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Guideline for the Assessment of Existing Structures

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CONTENTS

1 INTRODUCTION

As structures are aging, the assessment of buildings, bridges, tunnels, dams and industrial structures is becoming increasingly important. Structural codes have been developed for new design, but they often are not appropriate for assessment since there are significant differences between design and assessment. Design uncertainties arise from the prediction of load and resistance parameters of a new structure. These uncertainties represent the variability of a large population of structures caused by the unequal qualities of material, the different construction practices, and the variability of site specific live loads. Also a conservative design does not result in a significant increase in structural cost while a conservative assessment may result in unnecessary and costly repairs or replacement.

Therefore, there is a clear need for technical rules for the assessment of existing structures. In some countries, participating at SAMCO, especially the UK, assessment codes and guidelines are available, but in most European countries only single assessment routines are discussed within the scientific community, but are rarely applied by practicing engineers. This guideline is developed within the work package 3 "codes and recommendations" and offers a methodological framework for assessment, assuming a stepwise procedure, beginning with simple methods and going on more sophisticated, if necessary.

In the first chapter the general scope of the guideline is defined. The second chapter describes in detail the principals of structural assessment. A structure of the assessment methodology is introduced and the single procedures for data acquisition, structural analysis and safety verification are described. In the third chapter the proposed assessment levels (Level 0 – Level 5) and associated procedures are explained.

2 GENERAL

2.1 Scope

Structural assessment can be initiated, when there has been a change in resistance. Such as structural deterioration due to time-depending processes (e.g. corrosion, fatigue) or structural damage by accidental actions. Also when there will be a change in loading (e.g. increased traffic load) or an extension of the design working life.

Assessment can also be carried out to analyse the current structural reliability (e.g. for environmental hazards like earthquake or extreme winds and/or waves).

The proposed guideline presents a methodological framework of the assessment of existing structures and a summarisation of the manifold methods developed in recent years for structural assessment. It is intended to describe the coherency and difference between methods and to provide an understanding and to help practising engineers finding the suitable assessment procedure depending on the assessment objectives as well as on different boundary conditions.

Within management of groups of structures it is necessary to assessment unify, so that different structures are assessed in the same way and results are comparable between authorities, regions or countries. The guideline is meant to provide a framework to achieve that goal.

It is intended to explain the principles of structural assessment and to feature the several levels of structural assessment, starting with simple but conservative methods and progressing to more refined but also more expensive methods.

The guideline can be applied to all kind of existing structures (e.g. bridges and tunnels, buildings, industrial structures on- and offshore) of any type of structural material (concrete, steel, timber, masonry, composite material).

The structures to be assessed can be designed based on accepted engineering principles or design rules as well as on good workmanship, historic experience and accepted professional practice.

Since fire resistance requires properties different from those of structural safety and integrity, the assessment of fire resistance is not part of the guideline.

3 PRINCIPLES OF STRUCTURAL ASSESSMENT

3.1 Objectives

In general structural assessment is a process to determine, how reliable the existing structure is able to carry current and future loads and to fulfil its task for a given time period.

The first step of the assessment process must always be the clear specification of the assessment objective. This is essential to identify the most significant limit states. Associated with the limit states are the structural variables to be investigated and with those the assessment procedure to be applied.

A wide range of different assessment procedures exists with varying complexity and the choice of the appropriate procedure depends highly on the specified requirements of assessment.

There are two main objectives to conduct assessment of existing structures, the assurance of structural safety and serviceability and the minimisation of costs.

3.1.1 Structural safety and serviceability

The main task of assessment is to ensure that the structure or parts of the structure do not fail under loading. The assessment is carried out for ultimate limit states, which are [1]:

- loss of equilibrium of the structure or parts of it as a rigid body (e.g. overturning)
- attainment of the maximum resistance capacity
- transformation of the structure or part of it into a mechanism
- instability of the structure of part of it
- sudden change of the assumed structural system to a new system (e.g. snap through)

A reduction of serviceability may lead to a limitation of use and therefore serviceability assessment might become necessary. Serviceability limit states include [1]:

- local damage which may reduce the working life of the structure
- unacceptable deformations which affect the efficient use
- excessive vibrations which cause discomfort to people

Safety and serviceability can be evaluated for a variety of reasons, among others for changes in use or increase of loads, effects of deterioration, damage as result of extreme loading events and concern about design and construction errors and about the quality of building material and workmanship.

Increases of the maximum live load limits and changes of use are probably the main reasons for structural assessment. For buildings such changes could result in the need to support higher floor loadings. For bridges there is a world wide demand to raise the limits for traffic loads.

Any structure is undergoing some degree of deterioration. The effects of deterioration are structure and site specific. Concerning structural strength, corrosion and fatigue are the main deterioration processes. Spalling, cracking, and degraded surface conditions are typical indications of deterioration.

Impact, earthquake or wind storms can result in structural damage. The remaining load carrying capacity needs to be analysed after such events.

It may be necessary to assess an existing structure, after concerns about the correct design and constructions arise, including low quality building material or workmanship [6].

3.1.2 Cost minimization

In the last decades systems for managing single structures such as bridges or a whole stock have been developed for minimising the overall cost by optimising inspection, maintenance and repair work. A main task within this decision making process is the assessment of the structural conditions to determine the current state and to assume the future performance of a structure depending

The objective of assessment within structure management is to provide information about the structural state for optimisation of the point in time and the extent of inspection, maintenance and repair work (maximum operation effect at minimum costs) and for priorisation of maintenance and repair work within a stock of structures or parts of a structure. Further on it needs to be achieved to minimise economic losses by disruption of operation of the structure.

The assessment results should be available in a form, useable in the structure's management. It means that input values, calculations and results should be archived for future reference and reassessment. Also the applied assessment routines should be unified within a stock to make results comparable and so to make the right inspection, maintenance and repair decisions.

3.2 Methodology

The assessment of existing structures can be carried out with methods of varying sophistication and effort. The core objectives, as described above, are to analyse the current load carrying capacity and to predict the future performance with a maximum of accuracy and a minimum of effort. Unduly conservatism but also too lax restrictions should be avoided.

In most cases it judiciously to start with simple conservative routines and use more sophisticated routines only when the evaluated load carrying capacity is insufficient.

Generally structural assessment should be carried out using limit state principles with characteristic values and partial safety factors. If more refined methods are necessary, the probabilistic approach has to be applied, if economic.

If structures have failed assessment to an acceptable capacity, the engineer can make a recommendation, but the technical authority is likely to be ultimately responsible for public safety and therefore has to do the final decision. A structure, failed in assessment, may remain in service if it presents a low risk, subject to monitoring.

3.2.1 Classification

In general assessment procedures can be classified into three groups: measurement based assessment, model based assessment and non-formal assessment.

<u>Measurement based serviceability assessment</u>

In the category fall those assessment routines, where the load effects are not determined by structural analysis but direct by measurement (e.g. performance monitoring, proof load tests). Since only serviceability measures can be determined directly, the method is only able to verify structural sufficiency within the Serviceability Limit State. It is a two-component procedure where the components are the follow:

(1) measurement of load effects

(2) serviceability verification

Measurement based assessment routines are in general not complex. An example application is the evaluation of serviceability measures like displacement or dynamic behaviour after a new utilisation or the structure. The assessment of nearly insufficient and monitored structures may also be based on this method. Measurement based assessment is of little significance and will therefore not be described in detail within this guideline.

<u>Model based safety and serviceability assessment</u>

In this category fall all those assessment routines, where the load effects are determined by model based structural analysis. Using this method Ultimate Limit State and Serviceability State can be modelled and therefore assessed. It is a three-component procedure where the components are the follow:

(1) acquisition of data of loading and resistance

(2) calculation of load effects on structural models

(3) safety and serviceability verification

Most assessment applications are processed based on a structural model, exceptions are only the above mentioned measurement based serviceability assessments. This guideline will focus mainly on these assessment procedures. A detailed description of the specific components and a selection of methods within each component is given in the next section.

Non-formal assessment:

In this category fall assessment routines which are based on the experience and the judgement of the assessing engineer. They are more or less subjective and are applied only exceptional.

Most non-formal assessment takes place within structure management, where the structural condition is evaluated on the base of visual inspections.

3.2.2 Assessment levels

As mentioned before, assessment procedures vary in sophistication. It is recommended to start the assessment with simple but conservative low level methods and, in case the assessment failed, move on with more refined upper levels. The grading concerns the specific methods of all three components (Annex A).

In general within one applied assessment procedure the sophistication of the individual components should be of around the same level. For instance, there is no sense to achieve resistance and load parameters with simple but imprecisely methods and use full probability based methods for the verification.

Admittedly, there may be cases were a mixture of methods with low and high complexity is advisable. For instance when a first step low level assessment failed and in a second step the structure specific resistance and load parameters are achieved by more refined investigation methods like NDT, the structural analysis and the verification can be carried out with the same simple methods as in the first step and the assessment may now result in sufficiency.

The proposed assessment levels are established for structuring the assessment process. They are not imperative and the boundaries of the levels are flexible. The levels adhere roughly to those, established in the UK for the assessment of highway bridges [7].

The proposed assessment levels are described below and an outline of the structure shows Fig.1.

Level 0: Non-formal qualitative assessment:

Assessment, based on experience of the engineer, is mostly used for a pre-evaluation of the structure. One is able to evaluate visual deterioration effects like corrosion of steel members or visual signs of damage (cracks, spalling).

Level 1: Measurement based determination of load effect:

Assessment of serviceability by measurement of performance values and comparison with threshold values. There is no structural analysis carried out. The threshold values can be given in codes or individually specified. Details are described in chapter 3.1.

