \frac{f(\bar{\mathbf{x}})}{h(\bar{\mathbf{x}})} \right] \tag{8.21}$$

The probability of failure is then estimated from

$$v = \frac{1}{m}\sum_{k=1}^{m} \mathrm{I}(g(\bar{\mathbf{x}}^{(k)}) \le 0)\frac{f(\bar{\mathbf{x}})}{h(\bar{\mathbf{x}})} \tag{8.22}$$

This estimator of the failure probability is again unbiased, i.e.

$$\mathrm{E}[\mathrm{I}(g(\mathbf{x}) \le 0)] = \mathrm{E}\left[\mathrm{I}(g(\bar{\mathbf{x}}) \le 0)\frac{f(\bar{\mathbf{x}})}{h(\bar{\mathbf{x}})}\right] \tag{8.23}$$

with variance

$$\mathrm{Var}[v] = \frac{1}{m}\left\{\mathrm{E}\left[\mathrm{I}(g(\bar{\mathbf{x}}) \le 0)\left(\frac{f(\bar{\mathbf{x}})}{h(\bar{\mathbf{x}})}\right)^2\right] - P^2(F)\right\} \tag{8.24}$$

As can be seen from above equations, the utilization of the density function $h(\bar{\mathbf{x}})$ does not change the value of the estimate of $P(F)$, but only the variance of the estimate. Therefore, a useful choice of $h(\bar{\mathbf{x}})$ can be based on minimizing the variance. Ideally, such a weighting function should reduce the sampling error. It has been shown, that by centring the sampling density $h(\mathbf{x})$ at the design point $\mathbf{x}^*$ the sampling error can be reduced quite efficiently. In other words, the weighting function (sampling density) $h(\bar{\mathbf{x}})$ is chosen with mean value vector $\mathrm{E} = \lfloor\bar{\mathbf{X}}\rfloor = \mathbf{x}^*$ and covariance matrix $\mathbf{C} = \mathrm{Cov}\lfloor\bar{\mathbf{X}}, \bar{\mathbf{X}}\rfloor = \mathrm{Cov}[\mathbf{X}, \mathbf{X}]$ in the following form (n-dimensional normal distribution)

$$h(\bar{\mathbf{x}}) = \frac{1}{\sqrt{2\pi^n \det \mathbf{C}}}\exp\left[-\frac{1}{2}\left(\mathbf{x} - \mathbf{x}^*\right)^{\mathrm{T}}\mathbf{C}^{-1}\left(\mathbf{x} - \mathbf{x}^*\right)\right] \tag{8.25}$$

The efficiency of this concept depends on the geometrical shape of the limit state function. In particular, limit state functions with high curvatures or almost circular shapes cannot be covered very well. As can be seen from Figure 8.3, approximately half of the samples hit the failure domain, contributing therewith to the estimate of the probability of failure. Since, furthermore, the samples are centred around the most likely point leading to failure, the variance of the estimate can be considerably reduced.

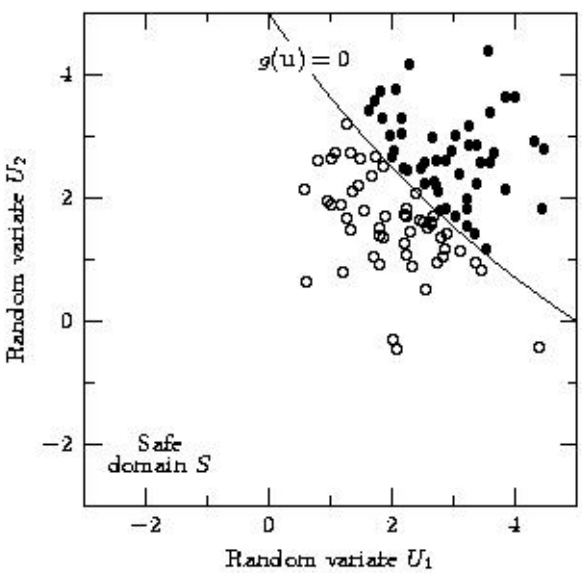


Figure 8.3: Importance sampling at the design point $\mathbf{u}^*$.

8.3.3 Adaptive Sampling

Another Monte Carlo method for estimating the probability of failure is **adaptive sampling**. This method also utilizes an importance sampling density function $h(\cdot)$, however, the necessary parameters like, e.g., the mean values and the covariance matrix—are estimated from the results of a previous Monte Carlo simulation run.

Given is an importance sampling density function $h(\mathbf{x}, \boldsymbol{\mu}, \mathbf{C})$ (e.g., a normal joint distribution) with unknown mean values $\boldsymbol{\mu}$ and covariance matrix $\mathbf{C}$. As we already know, an optimal choice of these parameters is given when the variance of estimator of the failure probability becomes minimal. In other words, for determining the mean values $\boldsymbol{\mu}$ and $\mathbf{C}$ the following optimisation problem has to be solved

$$\min_{\boldsymbol{\mu},\mathbf{C}} \left[\int_D \mathrm{I}(g(\mathbf{x}) \le 0) \frac{f^2(\mathbf{x})}{h(\mathbf{x}, \boldsymbol{\mu}, \mathbf{C})} d\mathbf{x} \right] \tag{8.26}$$

In general it is difficult to find a solution of above equation. Therefore, via a first initial guess of the parameters $\boldsymbol{\mu}_0$ and $\mathbf{C}_0$ samples in the failure domain are generated which allow to estimate these parameters directly and refine them in subsequent steps. In the $(\gamma+1)$-th iteration step the parameters $\boldsymbol{\mu}_{\gamma+1}$ and $\mathbf{C}_{\gamma+1}$ are estimated by

$$\boldsymbol{\mu}_{\gamma+1} = \frac{1}{m} \sum_{k=1}^{m} \bar{\mathbf{x}}^{(k)} \, \mathrm{I}(g(\bar{\mathbf{x}}^{(k)} \le 0)) \frac{f(\bar{\mathbf{x}}^{(k)})}{h(\bar{\mathbf{x}}^{(k)}, \boldsymbol{\mu}_\gamma, \mathbf{C}_\gamma)} \tag{8.27}$$

and

$$\mathbf{C}_{\gamma+1} = \frac{1}{m} \sum_{k=1}^{m} (\bar{\mathbf{x}}^{(k)} - \boldsymbol{\mu}_{\gamma+1})(\bar{\mathbf{x}}^{(k)} - \boldsymbol{\mu}_{\gamma+1})^{\mathrm{T}} \, \mathrm{I}(g(\bar{\mathbf{x}}^{(k)} \le 0)) \frac{f(\bar{\mathbf{x}}^{(k)})}{h(\bar{\mathbf{x}}^{(k)}, \boldsymbol{\mu}_\gamma, \mathbf{C}_\gamma)} \tag{8.28}$$

In other words, adaptive sampling utilizes previously gathered information about the failure domain to improve the estimated parameters and therewith the efficiency of the importance sampling technique. In Figure 8.4 samples after an adaptation are shown. As can be seen, the samples are distributed along the limit state surface, allowing, in general, a further variance reduction than possible with importance sampling at the design point.

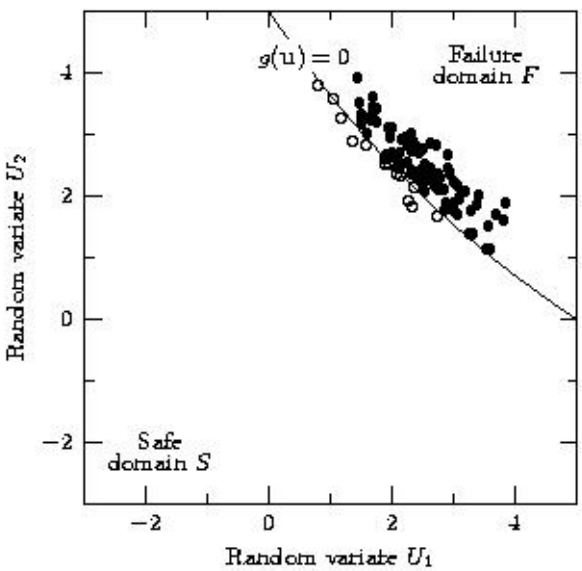


Figure 8.4: Adaptive sampling in standard normal space.

8.4 System Reliability Analysis

8.4.1 Classification of Systems

Engineering systems that are composed of multiple units can be classified as series or parallel systems or combinations thereof. However, not all systems are composed of multiple units or can be decomposed into such series or parallel systems. This is especially true for structural systems. Nevertheless, even in the latter case the (general) system may be represented as a combination of failure events (e.g., in the case of multiple failure modes) which are logically connected in series or in parallel. However, these (structural) failure events are not simply the failures of the individual units or sets of such units and therefore, in general, quite difficult to quantify.

Systems that are composed of units connected in series are such that the failure of any one or more of these units constitutes the failure of the entire system. These systems have no redundancy and the reliability or safety of the system requires that none of its units fail. In other words, the failure F of the **series system** is the union of the unit failures B_i ($i = 1, \ldots, m$) and the failure probability is given as

$$P(F) = P(B_1 \cup B_2 \cup \cdots \cup B_m) \tag{8.29}$$

Systems that are composed of units connected in parallel are such that the failure of the entire system requires the failures of all of its units. Such systems are redundant and the failure of the **parallel system** is the intersection of the unit failures and the failure probability is given as

$$P(F) = P(B_1 \cap B_2 \cap \cdots \cap B_m) \tag{8.30}$$

In general, we have combined series-parallel systems, i.e. we have a combination of series-connected and parallel-connected units. Consequently, the failure probability of **parallel-connected series systems** is given as

$$P(F) = P((B_{11} \cup \cdots \cup B_{1j}) \cap (B_{21} \cup \cdots \cup B_{2k}) \cap \cdots \cap (B_{m1} \cup \cdots \cup B_{ml})) \tag{8.31}$$

whereas the failure probability of **series-connected parallel systems** is

$$P(F) = P((B_{11} \cap \cdots \cap B_{1j}) \cup (B_{21} \cap \cdots \cap B_{2k}) \cup \cdots \cup (B_{m1} \cap \cdots \cap B_{ml})) \tag{8.32}$$

Each system can be represented by one of the above descriptions by utilizing the distributive law of set theory. Moreover, the above sets can be made minimal by absorbing subsets leading to **minimal cut sets** or **minimal path sets**.

Since parallel systems are composed of redundant units, its reliability depends on whether the redundancies are active or standby. For systems with **active redundancies**, failures of units will occur sequentially, unless the capacities of the components are perfectly correlated and identically distributed. Therefore, for an active redundant system, all subsequent unit failures will involve conditional probabilities. For systems with **standby redundancies**, however, the unit failure events may be statistically independent. Moreover, standby redundancy will invariably increase the reliability of a system as long as the redundant units are not perfectly correlated. In contrast, active redundancy may or may not be very effective in improving the reliability of a system. Redundancy in a structural system is invariably of the active type. In other words, all components in a structural system carry loads or are subject to the effects of the applied loads.

8.4.2 Bounds on Failure Probabilities

To derive bounds on the failure probabilities we utilize the assumption of statistical independence and perfect correlation among the failures of the individual units. The failure probability of series system with independent failures of the individual units is given as

$$P(F) = P(B_1 \cup B_2 \cup \cdots \cup B_m) = 1 - \prod_{i=1}^{m} (1 - P(B_i)) \tag{8.33}$$

whereas the failure probability of parallel systems with independent failures of the individual units is given as

$$P(F) = P(B_1 \cap B_2 \cap \cdots \cap B_m) = \prod_{i=1}^{m} P(B_i) \tag{8.34}$$

In case of complete dependency between the failures of the individual units, the failure probability of the series system is

$$P(F) = P(B_1 \cup B_2 \cup \cdots \cup B_m) = \max P(B_i) \tag{8.35}$$

and the failure probability of the parallel system is

$$P(F) = P(B_1 \cap B_2 \cap \cdots \cap B_m) = \min P(B_i) \tag{8.36}$$

Therewith, the **first-order (linear) bounds** of the failure probability for a **series system** are

$$\max P(B_i) \le P(F) \le \sum_{i=1}^{m} P(B_i) \tag{8.37}$$

and for a parallel system

$$0 \le P(F) \le \min P(B_i) \tag{8.38}$$

In practical applications these bounds are too wide to be useful. E.g., whereas the demand on the different units in a parallel system is in general not independent, the independence of the capacities among the units is certainly desirable in order to increase the reliability of the system. An **improved upper bound** for a **parallel system** is

$$P(F) \le \min_{j<i} P(B_i \cap B_j) \tag{8.39}$$

For a series system the second-order (linear) bounds are given as

$$\begin{aligned} &P(B_1) + \sum_{i=2}^{m} \max[0, (P(B_i) - \sum_{j=1}^{i-1} P(B_i \cap B_j)] \le P(F) \\ &\le P(B_1) + \sum_{i=2}^{m} (P(B_i) - \max_{j<i} P(B_i \cap B_j)) \end{aligned} \tag{8.40}$$

The narrowness of these bounds depends also on the ordering of the events. Useful bounds can be derived when the unit failures B_i are ordered with respect to the values of $P(B_i)$. In general, higher-order bounds do not improve the above results significantly.

8.4.3 System Analysis by FORM

As has been shown above, the system failure F can be described as a union or intersection of unit failures B_i ($i = 1, \ldots, m$). Let us assume that

$$F = \bigcap_{i=1}^{m} B_i \tag{8.41}$$

with

$$B_i = \left\{ \boldsymbol{\alpha}_i^{\mathrm{T}} \mathbf{U} + \beta_i \le 0 \right\} = \left\{ Z_i \le -\beta_i \right\} \tag{8.42}$$

i.e., the unit failure events are described by half spaces. The covariance matrix of $\mathbf{Z} = [Z_1, Z_2, \ldots, Z_m]$ is given by

$$\mathrm{Cov}[\mathbf{Z},\mathbf{Z}] = \left\{\boldsymbol{\alpha}_i^{\mathrm{T}}\boldsymbol{\alpha}_j;\, i,\, j = 1,2,\ldots,m\right\} \tag{8.43}$$

which is nothing else then the correlation matrix $\mathbf{R}$, since $\mathbf{Z}$ is a vector of zero-mean, standard normal random variables. The probability of system failure F is

$$P(F) = P\left(\bigcap_{i=1}^{m}\{Z_i \le -\beta\}\right) = \Phi_m(-\boldsymbol{\beta},\mathbf{R}) \tag{8.44}$$

with $\Phi_m(\cdot)$ as the m-dimensional cumulative normal distribution and $\boldsymbol{\beta} = [\beta_1, \beta_2, \ldots, \beta_m]$ as the reliability index vector. Consequently, when system failure F is described as a union of unit failures B_i ($i = 1,\ldots,m$), i.e.,

$$F = \bigcup_{i=1}^{m} B_i \tag{8.45}$$

$P(F)$ can be determined by the following FORM result

$$P(F) = P\left(\bigcup_{i=1}^{m}\{Z_i \le -\beta\}\right) = 1 - P\left(\bigcap_{i=1}^{m}\{Z_i \le -\beta\}\right) = 1 - \Phi_m(-\boldsymbol{\beta},\mathbf{R}) \tag{8.46}$$

9 DECISION-MAKING AND COST-BENEFIT-ANALYSIS

The management of existing systems requires a lifetime optimisation methodology. Thereby, each effort in having a safe/reliable state of operation has to be weighted by the costs raised by restoring/preserving such a state and the benefits derived from the operation of the system. Quite typically, each system has certain parameters which can be controlled by preventive maintenance or changed by re-construction efforts. Lifetime optimisation of existing systems intends to find the cost-optimal set of actions required to extend the service lifetime. However, this task can only be fully accomplished when that cost-optimal solution does not exceeds the benefits, i.e. when the expected cost although minimal is not greater the expected gain.

9.1 Cost-Benefit analysis

In re-assessment of structures and engineering systems, engineers are often in the situation to be involved in decisions on repair and/or strengthening of an existing system / structure where some statistical information is available. In the following it is shown how Bayesian statistical decision theory can be used for making such decisions in a rational way. The theoretical basis is detailed described in e.g. [72] and [37]. It is assumed that the decision is taken on behalf of the owner of the structure, and that a cost-benefit approach is used with constraints related to minimum safety requirements specified by national / international codes of practice and / or the society. The same principles can be applied in case of other decision makers. It is noted that the optimal solution from the cost-benefit problem should be used as one input to the decision process.

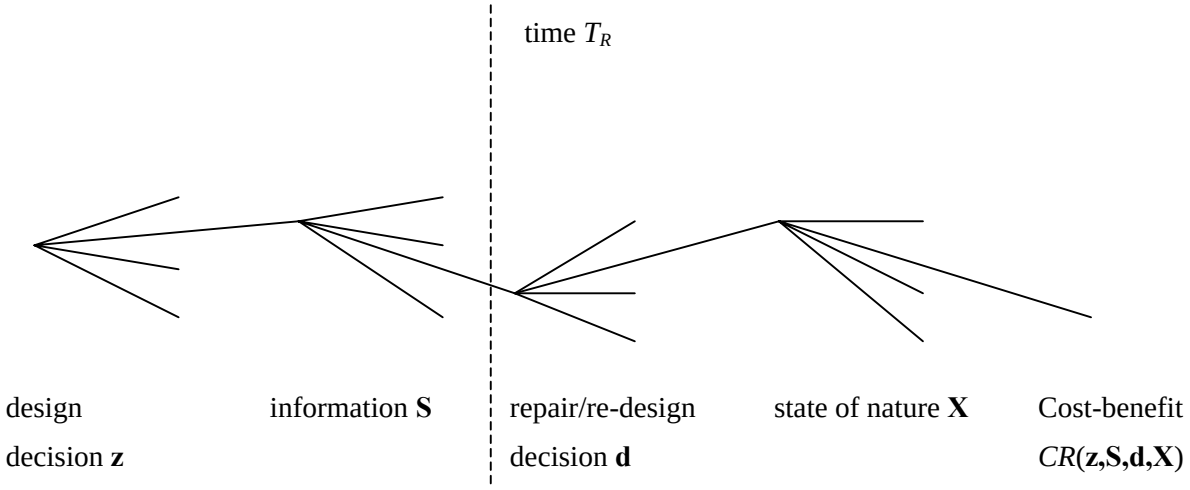


Figure 9.1. Decisions in re-assessment with given information. The vertical line illustrates the time of re-assessment.

The decision problem on possible repair and/or strengthening in a re-assessment situation is illustrated in Figure 9.1. It is assumed that the design variables in the initial design situation are denoted $\mathbf{z}$. After the initial design information about the uncertain variables influencing the behaviour of the structure is collected, and are denoted $\mathbf{S}$. Often this information will be collected in connection with the re-assessment.

The decision variables at the time T_R of re-assessment are denoted $\mathbf{d}$. The uncertain variables describing the state of nature are denoted $\mathbf{X}$.

The decision problem can be described as a decision with given new information.

An important difficulty in Bayesian statistical decision theory when applied in civil and structural engineering is that it can be difficult to assign values to cost of failure, or not acceptable behaviour, especially when loss of human lives is involved. One solution is to calibrate the cost models to existing structures or to base the decisions on comparisons with alternative solutions.

Further, organizational factors can have a rather significant influence in the decision process. These factors often have an influence, which is not rational from a cost-benefit point of view. Examples are the influence of the organizational structure, personal preferences and organizational culture.

The density function of the n stochastic variables $\mathbf{X} = \{X_1, X_2, ..., X_n\}$ is denoted $f_{\mathbf{X}}(\mathbf{x}, \theta)$ where θ are statistical parameters, for example mean values, standard deviations and correlation coefficients. Further, it is assumed that a decision has to be taken between a number of alternatives which can be modelled by design/decision variables $\mathbf{d} = \{d_1, d_2, ..., d_N\}$. The decision model in Figure 1.1 illustrates the case with one discretized variable d. The decision is taken before the realization by nature of the stochastic variables is known. Besides the decision variables $\mathbf{d}$ and the uncertain variables $\mathbf{X}$ also a cost-benefit function $CB(\mathbf{Z}, \mathbf{S}, \mathbf{d}, \mathbf{X})$ is introduced in the decision model. When a decision $\mathbf{d}$ in the re-assessment problem has been taken and a realisation $\mathbf{x}$ of the stochastic variables appears then the value obtained is denoted $CB(\mathbf{z}, \mathbf{s}, \mathbf{d}, \mathbf{x})$ and represents a numerical measure of the consequences of the re-assessment decision and the realisation obtained. $CB(\mathbf{z}, \mathbf{s}, \mathbf{d}, \mathbf{x})$ is assumed to be measured in monetary units and represents in general costs minus benefits, if relevant.

Illustrative examples of the decision variables **z** and **d**, and the stochastic variables **S** and ***X*** are:

- **z**: design parameters, e.g. geometrical parameters of a structural system (cross-sectional dimensions and topology). The design parameters are already chosen at the initial design, and are therefore fixed at the time of re-assessment.
- **S:** information collected, e.g. concrete compression strengths obtained from samples taken from the structure, measured wave heights, non-failure of the structure, no-find of defects by an inspection.
- **d**: design parameters in the re-assessment, e.g. geometrical parameters of a repair (cross-sectional dimensions and topology).
- ***X***: stochastic variables, representing e.g. loads and material strengths.

In some decision problems it can be difficult to specify the cost function, especially if the consequences not directly measurable in money are involved, for example personal preferences. However, as described in von [82] rational decisions can be taken if the cost function is made such that the expected value of the cost function is consistent with the personal preferences.

If the information **S** is related the stochastic variables $\mathbf{X}$ then a **predictive density function** (updated density function) $f''_{\mathbf{X}}(\mathbf{x}|\mathbf{s})$ of the stochastic variables $\mathbf{X}$ taking into account a realization $\mathbf{s}$ can be obtained using Bayesian statistical theory, see [64] and [35]

If the decision-maker wants to act rationally, taking into account the information $\mathbf{s}$ the strategy $\mathbf{d}$, which maximizes the expected cost-benefits, has to be chosen from

$$CB^* = \max_{\mathbf{d}} E''_{\mathbf{X}|\mathbf{s}}\left[CB(\mathbf{z}, \mathbf{s}, \mathbf{d}, \mathbf{X})\right]. \tag{9.1}$$

$E''_{\mathbf{X}|\mathbf{s}}[-]$ is the expectation with respect to the predictive (updated) density function $f''_{\mathbf{X}}(\mathbf{x}|\mathbf{s})$. In the following the initial design variables **z** are not written explicitly. CB^* is the maximum cost-benefit corresponding to the optimal decision. If the benefits are not dependent on the stochastic variables then the optimisation problem can be written:

$$CB^* = \max_{\mathbf{d}} CB(\mathbf{d}) = \max_{\mathbf{d}} \left\{B(\mathbf{d}) - E''_{\mathbf{X}|\mathbf{s}}\left[C(\mathbf{s}, \mathbf{d}, \mathbf{X})\right]\right\} \tag{9.2}$$

where the future benefits are denoted *B* and the future costs are denoted *C*. Both benefits and costs should be discounted to the time of the re-assessment. The optimisation formulation can also be generalised to include decision variables related to experiment planning.