Level 2: Partial factor method, based on document review:

Assessment of load-carrying capacity and serviceability using information from design, construction and inspection documentation. Structural analysis is generally carried out using simple methods. Safety and serviceability verification is based on partial factors. Details are described in chapter 3.2.1.1.

Level 3: Partial factor method, based on supplementary investigation:

Assessment of load-carrying capacity and serviceability using information from site-specific detailed non-destructive investigations. Structural analysis is carried out using refined methods and detailed models. Safety and serviceability verification is based on partial factors. Details are described in chapter 3.2.1.2.

Level 4: Modified target reliability, modification of partial factors:

Verification of the load–carrying capacity with site-specific modified partial safety factors. Structural properties as well as external circumstances can influence the safety measure. Practically, modifying of partial factors is carried out for groups of structures with similar structural behaviour or load influences. Details are described in chapter 3.2.2.

Level 5: Full probabilistic assessment:

Assessment, taking into account all basic variables with their statistical properties. Structural reliability analysis is used directly and instead of partial factors. Uncertainties are modelled probabilistically. Details are described in chapter 3.3.

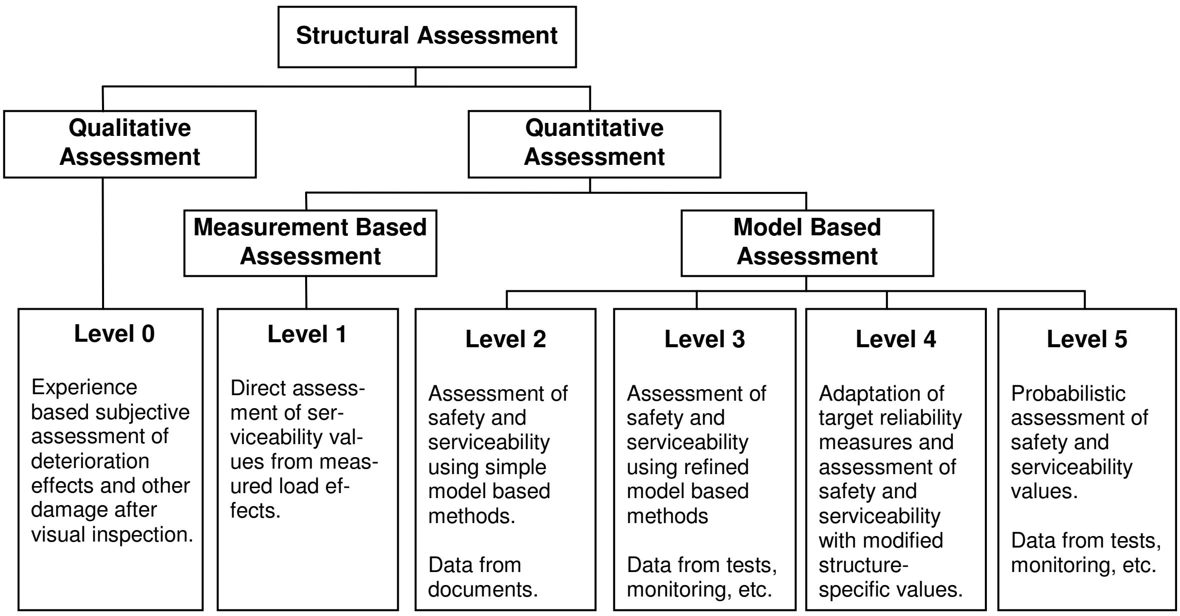

Fig.1: Context of the Structural Assessment Levels

3.3 Methods of data acquisition

To determine load effects, in most cases of assessment it is necessary to gather information about material and structural properties and dimensions as well as about the previous, current and/or future loading on the structure. Environmental conditions are of physical, chemical or biological nature and can have an effect on material properties.

The main difference between design and assessment is, that in the latter uncertainties can be reduced significantly by site specific data from the real structure.

There is a wide range of methods with varying expense and accuracy. The choice of the data acquisition method highly depends on the assessment objective and with that on the assessment procedure. Usually simple methods like the study of documents are applied in the beginning. To reduce uncertainty within higher assessment levels more sophisticated test methods need to be applied. Non-destructive methods are to prefer to destructive methods, whenever this is possible.

Beside the provision of data which describes the current state of the structure, also information about time depending processes like deterioration need to be acquired. This can take place with periodic or permanent measurement (i.e. structural health monitoring).

The results of the data acquisition should be of the same form, to be able to compare data from different methods and to be able to use data in future assessment procedures.

3.3.1 Study of documents

To review documents from design and construction process as well as inspection and maintenance reports is in general the easiest way of gathering data about the structure to be assessed. It has to be assured that the reviewed documents are correct.

Loads can be usually determined from current loading codes and environmental conditions may be obtained from inspection reports.

Resistance properties like material and structural properties and dimension can be obtained from codes, drawings and other design specifications (e.g. static calculations, subsoil condition report), from construction documents (e.g. material delivery documentation) and from reports of earlier inspection and maintenance.

3.3.2 Inspections and material testing

To reduce uncertainties about load and resistance, site specific data should be used within the assessment process. Very effective methods are site inspection and material testing.

Under inspection one understands the in situ investigation of load and resistance parameters. There is a large variety of methods, starting with simple visual inspection and ending with several high-end non-destructive techniques.

Inspections are especially for detection and investigation of deterioration processes like corrosion and fatigue and for detection of changes in the structural systems. Therefore it is necessary to conduct inspections repeatedly.

Material tests are done for determining strength parameter of the used building material. The tests can be destructive and non destructive. They can be conducted on site or at a laboratory.

Parameters to be investigated and corresponding investigation methods are:

- cross sectional and longitudinal geometry changes (damages) from overloading (e.g. cracks, ruptures) and from deterioration processes (e.g. corrosion, spalling, fatigue cracks) using laser, ultrasonic devices, slide gauge, electronic gauges, etc.;
- structural integrity (e.g. for hidden damage or inhomogeneity) using for instance impact echo testing;
- material strength using tension and compression tests on samples, sclerometer method, pull-out tests, pull-off tests, etc.;
- parameter, influencing the dead load and the superimposed dead load (e.g. material densities, permanent equipment)
- duration influencing parameters of the structure (e.g. environmental conditions, carbonation and chloride content of concrete) using pH-test, phenolphthalein test, quantitative chloride analysis on samples, etc.;
- serviceability matter (e.g. crack widths, surface conditions of roads)

3.3.3 Performance testing and monitoring

In the case, the structural behaviour cannot be understood sufficiently or data acquisition using the methods discussed before did not bring the expected results, the performance of the structure should be tested. That means, that the static and/or dynamic behaviour is measured once, periodically or permanent to receive information about required structural properties.

It is important to know, that measurement data does not necessarily display reality. So there need to be care and caution with the installing of the sensors, within the measurement and during interpretation of the data. Whenever possible, measurement data should be redundant.

Structural health monitoring

Because it has become more and more affordable for owners or responsible authorities, long-term monitoring of structures or structural elements is nowadays a common tool for the permanent observation of the structural integrity.

Under structural health monitoring one understands the permanent or periodical measurement of time variant measures like displacement, strains and stresses, damage evaluation (e.g. crack width) and vibration characteristics with the aim to detect changes in the structural properties and in some cases to be alarmed, when limit states are reached or exceeded.

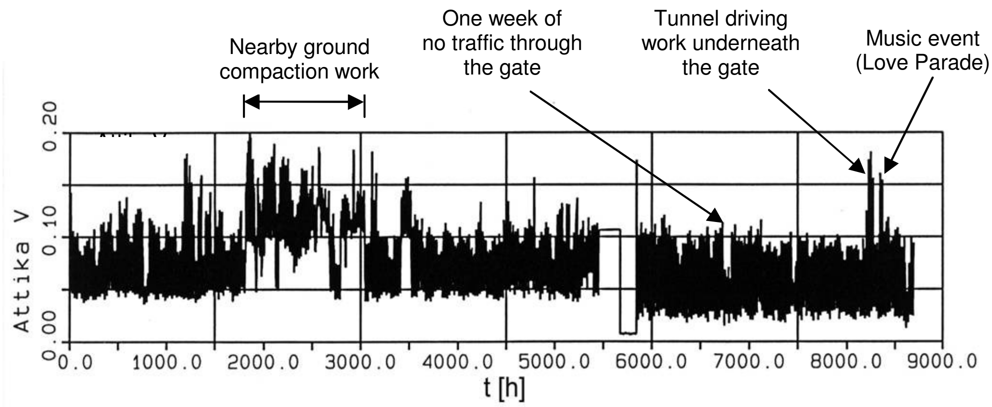

Fig.2: Monitoring of load effects (here dynamic effects) at the Brandenburg Gate in Berlin. Different load periods and single events are clearly detectable.

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It can be applied, when older or damaged structures are assessed as "just sufficient" or "just insufficient" and the structure should not be demolished or displaced but be observed carefully.

Structural health monitoring can also be applied to new built structures (i.e. life time structural health monitoring). Here the advantage is that the structural properties of the undamaged state will be known and future data implies the changes in the properties directly which makes predictions of future performances possible or easier.

System identification by static and dynamic measures

If dimensions and material properties of the real structure cannot be received by measurement and testing (e.g. inaccessibility, hidden damages) structural properties like stiffness of structural members and joints, flexibility of hinges or bearing conditions can be obtained by system identification. It is also an efficient tool for damage detection and monitoring of damage evaluation.

With this procedure static properties like displacements (e.g. deflections, inclinations) under defined loads as well as dynamic properties like natural frequencies and mode shapes are measured at the real structure. The structural system model becomes refined then in a way, so that the model reflects the same characteristic behaviour as the real structure.