In the following time-invariant reliability problems are considered. It is assumed that there is no systematic reconstruction of the structure in case of failure and discounting can be ignored. The total expected cost-benefits can then be written

$$CB(\mathbf{d}) = B(\mathbf{d}) - C(\mathbf{d}) = B(\mathbf{d}) - C_S(\mathbf{d}) - C_f P''_f(\mathbf{d}), \tag{9.3}$$

where $C_S(\mathbf{d})$ and C_f models the costs due to repair/strengthening after the re-assessment and due to failure, $B(\mathbf{d})$ models the benefits and $P_f^{''}(\mathbf{d})$ is the probability of failure updated with the information $\mathbf{s}$. Failure / no failure should here be considered in a general sense as satisfactory / not satisfactory behaviour.

In the case the information *S* models (one or more) events modelled by an event margin $\{h(\mathbf{d},\mathbf{X}) \le 0\}$, and failure is modelled by a limit state function $g(\mathbf{d},\mathbf{X})$, the updated probability of failure is obtained from:

$$P_f^{''}(\mathbf{d}) = P\left(g(\mathbf{d},\mathbf{X}) \le 0 \middle| h(\mathbf{d},\mathbf{X}) \le 0\right) \tag{9.4}$$

In the case the information *S* is related to the measurements of the stochastic variables **X** then the (updated) density function $f_{\mathbf{X}}^{''}(\mathbf{x}|\mathbf{s})$ is used, see Eq. (9.1).

The optimal design $\mathbf{d}*$ is obtained from the optimisation problem

$$\max_{\mathbf{d}} CB(\mathbf{d}) = \max_{\mathbf{d}} \left\{B(\mathbf{d}) - C_S(\mathbf{d}) - C_f P_f^{''}(\mathbf{d})\right\} \tag{9.5}$$

Eq. (9.5) can equivalently be formulated as a reliability-constrained optimisation problem

$$\begin{aligned} &\max_{\mathbf{d}} && B(\mathbf{d}) - C_S(\mathbf{d}), \\ &\text{subject to} && \beta^{''}(\mathbf{d}) \ge \beta^{\min}, \end{aligned} \tag{9.6}$$

where the generalised reliability index is defined by $\beta^{''}(\mathbf{d}) = -\Phi^{-1}\left(P_f^{''}(\mathbf{d})\right)$. $\beta^{\min}$ is a code specified minimum acceptable reliability level related to annual or lifetime reference time intervals. How to choose $\beta^{\min}$ is further considered in Part II. Other design constraints can be added to Eq. (9.6) if needed. Eqs. (9.5) and (9.6) give the same optimal decision if $\beta^{\min}$ is chosen as the reliability level corresponding to the optimal solution $\mathbf{d}*$ of Eq. (9.5): $\beta^{\min} = \beta^{''}(\mathbf{d}^*)$, i.e. there is a close connection between $\beta^{\min}$ and C_f / C_S. This can easily be seen considering the Kuhn-Tucker optimality conditions for Eq. (9.5) and Eq. (9.6).

The basic decision problems considered above can be generalized to be used in reliability-based experiment and inspection planning. If $\mathbf{d}$ and $\mathbf{e}$ model the design and the inspection / repair decision variables the optimisation problem can be written:

$$\max_{\mathbf{d},\mathbf{e}} \ \mathrm{E}[B(\mathbf{d},\mathbf{e})] - \left\{E[C_{IN}(\mathbf{e})] + \mathrm{E}[\,\mathrm{C_S}(\mathbf{d})] + C_R(\mathbf{d},\mathbf{e})P_R(\mathbf{d},\mathbf{e}) + C_f P_F(\mathbf{d},\mathbf{e})\right\} \tag{9.7}$$

where $E[B]$ models the expected benefits, $E[C_{IN}]$ models the expected inspection costs, $E[C_S]$ models the expected costs due to repair/strengthening after the re-assessment, C_R models the repair costs, P_R is the probability of repair and P_F is the probability of failure.

Eq. (9.7) can be further generalised if a constraint related to a maximum annual (or accumulated) failure probability $\Delta P_F^{\max}$ is added. If the inspections performed at times $T_1, T_2, ..., T_N$ are part of $\mathbf{e}$ the optimisation problem can be written

$$\begin{aligned} &\max_{\mathbf{d},\mathbf{e}} && \mathrm{E}[B(\mathbf{d})] - E[C_{IN}(\mathbf{d},\mathbf{e})] - E[C_S(\mathbf{d})] - E[C_{REP}(\mathbf{d},\mathbf{e})] - E[C_F(\mathbf{d},\mathbf{e})] \\ &\text{s.t.} && d_i^l \le d_i \le d_i^u, && i = 1,...,N \\ & && \Delta P_{F,t}(\mathbf{d},\mathbf{e}) \le \Delta P_F^{\max}, && t = T_R, T_{R+1}, ..., T_L \end{aligned} \tag{9.8}$$

where, $E[C_{REP}]$ is the expected cost of repair and $E[C_F]$ is the expected failure costs. T_R is the time of re-assessment. The annual probability of failure in year t is $\Delta P_{F,t}$. The N inspections are assumed performed at times $0 \le T_R \le T_1 \le T_2 \le ... \le T_N \le T_L$.

The total capitalised expected benefits are written

$$E[B(\mathbf{d})] = \sum_{t=T_R+1}^{T_L} B_t \left(1 - P_F(t, \mathbf{d}, \mathbf{e})\right) \frac{1}{(1+r)^{t-T_R}} \tag{9.9}$$

The *t*-th term represents the capitalized benefits in year t given that failure has not occurred earlier, B_t is the benefits in year t, $P_F(t, \mathbf{d}, \mathbf{e})$ is the updated probability of failure in the time interval $[T_R, t]$ and r is the real rate of interest. The updating is based on the information **s**.

The total capitalised expected inspection costs are written

$$E[C_{IN}(\mathbf{d}, \mathbf{e})] = \sum_{i=1}^{N} C_{IN,i}(\mathbf{e}) \left(1 - P_F(T_i, \mathbf{d}, \mathbf{e})\right) \frac{1}{(1+r)^{T_i - T_R}} \tag{9.10}$$

The *i*-th term represents the capitalised inspection costs at the *i*-th inspection when failure has not occurred earlier, $C_{IN,i}(e_i)$ is the inspection cost of the *i*-th inspection,

The total capitalised expected repair costs are

$$E[C_{REP}(\mathbf{d}, \mathbf{e})] = \sum_{i=1}^{N} C_{R,i} P_{R_i}(\mathbf{d}, \mathbf{e}) \frac{1}{(1+r)^{T_i - T_R}} \tag{9.11}$$

$C_{R,i}$ is the cost of a repair at the *i*-th inspection and $P_{R_i}(\mathbf{d}, \mathbf{e})$ is the probability of performing a repair after the *i*-th inspection when failure has not occurred earlier and no earlier repair has been performed.

The total capitalised expected costs due to failure are estimated from

$$E[C_F(\mathbf{d}, \mathbf{e})] = \sum_{t=T_R+1}^{T_L} C_F(t) \Delta P_{F,t}(\mathbf{d}, \mathbf{e}) P_{COL|FAT}(\mathbf{d}) \frac{1}{(1+r)^{t-T_R}} \tag{9.12}$$

where $C_F(t)$ is the cost of failure at the time *t*. $P_{COL|FAT}(\mathbf{d})$ is the conditional probability of collapse of the structure given failure of the considered component.

Numerical solution of the decision problems requires solution of one or more optimisation problems. Since the optimisation problems formulated are generally continuous with continuous derivatives sequential quadratic optimisation algorithms such as NLPQL [76] and VMCWD [70] can be expected to be the most effective, see [49]. These algorithms require that values of the objective function and the constraints be evaluated together with gradients with respect to the decision variables.

The probabilities in the optimisation problems can be solved using FORM techniques, see [68]. Associated with the FORM estimates of the probabilities also sensitivities with respect to parameters are obtained. If the decision problem includes analysis of a structural system the finite element method in combination with sensitivity analyses can be used.

9.2 Example Application

A road bridge with concrete columns is considered. The total expected lifetime is assumed to be T_L. The concrete columns are exposed to chloride ingress due to spread of de-icing salts on and below the bridge. There are some indications that chloride has penetrated the concrete and that corrosion of the reinforcement could be expected within the next few years. Therefore a re-assessment is performed at time T_R as illustrated in Figure 9.2.

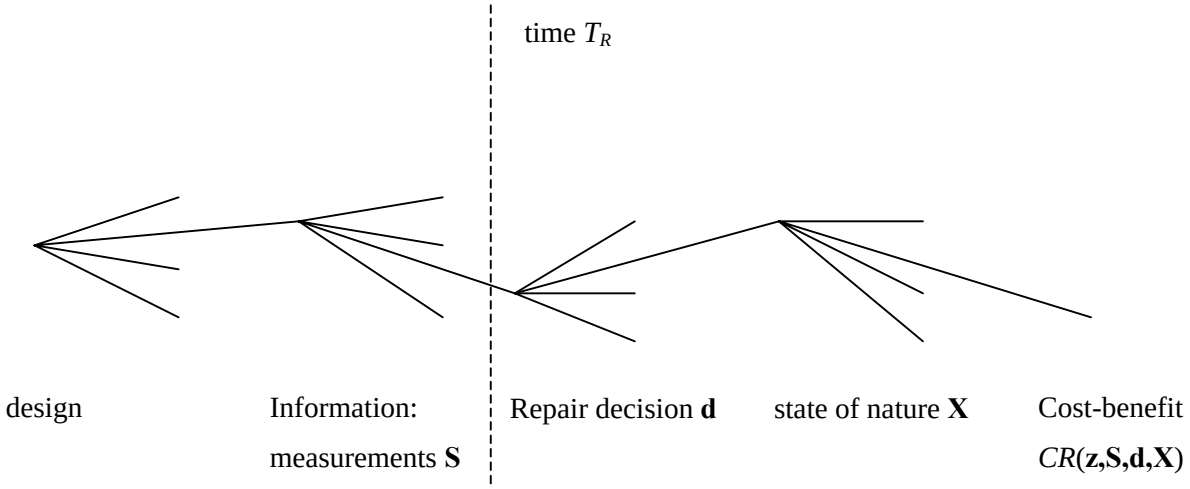


Figure 9.2. Example: decisions in re-assessment of concrete bridge.

Chloride ingress is one of the most common destructive mechanisms for this type of structures. The most typical type of chloride initiated corrosion is pitting corrosion which may locally cause a substantial reduction of the cross-sectional area and cause maintenance and repair actions which can be very costly. Further, the corrosion may make the reinforcement brittle, implying that failure of the structure might occur without warning. Probabilistic analysis of the time to initiation of corrosion in concrete structures has been treated in a number of papers. The models used in this example are based on equivalent models described in [44].

9.2.1 Information from measurements S

At the time of re-assessment it is assumed that chloride profiles are taken from representative parts of the concrete columns. The estimation of the time to initiation of corrosion is based on these chloride profiles combined with prior knowledge. A chloride profile consists of a number of measurements of the chloride concentration as a function of the distance to the surface, y. Using the chloride profiles, the surface concentration and the diffusion coefficient can be estimated.

It is assumed that diffusion (transportation) of chlorides into the concrete can be described by a one-dimensional diffusion model where $C(y,t)$ is the content of chloride at time *t* in the depth *y*, $D(y,t)$ is the coefficient of diffusion (transportation) at time *t* in the depth *y*, C_S is the surface concentration and C_{init} is the initial chloride concentration.

It is assumed that the diffusion coefficients can be written:

$$D(y,t) = D_0(y)\left(\frac{t_0}{t}\right)^a \tag{9.13}$$

where $D_0(y)$ is the reference diffusion coefficient at the reference time t_0 and *a* is an age coefficient (0<*a*<1). Models for the diffusion coefficient can include different diffusion coefficients in different depths.

Based on *n* measurements in one chloride profile the surface concentration c_S, the coefficient of diffusion D_0 and the age coefficient *a* can be estimated using the Maximum Likelihood method. If the number of measurements is sufficiently high asymptotic estimates of the covariance matrix $\mathbf{C}$ of the estimates of (c_S, D) can be obtained from, see [64]

$$\mathbf{C} = \left[-\mathbf{H}\right]^{-1} \tag{9.14}$$

where $\mathbf{H}$ is the Hessian matrix of the Log-Likelihood function. One estimate of μ and $\mathbf{C}$ are obtained from each chloride profile. If a number of chloride profiles is available Bayesian statistics can be used to model these and subjective information in the form of prior knowledge can easily be included, see [44].

If M chloride profiles are available the estimates μ and $\mathbf{C}$ from each profile can be combined with prior knowledge using Bayesian statistics to obtain predictive distributions for the surface concentration c_S, the coefficient of diffusion D_0 and the age coefficient a, see [44]. If $\mathbf{x} = (C_S, D_0, a)$ the following M sets of estimates are obtained: $(\hat{\mathbf{X}}_1, \mathbf{C}_{\hat{X}_1}), (\hat{\mathbf{X}}_2, \mathbf{C}_{\hat{X}_2}), ..., (\hat{\mathbf{X}}_M, \mathbf{C}_{\hat{X}_M})$. A lognormal distribution is assumed to describe both the surface chloride concentration and the diffusion coefficient parameters. These parameters are then transformed into normally distributed variables, i.e. the n-dimensional stochastic vector $\boldsymbol{X}$ with expected value $\boldsymbol{\mu}$ and covariance matrix $\mathbf{C}_{\mathbf{X}}$ can be assumed to be normally distributed.

Further it is assumed that the covariance matrix $\mathbf{C}_{\mathbf{X}}$ can be written

$$\mathbf{C}_{\mathbf{X}} = \frac{1}{h}\eta^{-1} \tag{9.15}$$

where $\boldsymbol{\eta}$ is a matrix of dimension $n \times n$ with determinant $|\eta| = 1$. h can be considered as the precision of the stochastic vector $\boldsymbol{X}$. It is assumed that $\boldsymbol{\mu}$ and h are uncertain and are modelled by stochastic variables, i.e. it is assumed that the individual standard deviations of the stochastic variables and the correlation coefficients are known.

From the M available realisations $(\hat{\mathbf{X}}_1, \hat{\mathbf{X}}_2, ..., \hat{\mathbf{X}}_M)$ of the stochastic vector $\boldsymbol{X}$ the following quantities can then be calculated

$$\mathbf{m} = \frac{1}{M}\sum_i \hat{\mathbf{X}}_i \tag{9.16}$$

$$\mathbf{D} = M\eta \tag{9.17}$$

$$\nu = n(M-1) \tag{9.18}$$

$$\mathrm{v} = \frac{1}{\boldsymbol{\nu}}\sum_i (\hat{\mathbf{X}}_i - \mathbf{m})^T \boldsymbol{\eta}(\hat{\mathbf{X}}_i - \mathbf{m}) \tag{9.19}$$

Initially it is assumed that (***m, h)*** is Normal-Gamma-2 distributed, see [72].

$$f_{\mu,h}(\mu, h|\mathbf{m}, \nu, \mathbf{D}, \mathrm{v}) = f_\mu(\mu|\mathbf{m}, \sqrt{\tfrac{1}{h\cdot\mathbf{D}}})\, f_{\gamma_2}(h|\mathrm{v}, \nu) \tag{1.20}$$

where $f_{\boldsymbol{\mu}}$ and $f_{\boldsymbol{\gamma}_2}$ indicate an n-dimensional Normal distribution and a Gamma-2 distribution, respectively.

If prior knowledge in the form of n^{pr} observations $(\hat{R}_1^{pr}, \varepsilon_1^{pr}), (\hat{R}_2^{pr}, \varepsilon_2^{pr}), ..., (\hat{R}_{n^{pr}}^{pr} \varepsilon_{n^{pr}}^{pr})$ of R for given experimental designs are available the corresponding prior parameters m', D', ν' and ν' can be determined from the above equations.

If a prior Normal-Gamma-2 distribution of (***m, h)*** is assumed with parameters $(\mathbf{m}', \nu', \mathbf{D}', \mathrm{v}')$ then the posterior distribution of (***m,*** h) will also be a Normal-Gamma-2 distribution with the following parameters

$$\mathbf{D}'' = \mathbf{D}' + \mathbf{D} \qquad \mathrm{p}'' = \mathrm{rank}(\mathbf{D}'') \tag{9.21}$$

$$\mathbf{m}'' = (\mathbf{D}'')^{-1}(\mathbf{D}' m' + \mathbf{D} m) \tag{9.22}$$

$$\nu'' = (\nu' + p') + (\nu + p) - p'' \tag{9.23}$$

$$\mathrm{v}'' = \frac{\nu'\mathrm{v}' + \mathbf{m}'^{\mathrm{T}}\mathbf{D}'\mathbf{m}' + (\nu\mathrm{v} + \mathbf{m}^T\mathbf{D}\mathbf{m}) - (\mathbf{m}''^T\mathbf{D}''\mathbf{m}'')}{\nu''} \tag{9.24}$$

Finally the (updated) **predictive distribution** of $\boldsymbol{X}$ given the prior parameters $(\mathbf{m}', \nu', \mathbf{D}', \nu')$ and the observations $(\hat{\mathbf{X}}_1, \hat{\mathbf{X}}_2, ..., \hat{\mathbf{X}}_M)$ is an *n*-dimensional Student-t distribution

$$f_{\mathbf{X}}(\mathbf{X}|\mathbf{m}'', \sqrt{\frac{\mathbf{D}'''}{\nu''}}, \nu'') \tag{9.25}$$

where

$$\mathbf{D}''' = (\mathbf{D}''^{-1} + \eta^{-1})^{-1} \tag{9.26}$$

The Student-t distribution can be written

$$f_S(\mathbf{X}|\mathbf{m}'', \sqrt{\frac{\mathbf{D}'''}{\nu''}}, \nu'') = \int_0^\infty f_N\left(\mathbf{X}|\mathbf{m}'', \sqrt{\frac{\mathbf{D}'''^{-1}\nu''}{h}}\right) f_{\gamma 2}(h|1, \nu'')dh \tag{9.27}$$

A Gamma-2 distribution with parameters (1, ν) corresponds to a Gamma-1 distribution with parameters (ν/2, ν/2). Instead of generating outcomes of $\boldsymbol{X}$ from a Student-t distribution (with mean **m**”, standard deviation $\sqrt{\frac{\mathbf{D}'''}{\nu''}}$ and parameter ν”), outcomes of $\boldsymbol{X}$ can be found by generating outcomes of *h* from a Gamma-1 distribution (with parameters ν”/2 and ν”/2) and for each given *h* be generating outcomes of $\boldsymbol{X}$ from a Normal distribution (with mean **m**” and standard deviation $\sqrt{\frac{\mathbf{D}'''^{-1}\nu''}{h}}$).

9.2.2 Repair / maintenance alternatives

On the basis of the available information described above the decision maker has to decide which repair/maintenance strategy should be applied. As an example, three different strategies are described below based on the models in [44]. All the costs given below are in some monetary unit. It is assumed that the repair is carried out before the probability of any critical event such as total collapse of the bridge. Therefore, in the following the optimisation problem is solved without any restriction on the probability of some critical event.

Strategy 1: consists of a cathodic protection. This strategy is implemented when corrosion has been initiated at some point. In order to determine when corrosion is initiated, inspections are carried out each year, beginning five years before the expected time of initiation of corrosion. The cost of these inspections is 25 each year except for the last year before expected initiation of corrosion where the cost is 100. The cost of the cathodic protection is 1000 and the cost of running the cathodic protection is 20 each year.

Strategy 2: is implemented when 5% of the surface of the bridge columns shows minor signs of corrosion, e.g. small cracks and discolouring of the surface. The repair consists of repairing the minor damages and applying a cathodic protection. As for strategy 1 the costs related to this strategy are the costs of the repair and the costs of an extended inspection programme which starts three years before the expected time of repair. However, by this strategy, also the costs related to running the cathodic protection must be taken into account. The cost of repair is 2000, the cost of inspection for three years before the repair is 100 each year and the cost of running the cathodic protection is 30 each year.

Strategy 3: repair is performed as a complete exchange of concrete and reinforcement in the corroded areas. The strategy is implemented when 30% of the surface at the bridge columns shows distinct signs of corrosion, such as cracking and spalling of the cover. The cost related to this strategy are the cost of the repair and the cost of an extended inspection programme which starts three years before the expected time of repair. The cost of repair is 3000 and the cost of inspection in the three years before repair is 200 each year. Traffic restrictions in the year of repair he bridge decrease the benefits with 1000.

9.2.3 Cost – benefit models

The total expected costs for maintenance/repair is determined from

$$C_S(d_1, d_2, d_3) = \sum_{i=T_R}^{T_L} P_i(\mathbf{d}) C_i(\mathbf{d}) \tag{9.28}$$

where $\mathbf{d} = (d_1, d_2, d_3)$ is the three repair/maintenance options, $P_i(\mathbf{d})$ is the probability that repair/maintenance is performed in year *i* and $C_i(\mathbf{d})$ is the total costs of the repair strategy if the repair is performed in year *i*:

$$C_i(\mathbf{d}) = \sum_{j=T_R}^{T_L} C_{i,j}(\mathbf{d}) \frac{1}{(1+r)^{j-T_R}} \tag{9.29}$$

$C_{i,j}(\mathbf{d})$ is the repair/maintenance cost in year *j* if the repair is performed in year *i*. These costs can be found in the descriptions of the repair strategies. The costs are discounted to the time of re-assessment T_R using the real rate of interest *r*.

The expected benefits in the remaining lifetime are determined from

$$B(d_1, d_2, d_3) = \sum_{i=T_R}^{T_L} B_0 \frac{1}{(1+r)^{i-T_R}} - \sum_{i=T_R}^{T_L} P_i(\mathbf{d}) B_i(\mathbf{d}) \tag{9.30}$$

where

$$B_i(\mathbf{d}) = \sum_{j=T_R}^{T_L} \Delta B_{i,j}(\mathbf{d}) \frac{1}{(1+r)^{j-T_R}} \tag{9.31}$$

B_0 is the basic annual benefit from use of the bridge and $\Delta B_{i,j}(\mathbf{d})$ is the loss of benefits in year *j* due to repair in year i, e.g. due to traffic restrictions.

9.2.4 Optimal decision

The optimal repair strategy is obtained solving the optimisation problem

$$\max_{\mathbf{d}} \qquad B(\mathbf{d}) - C_S(\mathbf{d}) \tag{9.32}$$

The expected costs are determined using the predictive stochastic model for the surface concentration c_S, the coefficient of diffusion D_0 and the age coefficient *an* obtained using the available information.

10 LIFE TIME EXTENSION

10.1 Breakdown of Retrofit Steps

Lifetime extension of existing systems aims at maintaining long-term serviceability and durability through practical and effective measures. Uniform safety/reliability of existing systems has to be gained through **retrofitting**, whereby components with low rating factors and components susceptible to deterioration should be considered in particular. The retrofit procedure, thereby, can be split in two parts: **restoration** and **preservation**. Whereas restoration is aimed at providing an acceptable level of system safety/reliability with a minimum set of activities, preservation is intended to preserve this level during the extended service life.

In general, an economically balanced amount of restoration and preservation steps should be striven for. In other words, all restoration and preservation steps should be based on a **cost-benefit analysis** for the extended service life cycle. This allows, on the one hand, identifying the cost-optimal solution out of a plethora of possible retrofit efforts for a certain system subjected to certain demands. And, on the other hand, it also allows quantifying the retrofit effort as such, i.e. if the retrofit will provide any kind of benefit at all, or a complete renewal of the system would be a more adequate strategy.

10.2 Restoration

Restoration is defined as an essential or **minimum set of retrofit steps** such that the service life can be extended for a **specified time period**. In general cases, the exact time period, for which the service life of an existing system should be extended, is not known precisely. Quite often the decision about further usage of a system depends on how the system can be operated in the extended life period. In this cases, a minimum time period has to be specified, until when steps of preservation should be undertaken.

In the following the essential restoration steps are itemized. Whereas a safe/reliable condition of operation is mandatory for all systems, further steps–like, e.g., the reassessment of the system capacity and demand, or measures against deterioration–are dependent on the results of the system analysis. The restoration steps are:

Assuring that the demand on the system is met with an acceptable level of confidence: All steps taken for retrofit have to assure that the system can be operated safely/reliable. Even if there is no deterioration present, it has to be certified by **reanalysis** that the demand on the system is met by the system capacity with a certain confidence. The analysis should further identify units with low rating factors, units susceptible to deterioration, as well as system deficits.

Re-assessing the system capacity: If the demand is not met by the system, or units have been identified as vulnerable, **reconstruction** efforts should be undertaken. These steps become, e.g., necessary if a system has to meet more different demands in the extended service period than it has met in the previous service period. Typical examples are warehouses which are re-constructed as exhibition spaces, thereby inducing different load scenarios and possible adaptations of the sizes of the available rooms, or processing units which are adapted to modified manufacturing methods.

Re-assessing the system demand: In addition to modifying the system, also **restrictions on controllable demands** can be imposed. The success of such a re-assessment is dependent on the type of system, i.e. the ratio of controllable to non-controllable demands. Whereas civil engineering structures are characterized, in general, by a high degree of exposure to non-controllable demands like, e.g., wind, snow and earthquake loads, processing units, e.g., show a high degree of insularity with regulated in- and outflows. Therefore, typical examples for imposing restrictions on demands are the modification of the operating temperature range of plants, or traffic regulations for infrastructure systems (e.g. speed and weight limits).

Restoring the integrity of significantly deteriorated components: This can, on the one hand, be achieved by **repair** work, e.g. welding of cracks, renewing of protective covers, overhauling of engine components or modification/optimisation of the flow of traffic. On the other hand, the integrity can also be restored by **replacement** or **addition** of units, as, e.g., additional reinforcements like metal jackets on reinforced concrete columns, hot standby vessels and turbines, additional traffic lanes, or traffic guidance systems.

Correcting details which will pose further or accelerated deterioration: Quite regularly the reasons for the occurrence of extensive deterioration are errors or a matter of ignorance in the design or construction process. In other words, in these cases there exists a discrepancy between the system model and the true system. Therefore, restoration does not only mean to repair or replace deteriorated units, but also to identify sources of extensive deterioration and to take steps for correction. Typical examples are the change of the dynamic behaviour of a structure by active/passive dampers or base-isolation devices, or measures against corrosion.

All the restoration steps allow to extend the service life of the system for a minimum time period. Further service life extensions can be achieved by preserving the restored system.

10.3 Preservation

Preservation defines all activities which allow keeping the system in a state such that a continuous safe/reliable operation is guaranteed during the entire service life. This is of paramount importance for systems which are subject to deterioration with usage and age, as well as for a further extension of the lifetime. Preservation encircles different activities: information updating, re-assessment and, most notably, maintenance. Maintenance allows an active exertion of influence on the system capacity. Thereby, two broad strands of maintenance can be distinguished:

Corrective maintenance: Corrective maintenance is the maintenance that occurs when a system or unit fails. Thus, corrective maintenance denotes all actions performed as a result of failure, to **restore a system or unit to a specified condition**.

Preventive maintenance: Preventive maintenance is the maintenance that occurs when a system is operating. Thus, preventive maintenance denotes all actions performed in an attempt to preserve a system in a specified condition by providing **systematic inspection**, **detection**, and **prevention** of incipient failures of units.

Quite obviously, the most common strategy for preservation of existing systems is preventive maintenance. The procedure of preventive maintenance can be further distinguished along certain criteria influencing the maintenance schedule:

Age-dependent preventive maintenance: This policy has its origin in the age replacement policy, where a unit is always replaced when it reaches a certain age or at failure, whichever occurs first. Extensions thereof led to the age-dependent preventive maintenance policy, where the unit is preventively maintained at a specified time. The preventive maintenance can be minimal, imperfect or perfect–the latter being simply replacement. The maintenance time is measured from the last replacement. The specified time may be also random, since quite often it is impractical to maintain a unit in a strictly periodic fashion. A typical example is a processing unit, where maintenance can be only performed after completing a full work cycle.

Periodic preventive maintenance: Utilizing this policy, the unit is preventively maintained at fixed time intervals independent of the failure history of the unit, and repaired at intervening failures. Most common in this class are periodic replacement with minimal repair and imperfect maintenance with minimal repair. Also combinations thereof are utilized, e.g., an imperfect repair until a specified number of repairs are reached and the unit is replaced. This is typical for deteriorating systems, where in the beginning only minor repairs are necessary, whereas in the later stages the unit is in such a worse operating condition that major maintenance at a higher cost has to be undertaken.

Sequential preventive maintenance: In opposition to the periodic preventive maintenance policies, sequential preventive maintenance policies are based on the state of the unit after the last maintenance and, therefore, these policies are in general performed at unequal time intervals. Typical examples are deteriorating units, which require more frequent maintenance with increased ages. Consequently, sequential preventive maintenance allows a more flexible scheduling then periodic preventive maintenance and, therewith, also more cost-effective maintenance solutions.

Failure limit preventive maintenance: Under this policy, a preventive maintenance is performed when the failure rate or other safety/reliability indices of a unit reach a predetermined level, whereas intervening failures are corrected by repair. This policy allows operating a unit or system above the acceptable level of safety/reliability. The policy can be combined with inspections, when the maintenance is based on measure-

ments of some increasing state variable like, e.g., wear, accumulated damage or accumulated stress, and the proneness to failure is described by an increasing state-dependent failure rate function.

Repair limit preventive maintenance: This policy takes into account the cost inclined with repair. If one unit fails, the repair cost (rate) is estimated and repair is undertaken if the estimated cost (rate) is less than a prescribed limit. If the limit is exceeded the unit is replaced. One should note, that the repair cost does include also the repair time or downtime cost.

The above itemized preventive maintenance policies are designed for systems composed of a single stochastically deteriorating unit or multiple stochastically deteriorating units, when there exists neither failure dependence, nor structural dependence or economical dependence. If there exists any kind of dependence, then the optimal maintenance policy for a single unit is not necessarily the optimal maintenance policy of a multi-unit system, since the maintenance decisions are no longer independent. In other words, the optimal maintenance policy for a unit of the system depends on the state of all other units. E.g., structural dependence means that the state of one unit influences the demand on other units, whereas economic dependence means that performing maintenance on several units costs less money or needs less time than performing maintenance on each unit separately. These facts are taken into account in group maintenance policies which are mainly based on the specific structure of the system dependence (e.g., series or parallel) or the degree of redundancy inherent in the system. Typical examples are systems with units operating in series, where a failed unit has to be repaired immediately to minimize downtimes of the entire system, whereas for a system with units operating in parallel, the failed unit has not to be repaired immediately, but the maintenance can be performed when more units need repair or at the scheduled system maintenance time. Nevertheless, group maintenance policies utilize the above mentioned single unit policies as building blocks.

11 CONCLUSIONS

This document presents an overview of all aspects regarding assessment and life extension of existing structures and facilities. It focuses on the construction industry with its applications in various industrial sectors, like oil & gas (offshore platforms, pipelines), infrastructure (bridges, roads, railways, marine transportation), building, chemical industry (facilities and chemical plants), power industry (plants, power distribution facilities), aircraft. But nevertheless it is transmissible to nearly every kind of structure or facility.

Assessment strategies for existing structures are currently applied at a high level across all industrial sectors. However, each industrial sector works for its own profit, which leads to many standards and guidelines for specific problems. A harmonisation of basic rules certified in a standard would be adjuvant to use the best practice of each industrial sector. A suggestion for such a framework is introduced in this report. Specific rules and standards are still required to cover specific problems. Of course, an improvement and extension of methods and processes is necessary. Especially, case studies to verify and validate the proposed framework are essential.

The presented state-of-the art report also shows, that a cost-optimal assessment and life extension is not possible without considering the whole life cycle of the structure or facility. Therefore, the assessment process has to be involved next to other aspects, like risk assessment, risk management, and maintenance processes to obtain a cost-optimal life cycle. During the life of a structure or industrial plant, many decisions have to be made. It is recommended to establish the decision on probabilistic long-term cost-benefit criteria. Those decisions are objective and can be easily integrated in software supported decision tools. However, such an advanced decision-making process is rare in the current praxis.

The report on state-of-the art of assessment and life extension of existing structures and industrial plants identifies the current best practice across all industrial sectors. Nevertheless, more research, advertisement, and practical experience in needed to reach the aim of establishing a well-proven cost-optimal framework for the assessment and life extension of existing structures and industrial plants. The identified future needs are intensively described in [3].

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leaded by

Helen Bolt (BOMEL Ltd UK)

helenbolt@bomelconsult.com

PART II

Acceptable Safety Levels for Reassessment

Document Status

Authors

Name	Company
John Williams (complete report 6.2)	BOMEL
Helen Bolt (complete report 6.2)	BOMEL
Ole Tom Vardal	Aker Offshore Partner, Norway (AOP)
Christian Bucher	Bauhaus University Weimar (BUW)
Maik Brehm	Bauhaus University Weimar (BUW)
Oliver Kuebler	Swiss Federal Institute of Technology (ETHZ)

Reviewing/Approval of report

Name	Company	Signature	Date

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Table of Contents

1 INTRODUCTION

As existing facilities / structures are modified, as engineering knowledge advances and as the requirements to extend operational life increase, industries must demonstrate that operations can continue safely and economically.

It is widely acknowledged that a facility / structure may not meet current design standards, but may still be fit-for-purpose. In many cases, the cost of enhancing an existing facility / structure so as to meet new design standards is prohibitive. Equally, the societal cost of disqualifying an existing installation through loss of employment, revenue, etc. must be recognised and therefore a higher level of risk may be acceptable for an existing structure than may be inherent in new designs.

Traditionally, risk levels were not defined in terms of failure probability and a deterministic approach was used for design. In the simplest form of deterministic design, the maximum load that would be applied during the life of a facility / structure and the minimum corresponding strength of the design option (or the existing facility / structure) are estimated. If the latter is greater than the former by the required margin, the structure is deemed to be 'safe'.

However, reassessment is a different activity to design. It is aimed at assessing the real condition of the existing facility / structure and answering the questions: 'is the facility / structure safe now; will it remain so in the future; and can it be used for its intended purpose for its remaining operational life?' A probabilistic, rather than deterministic, approach provides an evaluation tool which assists in answering such questions.

With a probability approach, the economics associated with risk of failure can be assessed and precedents have been set in a number of industries for the acceptance of higher probabilities of failure offsetting the costs and risks of in-service strengthening / upgrading. Clearly, similar arguments balancing economic failure probabilities could be applied to new designs, although the cost of incorporating additional reliability (lower risk) is often relatively small if planned at the outset.

In considering risk acceptance criteria, both the probability of failure (where different uncertainties are associated with design and with reassessment) and the consequences of failure (in economic, societal and environmental terms) must be considered.

1.1 Probability of Failure

The probability of failure in existing facilities / structures increases where there is uncertainty in:

- Quality of design and manufacture
- Service history
- Operating conditions / environment
- Failure mechanics
- Rate of degradation.

1.1.1 Quality of design and manufacture

'Quality' is governed both by the level of compliance to procedures and standards as well as by the understanding of structural or mechanical behaviour at the time of design or manufacture.

More detailed design information is likely to survive, and to be readily available, for 'recent' structures / facilities than 'older' structures / facilities. Also, with the greater implementation in recent years of company Health and Safety Management Systems and associated quality control procedures, the design criteria, assumptions and checking performed tend to be more rigorously recorded and more formally documented (allowing the 'baseline condition' to be more easily established).

For manufactured components, an audit trail (identifying where, when, under what controlled conditions and the inspections performed) is important in establishing the as-built baseline reliability for the facility / structure.

1.1.2 Service history

Information on the frequency, nature and findings of in-service inspections add considerably to the level of confidence (or otherwise) in the integrity of a facility / structure. For example, components predicted during design as fatigue sensitive and with relatively low fatigue lives should have been prioritised for periodic inspection. If no anomalies are identified in practice over time, uncertainty in the fatigue performance of those components is reduced.

Depending on the age of a facility / structure, it could have been subjected to one or more inspections throughout its life. Anomalies and observations should be logged and remedial actions taken. If periodic inspections are performed, trends can be monitored and confidence gained provided behaviour generally follows theoretical predictions. Conversely, unexpected findings from the last inspection add to uncertainty.

1.1.3 Operating conditions / environment

Existing facilities / structures may have been modified throughout their operational life. New components could have been added or redundant components could have been removed (emphasising the need for rigorous weight control documentation). Increased or additional functional requirements may have been introduced (e.g. higher operating temperature / pressure, greater corrosion resistance or protection from fire / blast scenarios), and these may have altered the baseline reliability against which reassessment is to be benchmarked.

More accurate imposed loading data is constantly being generated, including metocean (wind and wave) data for fixed and floating offshore structures and seismic spectral data for major onshore facilities. In some instances, this latest data is **less** onerous than that used for the original design. In addition, it may be appropriate to remove or reduce partial safety factors introduced to cover uncertainty in earlier measurements or generic criteria adopted during design.

1.1.4 Failure mechanisms

Large scale model testing and advanced analytical modelling have added significantly to the overall understanding of failure mechanisms and ultimate loads of facilities / structures. This information may not have been available to engineers at the time when the facilities / structures were conceived, designed and fabricated / constructed. However, whilst an increased understanding of behaviour reduces current uncertainty, the identified failure may not be a mechanism previously evaluated as a potential risk.

1.1.5 Rate of degradation

Ageing structures tend to be evaluated as if they were new, possibly with a reduction in member size to reflect obvious corrosion or other surface degradation loss. Often, there is little recognition that degraded or deteriorated structures may not behave in the same manner as structures before deterioration. This is a significant issue with concrete bridges where corrosion and deterioration due to chlorides in de-icing salts are one of the biggest problems associated with reassessment of such structures (see Section 5).

1.2 Consequences of Failure

The types of risk encompass:

- Financial / business
- Societal
- Environmental.

The consequences of failure of a facility / structure may have an impact on one or more of the above and may be either direct or indirect.

1.2.1 Financial / business

Failure of a facility / structure will obviously have direct financial consequences. The business operated from the premises or the product manufactured at the facility will be seriously disrupted and revenue will be lost, as well as the cost of reinstating the facility / structure.

However, in addition to the direct costs, there are consequential costs associated with the failure of most major installations. For example:

- **Offshore Production Platforms:** an explosion on an offshore platform may have direct consequences such as loss of life or injury and repair / replacement of the facility. However, the Piper Alpha disaster in the UK Sector of the North Sea in 1988 led to the introduction of new regulations which included the requirement that the owner / operator of any offshore installation must demonstrate the adequacy of preventative and protective measures. Blast and fire walls were retrofitted, escape routes and shelters were enhanced, and operational procedures were tightened. The overall cost to the industry was considerable.
- **Maritime Transport:** the sinking of oil tankers such as the 'Prestige' off the Spanish coast in 2002 or the 'Exxon Valdez' off Alaska in 1989 has not only an obvious direct environmental impact from oil pollution but also associated costs for clean up and loss of livelihood for fishermen and those in the tourism industry.
- **Railways:** accidents on railways resulting in multiple deaths and injuries are given extensive coverage in the media and can severely damage public confidence in the transport system. Passenger numbers drop both from fear of a similar incident as well as from the disruption and restrictions imposed during remedial works to minimise the probability of such an incident reoccurring.
- **Motorways and Bridges:** indirect costs associated with rehabilitation of motorways and bridges are borne not by the owner but by users. Damage to a bridge, for example, can lead to restrictions of the permitted load (imposing detours on heavy trucks), closure (imposing detours on all traffic) or reduced traffic capacity (resulting in waiting time and reduced speed). It is necessary to include these indirect costs when determining which strategy is optimum for society.
- **Building Sector:** in the commercial sector, severe damage to a commercial building will disrupt business through the possible requirement for relocation, loss of documents and records (electronic and paper) and staff morale. In the domestic sector, temporary or permanent re-housing may be necessary.

1.2.2 Societal

Societal risk is often perceived in terms of the Potential for Loss of Life (PLL) of an individual incident. As society in general has an aversion to single accidents which result in multiple fatalities, a number of accidents on the roads (each entailing one or two fatalities) tend not to be given the same media coverage as a single rail or air crash. Hence, annualised risk to a group or an individual passenger / employee presents a less emotive measure for comparison between industries / operations.

When considering low probability / high consequence events, society also has a different perspective depending on whether the risk is to the public at large or to industrial workers. For example, the first generation of offshore structures in the UK Sector of the North Sea did not consider seismic events as a design criterion. Even today, whilst earthquake scenarios are evaluated, it is generally in terms of the effects on certain structural components (cranes, flare towers, etc) and process plant to pipework connections. However, in the nuclear industry, seismic analysis is commonplace and has formed part of the power plant design procedure since the early years of the nuclear industry.

Measured seismic activity in Northern Europe, whilst more frequent than generally perceived by the public, is of relatively low severity compared to other parts of the world. For a manned offshore structure in the North Sea whose design is dominated by extreme environmental loading (waves and wind), the risk associated with an earthquake is extremely remote and the consequences in PLL are limited to the onboard

operatives (typically 100-200). Thus any requirement to retrospectively protect against seismic events for existing offshore facilities needs to be considered with caution since the cost of remedial works may be disproportionate to the potential benefits.

On the other hand, the extremely low probability of an earthquake causing severe damage to a nuclear installation is a very different scenario, with the potential to jeopardise the lives of the public over a large area.

The consequences of failure in terms of life safety can thus be grouped as follows:

- Component failure with PLL in single figures
- Catastrophic facility / structure failure with large PLL of operatives or users
- Catastrophic failure with PLL encompassing large sections of the public (e.g. from meltdown at a nuclear facility).

1.2.3 Environmental

The consequences of failure in terms of environmental risk may be linked with financial risk (e.g. clean-up of oil pollution from tanker or offshore facility failures) or societal risk (e.g. fallout from a nuclear disaster).

However, environmental risk may also arise from such activities as burning of fossil fuels (global warming and pollution) or improper use of pesticides. The consequences of such activities are extremely difficult to quantify and the risks may largely lie in the unknown impact.

2 TARGET SAFETY LEVELS

As discussed in Section 1, reliability methods are increasingly being used in most industrial sectors to make decisions regarding safety and life cycle costs of facilities / structures. Such methods, which deal with the uncertainties associated with design, fabrication and operation, may be classified[4] as:

- SRA (Structural Reliability Analysis[1]) to determine failure probability, considering variability and uncertainties
- QRA (Quantitative Risk Analysis[2]) to estimate the likelihood of fatalities, environmental damage or loss of assets.

SRA is sometimes used to provide input into a Quantitative Structural Risk Analysis (QSRA) and possibly a QRA. In particular, QSRA accounts for human factors which may lead to accidental loads and abnormal resistance.

The target safety level of a facility / structure should depend on the possible consequences of failure in terms of risk to life (or injury), economic losses and the level of social inconvenience. It also depends on the uncertainties considered (including the SRA or QSRA methodology), the initiating events and failure modes of components / systems, and the expense / effort required to reduce the risk of failure.

In principle, a target level which reflects all hazards (environmental loads, seismic events, etc) and all failure modes (ultimate, fatigue, foundations, etc.) as well as the different phases (including in-place operation and temporary conditions associated with construction and repair) could be defined with respect to each of the three ultimate consequences of failure outlined in Section 1.2, and the most severe would govern the decision making. Alternatively, if all consequences were measured in economic terms, a single target safety level could be established.

In practice, however, it is convenient to treat different hazards, failure modes and phases separately. A certain portion of the total (target) failure probability could then be allocated to each case, but often the simplification is made to treat each case separately[2, 3].

2.1 Reference Period for Target Failure Probabilities

The target failure probability must be linked with a given time period (e.g. a year or the service life of the facility / structure). If the relevant consequence is fatalities, annual failure probabilities are often adopted to ensure the same fatality risk to individuals at any time (see Section 1). This approach means that the target level does not depend upon the number of people at risk. However, with respect to environmental damage and economic loss, the target level should depend on the overall potential consequences.