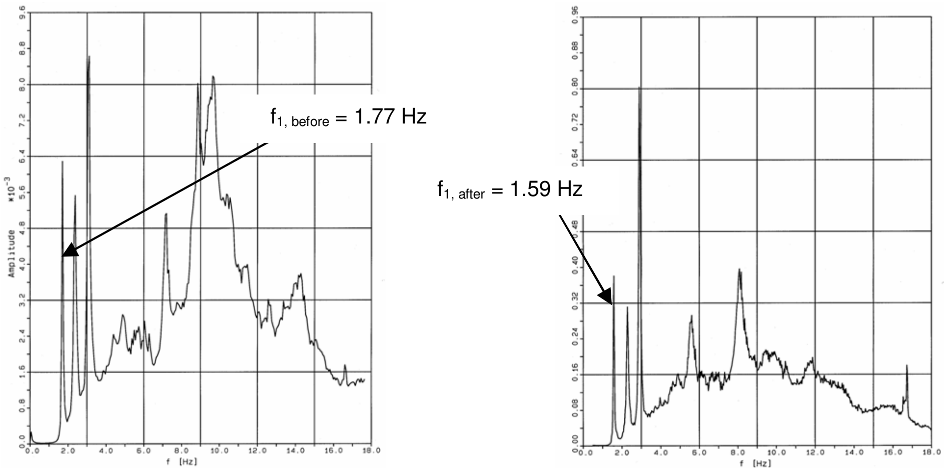

Fig.3: System Identification: Changes in the first natural frequencies of the Brandenburg Gate in Berlin before and after changes in the foundation conditions are shown by power spectra.

When system identification is applied periodically or even permanent, the time variance of structural properties by deterioration processes or other damage causing events can be identified and monitored. The structural model will than become updated according the obtained new measures.

It needs to be mentioned, that environmental conditions, especially temperature, can have a large influence on the static and dynamic measurands. This must be considered when evaluating the structural properties.

Proof load tests

The application of defined loads to a structure to verify the load carrying capacity is a strong tool for assessing existing structures.

There are different types of proof load tests depending on the investigated limit state. Within the serviceability limit state load effects can be measured after application of the proof load and in a second step there will be the verification, if the measure exceeds the limit state or not. Within the ultimate limit state the fact that the structure or elements of the structure don't fail during the test verifies the no-exceeding of the limit state.

It is also common to increase the proof load until the signs of plastification occur, for example with noise emission sensors on RC-structures. Of course it needs to be assured that the failure behaviour is ductile not brittle.

The difference of proof load test to system identification methods with defined loads is that at the former the results will be used to verify load carrying capacity or serviceability direct whereas at the latter the results are used to adjust the structural model on reality.

3.3.4 Monitoring of live loads and environmental conditions

Loads due to the intended use (e.g. floor loads in buildings and traffic loads on bridges) as well as environmental loads (e.g. wind, waves, temperature, earthquake) are mostly site specific.

With data from monitoring site specific live load models can be developed and used for structural assessment instead load models from codes.

Load effects to the structure due to extreme load events like special traffic, extreme wind and earthquake can be determined and evaluated.

Environmental conditions are of physical, chemical or biological nature. Due to monitoring of environmental conditions future deterioration of the structure or parts of the structure can be predicted.

3.4 Methods of structural analysis

Structural performance shall be analysed using models that reliably represent the loading on the structure, the behaviour of the structure and the resistance of its components. The analytical model should reflect the actual condition of the existing structure [2].

3.4.1 Simple analysis methods

For lower assessment levels it is often effective to calculate load effects with basic conservative methods with simple structural models, provided that the approximately large uncertainty is regarded with an adequate safety measure.

Typical simple analysis methods are among others space frame and grillage analysis combined with a simple load distribution and linear elastic material behaviour, which result in a lower bound equilibrium solution.

3.4.2 Complex analysis methods

In case low level assessment failed, refined load effect calculation methods need to be accomplished.

Refined methods include mainly finite element analysis and non-linear methods such as yield line analysis, where these may lead to higher capacities. Particularly a specified modelling of the material behaviour such as time-variant behaviour (e.g. shrinkage and creeping of RC structures) and the consideration of interactions between material components (e.g. bond, tension stiffening in RC) will uncover hidden capacity reserves and reduce conservatism.

Applying full probability safety verification, stochastic finite elements can be used to model the structure. The difference to conventional finite element models is that the stochastic elements take the spatial correlation of the random variables into account.

3.4.3 Adaptive models

To avail new information about the structural behaviour within assessment, for example from long term monitoring, models need to be updated allowing for the new information.

Adaptive models are able to update automatically structural variables (e.g. stiffness parameters) using measurement data such as a change in displacements, strains or damage values (e.g. crack width).

3.5 Methods of reliability verification

While data acquisition and structural analysis are procedures to obtain information about the structural state the third component of the assessment process discusses the actual evaluation of the safety and serviceability margin which can be described as the distance between the actual real state of the structure and the limit state.

The verification of an existing structure should normally be carried out to ensure a target reliability level that represents the required level of structural performance. Current codes or codes equivalent to ISO 2394, which have produced sufficient reliability over a long period of application may be used. Former codes that were valid at the time of construction of an existing structure should be used as informative documents [2].

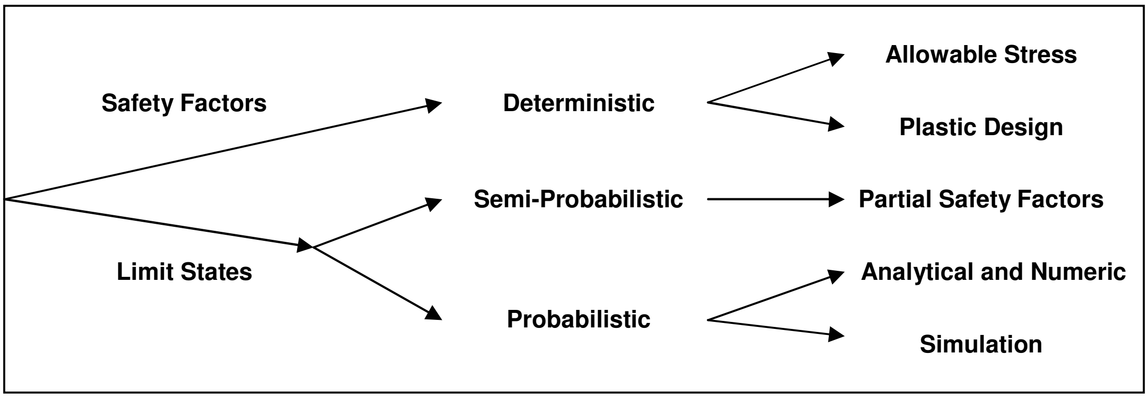

Fig.4: Reliability verification approaches [3]

3.5.1 Deterministic verification with global safety factors

The deterministic approach is the traditional way of defining safety. It is fully based on experience and the safety measures are of empiric nature respectively. Deterministic verification is characterised by simplifications and associated with those by conservative safety measures.

The most common deterministic safety measure is the global 'factor of safety'. It is the ratio of the resistance and the load effect and applied mostly on the resistance side. Basic variables are represented by deterministic constituted normative values. The concept of the permissible stresses is a typical deterministic verification method, where failure of the structure is assumed to occur, when any stressed part of it reaches the permissible stress. The accuracy depends on, how well the normative value of the permissible stress represents the failure stress of the real material and how well the calculated stress represents the actual stress in the real structure.

Another concept is the load factor method, where the safety measure is represented by the 'load factor', which is the ratio of the ultimate strength of a member to its working loads.

Deterministic verification methods with one global safety factor reflect reality insufficient and contain a considerable amount of uncertainty and for that reason should be used only exceptionally within the assessment of existing structures. For instance the scatter for variable load is much higher than for permanent loads. Applying an overall safety factor results in quite different safety levels for heavy structures like concrete structures compared to lightweight structures like steel structures.

3.5.2 Partial safety factors

The semi-probabilistic approach is based on the limit state principle. The primary concern is to ensure that failure does not occur in a component of the structure or the structure itself, which is described as *Ultimate Limit State* (ULS). For structural assessment it may also be important to analyse the serviceability performance where the structural effects of loading may be a serviceability failure, described as *Serviceability Limit State* (SLS).

As safety measure *partial safety factors* are established. They have been developed with reliability analysis for a specific target reliability and applied to the accordant design parameters. Partial safety factors guard against the extreme variations of the design parameters, which could possibly occur during their use on both, resistance and load side.

Within the classification of probabilistic based methods for structural design and assessment semi-probabilistic methods are ranked as *Level 1 Methods*, where the basic variables are specified with one *characteristic value*.

The semi-probabilistic verification method can much better reflect reality because uncertainties can be taken into account on those design parameters where they occur. Since partial factor based verification methods have been developed for design reasons, most design codes use them. To simplify verification routines, partial safety factors supply a wide range of structures and failure modes. Also in design a safe structural answer is more important than a realistic one and economic design means ease of construction instead of structural efficiency. For those reasons semi-probabilistic methods tend to be conservative for the majority of structures. The level of conservatism varies from structure to structure.

3.5.3 Probabilistic verification

Probabilistic verification procedures are also based on the principle of limit states as described above.

Within assessment it will be intended to identify the real values of the design parameters by inspection, testing, monitoring or other methods and therewith to minimise uncertainties. In the verification process the data is the basis to model all uncertainties in the underlying variables and to compute the actual probability of failure.

Probability of failure and structural reliability are directly associated. The measures of whether a structure is adequately safe or not, are the *probability of failure* and the equivalent *reliability index*.