2.2 Methods for Establishing Target Levels

Various methods(4) may be applied to establish the target level:

- The implicit safety or risk level from existing codes
- The experienced likelihood of fatalities, environmental damage or property loss associated with operations which are considered to be acceptable
- Cost-benefit criteria.

2.2.1 Safety level implied by existing codes

The target level for SRA is commonly taken to be the implied probability of failure in given codes or guidelines which are judged to be acceptable. To achieve a representative target level, several cases of geometries, material properties and load conditions should be considered. The implied failure probability will therefore vary and the target level should be based on the mean value or some other measure of the implied failure probability. Obviously, if a relaxation of safety level is desirable, a higher value than the mean is selected. Different levels may be used depending upon the mode of failure, consequences of failure etc. In particular, it is necessary to make a distinction between target level for components and the system. If a single target value for design is applied, the target value could be a weighted mean, with a weight factor which depends upon the consequence of failure for the different components considered.

A main issue in code calibration has been the assessment of uncertainties, especially in the environment loads. Old and obsolete codes tend to represent data available at the time the code was drafted, often reflecting a higher uncertainty level in yield strength and geometrical imperfections than current data (due to improvements in fabrication procedures achieved in later years).

The target level in code calibration is commonly based on the average failure probability implied by some existing code. This is particularly problematic when carrying out reassessment of a facility / structure designed (and constructed) during the regime of a now obsolete code.

It has been documented(2, 4) that different uncertainty assessments have been used by different organisations, and caution needs to be taken if target levels inferred by one organisation are applied by another. Target levels should therefore be related to specific procedures for uncertainty and reliability analysis.

It is also important that the reliability-based design or assessment is consistent with well established design / assessment practice. However, this is not always the case. For example, general guidance from NKB(5) and DNV(6) typically specify annual target values in the range 10^{-3} to 10^{-6}. Moan(7) reviewed the probability of failure inherent in offshore oil and gas codes for primary jacket components and annual values were found to be in the range $2x10^{-3}$ to 10^{-5}.

2.2.2 Target level based on accident experience

As outlined earlier, the probability of fatalities, pollution and property loss can be estimated using risk assessment and compared with target levels for each. Generally, the focus has been on fatalities and the reference value for annual death rate in society proposed by Flint(8) and Jordaan and Maes(9) is of the order of 10^{-4}. Paté-Cornell(10) suggests 10^{-3} to 10^{-4} as the upper bound of acceptability. The target structural annual failure probability, p_{FT}, based on fatality rate may therefore be taken as:

$$p_{FT} = \frac{1}{f(n_r)} K_s \cdot 10^{-4}$$

where K_s is a societal criterion factor which relates to the extent to which the activities associated with the structure is hazardous and voluntary[8] and $f(n_r)$ is a risk aversion function of the total number of people, n_r, at risk. Obviously if the concern is individual risk, $f(n_r)$ is equal to 1.0. However, Flint[8] proposed that $f(n_r)$ is equal to n_r and Allen[11] suggests that $f(n_r) = n_r^{½}$.

Reference 4 presents frequency-consequence diagrams from worldwide data collected for offshore and ship structures, considering all kinds of hazards including those from human error. It is inferred that the annual probability of total loss of fixed platforms worldwide is about $4x10^{-4}$. Similarly, the annual probability of a single loss of life is approximately 10^{-3} and a multiple fatality is approximately 10^{-4} to 10^{-5}.

2.2.3 Target level based on economic criteria

The single measure of risk which includes all consequences (loss of life or injury, business / environmental loss and societal loss / inconvenience) is an economic target. The cost of failure includes such items as repair / replacement of the facility, delay and disruption to business activities and pollution as well as fatalities (see Section 1). The issue of attributing a value to a life is obviously emotive. The value may be based on compensation obtained in courts or it may be an industry 'equivalent' such as that used by the UK Railway Group[12].

The Railway Group considers the number of equivalent fatalities (where ten major injuries or 200 minor injuries are equivalent to one fatality) and attributes a value to each prevented fatality when carrying out cost-benefit analysis of safety measures. The industry typically uses two values, one from the UK Department for Transport (DfT) and the other sometimes used by the industry to help guide decision-making. The DfT figure is based on a formula that calculates the cost of a road fatality and it currently stands at £1.30m. The industry figure, which currently stands at £3.64 million, is used if: the impact may produce an adverse response from society; or large numbers of people may be killed at one time; or when potential victims are particularly vulnerable; or the potential of the risk inspires dread.

Cost-benefit analysis is a useful tool when evaluating the introduction of new safety features or operational / organisational changes, and can provide support when making decisions regarding the relative safety of new versus existing facilities / structures.

2.3 Target Safety for Ageing Structures

Ideally, the same target safety level associated with individual fatalities and environmental damage should be applied to both existing / ageing and new facilities. However, any cost-benefit analysis of safety measures will need to reflect the remaining life of the facility. In addition, it is recognised that the expenditure required to reduce the failure probability by a factor of ten for an existing structure may be several times the cost of a new structure[4].

In practice, therefore, even the individual risk criteria (for both on-site workers and off-site members of the public) are usually relaxed for existing plant since upgrading to meet current safety levels is uneconomic. The only alternative may be to close the plant, increasing unemployment (with potentially greater risk for the unemployed than the occupational risk whilst working) and moving the risk to another location. For a variety of social-economic reasons, there is a strong argument that it is appropriate to consider lowering the target safety levels related to individual risks for older plant.

The following sections discuss how this matter has been (or is being) handled in various industrial sectors.

3 OFFSHORE OIL AND GAS

A number of approaches have been considered to set target levels for existing offshore structures. Bea[13] proposed cost analysis, which implies an acceptance of lower reliabilities for old systems compared to new. Iwan[14] proposed a target level based on separate considerations of the fatality rate and environmental damage – this also resulted in reliability levels which implied a slightly lower target for existing than for new offshore facilities.

The most widely used approach in the offshore industry, however, originates from an American Petroleum Institute (API) task group which was established in 1992 to produce guidelines for assessment of existing structures[15]. The philosophy behind the consequence-based approach developed by the task group has been incorporated into the API 'Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms' (API RP2A[16, 17]) and has formed the framework for the proposed ISO International Standard for 'Fixed Steel Offshore Structures' (ISO/CD 19902[18]).

The guidelines drafted by the task group recognised that some uncertainties which were present at design (e.g. material properties and precise equipment weights / layouts) were known for the fabricated / installed structure, whilst other anomalies (e.g. defects, damage and corrosion) may be introduced as the structure matured. They also, in essence, allowed a relaxation in the target reliabilities in cases where the economic and societal costs of upgrading a fixed facility were beyond the bounds of reasonable practicability.

The following sections chart the development of the philosophy behind acceptance criteria for the reassessment of structures already in the field, and show how the consequence-based approach (intended for reassessment purposes only) has now also been embraced for design purposes – a reversal of the traditional scenario of trying to adapt design methodologies for reassessment.

3.1 Development of the Assessment Process

An API task group was established in 1992 to develop guidelines for assessment of existing platforms. Concerns had been raised by the US Minerals Management Service regarding the adequacy of older structures, prompted also by the expiry of initial operating permits and by the occurrence of significant environmental events such as hurricane Juan (1985), the Loma Prieta earthquake (1989) and, in particular, hurricane Andrew (1992). The task group developed a supplement to API RP2A[16] for the assessment of fixed offshore platforms in US waters, addressing metocean, earthquake and ice loading.

The assessment process developed by the API task group was initially modelled on that derived by Iwan et al[19] for seismic safety assessment of offshore platforms. It adopted consequence-dependent assessment criteria and categorised platforms in terms of life safety and environmental impact. Platforms were considered 'manned' if they are continually occupied by persons living on them (i.e. living quarters in use) and a differentiation was made between those where there is sufficient time to safely evacuate all personnel prior to a design event and those which are considered to be 'non-evacuated'. The consequence-based criteria were initially intended specifically for assessment purposes.

The API assessment process also promoted two levels of analytical complexity: design level analysis and ultimate strength analysis. The assessment criteria are specified so that a platform which passes the design level analysis will also pass the ultimate strength check. It was proposed that each level of analysis was conducted using the current (20th Edition) API procedures and considered that outdated versions of RP2A were inappropriate.

The design level analysis proposed was as for new platforms but with less stringent environmental conditions (see Section 3.1.2).

For the ultimate strength analysis, inelastic static pushover analysis or design level analysis with all safety factors removed was suggested.

3.1.1 Loading and response terminology

Three key terms associated with acceptance criteria were identified and introduced by the task group:

- **Load Reduction Factor (LRF):** for an existing structure, the LRF was defined as the ratio of global load (e.g. base shear) causing a unity check or a design utilisation ratio of 1.0 to the 100-year global design load where the loads and member utilisation values are calculated according to the 20th Edition of RP2A. For platforms designed according to the 9th – 19th Editions of RP2A, the LRF approximates to the ratio of the original 100-year design load to the 20th Edition design load (since resistance equations had remained largely unchanged). By this argument, it was shown(15) that platforms designed according to the 9th – 19th Editions had design global loads that were 60 – 65% of the 20th Edition loads. Thus, if 19th Edition designed structures were 'acceptable', then using an LRF of 0.6 – 0.65 in conjunction with the then current code would provide a consistent level of safety.
- **Ultimate to Linear Ratio (ULR):** the ULR was defined as the ratio of the ultimate resistance load to that causing a unity check of 1.0 in the original design. The ULR thus reflects implicit conservatism in code design equations, the difference between the true material yield and nominal yield, reserve strength from system redundancy, etc. Given minimum factors of safety in API RP2A of 1.2 – 1.3, a ratio of mean to nominal yield of 1.2 for mild steel and typical system redundancies of between 1.0 (for a minimal non-redundant platform) to 1.3 (for a redundant multi-leg platform), the ULR was considered to be in the range of 1.6 – 1.8(20).
- **Reserve Strength Ratio (RSR):** the RSR was defined as the ratio of the ultimate strength load to the 20th Edition (100-year return period) design load. It is a measure of the platform reliability in a given environmental area.

Thus the RSR = ULR x LRF.

It is important to note that the above values for LRF, ULR and RSR were validated in a Joint Industry Project (see Section 3.1.3 and Reference 21) involving analysis of many different platforms in the Gulf of Mexico – less rigour was expended for other US waters.

3.1.2 Acceptance criteria for Gulf of Mexico

With two categories for environmental impact identified (significant and insignificant) and three categories for safety (manned, non-evacuated; manned, evacuated; and unmanned) there are potentially six exposure categories. However, in the Gulf of Mexico, industry practice is to evacuate for hurricanes (eliminating the manned, non-evacuated categories). Platforms considered as having significant potential environmental impact if collapse occurred were qualitatively considered in terms of the amount of oil spilled, the proximity to environmentally sensitive areas and the availability of containment equipment – no quantitative guidelines were specified.

LRF and RSR values for three exposure categories are presented by the API task group. These categories are:

- **Significant Environmental Impact Category**
  The LRF and RSR values are based on obtaining a similar level of integrity when designing to the 20th Edition as that for platforms designed to the 9th – 19th Editions (as discussed above). Reviewing failure and survival data from hurricane Andrew, the acceptable RSR was set at the conservative end of the range (i.e. 1.2).
- **Manned, Evacuated Category**
  For this category, the objective is to avoid extensive damage or failure in a sudden storm – in the Gulf of Mexico this includes hurricanes which dominate the risk. The acceptance criteria were based on two objectives:
  - The platform should have the same standard of safety as a new design, but with respect to only the sudden storm loads
  - The probability of platform failure for sudden storms should be around 10^{-4} per year. This was consistent with the target risk of seismic events offshore California.

With respect to the population of sudden storms, the LRF was set at 1.0 (i.e. 100-year return period sudden storm) with a ULR as before of 1.8, and hence an RSR also of 1.8. Relative to the full population of hurricanes in the Gulf of Mexico, the LRF was taken as 0.45 with an RSR of 0.8. Using probability distributions of load and resistance for a generic platform, the risk was found to be consistent with the target value of 10^{-4} per year.

- **Minimum Consequence Category**
  Although life and environmental safety are not issues for this category, there must be sufficient strength to ensure life safety during personnel visits under operational storm conditions. The question of 'how safe is safe?' becomes more subjective for such structures. The consensus of the API task group was that an LRF of 0.3, with a ULR of 1.6 (since such platforms are likely to have lower ultimate to linear ratios), was appropriate. This gives an RSR of 0.5.

The API task group also considered appropriate LRF and RSR values for platforms in other US offshore areas, but not the North Sea or any non-US locations. However, subsequent work in Europe(22) has investigated the relationship between probability of failure and RSR, and the sensitivity of the relationship to environmental conditions.

3.1.3 Calibration of target probabilities of failure

Hurricane Andrew passed through the Gulf of Mexico in 1992 with environmental conditions which affected thousands of platforms. A Joint Industry Project(21) was subsequently organised to study the effects of the hurricane and calibrate conclusions with the assessment procedures proposed by the API task group.

Simple reliability calculations were performed for generic platforms in the Gulf of Mexico (GoM) and the West Coast (WC) of the US. The conclusions from this work were that:

- New designs on the WC of the US barely meet a 10^{-4} per year probability of failure
- Recommended criteria for high consequence platforms on the WC lead to failure probabilities roughly twice those for new designs
- Manned platforms in the GoM lead to target failure probabilities of 10^{-4} per year relative to sudden hurricanes and winter storms.

3.2 Exposure Levels in the International Standard

As proposed by the API task group and incorporated into API 'Recommended Practice', the International Standard for 'Fixed Steel Offshore Structures' (ISO/CD 19902) categorises structures by various levels of exposure to determine criteria that are appropriate for the intended service. The categories have been expanded from those in API RP2A to form a 3x3 matrix of 'Life Safety' (S-1, S-2 and S-3) against 'Consequence of Failure' (C-1, C-2 and C-3):

S-1 = manned – nonevacuated

S-2 = manned – evacuated

S-3 = unmanned

C-1 = high consequence of failure

C-2 = medium consequence of failure

C-3 = low consequence of failure

A platform is categorised as S-1 unless either reliable forecasts and evacuation planning allows category S-2 status or visits are short and not undertaken when severe weather is predicted (allowing category S-3 status). High consequence of failure refers to platforms with high production rates and / or large processing capability, or those platforms that have potential for well flow of either oil or scour gas in the event of platform failure.

This leads to nine potential exposure categories. However, the level to be used (L1, L2 or L3) for structure categorisation is the more restrictive level for either life safety or consequence, resulting in three exposure levels as in Table 1.

Life Safety Category	Consequence of Failure Category		
	High consequence of failure	Medium consequence of failure	Low consequence of failure
Manned – nonevacuated	L1	L1	L1
Manned – evacuated	L1	L2	L2
Unmanned	L1	L2	L3

Table 1 Level of Consequence of Failure

3.3 Assessment Process in the International Standard

Clause 25 of the International Standard presents procedures for assessment of existing platforms. It recognises that some existing / ageing structures will not comply with current design criteria since design guidelines have altered over the years in response to increasing knowledge and understanding of the conditions under which offshore installations are operated. A structure that complies with the other clauses of ISO/CD 19902 is automatically considered 'fit for purpose'. However, for structures which do not comply with the other clauses of the International Standard, an assessment process is given in Clause 25 whereby the owner of the facility must demonstrate 'fitness for purpose' for its **specific site** conditions and operational requirements.

The International Standard states that, for existing structures, it is permissible to accept limited individual component 'failure' provided that both the reserve strength against overall system failure and deformations remain acceptable. The assessment process may involve detailed review, analysis, testing or calculation of those aspects of the design that do not comply with the other clauses of the International Standard. It is also stressed that prevention and mitigation measures to reduce the occurrence of (and consequences from) structural failure should be considered during all stages of the assessment process.

The assessment consists of six main elements as outlined below. It should be noted that, whilst the principles are presented in the International Standard, the equivalent criteria to those in Section 3.1 for API are not, and only appear in a Regional Annex. No definitive RSRs are presented (since it was considered inappropriate to dictate applicable values throughout the world), although values for design of new structures are discussed.

3.3.1 Assessment initiators

Initiators of the assessment process include exceedance of intended design life (life extension), damage or deterioration to a primary structural component or revised design criteria (changed exposure level, inadequate deck height, more onerous environmental or geotechnical criteria, etc).

3.3.2 Screening criteria

Screening criteria for assessment in ISO/CD 19902 may be based on the following methods:

- Assessment with explicit probabilities of failure. The method of computation is left to the discretion of the duty holder / owner but must be substantiated.
- Assessment adopting risk based reserve strength factors similar to the approach of the API task group.
- Assessment of similar platforms by comparison. Design level or nonlinear ultimate strength performance characteristics from an assessment of one platform may be used to infer 'fitness for purpose' of other similar platforms, provided the platform's framing, foundation support, service history, structural condition and payload levels are not significantly different.

- Assessment based on prior exposure. Prior storm exposure may be used, provided the platform has survived with no significant damage. The procedure would be to determine, either from measurements or calibrated hindcasts, the expected maximum base shear / overturning moment that the platform has been exposed to and then check if it exceeds, by an appropriate margin, the base shear / overturning moment required in the ultimate strength analysis check.

3.3.3 Platform condition assessment

Information from structural surveys (above and below water), or more accurate foundation data should be collected and utilised, where available. Many older platform foundation designs were based on soil boring information taken a considerable distance away from the installation site.

3.3.4 Action (load) assessment

The metocean data required for an assessment are the same as for design (except where more recent data have been acquired). However, the current deck height and an allowance for any future subsidence within the platform life should be determined.

3.3.5 Resistance assessment

The assessment should be based on its current condition (or future intended condition), accounting for any damage, repair, scour, modifications or other such factors. Structural component resistance is performed as for new structures.

As recommended by API, a two stage approach is suggested: an initial design level analysis followed, if necessary, by a nonlinear ultimate strength analysis.

Design Level Analysis

The procedure is identical to that adopted for new structures, with all partial load and resistance factors applied. If all components within the structure and foundation are assessed to have utilisations less than or equal to unity, the structure is considered ‘fit for purpose’ and no further analysis is required.

For fatigue evaluation, inspection history may be used to demonstrate future fatigue performance, giving consideration to both the inspection reliability and crack growth behaviour in lieu of analytical procedures used in original design.

For foundations, the use of increased pile capacities due to ageing effects are acceptable at the specific platform location. Individual pile foundation failure is acceptable if the total foundation system can be shown to have adequate reserve strength against failure.

Nonlinear Ultimate Strength Analysis

Local overstress and potential local damage is accepted, but total collapse or excessive / damaging deformations must be demonstrated not to occur.

The analysis should determine the best estimate of the structure’s system strength for the unfactored 100-year return period global environmental loading. The most common measure of system reserve strength is the RSR.

3.3.6 Prevention and mitigation

Prevention measures (structural strengthening, load reduction, etc) and mitigation measures (de-manning, hydrocarbon inventory reduction, etc) should be considered at all stages in the above process.

4 BUILDINGS (STEEL AND CONCRETE STRUCTURES)

The UK Institution of Structural Engineers has published a document[24] entitled 'Appraisal of Existing Structures' in which it is stressed that there is no absolute measure of adequate safety and even less of serviceability. Design and construction in accordance with current regulations and codes of practice provides a generally accepted level of safety which is a useful datum but, when assessing existing structures, 'engineering judgement should take precedence over compliance with the detailed clauses of codes of practice for structural design'.

The appraisal cycle presented in Reference 24 consists of a series of assessments of the strength and future serviceability of the structure. For each stage, information is collected and assessed and, if the result shows that the structure is adequate, the process can be terminated. If the result is inconclusive, more information can be collected and assessed more thoroughly. The three stages entail:

1. A preliminary, broad assessment of apparent physical condition, robustness and strength (including simple calculations where necessary). If these checks indicate a dangerous situation, some temporary safety measure may have to be taken pending further investigation.
2. A complete reassessment, including numerical checks on stability and integrity of the whole structure as well as the strength of individual components. A conventional design approach is usually employed for such checks.
3. A deeper analysis based on best knowledge of loads and material strengths that can be practically obtained from measurements, tests or other investigations. Such, more precise, knowledge 'may justify reduction of the (partial) safety factors used in the calculations'.

4.1 Reduction of Safety Factors During Reassessment

4.1.1 Principles

The European Commission initiated the preparation of a set of technical rules for the structural design of building and civil engineering works which currently serve as an alternative to those in force in member states but will ultimately replace them. EN 1990[41] describes the overall principles and requirements for safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with a partial factor method.

Any design code limit state safety format (and the implicit notional failure probability, p_c) applies to the broad class of structures covered by that design code. Because use is made of generic rather than structure-specific material and structural properties in code calibration, p_c does not necessarily give a good estimate of the failure probability of any particular structure designed to the rules. This is important, and particularly explains why design rules tend to be conservative for checking the safety of an existing structure. Equally, risk control in structural design is based on avoiding failure of elements (beams, columns, slabs, etc) rather than considering final consequences.

When considering the factors of safety (partial or global) to be used in the final reassessment stage described above, care needs to be taken to ensure that compatible factors are used throughout. It should be noted, for example, that the partial factor format used in the UK structural design codes (such as BS 8110[23] or BS 5950[47]) differs from that in Eurocodes, although the principles are similar.

As an illustration of potential reduction of partial safety factors, Reference 24 considers the UK design code formulations, with the basic equation:

$$\gamma_f \text{ x load effects} \leq \text{structural resistance} / \gamma_m$$

where γ_f is the partial factor for loads and γ_m is the partial factor for material strength, etc. In turn, γ_f is made up from three factors:

- γ_{fl} : load variation factor (i.e. allows for unfavourable load deviation)

- γ_{f2} : load combination and sensitivity factor (i.e. allows for probabilities of various load maximums acting together)
- γ_{f3} : structural performance factor (i.e. allow for effects such as unforeseen stress redistribution).

Load Variation Factor

For reassessment, it may be feasible to measure structural dimensions and densities and calculated dead loads accurately. In such circumstances, it is suggested that the design value of γ_{f1} = 1.15 could be reduced to 1.05.

For imposed load, due consideration needs to be given not only to the current situation (e.g. level and distribution of storage) but also to safeguards on possible future use. Generally the imposed load safety factor will remain unchanged for reassessment, although evaluation of liquid storage (using the maximum possible head) could for example justify a reduction of γ_{f1} from 1.35 to 1.05.

For wind loads, no reduction of γ_{f1} is suggested except possibly for reassessment of certain secondary elements which are not critical to overall structural failure or human safety.

Load Combination and Sensitivity Factor

As the considerations that govern this factor are not affected by the difference between design and reassessment, no reduction in γ_{f2} is justifiable.

Structural Performance Factor

The design value of γ_{f3} = 1.2 covers inaccuracies of construction, inaccuracies of analytical techniques and the severity of the consequence of collapse.

If measured dimensions (including eccentricities) are used in the reassessment and realistic load transfer mechanisms are identified, it is suggested that the following reduced values of γ_{f3} could be adopted:

- γ_{f3} = 1.05 for secondary elements, failures of which will not lead to progressive collapse
- γ_{f3} = 1.15 for primary members supporting other parts of the structure and for secondary members, failure of which might cause loss of life and / or substantial material damage.

Material Factors

The factor γ_m allows for the difference between the strength of the material actually in the structure and the strength of test specimens. It also allows for the accuracy of the formulae used for predicting behaviour. γ_m therefore varies for different modes of failure.

The test specimens will either have been made from similar, but not identical, material (as is the case for steel where one tensile test specimen is typically taken for every 40 tonnes rolled).

The reduction of γ_m that can be permitted depends on how well understood the failure mechanism is, and on the extent of visual warning of impending failure that can be expected. If the failure will be ductile and the member is visible (giving warning by excessive deflection), a significant reduction may be acceptable. If the member is hidden and / or failure will be brittle, little or no reduction should be allowed. For example, for a visible under-reinforced concrete beam, γ_m for bending could reasonably be reduced from 1.5 to 1.25. For a hidden over-reinforced beam, it is doubtful if any reduction should be contemplated (particularly if shear were the governing criterion).

4.1.2 Bayesian Updating of Partial Safety Factors

The validity of the above described updating procedure for partial safety factors can be explained on the basis of Bayesian updating. To illustrate this, a steel tensile bar is considered of which the yield strength distribution is *a priori* modelled to be normally distributed with a known standard deviation of σ_{fy} = 15 (all values are given in MPa). Based on experience, the mean value is modelled as a normally distributed variable with the mean value $\mu_{\mu_{fy}}$ = 345 and standard deviation $\sigma_{\mu_{fy}}$ = 10. In addition, experiments are

carried out leading to the following observations: $\mathbf{o} = (364, 338, 356, 366, 351)^{\mathrm{T}}$. Based on Bayes' theorem and the observation $\mathbf{o}$, the characteristic value (the 5% fractile) can be updated from $f_{yk} = 315$ MPa to $f_{yk}^{u} = 325$ MPa, as seen on the right hand side in the following figure.

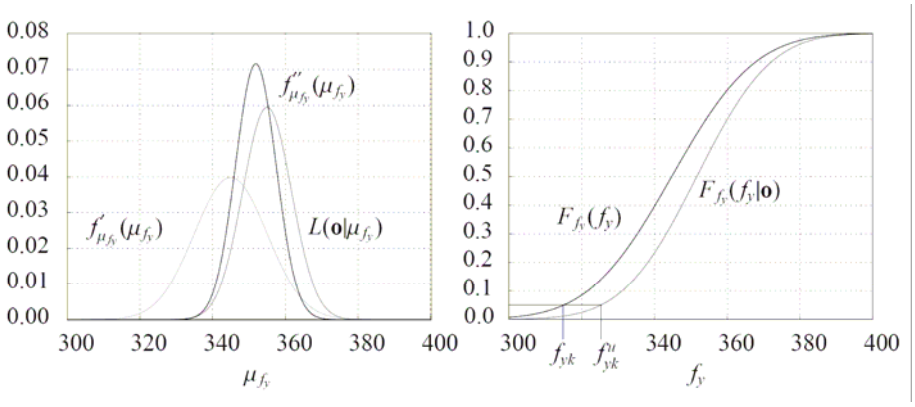

Figure 1 Prior, posterior and predictive distribution.

The left side shows the prior, likelihood and the posterior of the yield strength's mean and in the right hand side the predictive distribution of the yield strength is drawn.

In addition, to the updating of characteristic values, the partial safety factors can be updated because the coefficient of variation is often reduced if observations are made; in the example from $V_{yk} = 0.052$ to $V_{yk}^{u} = 0.046$. This means that by updating, not only is the characteristic value better predicted, but also variation may often be reduced, which leads to a higher reliability if the same partial safety factors are applied. On the other hand, safety factors may be updated as well to maintain the same reliability. With $a_{yk} = 0.8$, $\beta = 4$, $k_{yk} = 1.64$ and the classical definition of partial safety factors for normally distributed resistances the partial safety factors as: $\gamma_M = 1.10$ and $\gamma_M^{u} = 1.08$. Finally, the increase in the characteristic value and the decrease of the partial safety factor lead to a 4.8% reduction of the required design variable, e.g. cross-section.

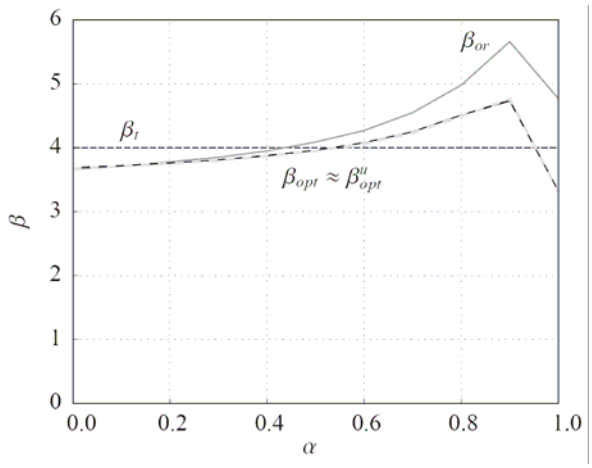

Figure 2 Code calibration using CodeCal03.

Figure 2 illustrates the results using the more refined reliability-based code calibration procedure which is implemented in the JCSS code calibration program CodeCal[62]. The figure shows the original distribution of the reliability index β_{or} for partial safety factors as applied in the current Eurocodes. The reliability index is shown for different values of α, the ratios of permanent load G to total load $G + Q$. Calibrating the partial safety factors using CodeCal, the distribution of β_{opt} is obtained. Especially for high fractions of permanent load (α close to one), it is seen that the distribution obtained is much closer to the target reliability

$\beta_t = 4.0$. Table 2 shows that, depending on the choice of approach, the partial safety factors may differ considerably. The table also shows that updating may reduce the safety factor of the resistance which is achieved by an increase of the load factors. In Figure 2 it is seen that the reliability obtained after updating and optimization is practically identical to the optimized level without optimization. For the calibration procedure, the relative frequencies w_i are assumed to be uniformly distributed, the permanent load is taken to be normally distributed with a coefficient of variation of $V_G = 0.10$, and the variable load is Gumbel distributed with $V_Q = 0.4$.

	original	optimized	updated & optimized
γ_R	1.10	1.03	1.02
γ_G	1.35	1.25	1.27
γ_Q	1.50	1.61	1.65

Table 2 Optimized partial safety factors using CodeCal.

4.2 Probability and Consequences of Failure

The traditional approach to ensuring structural safety through design code safety formats, adapted by reduced partial safety factors for reassessment as appropriate, has served the structural engineering profession very well (despite occasional failures). However, the actual probability of structural failure (in safety terms) remains largely unknown. It is not quantified by the 'safety indices' used for calibrating modern limit state design codes since these represent 'nominal' probabilities within a self-contained, structurally orientated, safety culture. This culture is largely concerned with structural failure as the undesirable outcome rather than with possible societal consequences of such failures.

However, increasingly, structures are viewed as part of wider infrastructure systems. This requires that their risk assessment is approached in a manner which is more compatible with that for petrochemical or nuclear facilities.

Risk assessments would be in additional to the usual design rules for components and to quality assurance programmes for documentation, construction and commissioning. The assessments could be carried out ahead of detailed system design, updated periodically as subsequent design / construction / operational phases proceed, and be used to assess the safety of the mature, perhaps deteriorated, facilities. This 'life cycle' model would inform inspection planning for life extension and the possible structural repairs / enhancements required could be compared in a rational manner to other options.

EN 1990 Eurocode[41], which is the head Eurocode for the European harmonised set of structural design standards and is 'material-independent', does provide some guidance for adopting a different level of reliability for structural safety or serviceability by considering:

- The cause and mode of failure
- The possible consequences of failure in terms of risk to life, injury, potential economic losses and the level of social inconvenience
- The expense and procedures necessary to reduce the risk of failure
- The different degrees of reliability required at national, regional or local level.

However, the Eurocode is primarily for design rather than reassessment and, for the majority of structures, use of the limit state approach in accordance with EN 1990 and associated Eurocodes 1-9 (together with appropriate quality control measures) is considered to ensure 'an appropriate degree of reliability' for **new** structures.

5 ONSHORE CONCRETE STRUCTURES (INCLUDING BRIDGES)

In addition to changes to codes and construction practices over the years, the assessment of existing concrete structures needs to address the issue of deterioration. At present, ageing concrete structures tend to be evaluated as if they were new with, possibly, a few refinements to reflect deterioration.

The assessment of deteriorated concrete structures is a relatively new area and most documents covering the topic are associated with bridges. There are three main reasons for this: firstly governments are generally the main owners of bridges and can insist on inspections / assessments as a statutory requirement; secondly the development of assessment codes is only economic if a large population of structures is affected; and lastly the use of de-icing salts (used on bridges in the Northern Hemisphere) is the main source of deterioration of concrete structures.

5.1 UK Bridge Reassessment

In the UK, the assessment of existing bridges has been a requirement for many years. It was recognised that assessment against more onerous current design codes rather than those to which the bridges were designed would lead to many failing to comply whilst still appearing to perform adequately in service. Because of this, the UK Highways Agency published its own assessment code BD 44/90[25] (updated in 1995 to BD 44/95[26]). Simple amendments were made to the design code (BS 5400: Part 4[40]) to produce this assessment code, but much of the conservative nature of BS 5400 remains in BD 44/95. The amendments consist of reductions to partial safety factors, principally in terms of materials, similar to those proposed by the UK Institution of Structural Engineers[24] for steel and concrete buildings as discussed in Section 4.

The UK Highways Agency's loading standard BD 21/93[27] used in conjunction with BD 44/95 introduces a condition factor (with a value of between 0 and 1) which is applied to the load-carrying capacity. This allows the assessing engineer to reduce the resistance (bending, shear, etc) to allow for any deficiencies noted in the inspection that cannot be allowed for in calculations. This condition factor is determined on the basis of engineering judgement and is a crude means of assessing the effects of deterioration.

Recognising that the corrosion of bridges due to chlorides (in de-icing salts) is the biggest problem facing UK bridges, the Highways Agency produced Advice Note BA 51/95[28] to provide guidance on the effects of corrosion on load-carrying capacity and safety. This was the first codified attempt at quantifying the structural effects of corrosion in the UK, and provides reduction factors that engineers can include in their calculations. Simple qualitative guidelines for assessing the structural effects of corrosion are also provided. The main quantitative recommendation is that bond should be reduced by 30% when cracking has occurred.

A 15-year bridge 'rehabilitation' programme for trunk road bridges in the UK was launched in 1987. A similar programme for local bridges was launched soon after. In addition, under an EU Directive for international transport, 40 tonne lorries were allowed on UK roads from 1999. These three activities have meant that a large number of UK bridges have been inspected and assessed in recent years. As the rehabilitation programme progressed it became increasingly apparent that there were fundamental limitations in the current assessment rules. These were leading to strengthening being proposed for bridges which had been carrying traffic with no apparent sign of distress for many years. It also led to situations where it was unclear what the actual level of safety in a bridge was, particularly where deterioration was present. In order to prioritise bridge maintenance funds effectively and to be able to forecast the future needs of the bridge stock rationally, it was considered necessary to revise the existing assessment rules. The Highways Agency now recommends a five level assessment procedure in BA 79/98[29] as shown overleaf:.

The procedure is a progressive one, starting at level 1 and progressing up to level 5 depending on the importance of the bridge. A level 1, assessment is likely to be fairly crude and based on existing assessment codes. If a bridge is adequate at level 1, then it is likely to be safe. However, if it does not pass a level 1 assessment then there is the option to progress to a more sophisticated level 2 assessment. This process can continue up to a level 5 assessment. At the higher levels of assessment, the emphasis is on a reliability approach such that inherent differences in robustness between bridges can be identified in order to achieve similar safety levels. However, little mention is made of deterioration and reliability analysis is not always appropriate to the assessment of deteriorated structures, as the effects of deterioration are not necessarily

random. Basing assessment on reduced cross-sections, and not recognising that some of the design code rules will not necessarily be appropriate to deteriorated structures, could lead to problems.

Level	Procedure
1	Assessment using simple analysis and codified requirements and methods
2	Assessment using more refined analysis
3	Assessment using better estimates (bridge specific design values of load and resistance, using probabilistic estimates where possible)
4	Assessment using bridge specific target reliability
5	Assessment using full-scale reliability analysis

5.2 Concrete Building Structures

BS 8110[23] Part 1 is the main code for the design of concrete building structures in the UK. Part 2 of the code is intended for special circumstances and contains a section on appraisal / testing (although this is intended primarily for the construction phase). BS 8110 is not an assessment code but, as there is no formal concrete building assessment code in the UK, it is commonly used for that task. The Institution of Structural Engineers has published an assessment report[24] on the appraisal of existing structures which can be used in conjunction with BS 8110 to improve the assessment procedure. A considerable amount of background information is provided in the report on materials, forms of construction, testing, and reduced safety factors which can be used without altering the overall reliability (by eliminating some unknown factors present at the design stage). Again, as in Section 5.1, the major omission in this approach is the treatment of the structural effects of deterioration.

5.3 Deterioration of Concrete

Comité Euro-International du Beton (CEB) Bulletin 243[30] provides a means of assessing the reductions in load-carrying capacity on the basis of:

- Condition Rating (CR)
- Level of inspection
- Extent of deterioration
- Extent of maintenance
- Required service life
- Method of determining resistance (load-carrying capacity)
- Redundancy in structure.

The Bulletin is somewhat contradictory in that it says the method is applicable only to those structures where no visible deterioration is observed. However, the elements used in determining the resistance reduction factor cover severe deterioration which is likely to be highly visible. The reduced resistance is given by:

$$R_{red} = \Phi R_d$$

where: R_{red} = reduced resistance (bending, shear, axial etc) of a deteriorated structure

Φ = resistance reduction factor based on the current condition of the structure

R_d = design resistance of the section being considered.

The concept of a Φ value is common in US and Canadian codes where it typically represents the uncertainty associated with each type of resistance. In Bulletin 243, it is derived from the following expression:

$$\Phi = B_R e^{-\alpha_R \beta_C V_R}$$

where: B_R = bias ratio between the true and mean resistance of the member

V_R = coefficient of variation relating to the reliability level of the test and inspection data

α_R = deterioration factor obtained from the CR

β_C = target value of the minimum acceptable safety level. Two levels are foreseen: 3.5 for normal expected service life and 2.5 for a limited period (i.e. until the next inspection).

The value of Φ can vary from around 0.5 for severely deteriorated structures with little redundancy that have not been inspected regularly or maintained properly, to more than 1.0 for redundant structures in good condition that have been inspected regularly and properly maintained.

CEB 243 provides a means of reducing the load-carrying capacity based on deterioration. It uses test and inspection data obtained from the structure. However, it does not recognise that some elements of deteriorated structures behave differently to new structures. For instance, the design code resistance formulae may not be appropriate for use with deteriorated structures. Such an approach also assumes that all load-carrying mechanisms will reduce by the same amount for the same observed level of deterioration. This has not been established in any tests.

5.4 Reassessment Approaches

All the above codes and recommendations are aimed at establishing the current state of a structure without any consideration given to the future performance. Little mention is made of deterioration mechanisms, and no attempt is made to tie in material deterioration to structural deterioration. In addition, no criteria are available to define minimum performance criteria. This leaves the assessing engineer with no choice but to make crude assumptions, which may or may not be conservative. As a result it is not possible to provide the structure's owner with a confident answer to the question: 'How long will my structure last?'.

There are three approaches to modifying codes to account for the effects of corrosion on concrete structures:

1. Leave the code unchanged, and reduce section areas. This is what the UK Highways Agency has proposed for bridge assessment. It has the problem that it does not recognise that the reduction in load-carrying capacity may not be directly related to the material deterioration.
2. Leave the code unchanged, and introduce a capacity reduction factor. This is the approach taken in BD 21/93 and in CEB Bulletin 243. Again, there is the problem that it does not recognise that the reduction in load-carrying is not directly related to the material deterioration. In the case of BD 21, it also requires some significant judgement from the engineer.
3. Modify the code resistance formulae to reflect the behaviour of deteriorated structures. This has been attempted, to an extent, by Rodriguez et al[31] for EuroCode 2[32]. Unfortunately, the results are based on limited laboratory tests where accelerated corrosion was used.

Option 3 is the preferred approach as it is more realistic and logical. However, it requires good quality data to be available to develop reliable resistance formulae. This route will not be fully achievable until sufficient data are available in the public domain (perhaps not for 10 to 20 years).

5.5 Bridge Management in Europe: BRIME

The 'Bridge Management in Europe (BRIME)' project, funded by the European Commission, aims to develop a management framework on the European road network that enables bridges to be maintained at minimum overall cost (taking into account all factors including the condition of the structure, load carrying capacity, rate of deterioration, effect on traffic, life of the repair and the residual life of the structure). The project has been carried out by the national transport research laboratories in Germany, France, Norway, Spain, Slovenia and the UK.

One of the deliverables from the project (reference D7, P97-2220) considers decision criteria for repair or replacement. The proposed method is based on a global cost analysis that reviews all the costs involved in designing, constructing, inspecting, maintaining, repairing, strengthening and demolishing a bridge, as well

as the road user costs associated with the service life of the bridge. The objective has been to develop a strategy that minimises the global cost whilst keeping the lifetime reliability of the structure above a minimum allowable value.

The work includes a detailed evaluation of two commercial bridge management systems (PONTIS[48] and DANBRO[49]) and two theoretical models (University of Colorado[50] and University of Lisbon[51]). From these evaluations, an extensive literature review and completed questionnaires on decision procedures, a proposed methodology has been developed that assists decision-making regarding possible alternative actions for a deteriorated bridge.

The global cost of each alternative action is evaluated through a series of factors. The selection of the most suitable repair / replacement alternative is based on the comparison of these costs. The method permits the choice among alternatives that depend on numerous factors that can be of a very different nature and are considered independent or, at least, semi-independent.

The method proposed is this work is structured in the following phases:

1. Identification of the factors
2. Evaluation of the factors
3. Comparison of alternatives and selection.

Identification of Factors

The identification of the factors to be taken into account for the comparison of repair / replacement alternatives is of great importance and establishes the degree of detail of the evaluation (i.e. a general study with a few highly aggregated factors or a detailed study with many specific and highly disintegrated factors). Table 3 presents a list of factors to observe in a generic bridge and these should be adapted to each specific case.

C_I	**Inspection costs**
C_M	**Maintenance costs**
C_R	**Repair costs**
C_{RA}	Structural assessment costs
C_{RR}	Structural repair costs
C_F	**Failure costs**
C_U	**Road user costs**
C_{UD}	Traffic delayed cost
C_{UR}	Traffic rerouted cost
C_{URT}	Time costs
C_{URO}	Vehicle operating costs
C_{URA}	Accident costs
V_S	**Salvage value**
C_O	**Other costs**

Table 3 General Factors for Repair / Replacement Evaluation

Evaluation of Factors

It is necessary to establish a common value unit that will serve as a reference to evaluate the factors. The economic cost has been selected in this methodology as the common unit.

The value of most factors tends to have an objective base but, in spite of this objectivity at the outset, difficulties can arise (e.g. the increase in the accident rate when the lane width is reduced by 15%).

In some instances, the value of specific factors is more subjective. This is particularly the case for certain social factors (e.g. the value of life, the destruction of the artistic or cultural patrimony or the social impact produced by restricted access).

Most factors listed in Table 3 are composed of a number of sub-factors. Consider, for example, failure costs (C_F) which include all costs resulting from structural collapse of a bridge or a situation in which such collapse is imminent and the bridge has to be closed to traffic. The cost associated with the structural failure can be obtained from the probability of failure and the various costs associated with collapse:

- **The cost collapse** can be divided into bridge replacement costs, loss of lives and equipment, and architectural, cultural and historical costs
- **The bridge replacement costs** include all the extra expense involved in having to replace a bridge that theoretically still has a few years of service. To estimate this, the situation of the immediate collapse and replacement of the bridge has to be compared with the situation of replacement in a few years. The replacement costs are essentially those for constructing a new bridge and for the traffic impairment during that period
- **The loss of lives and equipment costs** comprise the value of the lives of the people using the bridge (or what society is prepared to pay to save them) and the value of their vehicles supposing there is a sudden collapse of the bridge. These costs can be estimated from current traffic values and normal insurance values for vehicles and people
- **The architectural, cultural and historical costs** are a way of weighting bridges which are especially important from these points of view.

Other costs (C_O) identified in Table 3 which may be accrued with some alternatives and influence the selection process include:

- Reduction of gauges or dimensions of deck platforms (lanes, shoulders, etc)
- Influence of the proposed alternative in other traffic (pedestrian, bicycles, etc)
- Absence of alternative routes for light and / or heavy traffic
- If the bridge is used for public transport (buses, coaches, school transport, etc), absence of other alternative public transport to cover that route (for example, railway)
- Influence on other ways of transport (railway, high speed, etc) that can provoke traffic disruption, limitation of working hours, need of night working hours, etc
- Economic influence of the works in nearby localities: for example, the interruption of a road can affect shops, industries and the lives of the inhabitants of those localities crossed by that road
- Environmental impact of the works in the locality where they take place: noise, dust, contaminants, etc
- Loss of the historic, patrimonial, aesthetic, religious and traditional values of the bridge at all levels: national, provincial, local, etc
- Appearance of additional functioning expenses: personnel, boards, beacons, other signalling, etc
- Convenience of a given alternative from the point of view of the use of available equipment, stocked materials, similar actions in nearby places, etc.

Comparison of Alternatives and Selection

The selection of the preferred alternative(s) is based on the minimisation of the generalised total cost over the analysis period. Every repair or replacement decision is made according to the repair index (RI) of each alternative, where the RI indicates how much the proposed repair will cost compared with a no-action option (or any other baseline taken as a reference).