Probabilistic verification routines are by now well developed and become more and more used in design and assessment of buildings, bridges and industrial structures. Nevertheless, the procedure is highly sensitive to the chosen probability distributions which represent the basic random variables and also to the analysis methods and models for calculating the load effects (e.g. grillage analysis, FE analysis). Therefore while using this especially for structural assessment effective tool it is necessary to take great care and have an adequate expert view on the variables, sensitive to the result.

3.5.4 Target reliability

In a probability based approach to assessment the structural risk acceptance correspond to a required minimum structural reliability, defined as target reliability. The requirements to the safety of the structure are consequently expressed in terms of the accepted minimum *reliability index* β or the accepted maximum *failure probability* P_f.

The target reliability level, used for verification of an existing structure can be determined based on calibration to existing practice (i.e. on existing codes), assuming that existing practice is optimal. Further possibilities to optimise the necessary reliability level are the concept of the minimum total expected cost and/or the comparison with other social risks (see Annex C).

The performance requirement should also reflect the type and importance of the structure, possible failure consequences and socio-economical criteria, which need to be considered when determining the target reliability level. [2, 8].

There are fundamental differences between the assessment of existing structures and the design of new structures, which affect the requirement on the structural performance and thus may affect the used target reliability in individual cases. The differences are as follows (ISO 13822):

- *economic considerations:* the cost between acceptance and upgrading the existing structure can be very large, whereas the cost of increasing the safety of a structural design is generally very small, consequently conservative generic criteria are used in design but should not be used in assessment,
- *social considerations:* these include disruption (or even displacement) of occupants and activities, also heritage values, considerations that do not affect the structural design, but assessment,
- *sustainability considerations:* reduction of waste and recycling, considerations of less importance in the design of new structure, but in assessment.

Target reliability values are denoted in several codes and guidelines. Annex C gives an example of calibration life time target reliability values, depending on the consequences of failure and the relative cost of the safety measure from ISO 2394.

4 STRUCTURAL ASSESSMENT ROUTINES

4.1 Performance assessment (Level 1)

The most straightforward formal assessment routine is the comparing of directly measured performance values x_m with defined threshold values x_t :

$$x_m \leq x_t$$

The assessment can be carried out under *working load* or under defined *proof load*.

Common applications are:

- *Serviceability tests after construction.*

  Measures are static and dynamic parameters (e.g. deflection of bridge decks, acceleration and natural frequency of foot bridges).

- *Serviceability and transport safety tests preliminary to utility changes.*

  Measures are static and dynamic parameters (e.g. deflections of ceilings in buildings; deflection, inclinations, acceleration and natural frequencies of railway bridge decks for an increased traveling speed of trains).

- *Monitoring of dynamic loading.*

  Measures are dynamic parameters (e.g. increase of vibration amplitudes at bad surface conditions of road bridges).

- *Performance monitoring of nearly insufficient structures.*

  Measures are static and dynamic parameters (e.g. deflection, crack growth)

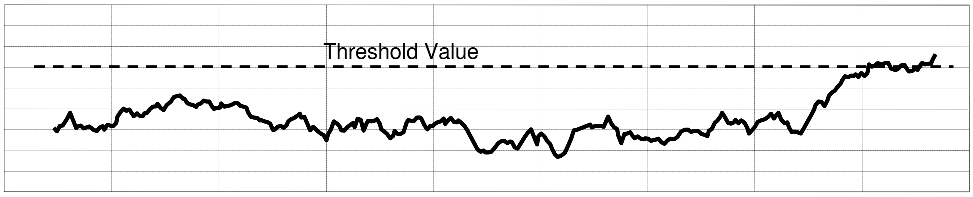

Fig.5: Direct measured serviceability value over the time and distance to the threshold value

4.2 Partial factor based assessment routines (Level 2, 3, 4)

Within the partial safety factor approach the safety and serviceability of a structure or component is validated by comparing the characteristic values for the action S_k and the resistance R_k:

$$\gamma_S \cdot S_k \leq \frac{R_k}{\gamma_R}$$

where γ_S and γ_R are the partial safety factors for the action effects and the resistances respectively.

While the characteristic values are based on statistical parameters, the partial safety factors have to be determined by calibration. This used to be based on engineering judgment, now there exist probabilistic tools for calibration.

4.2.1 Codified partial safety factors (Level 2, 3)

Generally, partial safety factors are constituted in codes and guidelines. If there are no specific codes and guidelines for assessment with adapted safety factors, safety factors calibrated for design from according design codes and guidelines (e.g. EC 1 for loading; EC 2 –EC 7 for resistance) have to be used.

4.2.1.1 Assessment based on documents and visual inspection (Level 2)

Information about load and resistance is taken from design documents and codes with exception of specific values, leading to the assessment (damage, extraordinary load). Knowledge about structural condition from visual inspection may also enter the assessment.

Within this routine the *structural analysis* is based on those used in the design process. If necessary, more refined analysis techniques (e.g. FE analysis) may be used.

Common applications are:

– *validation of safety and serviceability after extreme load damage* (damages can result from extreme traffic or floor load, impact, earthquake, windstorm, etc.),

– *validation of safety and serviceability after deterioration damage* (damages are results of fatigue, corrosion or other deterioration processes),

– *determination of safety and serviceability for utility changes* (utility changes can be a new use of buildings (e.g. to warehouses) and extreme heavy vehicle crossing of bridges, not designed for that load).

4.2.1.2 Assessment based on supplementary investigations (Level 3)

Detailed investigation on material properties and dimensions

When a structure or components could not be assessed as sufficient with Level 2 methods, accurate information about material properties and dimensions from in situ and lab tests may lead to a successful verification of safety and/or serviceability.

Typical parameters to be analysed are:

- geometric dimensions,
- mechanical and chemical material properties,
- hidden damage or inhomogeneities.

Investigation methods are listed and explained in 2.3.1.2.

To be able to make a sufficient conclusion about a material property, tests have to be carried out on an adequate number of samples. Statistics of the test results can than be used for determining the site-specific characteristic value of the tested property. Geometrical measures are generally applied with their nominal value.

Measurement based system identification

When it becomes necessary to use more refined analysis methods and models (e.g. FE models) the structural properties of the model need to be adapted to the reality. For that reason it is useful to determine performance measures and compare them to those of the model. Structural parameter need then to be adapted at the model to reflect the real structure in a sufficient manner.

Performance values and method of data acquisition:

- displacements (e.g. midspan deflection) – load tests
- strain / stress values – load tests, monitoring
- natural frequency and mode shapes – monitoring

Data acquisition methods are described in detail in section 2.3.1.3.

Structural parameters within model adaptation are:

- cross sectional stiffness (e.g. module of elasticity, dimensions, stiffness of superstructures and attachments)
- hinge and bearing flexibility
- mass distribution and damping behaviour
- constraints which affect structural performance (e.g. external prestressing)

Site specific live load models

Generally assessment is carried out with the same code based loading requirements for all bridges. These load models allow for the worst credible happenstance, in case of bridges for instance based on estimated maximum dynamic impact effects, worst overloading of vehicles and maximum lateral bunching of vehicles on a bridge. Also, they often allow for future increases in live loading and are inappropriate for safety evaluation of short time periods

With bridges taking into account different traffic conditions on different types of roads, different impact characteristics, or the lower probability of maximum impact effects occurring at the same time as lateral bunching leads to site specific live load models with reduced magnitudes [7].

To develop a site specific live load for assessment, statistical data needs to be collected from a range of structures of the same type. Then probabilistic load models have to be generated based on the collected data. With reliability analysis the load model will be calibrated for defined sets of structures. This work needs to be done in preliminary stages of the assessment by a technical authority.

4.2.1.3 Updating of measured quantities:

The investigation of the existing structure or structural component is intended to update knowledge about the structural resistance.

Additional information on structural resistance can be obtained from:

– on-site inspection and testing on structural elements
– performance checking of the whole structure

Based on this new data, the estimation of the previous ("prior") distribution functions can be updated and with them the characteristic and/or design values.

Different approaches within the partial factor format are introduced in Annex D.

4.2.2 Modified partial safety factors (Level 4)

Based on structure-specific safety characteristics, target reliability as assessment criteria can be modified. Within partial factor based safety format those factors need to be modified to represent the adjusted safety margin. Concerning resistance and load parameter, the actual assessment is still on a purely deterministic basis, since the same fixed nominal or characteristic values are used for each basic parameter.

Specific safety characteristics are loading history, consequences of failure, reserve strength and redundancy, warning of failure by inspection and monitoring.

<u>Loading history:</u>

For modifications based on loading history the underlying assumption is that if the bridge has been in service for a sufficiently long period of time, it can be reasonably assumed that it will have been subject to some extreme loading

<u>Consequences of failure:</u>

For modifications based on consequences of failure structures can be classified according to the consequences, if failure occurs (e.g. low, medium, high, very high hazards to life and economic consequences). The classification is based on the ratio ρ defined as the ratio between total costs (i.e. construction costs plus direct failure costs) and construction costs.

Reserve strength and redundancy:

For modifications based on reserve strength and redundancy structures can be classified according to the type of failure if failure occurs (e.g. ductile and redundant with high reserve strength, brittle failure).

Warning of failure:

For modifications based on warning of failure magnitude of the target reliability can be reduced, if early warning by periodic inspections or a monitoring system is intended.

Modifying partial safety factors based on adjusted target reliability is a probabilistic procedure and should be carried out for sets of structures and structural specifics by a technical authority.

4.3 Probabilistic assessment routines (Level 5)

The main result of a probabilistic assessment routine is the calculated probability of failure or the equivalent reliability index of a structure or structural member. Unlike in the partial safety factor concept, where design parameters are definite and uncertainties are guarded by safety factors, the probability of failure depends directly on the uncertainties in the load and resistance parameters (and on other factors such as gross errors, etc).