6 PETROCHEMICAL / PROCESS FACILITIES

Operators of ageing plants may not just need to consider future options for replacement / rejuvenation of the facilities at the end of their design life, but also the current impact that mature plant may have on securing new contracts. Shell, for example, has identified[52] that operators of ageing plant in the liquefied natural gas (LNG) industry (which is experiencing increasingly fierce competition and overcapacity) are now less likely to be able to secure long-term sales contracts such as those that dominated the first 30 years of the LNG market. Moreover, for the several LNG plants worldwide which are approaching the end of their design life (normally 20 years), rejuvenation may be necessary to reassure all parties that problems will not develop that might threaten their ability to fulfil commitments.

In the same publication[52], Shell believes that the situation for gas plants is similar to that for LNG. Whilst it is anticipated that the demand for natural gas and its products will increase dramatically in the next 10 years, 'only the suppliers with the best run plants will reap the rewards'. For those with ageing plant, upgrading is considered to be crucial.

6.1 Rejuvenation of LNG Plant

Reference 52 stresses the need for careful planning to ensure that opportunities for capacity increase or life extension are not missed. The rejuvenation procedure outlined in the document consists of:

- A high level review carried out by experts in operations, design and maintenance. The data gathering and evaluation considers the potential of the plant with respect to current limits on equipment and process parameters
- A more in-depth analysis, including benchmarking by a multidisciplinary team of the plant's operations against those of similar plant operating elsewhere, to reveal performance gaps
- A detailed equipment performance assessment by a team covering all relevant disciplines (rotating equipment, civil engineering, LNG technology, gas treatment, etc) aimed at the introduction of best-in-class techniques to enhance equipment and plant capacity. Risk reliability management and structured shutdown management may be implemented to increase plant availability.

A long-term view is important to ensure that the plant complies with any future trends or regulations. For example, an item of plant that is likely to have problems with spare parts in a few years should be upgraded and solutions that pre-empt increasingly stringent environmental legislation should be implemented.

The LNG plant on the coast of Brunei Darussalam, started-up in 1972, is an example of what can be achieved through rejuvenation. During rejuvenation in 1992, the 20-year old pneumatic instrumentation was replaced by a distributed control system, insulation was improved, critical electrical equipment was upgraded and new storage tanks were installed. The plant now operates at 140% of its original design capacity. A second rejuvenation programme is now being planned which will take place around 2010 and extend the life of the plant to 60 years.

6.2 Gas Turbine Life Monitoring

An industrial gas turbine can experience a wide variety of stop / start cyclic conditions. To maintain hot component integrity with ill-defined duty cycles, manufacturers often use the concept of equivalent operating hours. Each type of cycle is conservatively allocated an equivalent number of full power operating hours which are subtracted from the design life to estimate remaining life.

As an alternative to the equivalent operating hours approach, AEA Technology has developed a series of software modules which provide an estimate of hot component life usage based on actual duty cycles rather than the assumed worst case[53]. Using measurements of pressure and temperature along the gas path, a continuous assessment of component damage can be derived.

The system has been installed on two gas turbines at a gas compressor station, allowing maintenance to be carried out as needed rather than to a rigid schedule (thus reducing maintenance costs). Cost / benefit

decisions concerned with 'sweating the asset' can also be made, linking the operation of the turbine to periods of high or low gas price.

6.3 Structural Fatigue Failure in Process Equipment

A paper by Melchers[54] presents a hypothetical example of a safety assessment and the potential failure of a large pressure vessel.

The example involves the possibility of release of highly flammable material (liquefied petroleum gas: LPG) due to structural failure from fatigue or fracture. In Australia, LPG is a relatively popular automotive fuel and is often stored above ground on service station sites. In such instances, safety systems consist of crash barriers, multiple independent pressure relief valves, leak detection and emergency shutdown systems based on vacuum-traced pipework, electrical and mechanical shut-offs and well-signed shutdown activation points.

Conventional ASME[55] or equivalent design standards are often used for pressure vessels, pipelines, pumps and shutdown systems. However, these do not fully represent the safety of the system as a whole. The Australian Standard (AS 1596-1997), governing the site location and safety requirements to control exposure of the public to LPG hazards, was revised during 1991-1994 and requires a probabilistic risk analysis.

Consideration of the probability of fatigue failure is part of the analysis process. During typical facility operation, the tank contents are drawn down as fuel is sold. In addition, there is a daily fluctuation in temperature as the tank surface heats during the day and cools at night.

It is common in fatigue analysis to assume the pre-existence of a small defect or crack. In LPG storage, even a very small release will cause a very large temperature change around the crack due to the phase change from liquid to gas. This phase change will lead to freezing of LPG around the opening and, if ignition occurs, a jet fire will form. This scenario is unlikely to go unnoticed. Hence, whilst the fracture mechanics estimate of gradual fatigue failure may be of the order of 10^{-4} per year, the chance of the scenario going unnoticed and not being acted upon may be assumed to be less than 1 in 1000 and the overall probability of progressive fatigue failure can therefore be considered to be negligible.

It is also possible for an unusually high-level load cycle to occur and to lead to instantaneous tank failure. The potential consequences of failure for an LPG facility can be serious – these include boiling liquid expanding vapour explosions (BLEVEs), unconfined vapour cloud explosions (UVCEs), flash fires and jet fires. Risk assessments indicated that, fortunately, the probabilities of occurrence associated with BLEVE and UVCE consequences were negligible (less than 10^{-9} per year). The probabilities of occurrence of various forms of jet fire have been used to determine safe clearance distance based on maximum flame length.

The above example again illustrates that the assessment of plants / structures requires a realistic and wider approach to risk than is implicit in design code rules and quality assurance processes for documentation and construction. This is particularly important where the plants / structures form part of a wider infrastructure or industrial facility.

7 NUCLEAR INSTALLATIONS

The membership of the Nuclear Energy Agency (NEA) comprises 28 countries in Europe, North America and the Asia-Pacific region. The Committee on the Safety of Nuclear Installations (CSNI) of the NEA has set up a Working Group on Integrity of Components and Structures, with the mandate to advise on management of ageing plant. The programme of work encompasses the integrity of metal components / structures, ageing concrete structures and seismic behaviour.

In Europe, the average age of the 146 nuclear power plants, which generate one-third of the electricity supply in EU countries, is 23 years. Typically, nuclear power plants are licensed to operate for 30 to 40 years. Individual EU member countries may have different positions on whether they will seek license extensions. Essentially, there are two choices: to seek approval to continue operating or to close the plant down. While the assessment of which option to pursue may be partially a commercial decision by the nuclear power plant operator, public opinion and the resulting political environment are also involved. Extending the life of existing plant is not on the political landscape in several EU countries and governments have recently taken decisions to close plant early.

The effect of physical ageing such as corrosion in nuclear power plant may be degradation, which results in the reduction or loss of the ability of components, systems and structures to function within the expected design criteria. Typically, the ageing process may also reduce the reliability of components / systems / structures and may affect the safety of the facility unless preventative and corrective measures have been implemented. These measures are introduced through a programme of ageing management. However, in some cases, physical and non-physical ageing effects become so prevalent that it is impractical to correct them individually and complete refurbishment of the reactor facility is necessary. Equally, during the lifetime of a reactor, technical advances occur and components become obsolete (with associated difficulties in obtaining spares), also resulting in replacement of an entire system.

7.1 Technical Competency

One feature of life extension assessment highlighted at an International Atomic Energy Agency (IAEA) Scientific Forum in 2002[33] is the gradual loss of technical knowledge as the ageing nuclear workforces, both in the industry and in the regulatory authority, retire. This is becoming a growing concern, with nuclear physics disappearing from university curricula around the world, and could undermine the life extension process for facilities if solutions are not found.

7.2 Seismic Re-evaluation

Reassessment of the safety of existing nuclear power plants for a specified seismic hazard may be carried out if:

- No seismic hazard was considered in the original design
- Relevant codes and regulations have been revised
- The seismic hazard for the site has been reassessed as more onerous
- There is a need to assess the capacity of the plant for severe accident conditions and behaviour beyond the design basis.

In 1998, the CSNI produced a status report[34] on seismic re-evaluation which collated the approach and views of member countries. In the majority of countries, evaluation is carried out typically when plant is between 10 and 20 years old and then at 10 year intervals thereafter (often as part of a much wider ranging safety review). In over half the countries, there were reported to be no prescribed regulatory guidelines for seismic re-evaluation and the overall approach adopted (deterministic, probabilistic, margin methods and experience data) varied substantially.

In all cases, the input for re-evaluation was reported to be site specific (to avoid excessive conservatism) and the as-built situation was taken into account. However, input motion levels (seismic excitation), seismic categorisation, analysis methods and assessment criteria applied in re-evaluation were reported to be generally similar to those specified for design. Thus, there is a significant and consistent attempt to base the re-evaluation of plants, many of which were not originally designed for seismic loading, on the standards applied to new plant. Due to this reassessment philosophy, the resulting modifications required could be extensive: strengthening of buildings, walls, anchorages and equipment / pipework supports; and the removal of seismic interactions.

There was a high level of consistency among the member countries in the definition of the hazard probability level (generally set at 10^{-4}).

The degree of reliability incorporated into seismic re-evaluations was considered to be difficult to summarise. Responses from member countries generally referred to the reliability implied by the input or assessment levels. This probably reflects the fact that design conservatism is conventionally used as a surrogate for reliability, and the relationship between the two is difficult to quantify.

7.3 Probabilistic Targets

Probabilistic safety considerations are included in the IAEA International Safety Standards[35]. It is specified that 'a safety analysis of the plant design shall be conducted in which methods of both deterministic and probabilistic analysis shall be applied'. The objectives of both analyses are further specified and more detailed guidance is given in the supporting Safety Guide[36] on 'Safety Assessment and Verification'.

In terms of probabilistic targets, the Safety Guide refers to INSAG-3[37] and INSAG-12[38], and states that 'probabilistic targets can be set at a safety function or a safety system level. These are useful to check that the level of redundancy and diversity provided is adequate'. INSAG-3 and INSAG-12 differentiate between targets for existing and future plant as follows:

Core damage frequency: INSAG-3 has proposed the following objectives for core damage frequency:

- 10^{-4} per reactor year for existing plant
- 10^{-5} per reactor year for future plant

This is the most common measure of risk in nuclear power plant and, in many countries, these values have been adopted as probabilistic safety criteria (PSC).

Large release of radioactive material: PSC have also been proposed by INSAG-3 for a large radioactive release (which would have severe implications for society and would require implementation of off-site emergency arrangements):

- 10^{-5} per reactor year for existing plant
- 10^{-6} per reactor year for future plant

It is noted in the Safety Guide that, instead of these PSC, INSAG-12 states that 'another objective for future plant is the practical elimination of accident sequences that could lead to large early radioactive release, whereas severe accidents that could imply late containment failure would be considered in the design process'.

Health effects to members of the public: INSAG has no guidance on targets for such health effects. In some countries, the target for individual fatality risk is taken to be 10^{-6} per reactor year for members of the public.

7.4 Ageing of Nuclear Power Plant Concrete Structures

The performance of nuclear power plant concrete structures is reported to have been very good[39]. However, there have been several isolated instances that, if not remedied, could have challenged the capacity of the concrete containment and other safety-related concrete structures to meet future functional and performance

requirements. In general, many of these reported instances of degradation occurred early in the life of the structures and were primarily attributed to construction / design deficiencies, improper material selection or environmental effects.

Examples of some specific problems that have occurred due to age-related degradation include corrosion of steel reinforcement in water intake structures, corrosion of post-tensioning tendon wires, leaching of tendon gallery concrete, low prestressing forces, leakage of corrosion inhibitor from tendon sheaths into concrete, freeze / thaw damage, corrosion of containment metallic liners, and cracking of non-metallic liners.

New structures can be designed for improved durability based on operating experience (e.g. use of high performance concrete materials that exhibit improved durability by minimising the transport of fluids through the concrete). However, for existing structures, apart from the addition of barrier materials and sealants to accessible surfaces to prevent ingress of hostile environments, the most prudent approach to maintaining adequate structural margins is through a management programme that involves application of in-service inspection and maintenance strategies.

8 AIRCRAFT

The Joint Aviation Authorities (JAA) represent the civil aviation regulatory authorities of a number of European States who have agreed to cooperate in developing and implementing common safety standards and procedures. The UK, France and Germany make the largest contributions to the JAA. A Working Group (the European Ageing Aircraft Working Group: EAAWG) has been established by the JAA to specifically consider the continued airworthiness of ageing aircraft.

8.1 Regulatory View from Europe

A paper[42 and 43] prepared by the EAAWG provides recommendations and guidance for the safe operation of older aeroplanes throughout their remaining life. It covers all aircraft in commercial air transport, considering both design and maintenance aspects. The structural integrity of aeroplanes is of particular concern since factors such as fatigue cracking and corrosion are flight cycle and time dependent.

Service experience[42] has demonstrated that there is need to have periodically updated knowledge concerning the structural integrity of aeroplanes in a fleet, particularly as they become older. This knowledge of older aeroplane types can best be assessed on the basis of real time operational experience and the use of modern analytical techniques and testing. The guidance prepared by EAAWG presents inspection and evaluation programmes intended to ensure continuing structural integrity assessment and the incorporation of the results into maintenance programmes.

Initial focus has been on large transport aircraft (passenger or cargo) that are in revenue service, and eleven aircraft types were considered. For each type, a Design Service Goal (DSG) has been established. The DSG is the ‘period of time in flight cycles and / or flight hours envisaged at design and / or at certification during which the principal structure will be reasonably free from significant cracking and widespread fatigue damage is not expected to occur’.

Many civil transport aircraft were originally believed to be able to meet continuing structural airworthiness requirements for an 'indefinite' period. In practice, whilst this has not always proven to be the case, the structural life can be extended by maintaining an effective inspection and corrective maintenance programme. The EAAWG has established the necessary attributes of such programmes and the following five key structural issues should be addressed for each ageing aircraft type:

- Review all structurally related Service Bulletins and determine which require mandatory terminating action or the enforcement of special repetitive inspections
- Review and update the Supplemental Structural Inspection Programme for effectiveness
- Develop guidelines to assess the damage tolerance of existing structural repairs, which may have been designed without using damage tolerance criteria. Damage tolerance methodology needs to be applied to future repairs
- Review existing corrosion prevention programmes and develop a baseline Corrosion Prevention and Control Programme to maintain corrosion to an acceptable level
- Evaluate individual aircraft design regarding the susceptibility to widespread fatigue damage and develop a programme for corrective action.

The five ageing aircraft key issues are technically valid for all aircraft types, whether new designs or in-service aircraft. As the certification basis advances, less work is needed to show that the issues are covered. For aircraft certified to date, no certification basis has completely covered all the issues.

The Structural Inspection and Corrosion Prevention / Control Programmes are to be initiated no later than the time that the fleet leader (highest cycle / longest service aeroplane) reaches half the DSG. Equally, appropriate repair assessment arrangements and inspection details must be mandatorily incorporated into a Repair Assessment Document not later than at three-quarters of the DSG.

The Service Bulletin reviews include any aircraft structural components that require overly frequent repeat inspection (or which are difficult to inspect) and which will need to be modified or replaced. Selection of Service Bulletins for mandatory action is based on the reliability of inspection, the frequency of occurrence of failure and the associated adjacent structural damage if failure does occur.

The Supplemental Structural Inspection Programmes (SSIPs) are based on a thorough technical review of the damage tolerance characteristics of the aircraft structure using the latest techniques and allowing for changes in operational usage. They lead to revised or new inspection requirements, primarily for structural cracking. Fundamentals of SSIP development are considered(42) to be:

- Analyses made in respect to the continuing assessment of structural integrity should be based on supporting evidence, including test and service data. This supporting evidence should include consideration of the operating loading spectra, structural distributions and material characteristics. An appropriate allowance should be made for the scatter in life to crack initiation and rate of crack propagation in establishing the inspection threshold, inspection frequency and, where appropriate, retirement life. In certain instances, an inspection threshold may be based solely on a statistical assessment of fleet experience, provided that it can be shown that equal confidence can be placed in such an approach.
- It is essential to identify the structural parts and components that contribute significantly to carrying flight, ground, pressure or control loads, and whose failure could affect the structural integrity necessary for the continued safe operation of the aircraft. The damage tolerance or safe life characteristics of these parts and components must be established or confirmed.
- It is important to ensure that the extent of structure to be evaluated, the type of damage considered (fatigue, corrosion, in-service and production damage) and the inspection and / or modification criteria follow the damage tolerance principles of the ‘current’ standards.
- An effective method of evaluating the structural condition of older aircraft is inspection of selected details for individual aircraft with intensive use of non-destructive techniques, involving partial or complete dismantling (‘tear down’) of available structure.
- The effect on inspection of repairs and modifications should be considered. In addition, it may be necessary to consider the effect of repairs and operator approved modifications on individual aircraft. The operator has the responsibility for ensuring notification and consideration of any such aspects.
- An SSIP document should be produced and should be checked periodically against current service experience. Any unexpected defect occurring should be assessed as part of the continuing assessment of structural integrity to determine the need for revision of the document.

8.2 Fatigue Reliability

Fretting fatigue is one of the main mechanisms for the formation of cracks in riveted lap joint assemblies in ageing aircraft(44). Inspections have emphasised the importance of a time-dependent reliability estimation of this phenomenon. Probabilistic methods can compute the influence of the fundamental random variables to capture the major aspects of the fretting fatigue failure process, and form a rational basis for life prediction. Such methods are capable of simulating loadings and structural scenarios which are beyond the range of typical laboratory conditions traditionally used in the development of design data.

Analysis of the fatigue and fracture process for the purpose of making decisions relative to design and inspection rules is complicated by the significant uncertainties in the loading environment, material properties and inspection performance. Probabilistic and statistical methods can be used(45) to manage these uncertainties, providing estimates of reliability as a basic measure of structural performance and informing a maintenance programme of periodic inspection and repair.

8.3 Prediction of Corrosion Damage

To develop a comprehensive inspection and maintenance programme that takes advantage of life extension technologies, effective prediction of localised corrosion damage is essential.

Deterministic methods of corrosion prediction have traditionally been used for special cases in which the chemical environments, metallurgy, stresses and geometry are well understood and operate within relatively narrow limits. Due to the variability inherent in aircraft environments and structural configurations, deterministic expressions of corrosion rate and morphology that account for all of the important variables do not exist. Thus, it is necessary to introduce a probabilistic methodology for analysis of localised corrosion.

In addition to the variability associated with the corrosion processes, the challenges in dealing with natural environments must also be considered. Measurable atmospheric parameters such as temperature, dew point, annual rainfall and pollutant gas concentration may have an impact on the corrosion rate of simple materials. However, some of these parameters are difficult to measure accurately and success at correlation between the parameters and corrosion rates has been limited.

An alternative approach is the use of a database of corrosion rate and topography developed at each locale of interest, but this approach is difficult to extend to a new location and has limitations.

A combined probabilistic and database framework has been proposed for the management of the US Air Force fleet[46]. A probabilistic framework is used for the corrosion rate whereas a database is used for the characterisation of the corrosivity of air force bases.

All approaches aim to more accurately predict the effects of corrosion scenarios applicable to the specific operational environment, and hence provide a more accurate structural life assessment.

9 SUMMARY AND CONCLUSIONS

As identified in the previous sections, various technologies / industrial sectors have established specific safety / risk / reliability targets. Often the targets are fixed within a model which states that below a limit-line the risk is 'tolerable', between two limit-lines the risk should be minimised and above a limit-line the risk in 'intolerable' – this is normally called the ALARP (as low as reasonably practicable) principle as illustrated in Figure 3. However, in some industries the model is simpler, with a single limit-line below which the risk is tolerable and above which it is intolerable.

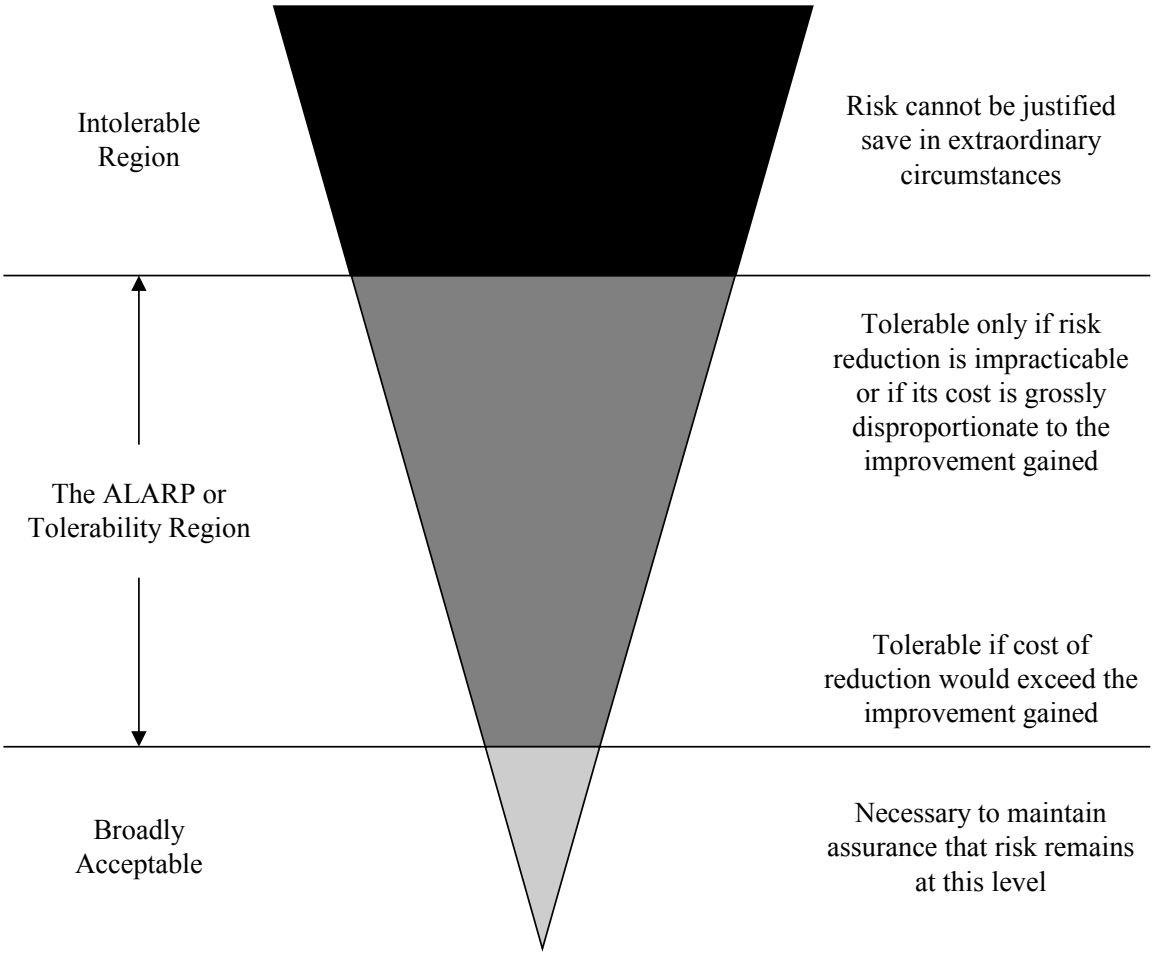

Figure 3 The ALARP Principle

Some risk models are two dimensional. These are based on the concept that the greater the consequence of an event, the lower should be the frequency of that event occurring. Such models may be in matrix form, characterised by classes of consequences and classes of probabilities. Alternatively, they may be formalised as an explicit cumulative probability distribution function versus the consequences.

Large differences in safety targets exist between technologies and between countries. A paper[56] entitled 'How safe is safe enough' considers this diversity and the need to harmonise targets. Table 4, overleaf, from the paper presents typical risk-based goals in various industries. The advances from harmonisation are considered to be a more effective distribution of resources to provide balanced risk across the global marketplace. Some technologies are required to spend greater and greater resources on progressively minimising potential risk, whilst others can operate at risk levels which are order of magnitude higher.

Technology	Considered Case	Reference
Marine Structures	Failure probability for the different accident classes: 10^{-3} – 10^{-6}	DNV 1992[57]
Aviation, Aeroplanes	Catastrophic failure: smaller than 10^{-9} per flight hour	JAR – 25[58]
Nuclear Power Plants (USA) (Canada)	Large scale core melt: less than 1 in 10,000 per year of reactor operation; individual risk from NPP 0.1% of the sum of cancer fatality risk from all other causes. Early fatality (public): 1 x 10^{-5} per site-year Large early release: 1 x 10^{-5} per site-year	Safety Goals USNRC 1998[59] AECB Canada Zeng 1998
Space Vehicles	Catastrophic consequence for Crew Transfer Vehicles (CTVs): smaller than 1 in 500 CTV missions	Preyssl 1996[60]
Process Industry (Netherlands)	Consequence > 10 fatalities: frequency must be smaller than 10^{-5} Consequence > 100 fatalities: frequency must be smaller than 10^{-7} Consequence > 1000 fatalities: frequency must be smaller than 10^{-9} Individual risk < 10^{-6} per plant year	Jones 1997 HSE Books 1997[61]
Electrical / Electronic Safety Systems (with embedded software)	Average probability of failure per demand: 10^{-1} to 10^{-5} for different Safety Levels 1 to 4 and low demand mode of operation	IEC 61508 1998

Table 4 Typical Risk-based Goals in Various Industries[56]

In the unlikely event that harmonisation was ever achieved, it would be a very difficult and long-term task involving political as well as technical dimensions.

In terms of existing facilities / structures, there is a general recognition across all industrial sectors that the reassessment process is different from the design process. As a minimum, the known conditions and specific functional requirements of existing facilities / structures need to be taken into account (with design 'uncertainty factors' removed where site-specific parameters are available from as-built information and inspections).

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leaded by

Christian Bucher (Bauhaus-University Weimar, Germany)

christian.bucher@uni-weimar.de

PART III

Case Studies

Table of Contents

EXECUTIVE SUMMARY

1 RELIABILITY ASSESSMENT OF PRIMARY SCHOOL

Cracking and other defects of a recently completed school building evoked serious speculations concerning the safety of school children. Reliability assessment of the school proved, that the load-bearing structure fulfils the requirements of Czech standards for the ultimate limit states, however it does not satisfy the requirements for the serviceability limit states. Furthermore the combination of different structural materials without an appropriate detailing was not properly considered in the design. The bad quality control of the process of execution also enhanced the unfavourable effects.

1.1 Introduction

The presented example of the reliability assessment of primary school indicates particular conditions of a structure and procedure of the assessment. It is necessary for the existing structure to assure an adequate level of structural safety and to predict its future durability. Various procedures are given in ISO 13822 [1]. Defects of existing structures are, as a rule, dependent on a number of different uncertainties. These might include various environmental conditions, workmanship and human error, way of previous exploitation of the structure including possible adverse time-dependent action effects, deterioration of materials and prediction of future events.

Table 1.1 indicates the proportions of various origins of structural failures chosen from basic activities during execution and service-life of structures, as it is given in [7]. The second line of the table indicates relations between these activities and two main causes: gross errors (about 80 %) due to the human activity and environmental effects (about 20 %), which are not directly dependent on the human activity.

Table 1.1: Proportions of the causes of structural failures.

Origin	Design	Execution	Use	Other
	20%	50%	15%	15%
Causes	Gross errors due to the human activity		Environmental effects	
	80%		20%	

The environmental influences include both random and hazard (accidental) situations, e.g. due to impact, explosion, fire and extreme climatic actions. Natural randomness causes only a small portion of the failures (about 10 %). Obviously, the development of more precise procedures based on the methods of structural reliability enabling to improve traditional approach of current standards for structural verification or their assessment has only a limited significance. Advanced engineering design methods including procedures for the assessment of structures and also evaluation of the possibilities for their intended future exploitation should therefore attempt to consider the actual causes of structural failures and the real state of existing structure, based on the methods of risk engineering. The risk assessment of a system (e.g. a structure in specific design conditions) attempts to cover all possible events that might lead to unfavourable effects related to the considered system. Regarding information given in Table 2 these events are caused mainly by gross errors in human activity and by accidental actions.

The reliability assessment of a structure is often a very complex task. Obtained results and their objectivity depend on many conditions, including the requirements of the owner (his expectation from the assessment) and experience of the appraising engineer. An advanced reliability assessment of a structure should also include risk analysis of the system formed by a structure located in the particular type of environment taking into account public perception of possible adverse effects. The presented study case of a public school illustrates the above mentioned findings.

1.2 Observed defects of a new public school

An example of the reliability assessment described here concerns a new building of the public school, which was recently completed in a north-western suburb of Prague to replace a three years old school that completely burn up during ordinary repair of the roofing. The new school consists of three separated buildings: the main four-storey building, gymnastic and dining hall. Shortly after its opening the first defects were observed in all these buildings. Dissatisfied local public started the long-term process of several reliability assessments and successive repairs of three separated parts of school building.

Various faults listed in Table 1.1 were detected in the new public school:

- insufficient design, e.g. inadequate simplifications of models during the structural analysis, ill-considered serviceability requirements, insufficient design documentation without appropriate detailing;
- errors during execution, e.g. an incorrect change of the floor layers, various changes of construction materials, heavier composition of terrace layers, poor quality of workmanship due to the lack of supervision;
- insufficient control during execution and consequently tendency of the responsible state owner to overlook the defects.

Many defects occurred during the execution, as the quality of workmanship was not sufficiently controlled and regularly documented. The supervision of the erection process was very poor, the supervisor of the school owner did not perform his job well and at last the site diary was lost. Many problems were caused by malfunction of heating, by faults in insulation and plumbing. The composition of the floor layers of gymnastic hall was changed during execution and the steam-proof insulation was omitted, although under the floor was directly located the school kitchen. Thus, the floor made of parquets was soon distorted due to the humidity of the school kitchen and consequently the gymnastic hall had to be closed, as the school children could be injured.

Visible deflections were soon observed and many cracks appeared in the reinforced concrete slabs of the dining hall due to the insufficient area of reinforcement specified in the design and also due to the heavier composition of the terrace layers over the slabs. Detail structural analysis proved that both the ultimate and serviceability limit states were not fulfilled. Consequently, it was necessary to close the dinning hall, to change the composition of the terrace layers and to add new supporting columns.

Another defect was the gradual damage of the protective layers against the atmospheric corrosion of the steel load-bearing structure of the separated gymnastic building, which started to flake off in a short time. Appearance of the rusting steel roof structure was publicly criticised.

New serious damages were observed one year after the school completion. Cracking in the partition walls and cladding components of the main building accompanied by malfunction of the doors and other defects were detected, as shown in Figures 2 to 4. Moreover the plasters of the ceilings were falling down and cracks were found throughout the main building. Parents became soon afraid about the safety of their school children and dissatisfied local citizens insisted on the quick refurbishment of the building. Finally, under the public pressure the owner of the school asked for the reliability assessment of the structure and for recommendations of possible measures. Thus, the public perception of observed defects was important for the initiation of the reliability assessment and for improvement or elimination of the series of defects, similar as in other cases described in [7].

1.3 Load bearing structure

The load-bearing structure of the main school building of four storeys consists of reinforced concrete slabs supported directly on columns within span distances of 6 m × 7,8 m. The slabs and columns have in some cases quite irregular shapes. The structure is founded on reinforced concrete foots, the non load-bearing walls of the basement are placed on the wall footings. The thickness of the reinforced concrete slabs is 0,20 m, the slabs are made of the concrete class C 20/25 and reinforcement of steel S 400. The partition masonry walls are made of solid clay masonry units of 0,15 m thickness.

The roof of the main school building is flat. The reinforced concrete load-bearing structure is linked in the part of roof with several steel skylights, there are also some ventilation devices on the roof. The central part of the roof is loaded by a penthouse for a boiler-room and for a lift machinery room. The walls of the penthouse are 0,45 m thick, made of solid clay masonry units. Two heavy boilers, a water cleaner and other devices are installed in the boiler-room, as it is shown in Figure 1.

1.4 Performance deficiencies

The non-structural walls of the main school building are impaired by many cracks of the width from 0,1 mm to 3 mm. The cracks damaged the tiling of the walls and floors in most parts of the school building, the net of cracks is clearly visible in the floor tiles of staircases. Moreover, big areas of plasters fell down from the walls and ceilings in the various parts of school building, as illustrated in Figures 2 and 3.

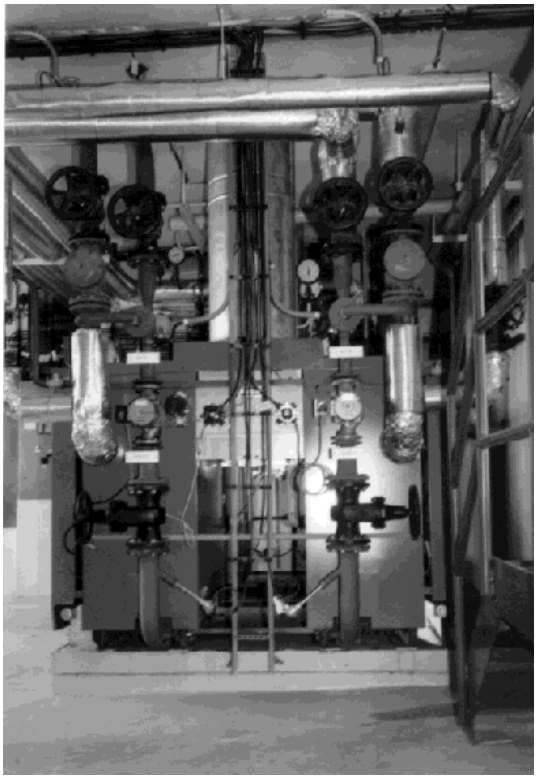

Figure 1.1: Boiler-room

Figure 1.2: Plasters of the staircase

The falling pieces of plasters from the ceilings invoked criticism of the children parents. The excessive deformations of the slabs caused malfunctioning of many doors. The installed crack indicators proved, that most of the cracks were still active (see Figure 4).

The greatest deflections and cracks of slabs were found out on the two adjacent reinforced concrete slabs of the roof, which were loaded by the heavy penthouse for boilers and lift machinery [1]. The penthouse itself was also damaged by cracks of a significant width.

Figure 1.3: Plasters of the classroom ceilings.

Figure 1.4: Cracks in partition walls.

1.5 Assessment of structure

The main aim of the reliability assessment of the new school building was to decide, if the load-bearing structure was reliable enough and if the school children were not endangered by the current state of the school building.

It was necessary to make an overall inspection of the school building and to detect the position and width of the main cracks. Some of the crack indicators, which had been set up in the previous year, showed that various cracks were still active. The project documentation was compared with the actual building. It was evident, that some columns had different shape and that the location of partition walls was rather inconsistent with the project. The partition walls were not built regularly and this caused different deflection of slabs in each floor. Thus, various partition walls were damaged by the system of cracks. Many cracks were also visible in the basement.

The load-bearing capacity of the slabs determined on the basis of the project documentation were compared with the internal forces and strengths obtained using the software Feat. The thickness of the concrete cover of reinforcement was not specified in the design and some parts of slabs had no auxiliary reinforcement, which could produce the shrinkage effects. Further, the designed area of reinforcement was in some cases insufficient regarding to the internal bending moment effects. This could easily result in the redistribution of moments leading to the amplification of slab deflections.

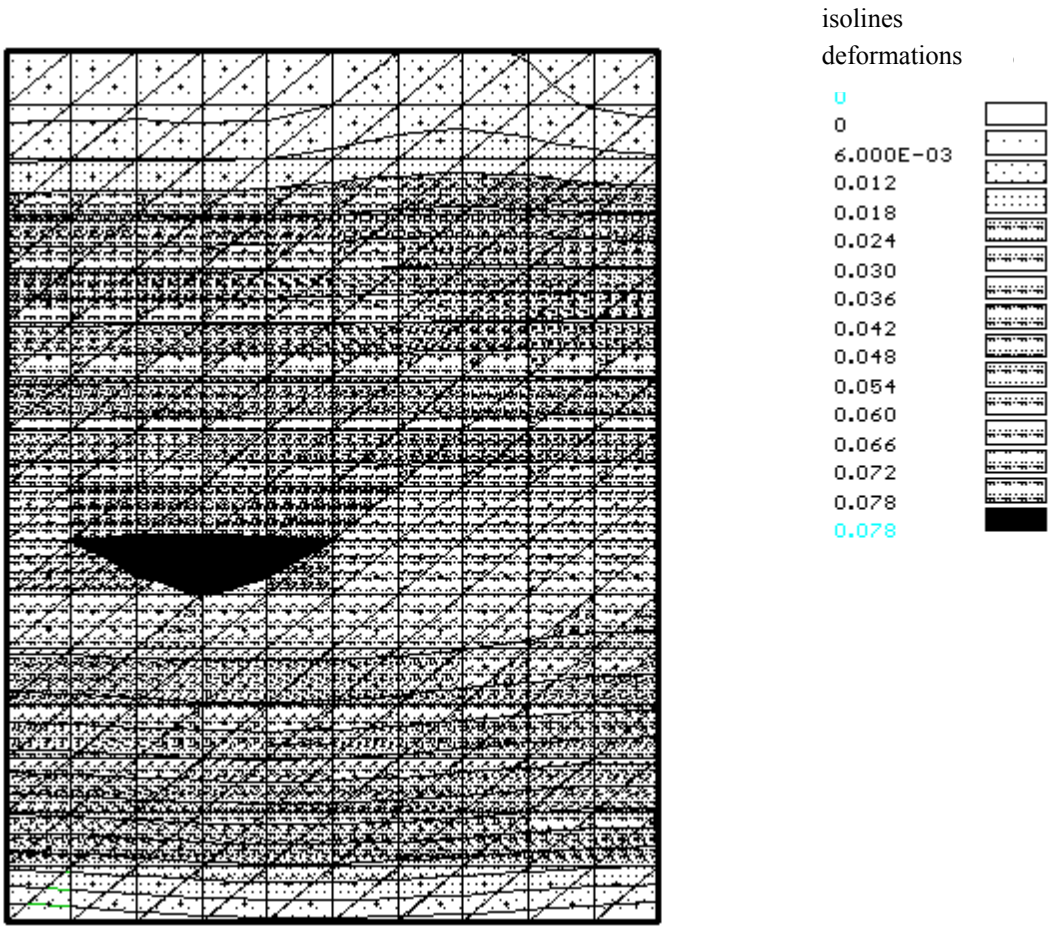

Fig. 5 Deflection of the roof slab.

The calculated short-time deflections of selected slabs are from 0,005 m to 0,035 m. The long-term deflections of the slabs considering creep effects are up to 0,08 m, as it is shown in Figure 5. The obtained results further indicate, that the deflections of the slabs including creep effects are too large and in some cases they may significantly influence the overall cracking of the structure. It is shown, that the resistance of slabs is sufficient, however the load-bearing system is not rigid enough. The non-regularly located masonry partition walls cause different deflections of the reinforced concrete slabs in adjacent floors above. The dilatations of partition walls from the load bearing structure were not considered in the design.

Verification of the main school building proves that the load-bearing structure fulfils the requirements for the ultimate limit states, however the reinforced concrete slabs are not sufficiently rigid. The main reason of the cracking in partition walls is the excessive deflection of reinforced concrete slabs, which does not fulfil the

requirements for the serviceability limit states of the Czech standards ČSN 73 1201 [2] and ČSN 73 1204 [3].

On the basis of appraisal of the poor state of new school building the school owner asked for the state financial support. The damaged plasters were removed, the cracks repaired and new plasters were made during one year.

It was obvious that most of the defects would not appear, if the adequate care had been devoted to the appropriate detailing in project, to the consideration of requirements for the serviceability limit states and to the requirements regarding the quality assurance during the execution.

1.5.1 Conclusions

Two main sources of structural damages are identified from the detail assessment of the school building:

- errors and inadequate quality in design documentation
- insufficient inspection and bad quality assurance of the construction processes.

The load-bearing structure fulfils the requirements for the ultimate limit states, however the reinforced concrete slabs are not sufficiently rigid.

The serviceability failure of the main school building is primarily caused by the lack of consideration of deflections due to loads, creep effects and shrinkage in design. Requirement for the minimum area of reinforcement needed for prevention of cracks is not in many cases satisfied.

The reliability assessment of the new school building shows that the structural defects are mainly caused by human errors (faults in structural analysis, in project documentation, in poor execution).

In spite the selected structural parts of school building have been fortified, plasters repaired, the new aspects of unfavourable behaviour of school building appears up to now. Regular inspections and maintenance of school building is needed.

1.6 References

[1] ČSN 73 0038, Navrhování a posuzování stavebních konstrukcí při přestavbách (Design and assessment of building structures subjected to reconstruction), 1986.

[2] ČSN 73 1201, Navrhování betonových konstrukcí (Design of concrete structures), 1986.

[3] ČSN 73 1204, Navrhování betonových deskových konstrukcí působících ve dvou směrech (Design of concrete two-way slabs), 1986.

[4] EN 1990 Basis of structural design, CEN, 2002.

[5] ISO 2394 General principles on reliability for structures, 1996.

[6] ISO 13822 Bases for design of structures – Assessment of existing structures, 2001.

[7] Holický M,, Prospects for Design Criteria Based on Acceptable Risks, 1st Czech/Slovac symposium Theoretical and Experimental Research in Structural Engineering, 2000.

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2 CONDITION MONITORING AND REMAINING LIFE ASSESSMENT APPROACH AT THE EDP GROUP

2.1 Introduction

This document deals with the condition monitoring and remaining life assessment concept and aims at presenting the technical background of this approach and its application to thermal power plants of the CPPE (the power generation company of the EDP Group). The use of condition monitoring and lifetime assessment in order to reduce forced outage risks is also mentioned, including some practical examples.

2.2 Technical Background

2.2.1 The Concept of Life of Industrial Plants

The life of industrial plants can be generally considered under different perspectives. Thus, it is usually referred to as useful life, which is the operating period of the plant as estimated in the design and set out in the call for tender documents. This concept of useful life is intimately associated to the concept of economical life, which is the payback time of the investment performed for the industrial plant. For fossil-fuelled thermal power plants, the useful life is generally considered to be in the range of 25-30 years, and takes into account the fact that after this period the equipment will become obsolete due to technical and/or economical reasons. In some countries, it is usual to consider the regulatory life, which is the period of time between two subsequent licences for operating the industrial plant, as granted by a legal authority. This period is usually set out by the legal authorities through the granting of the exploitation licence, following evidence from the side of the industrial plant owner, that it has fully capability to adhere to the safety and environmental rules in force during the extension period required.

For equipment operating at high temperature, where creep phenomena are active, it is also usual to define the technical life of materials as the period of 100,000 service hours at maximum allowable temperature. When the industrial plant follows a load cycling operation regime, it should be included in that figure the equivalent operating hours, which comprise the start-ups beside the normal operating hours.

2.2.2 Basic Reasons to Perform a Remaining Life Assessment

Until the beginning of 1970's, the costs of raw materials and manpower were relatively low and the fuel price allowed an electricity production at competitive prices. In addition, the lack of awareness of the population concerning the environmental impact produced by the operation of conventional thermal power plants enabled an easy selection of erection sites for new power plants.

The oil crisis of 1973 has produced a definitive change in the *status quo* that has dominated the post-2nd World War period. The raise on the primary energy sources were immediately reflected on the costs of raw materials and the manpower followed this evolution. As a consequence of this situation, the power plant equipment costs have raised significantly, as well as the generation costs, due to the higher fuel prices, which in turn had a very strong impact on the kWh price.

Power generation companies had to deal with this new situation rapidly, in order to keep its competitiveness. Regarding the generation costs, it was of utmost importance to optimise the performance of power plant units, in order to decrease the specific fuel consumption and consequently, to decrease the overall fuel consumption. With respect to the decision for the erection of new thermal power plants, which would replace the ones where the useful life was almost expired, a dilemma had emerged, making available two alternatives: from one side, to build new power plants with more efficient thermodynamic cycles but also involving higher costs or, on the other side, to defer investment costs and try to keep in service the old units, with less efficient thermodynamic cycles and with a high uncertainty regarding how further these units could be kept into operation without suffering a significant failure with serious consequences. To make the situation even more difficult to the power generation companies, a new subject brought up, which was related to the envi-

ronmental impact caused by thermal power plants. This was mainly due to the increasingly awareness of the public opinion, that started to demonstrate its opposition against the erection of new thermal power plants in the vicinities of urban areas.

The recognition of the environmental impact caused by the normal operation of thermal power plants, forced the power generation companies to seek for new technologies that would reduce this effect to acceptable levels. However, the costs incurred for the installation of this type of systems is an exponential function of the efficiency, which requires a very careful definition of the objectives to be met.