Uncertainties are modelled using appropriate probability distribution functions for each basic variable and for defined limit states the probability of failure or the equivalently the reliability index is calculated for structural components or the overall structure.

According to ISO 2394, three types of uncertainties may be identified:

- inherent random variability or uncertainty, subdivided into uncertainties which can, and cannot, be affected by human activities;
- uncertainty due to inadequate knowledge, subdivided into uncertainties which can, and cannot, be decreased by research activities;
- statistical uncertainty

It is very important for the assessing engineer to know, that the determined structural reliability or the equivalent probability of failure is a notional value and not an absolute measure of safety or serviceability. It should not be understood as a measure of the frequency of failure that might be expected during service. Instead the calculated probability of failure should be used for comparison with acceptance criteria to assess safety and serviceability.

Several assessment procedures based on the structural reliability analysis are described in that chapter. Detailed information are given in [6], [9-11].

4.3.1 Principles of probability based assessment

The overall procedure in probabilistic based structural assessment can be stated as shoed in Fig. 6):

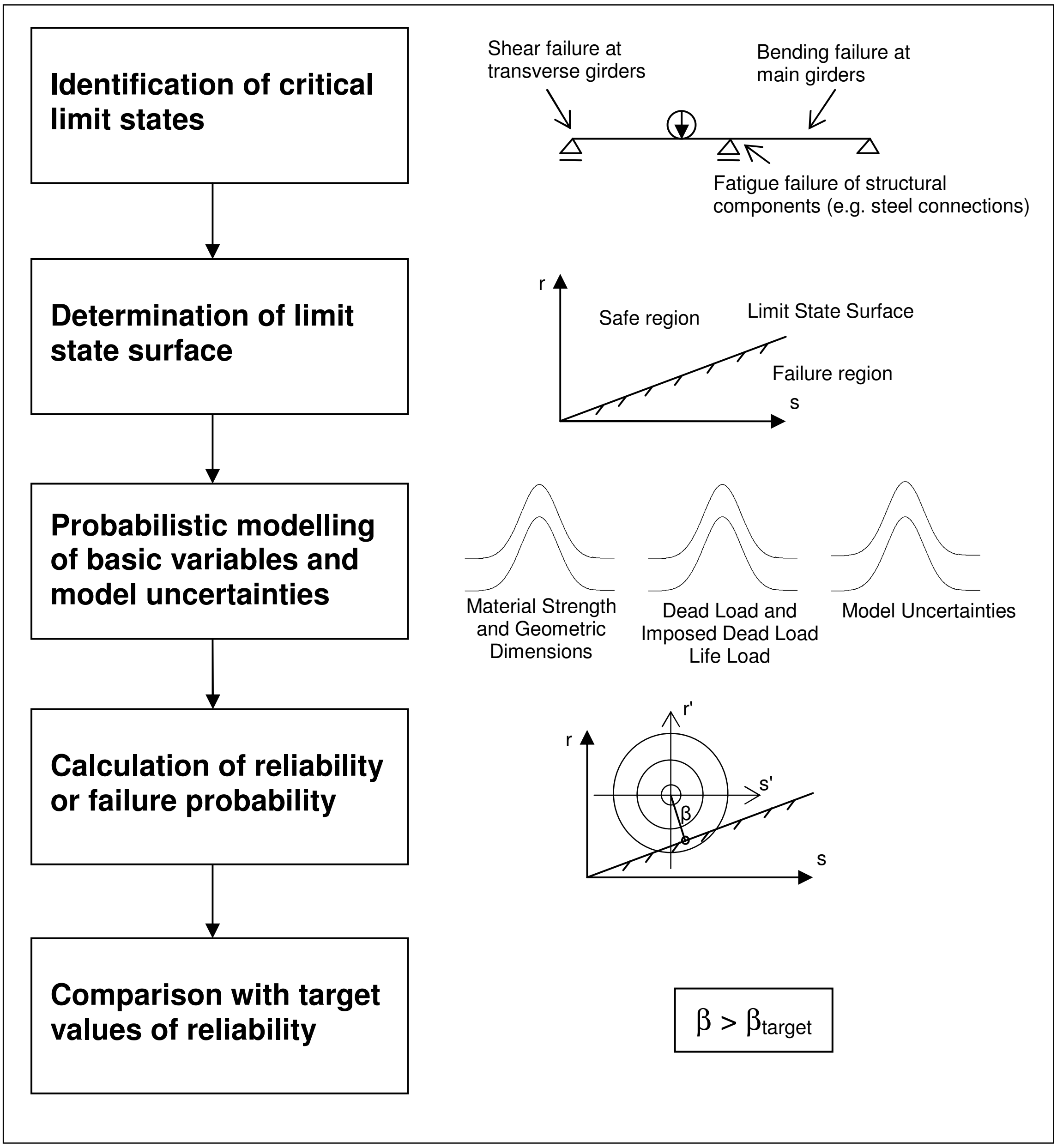

Fig.6: Probabilistic based assessment procedure

The critical failure modes have to be determined in a pre-evaluation process, applying traditional deterministic analysis. Information about location and extend of deterioration or mechanical damage can be acquired from inspections and monitoring. With sensitivity analysis it can be estimated whether the conclusion on identified critical failure modes is stable due to variations in information and modelling.

The limit state surface (or failure surface) is the limit state equation in the n-dimensional basic variable space:

$g(\boldsymbol{x}) = 0$

with $\boldsymbol{x}$ denoting the vector of n basic variables. It is describing the functional coherency of all basic variables within one failure mode and divides the basic variable space in a safe domain: $g(\boldsymbol{x}) \geq 0$ and a failure domain: $g(\boldsymbol{x}) < 0$ (Fig.7).

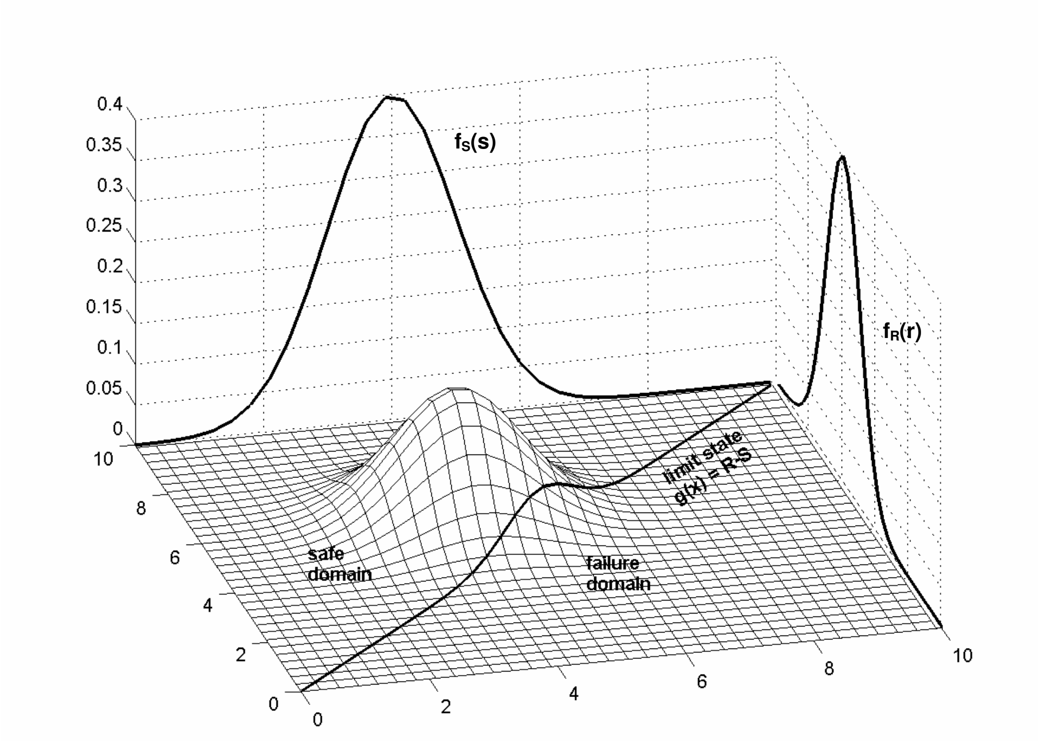

Fig.7: Schematic illustration of the probability density functions of the basic variables R and S, the joint p.d.f. and the limit state g(x)=R-S=0 [14].

All basic variables within the critical limit state related to load, resistance and modelling which imply uncertainty, are modelled as stochastic variables with corresponding statistical distributions in agreement with the knowledge about the variables from documents including codes, by inspection, testing and monitoring. It is especially convenient to model human natured live load specific to the structures in question, where the statistical distribution of the load strongly depends on the structures utilisation.

In a general case, the probability of failure P_f is defined by the limit state $g(\boldsymbol{x}) < 0$:

$$P_f = P\,(g(\boldsymbol{x}) < 0)$$

Several methods are developed to compute P_f (see Annex B):

- exact analytical methods
- numerical integration
- approximate analytical methods
- simulation methods.

The relation between the probability of failure P_f and the reliability index β is described by:

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

In a final step the calculated structural reliability is compared to the safety requirements, constituted in target values of the probability of failure or reliability index.

4.3.2 Time variant reliability assessment

Live load and resistance of a structure do change with time, particularly if the structure is subjected to deterioration advancing processes like environmental or chemical attack, or fluctuating stresses. A reliability analysis should therefore consider time-variance in basic variables describing load and/or resistance as processes.