In summary, it can be mentioned that technical, economical and environmental aspects have forced the thermal power plant operators to adopt life extension programmes on plants near the end of its useful life, mainly due to the following reasons:

- high investment costs involved in the erection of new thermal power plants;
- new plant erection sites very hard to find and thus very costly (e.g. compensations);
- the eventual need for conversions;
- the installation of systems to reduce the environmental impact (e.g., NO_x, SO_x);
- the increase of the safety conditions;
- the increase of the reliability and consequently the availability;
- operational flexibility (e.g., load cycling).

2.2.3 Adoption of the Remaining Life Assessment Approach at the EDP Group

Remaining life assessment of fossil-fuelled thermal power plant components operating at high pressures and temperatures became an important topic at EDP Group in the late 1980's, when it was introduced for the first time.

In fact, the first maintenance action that involved the application of a remaining life assessment methodology started with a cold reheat line of a steam generator of one of the CPPE's conventional thermal power plants.

This was the beginning of the implementation of a systematic maintenance approach based on this methodology to a set of power plant components considered as relevant to meet the criteria of availability, reliability and safety.

However, it should be mentioned that this approach was already present at the EDP Group before the 1980's, although in a latent way, and has influenced the solutions adopted in the different phases of the equipment life, namely:

- Conception;
- Design;
- Manufacture;
- Erection / Assembling;
- Commissioning;
- Operation / Maintenance;
- Decommissioning / Dismantling.

In practical terms, this latent concept was evident from systematic maintenance actions taken mainly on components which had a direct impact on the availability of the units.

2.3 Using Condition Monitoring and Lifetime Assessment to Reduce Forced Outage Risks

Power plant components operating at high temperatures and pressures are subjected to damage mechanisms (i.e., creep and fatigue) that limit its useful life and can ultimately cause its unexpected failure.

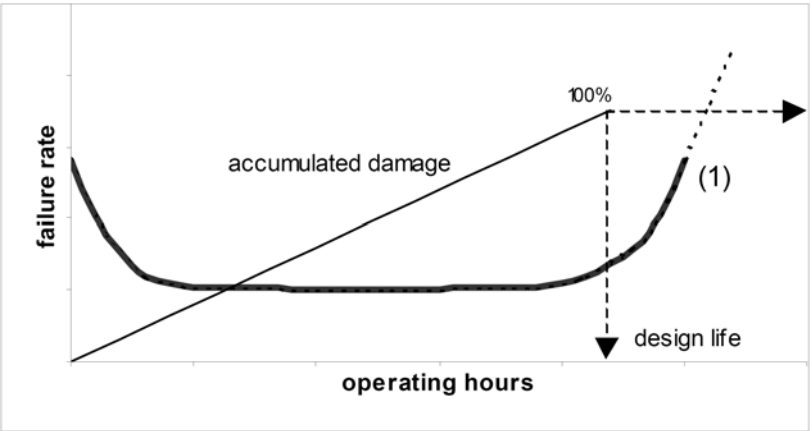
For this reason these components are designed (materials are selected and dimensions calculated) for a limited service life, usually from 100,000 to 150,000 hours, or for a limited number of start-ups.

In operating conditions components are then subject to two main types of deterioration:

- ”normal” wear;
- damage due to life limiting mechanisms.

The difference between these types of damage is more evident for turbines than for boilers, owing to the component characteristics and operating condition particularities.

Damage resulting from life limiting mechanisms undergoes an accumulation process during service life of component and should be significant, as regards maintenance, only in the “wear-out” stage of plant.



(1) Expected failure rate of plant components

2.3.1 “Normal” Wear

This type of damage originates the need for regular maintenance and it is usually detected and identified through condition monitoring techniques and performance tests, complemented with inspections carried out during maintenance work.

Plant critical components are not expected to be subject to "normal" wear in a large extent as it is expected to affect “parts” that can be refurbished or replaced by spares during overhauls. "Normal" wear is influenced by operating conditions, like start / stop cycling for instance.

In turbines the influence of operating conditions in “wear” is usually related to EOH (equivalent operating hours), determined by expressions like the following one:

$$t_a = t_{eff} + n_s \times t_s$$

where:

t_a = equivalent operating hours

t_{eff}= effective operating hours

n_s= number of starts in the period

t_s= operating hours that produce an amount of “wear” equivalent to the “wear” resulting from 1 start-up (t_s= 50 to 20 hours, depending on component).

Usually overhauls are carried out each time t_a reaches a determined number of hours, recommended by manufacturers and insurers, after last overhaul. The criterion used to compute the influence of start-up frequency on overhaul frequency is presented in the following table.

According to this criterion operating time between major overhauls is reduced by 46% when start-up frequency changes from a weekly to a daily start-up, and is reduced by 14% when start-up frequency changes from a monthly to a weekly start-up.

Operating time between minor overhauls is reduced by 59% when start-up frequency changes from a weekly to a daily start-up, and is reduced by 24% when start-up frequency changes from a monthly to a weekly start-up.

Start –up frequency	Operating time between overhauls (t_{eff})	
	Minor overhaul	Major overhaul
	t_a = 25,000 hrs t_s = 50 hrs	t_a = 75,000 hrs t_s = 25 hrs
Daily (1 start-up for 20 operating hours)	7,140 hrs	33,330 hrs
Weekly (1 start-up for 120 operating hours)	17,650 hrs	62,070 hrs
Monthly (1 start-up for 675 operating hours)	23,275 hrs	72,320 hrs

- *Minor overhaul: inspection of steam valves, bearings, safety and control devices, alignment check, visual inspection of LP last-stage blades.*
- *Major overhaul includes opening of turbine casings.*

2.3.2 Life Limiting Damage

Creep, fatigue, corrosion and erosion are the prevailing life limiting mechanisms of power plant components. Creep is time-dependent change in dimensions of a material under constant stresses, leading to failure after a determined time of exposure. For components used in power plants, this damage mechanism is related to continuous operation at high temperature.

Fatigue is originated when the material is under fluctuating stresses and promotes crack initiation and fracture after a determined number of strain / stress cycles. In power plants fatigue is related to transient operation (start–up /shutdown and load changes) and affects mainly heavy-walled components, where temperature differentials across the thickness can be regarded as relevant to promote cracking initiation and propagation.

Corrosion and erosion can accelerate both mechanisms. For this reason components are designed for a limited service life, usually for 100,000 to 150,000 hours, or a limited number of starts. Next, we will present some conceptual aspects that support life assessment program implemented in CPPE power plants.

Creep damage accumulation in service can be calculated using the Robinson's linear fraction rule:

$$E_c = \sum \frac{t_i}{t_{ri}}$$

where: t_i = time at temperature T_i

t_{ri} = time to failure at temperature T_i

Fatigue damage is determined using Miner rule:

$$E_f = \sum \frac{n_c}{N_c} + \frac{n_w}{N_w} + \frac{n_h}{N_h}$$

where: n= actual number of starts (subscripts c, w, h apply for cold, warm and hot)

N= number of starts to crack initiation

Total damage resulting from the combination of creep and fatigue is determined by the sum of creep damage and fatigue damage:

$$E_{total} = E_c + E_f \leq D$$

D represents de critical damage value. This value is ≤ 1, depending on component material and creep-fatigue interaction, i.e. the influence on each type of damage promoted by the presence of the other damage mechanism.

When E_{total} reaches the critical value D, the lifetime of the component is exhausted and failure is to be expected.

Component life expenditure (L_{exp}) is obtained from:

$$L_{exp} = \frac{E_{total}}{D} x100\%$$

Remaining Life is determined in remaining operating hours (t_{rem}) plus remaining start-ups (n_{rem}), as follows:

$$D\text{-}E = \sum \frac{t_{rem}}{t_{ri}} + \sum \frac{n_{crem}}{N_c} + \frac{n_{wrem}}{N_w} + \frac{n_{hrem}}{N_h}$$

Remaining Life depends on future service conditions, as shown in the following example.

Next table presents total damage and Life Expenditure calculated for a component with the design and service conditions indicated.

Design conditions	t_r = 200,000 hrs	N_c+N_w= 2,000 start-ups
Service condition	75,000 operating hours	110 (cold + warm start-ups) 1 monthly start-up pattern
Damage Present	E_{total} = 0.43	L_{exp} = 57% (1)

(1) *Assuming D = 0.75*

Remaining Life, after 75,000 service hours, for two different future operating patterns (weekly and monthly start-up patterns), will be as follows:

Start-up frequency	t_{rem}	n_{cr}+n_{wr}
Weekly (1 start-up for 120 operating hours)	34,910 hours	291 start-ups
Monthly (1 start-up for 675 operating hours)	56,580 hours	84 start-ups

For this particular component a change in future operating conditions from a monthly to a weekly start-up pattern will reduce remaining life from 56,580 hours to 34,910 hours, after 75,000 service hours, which represents a 38% decrease in remaining service hours.

2.3.3 Condition Monitoring and Lifetime Assessment

Based on calculations, the previous examples demonstrate the impact of operating conditions on component damage.

Taking into account such influence and the present service constraints and trends (most plants are running at operating conditions more severe than the design ones and also in excess of their original design lives), the implementation of RCM (Risk Centred Maintenance) or RBM (Risk Based Maintenance), requires the support of condition monitoring and remaining life assessment

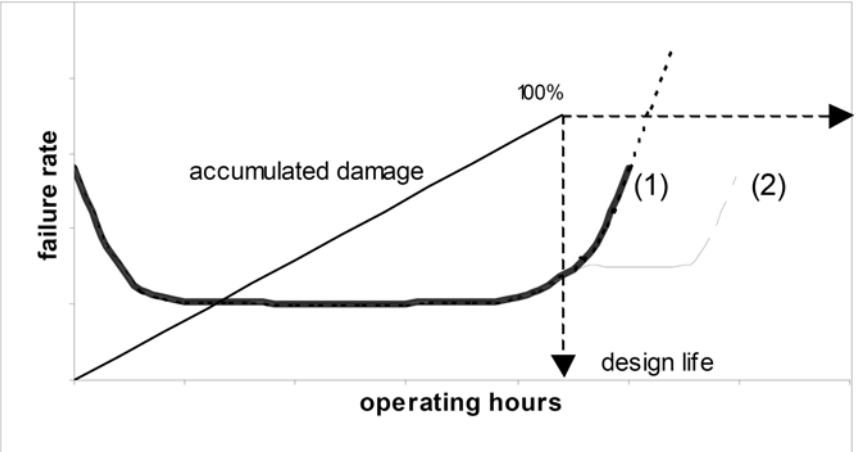
Condition monitoring is an important tool to detect and identify damage, evaluate actual condition of components and to predict damage progress and failure. It provides useful information to support maintenance decisions to reduce unavailability of plant and O&M costs, to support investment decisions and fulfil safety requirements.

Condition monitoring techniques for "normal" wear damage evaluation involve monitoring of operating parameters, performance and process variables and execution of performance tests, complemented with inspections and NDT (Non Destructive Examination), carried out during outages for maintenance work.

However, as plant approaches the end of design life or when operating conditions depart from design ones, life limiting damage can be the prevailing type of damage to take into account for future maintenance and investment decisions, and for risk-based decisions.

In this situation a methodology for remaining life assessment must be implemented to assess present damage condition of components and predict evolution, enabling:

- to identify failure modes and failure probability of critical components of plant, that can influence availability expectations and safety requirements,
- to support maintenance and investment decisions, by proper definition of maintenance intervals and type of repair/ replacement required or imposition of operating conditions to extend life of component and reduce risk of failure, for different operating scenarios.



(2) Failure rate of plant components with implementation of Remaining Life Assessment Methodology

Methods for lifetime assessment include two parts:

- a theoretical calculation (i.e., quantitative evaluation), which enables the determination of critical points taking into account anticipated failure modes;
- a qualitative evaluation based on non-destructive inspection, for validation of theoretical results and to extend the assessment to points not included in calculations.

Typical assessment procedures are based in a 3-stage approach:

Stage 1 – involves recalculation of original design life taking into account operational history and design dimensions. Critical points are determined in calculations.

Stage 2 – generally involves access to the component for condition monitoring performed in situ. Inspections and testing workscope and procedures are formulated taking into account results from calculation, potential damage mechanisms and failure modes, results of previous inspections and failure sand maintenance records.

Usually testing comprises conventional NDT techniques (MPI, UT, Dimensional Check, Visual Inspection) together with metallurgical inspection by replication techniques. A more refined calculation will be performed using actual dimensions and information from inspection results.

Stage 3 – requires the removal of material samples for laboratory tests and determination of actual materials property data and microstructural degradation characterisation. Advanced assessment techniques are required on this stage. It is not considered to be a routine and should be carried out only when results from Stages 1 and 2 have a level of uncertainty which is unacceptable in relation with strategic requirements.

2.3.4 Remaining Life Assessment at CPPE plants

Nowadays, remaining life assessment is part of routine maintenance work at all CPPE power plants.

Turbine remaining life assessment has been conducted in collaboration with the EDPP-EM (an engineering and maintenance company from EDP Group) and manufacturers.

Boiler and piping life assessment has been carried out in collaboration with independent specialist contractors.

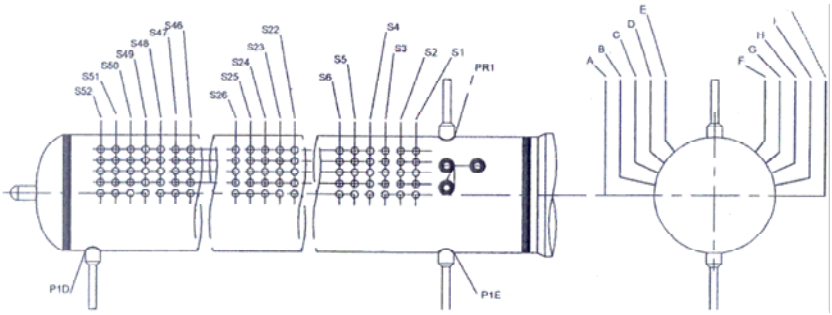
2.3.5 Practical Examples

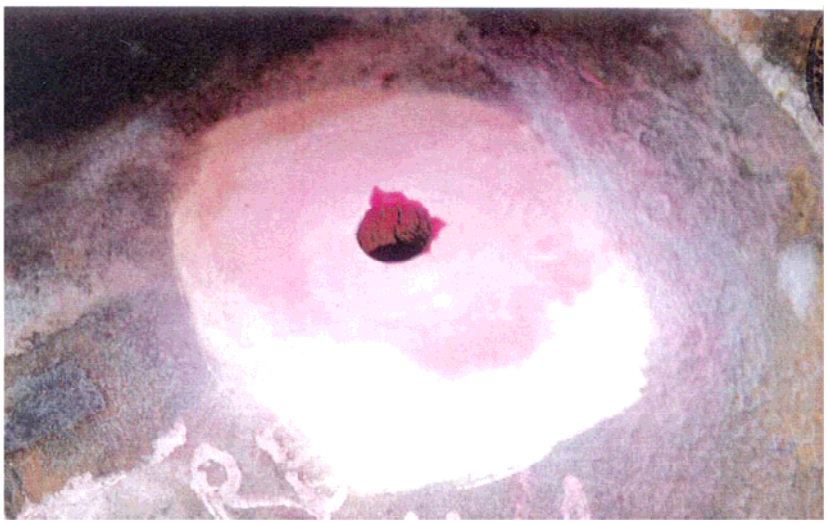
Critical components for lifetime assessment usually include drums, high temperature headers and tubing, in boilers, high-energy piping and special pieces like Y forgings, and rotors, casings and steam chests, in turbines. These components are subject to calculation and evaluation by non-destructive inspection at critical points determined by calculation or by experience, as previously mentioned.

Based on recorded history of one particular component or similar components operating in similar conditions, the experience is a very valuable factor to determine critical points for lifetime assessment.

Some examples of particular damage situations, whose detection was based rather on experience than on calculation, are presented.

<u>Example 1 – Reheater outlet header: cracking at vent opening (inner wall)</u>





Material : ASME SA 335 P22 (723.9 mm O.D. x 41.3 mm M.W.)

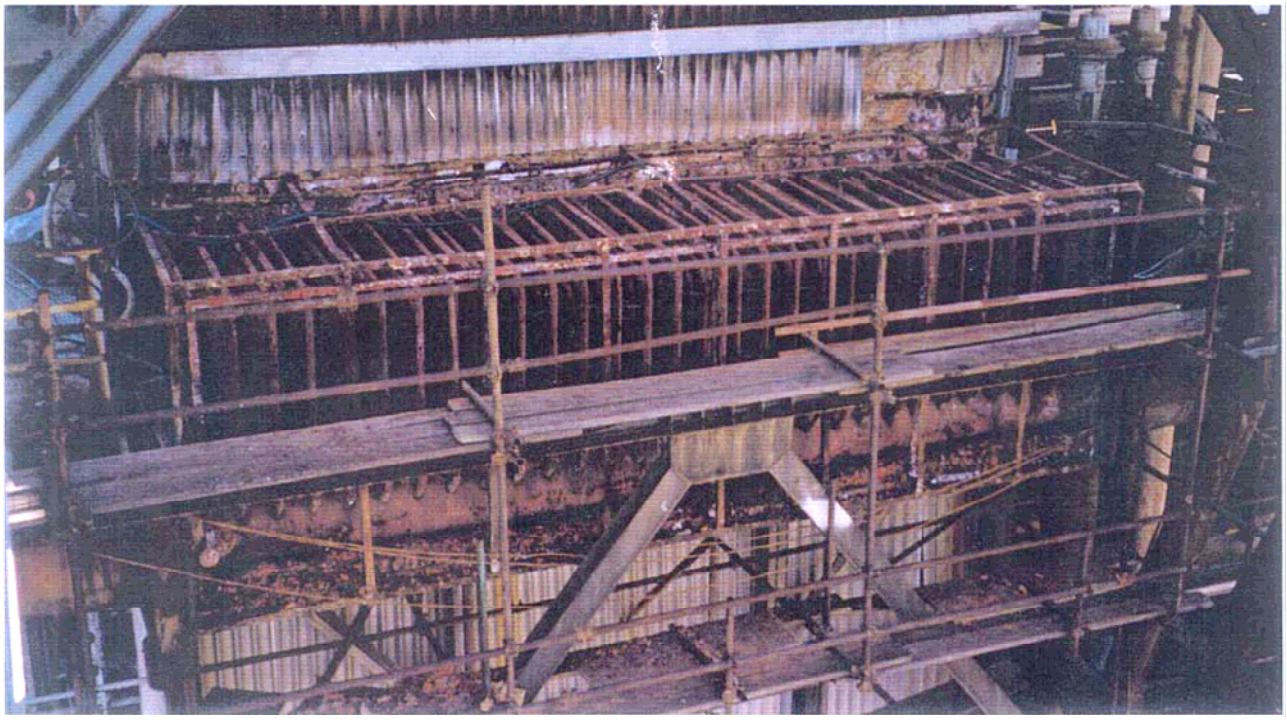
Damage mechanism: thermal fatigue.

Detected by ultrasonic testing (UT).

Cause: cooling and condensation of non-circulating steam in vent line, due to long pipe run before stop valve or lack of thermal insulation. Condensate drops down to vent opening in the header, at higher temperature, promoting thermal shocks.

Possible solutions: a) Shorten pipe run between opening at header and stop valve in vent line and improve thermal insulation. b) Periodic opening of vent valve.

Example 2 – Reheater tubing: corrosion in non-heated areas



Material: ASME SA 213 T22 (50.8 mm O.D. x 3.8 mm M.W.)

Damage mechanism: corrosion.

Detected by visual inspection and thickness measurement.

Cause: water infiltration through thermal insulation, during long shutdowns, in out door boiler. Water infiltration results from rain and from drain flashbox of nearby boilers.

When boilers have long outage periods this type of damage can be almost as significant as damage due to "hot-corrosion" in heated tubing, as heated tubes are usually thicker.

Example 3 – Drain lines: tube failure in drain piping of outdoor boiler and piping

Damage mechanism: same as for Example 2.

Drain piping is not usually considered a critical component for condition and lifetime assessment. However, drain piping possesses long runs and passes through places of passage for operators and workers.

In aged outdoor boilers with long outage periods it is important to inspect or replace, if it is more economical, the drain piping. Usually it is made of low thickness carbon steel and low alloy steel piping and corrosion can significantly reduce the pressure retaining capacity and failure can have a high impact on personnel safety. The same applies for low thickness connecting piping for instruments and controllers in thermal insulated equipment (drums and tanks) in out-door boilers.

2.4 Conclusions and Final Remarks

At CPPE power plants, a maintenance strategy has been implemented based in two programmes: *Periodic Preventive* and *Predictive Maintenance Programmes*.

These programmes are supported by several techniques, essentially complementary but corroborative in some cases.

Regarding the Predictive Maintenance Programme, it results from application of:

- *Condition Monitoring* by using on-line instrumentation or by the use of portable non-intrusive instrumentation and applying *fault sensibility* and *monitoring practicality* analysis, including

  - surveillance, collection and analysis of operating parameters (vibrations, temperatures) typical of each equipment;
  - surveillance, collection and analysis of process parameters (pressures, temperatures, flow rates), typical of equipment performance;
  - performance tests of equipment (efficiency, air leakage, air inleakage) and analysis of results;
- *Life time assessment*, taking into account the ageing of plants and operating conditions prospects, involving 3 steps:
  - recalculation of original design life;
  - access to components for condition monitoring performed in situ taking into account results from calculation, potential damage mechanisms and failure modes, results of previous inspections and failure and maintenance records; it includes conventional NDT techniques together with metallurgical inspection by replication techniques;
  - removal of material samples for laboratory tests and determination of actual materials property data and microstructural degradation characterisation which require advanced assessment techniques ;
- *Diagnostic Inspections*

Additionally, CPPE has implemented a program for the installation of new DCS (Data Control System). It became possible to design a large variety of monitoring functions, applied to several electrical-mechanical components, which are a support to maintenance strategies and programmes. This means:

- monitoring of control valves using a model based technique;
- detection of too high gradient concerning some variables correlated with unit trips or reduction of equipment life;
- monitoring of pumps or fans working out of the design curves;
- monitoring of components in what concern pressure losses;
- monitoring of closed loops by advanced fault detection algorithms;
- calculations to support life consumption assessment;
- calculations applied to the equipment, concerning efficiency, air leakage, air inleakage.

The new DCS facilities in control loops design have improved the dynamic performance of power plant operation with a positive impact on availability and efficiency, decreasing costs and the environmental impact.

The maintenance strategy we have been describing above has contributed to an improvement in CPPE power plants performance, a reduction of about 1.5% of Total Unavailability and about 30% of maintenance total costs.