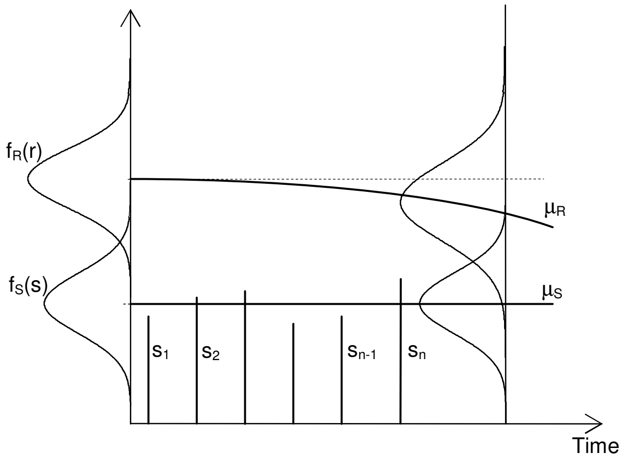

Fig.8: Schematic Representation of Load Processes and the Degeneration of Resistance [2]

In case of time dependent random variables (i.e. random processes), the limit state function should be considered with respect of time:

$$g(\mathbf{x}(t)) < 0 \text{ for } t \in [0,T]$$

where [0,T] denotes the reference period, which can be structural lifetime or other period of interest.

Then probability of failure becomes:

$$P_f(t) = P\,(g(\mathbf{x}(t)) < 0)$$

It is called "first passage probability", since it defines the probability that the limit state is "crossed" for the first time during the reference period [0,T]. Depending on the problem, there are several approaches for determination of failure probability at specific points in time taking into account the time dependence of load effects and / or resistance, e.g. [6]:

- time-integrated approach where the whole lifetime of a structure is considered
- discrete approach, where shorter periods are considered (e.g. wind storms)

Also time-variant problems can be transformed into time-invariant problems where e.g. a load process is replaced by a random variable with mean value equal to its expected maximum value over a chosen reference period. This method can be applied at overload failure and cumulative failure (e.g. fatigue, corrosion).

4.3.3 Appliance of additional information

When additional information has been gathered about an existing structure or its components the knowledge implicit in that information might be applied to improve any previous ("prior") estimate of the structural reliability of that structure or component [6].

Additional information on structural resistance and loads may be obtained from: on-site inspection or testing, monitoring, proof load testing and observations about satisfying past performances.

With additional information the updated failure probability P_f'' is given by Bayes' theorem. Several common procedures for updating the failure probability are described in Annex D.

In general two complementary approaches to updating exist:

– consideration of *individual structural properties* with allowance of new information from inspections, tests (e.g. material strength, structural dimensions, stiffness, bearing conditions, etc.),

– consideration of *the performance of the whole structures or structural members* with allowance of new information from load tests and satisfying past performance.

4.3.3.1 Additional information from inspections, tests and monitoring

The aim of inspections, tests and monitoring is to receive new information on individual parameters affecting the structural performance to update distribution functions of the parameters by Bayesian approach and to actualise the existing failure probability based on the updated distributions.

4.3.3.2 Additional information from proof load testing

Proof load testing is generally used to verify the resistance of an existing structure or structural members. Results of proof load testing may be analysed using a probabilistic approach. If a structure has survived a known proof load, then the distribution function of structural resistance can be truncated at this known load effect, as shown in Fig. 9 [12].

4.3.3.3 Additional information from past service loading

Satisfactory structural performance during T years in service means that the structural resistance is greater than the maximum load effect over this period of time. The distribution function of the structural resistance at time T can be updated based on the knowledge, that the used structure is a proven structure and, if undamaged, in many respects better then a new structure. This does not consider deterioration, but can be applied to many structures in early and useful life (i.e. when determination is controlled or not yet significant). In the value of the service load there is often uncertainty, but the load variance can be estimated from data on existing or past loads from inspection or monitoring [12].

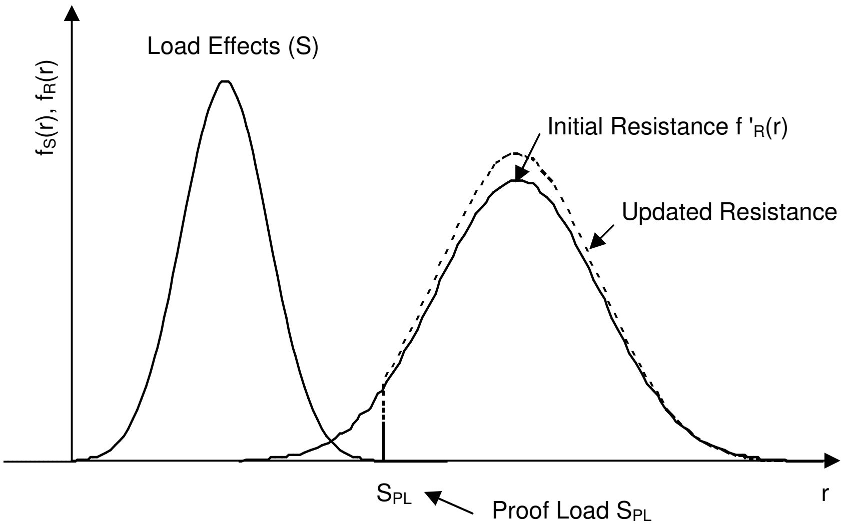

Fig.9: Effects of Proof Load on Distribution of Structural Resistance

4.3.4 Reliability of structural systems

Generally, a structure as a system consists of more than one single failure mode element. Systems behaviour has to be examined, since systems failure is usually the most serious consequence of structural failure. It is therefore of interest to assess the probability of failure of a system following initial element failure. In particular, it is necessary to determine the systems characteristics in relation to damage tolerance or structural integrity with respect to accidental events. A systems analysis may therefore be carried out to establish redundancy and the state and complexity of the structure (multiple-failure modes) [1].

The real structure needs to be modelled, by an equivalent system in such a way, that all relevant failure modes can be treated. The fundamental systems are series systems and parallel systems. In practice, usually mixed systems are found.

Series Systems:

In a series system the individual elements are connected in series in regard to their function. The failure of a single element causes the failure of the entire system ("weakest link"). Statically, determinate systems are series systems. If the elements are brittle, the system fails by element fracture; if the elements are ductile the system fails by excessive yielding. The failure probability can be calculated by "union" of the limit states of all system elements.

Parallel Systems:

In a parallel system the individual elements are connected in parallel in regard to their function. Only the failure of all elements causes the failure of the entire system. Statically indeterminate systems are parallel systems because of their redundancy, if the elements are sufficient ductile. Does a parallel system contain out of ideal brittle elements it may also fail as series system. The failure probability can be calculated by "intersection" of the limit states of all system elements.

4.3.5 Structure-specific safety characteristics

To avoid unnecessary strengthening due to over-conservative assessment it is essential to provide a consistent safety level which is adapted on structural specifics for all limit states being under observation. The target reliability is considered as such control parameter based on optimisation.

Target reliabilities depend on the analysed limit state class (ultimate, fatigue, serviceability) and on the reference time period. Parameters, affecting the target reliability are loading history, consequences of failure, reserve strength and redundancy and warning of failure by inspection and monitoring. They are described more detailed in 3.2.2.

Recommended target reliabilities for the assessment of existing structures are listed in Annex B.

4.4 Special cases of assessment

Previous design code based assessment

For older structures or details it might be necessary to consult previous design standards. The results should be reviewed using modern methods and up-to-date knowledge, and not be accepted unreserved [4].

Satisfactory past performance based assessment

Structures designed and constructed based on earlier codes, or designed and constructed in accordance with good construction practice when no codes has been applied, may be considered safe to resist loads other than accidental loads (including earthquake) in order to ISO 13822 provided that:

- careful inspection does not reveal any evidence of significant damage, distress or deterioration,
- the structural system is reviewed, including investigation of critical details and checking them for stress transfer,
- predicted deterioration taking into account the present condition and planned maintenance ensures sufficient durability,
- there have been no changes for a sufficiently long period of time that could significantly increase the loads on the structure of affect its durability and no such changes are anticipated,

and may considered serviceable for future use in order to ISO 13822 provided that:

- careful inspection does not reveal any evidence of significant damage, distress or deterioration or displacement,
- the structure has demonstrated satisfactory performance for a sufficiently long period of time for damage, distress, deterioration, displacement or vibration to occur,
- there will be changes to the structure or in its use that could significantly alter the actions including environmental loading on the structure or part thereof,
- predicted deterioration taking into account the present condition and planned maintenance ensures sufficient durability.

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ANNEX A – CLASSIFICATION AND STRUCTURE

The assessment of existing structures can be classified in routines with ascending sophistication. The structure of the three main categories is shown in table Tab.A-1. It needs to be stressed, that the order within one group of procedures must not be followed severe. Neither it is obligatory to apply procedures of the same complexity.

Classes of Assessment	Objectives	Methodology		
NON-FORMAL ASSESSMENT (Level 0)	qualitative condition assessment	Visual Inspections Experience Based Graduation		
MEASUREMENT-BASED ASSESSMENT (Level 1)	quantitative serviceability control and assessment	Determination of Load Effects: Measurement of Performance Values under Service Loading		Verification: Comparison with Threshold Values
MODEL-BASED ASSESSMENT	quantitative safety and serviceability assessment	Determination of Load Effects: Data Acquisition	Determination of Load Effects: Structural Analysis	Verification
(Level 2)		Document Review Inspections	Basic Structural Models	Deterministic (Permissible Stress) -- only exceptional -- Semi-Probabilistic (Partial Safety Factors)
(Level 3)		Monitoring of Static Load Effects and Deterioration (Deformations, Stresses, Cracks, Corrosion etc.) Monitoring of Live Load and Environmental Influences	Refined Models (FEM, Nonlinear Analysis)	
(Level 4)		Testing and Measurement of Material Properties and Dimensions	Adaptive FE – Models	Probabilistic Approximation Methods (FORM, SORM)
(Level 5)		Monitoring of Dynamic Load Effects (Eigenfrequencies, Mode Shapes)	Stochastic FE – Models	Probabilistic Simulation Methods (MCS, LHC)

Table A.1: Classification and Structure of the Assessment Process

A.1 Brief Abstract of Assessment Levels

A.1.1 Level 1: Measurement Based Assessment

OBJECTIVES
- Control of the performance of the structure over a certain time period - Control of adherence of serviceability and fatigue limit values (deformation, stress, stress range) - Control of variable loads and influences

DETERMINATION OF LOAD EFFECTS
- Measurement of performance values like deformations, stresses and dynamic values

RELIABILITY VERIFICATION
- Comparison of Measured Data with Threshold Values - Analysis of Trends and Correlations with External Influences

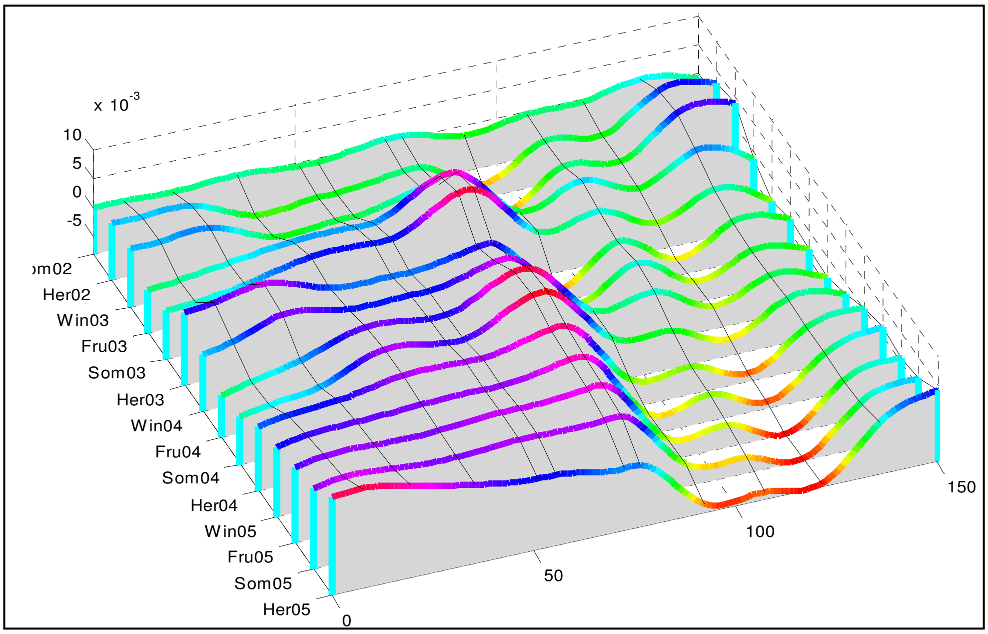


Fig.A1: Direct Assessment of Monitored Vertical Movements of a Railway bridge Deck during Construction of the Surrounding Station at Lehrter Bahnhof in Berlin

A.1.2 Level 2: Basic Model Based Assessment

OBJECTIVES
- Validation of safety and serviceability after extreme load damage or after deterioration damage - Determination of safety and serviceability for utility changes

DATA ACQUISITION
- Load and resistance data from design documents and codes - Inspections

STRUCTURAL ANALYSIS
- Methods and models as used in the design process - Refined models

RELIABILITY VERIFICATION
- Deterministic - Semi probabilistic

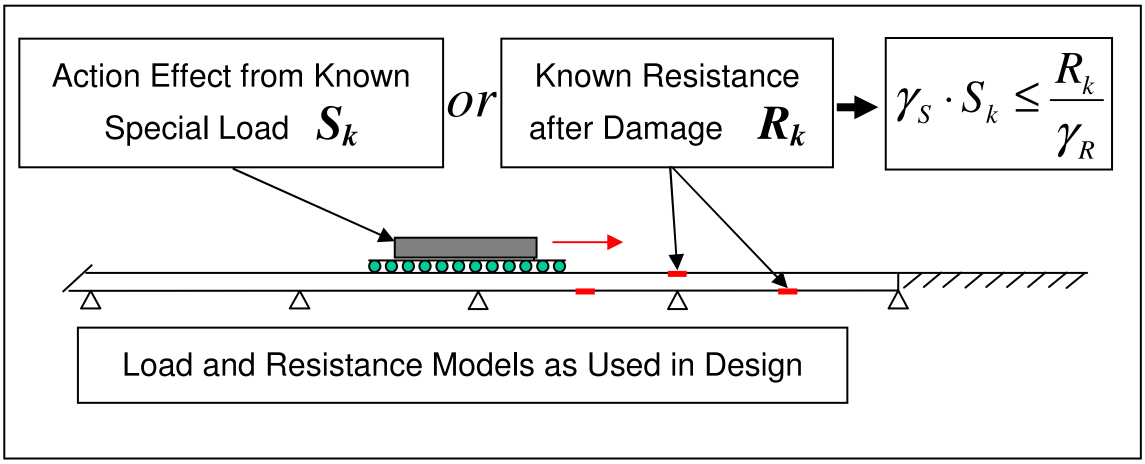


Fig.A2: Simplified Example for Level 2 Assessment

A.1.3 Level 3: Advanced Model Based Assessment

OBJECTIVES
- Determination of load carrying capacity and remaining life time of damaged structures

DATA ACQUISITION
- Investigation on Material Properties and Dimensions (NDT)
- Monitoring based System Identification
- Load Monitoring
- Proof Load Tests

STRUCTURAL ANALYSIS
- Refined Methods and Models (FEM, Nonlinear Analysis)
- Adaptive Models

RELIABILITY VERIFICATION
- Semi probabilistic

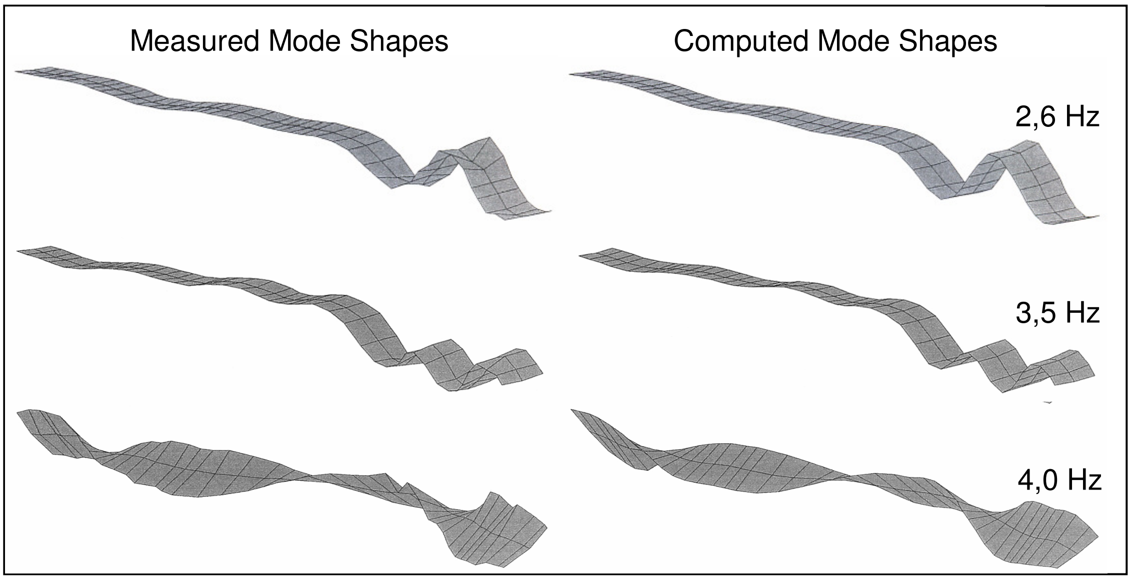


Fig.A3: System Identification and Model Improvement at the Deck of Westend Bridge in Berlin

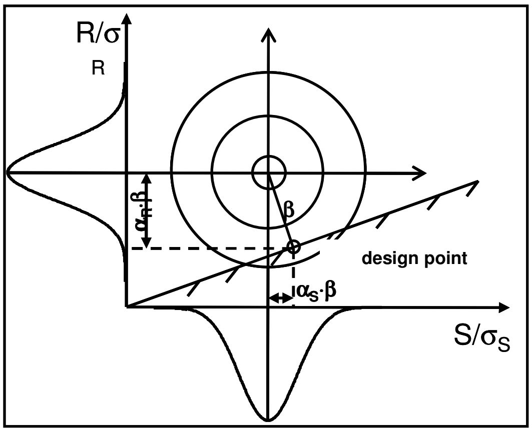
A.1.4 Level 4: Assessment with Modified Target reliabilities

OBJECTIVES
- Adapting of the target safety level in dependence of the consequences of a structural failure, which itself depends on the utility of the structure, redundancy and the failure characteristic.

DATA ACQUISITION
- Investigation on Material Properties and Dimensions (NDT)
- Monitoring Based System Identification
- Load Monitoring
- Proof Load Tests

STRUCTURAL ANALYSIS
- Refined Methods and Models (FEM, Nonlinear Analysis)
- Adaptive Models

RELIABILITY VERIFICATION
- Semi probabilistic



$$\gamma_S = \frac{S_d}{S_k} = \frac{1+\beta\cdot\alpha_S\cdot v_S}{1+k_S\cdot v_S}$$

$$\gamma_R = \frac{R_k}{R_d} = \frac{1-k_R\cdot v_R}{1-\beta\cdot\alpha_R\cdot v_R}$$

Fig.A4: Illustration of the Safety Index ß in the Standard Normal Space and Description of the Partial Safety Factors as Function of ß for Normal Distributed Values

A.1.5 Level 5: Full Probability Assessment

OBJECTIVES
- Determination of load carrying capacity and remaining life time of damaged structures under consideration of existing uncertainties

DATA ACQUISITION
- Investigation on Material Properties and Dimensions (NDT)
- Monitoring based System Identification
- Load Monitoring
- Proof Load Tests
- Statistical Characteristic of Data

STRUCTURAL ANALYSIS
- Simple Models and Methods
- Advanced Models and Methods

RELIABILITY VERIFICATION
- Probabilistic approximation methods (FORM, SORM)
- Simulation Methods (MCS)

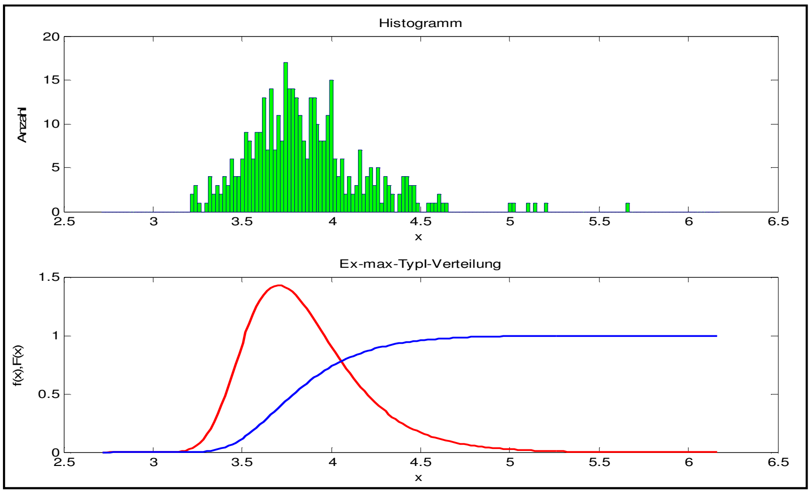


Fig.A5: Histogram and Distribution of Live Load (Simulation)

ANNEX B - METHODS FOR CALCULATION OF FAILURE PROBABILISTIES

Within probabilistic verification procedures it is necessary to calculate the convolution integral that takes on the following form

$$P_f = \int_{g(\boldsymbol{x}) \leq 0} f_X(\boldsymbol{x}) d\boldsymbol{x}$$

where $f_X(\boldsymbol{x})$ is the joint density function of the n random variables.

Several methods have been developed to compute P_f, e.g. exact analytical methods, numerical integration, approximate analytical methods and simulation methods or a combination these methods. In most cases P_f cannot be evaluated by exact analytical means and other procedures have to be employed, such as approximate methods (Level 2 methods) and simulation methods (Level 3 methods). They are described in this Annex.

B.1 Simulation methods (Level 3 - methods)

Simulation methods feature generality, simplicity and effectiveness on problems where the limit state equation is highly non-linear. The most serious drawback is the computation effort, especially, when the reliability level is high [9].

Monte Carlo Simulation:

With Monte Carlo Simulation (MCS), the probability density functions and the associated statistical parameters for all variables are defined and than random sampling, using a random number generator, is employed to obtain an outcome of the random vector. It is then checked for this set of values, whether the realisation lies within the failure domain or within the safety domain. This procedure is repeated many times and the probability of failure P_f is determined directly by:

$$P_f = \frac{N_{g(\boldsymbol{x}) \leq 0}}{N}$$

where $N_{g(\boldsymbol{x}) \leq 0}$ denotes the fraction of realisations leading to failure and N the total number of realisations. Because of the large number of samples, required in order to determine low failure probabilities, it is not likely be used in practical assessment of existing structures [5].

Importance Sampling:

Important Sampling methods reduce the number of samples required for a simulation based probabilistic analysis.

A widely employed importance sampling approach is to move the sampling centre from the origin in standard Gaussian space to the design point on the limit state surface (most likely failure point). About half the sample will than be located in the failure domain. The failure probability is calculated from the sum of the weight factors of all the realizations for which a failure occurs as

$$P_f = \frac{1}{N}\sum_{i=i}^{N}\left(W(\tilde{x}_i)\middle| g(\tilde{x}_i) \le 0\right)$$

where N is the number of realisations, x_i is a realisation for the importance sampling distribution, $g(x_i) \le 0$ indicates failure for realisation i and $W(x_i)$ is the corresponding weight factor calculated as

$$W(\tilde{x}_i) = \frac{\Phi_u(x_i)}{h_v(\tilde{x}_i)}$$

where Φ_u is the joint probability density function of the set u of stochastic variables u_j in standard Gaussian space, h_v is the employed importance sampling density function.

The most straight forward sampling density is the multivariate Gaussian distribution with standard deviations equal to one, but other sampling distributions and other sampling centres might be even more efficient. Within Adaptive Importance Sampling the sampling centre is moved according to information from a previous sampling. This approach is less efficient than the gradient methods, but does not require transformation into the standardised Gaussian space. Other methods, here not described, are the Radial IS, the Axis Orthogonal IS and the Directional IS.

<u>Latin Hypercube Sampling:</u>

The constrained Monte Carlo simulation called Latin Hypercube Sampling (LHS) is a highly recommended technique. It allows a small number of simulations under achieving of an acceptable accuracy. LHS is a special type of Monte Carlo simulation, which uses the stratification of the theoretical probability estimation of the first two or three statistical moments of structural response. It requires a relatively small number of simulations – repetitive calculations of the structural response resulting from adopted computational model (tens or hundreds). The utilisation of LHS strategy in reliability analysis can be rather broad and it is not restricted just to the estimation of statistical parameters of structural response. [10]

In LHS the region between 0 and 1 is uniformly divided into N non-overlapping intervals for each random variable; where N is the number of random numbers which need to be generated for each random variable, i.e., number of simulation cycles. The cumulative probability distribution function for all random variables are divided into N equivalent intervals (N = number of simulations), centroids or randomly chosen values within the interval are then used in the simulation process. This means, that the range of the probability distribution function $\Phi(x_i)$ of each random variable x is divided into N intervals of equal probability 1/N. The representative parameters of variables are selected randomly based on random permutations of integers 1,2,…N. Every interval of each variable must be used only once during the simulation. By repetitive calculations of the response function a set of response variables is obtained and evaluated by simple statistics. In case of Crude Monte Carlo Simulation such a process can be quite time consuming as thousands of simulations are needed for acceptable good estimations. Using a smaller number of simulations, LHS results in satisfactory estimates of basic statistical parameters of response. It is valid mainly for the first and second statistical moments, but quite good results can also be obtained for the third moment (skewness) [10].

B.2 Approximation methods (Level 2 - methods)

Because level 3 methods can be very time consuming for complex failure mechanisms with many random variables, level 2 methods try to provide quick and reliable approximations.

The most well known methods are the First Order Reliability Method (FORM) and the Second Order Reliability Method (SORM).

The first step consists to transform the problem into the standard Gaussian space. That means that all the initial variables are transformed in a set of independent normal random variables with zero mean and unit standard deviation.

In the standardised space, the nearest point from the origin to the transformed limit state surface $g_u(u) \leq 0$ is called *design point* and the distance from the design point to the origin is noted *reliability index* β. The location of the design point by a suitable search algorithm is the main task. It can for example be determined by iteration procedures.

The approximation of the limit state surface at the design point could be linear (FORM approximation) or with second order terms (SORM) methods. In the FORM approach, the failure surface $g_u(u) \leq 0$ is approximated by a tangent hyperplane at the design point. The probability of failure is then approximated by:

$$P_f = \Phi(-\beta) = 1 - \Phi(\beta)$$

where Φ is the probability function of the standard normal variable. In the SORM approach, the failure surface is approximated by a hyperparaboloid which crosses the design point and which has the same curvature.

The probability of failure is given by:

$$P_f = \Phi(-\beta)\prod_{i=1}^{n}(1-\kappa_i\beta)^{-1/2}$$

where κ_i is the different individual curvatures at the design point.

ANNEX C - TARGET RELIABILITY LEVELS

The required level of structural performance, the target reliability level in the ultimate limit state as well as in the fatigue and in the serviceability limit state is constituted as *target failure probability* and *target reliability index*.

The remaining working life determined at the assessment is considered as a reference period of an existing structure for serviceability and fatigue, while the whole design working life is often considered as a reference period for a new structure. A shorter reference period might be reasonable for the ultimate limit state. The target reliability indices may be chosen in accordance with current codes, if such are provided, otherwise the values given in Table C.1 are intended as illustrations for assessment of existing structures [2].

Limit States	ß	Reference period
Serviceability		
reversible	0,0	remaining working life
irreversible	1,5	remaining working life
Fatigue		
can be inspected	2,3	remaining working life
cannot be inspected	3,1	remaining working life
Ultimate		
very low consequences of failure	2,3	design working life (e.g. 50 years)
low consequences of failure	3,1	design working life (e.g. 50 years)
medium consequences of failure	3,8	design working life (e.g. 50 years)
high consequences of failure	4,3	design working life (e.g. 50 years)

Table C.1: Target reliability indices for assessment of existing structures (ISO 13822)

This numbers have been derived with the assumption of lognormal or Weibull models for resistance, Gaussian models for permanent loads and Gumbel extreme value models for time-varying loads. It is important that the same assumptions (or assumptions close to them) are used if the values given in table C.1 are applied for probabilistic calculations [1].

The relationship between the reliability index β and the probability of failure P_f is defined by $\beta = \Phi^{-1}(P_f)$ and given in table C.2.

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1,3	2,3	3,1	3,7	4,2	4,7	5,2

Table C.2: Relationship between β and P_f

Finally, it should be stressed, that a β-value and the corresponding failure probability are formal or notional numbers, intended primarily as a tool for developing consistent design rules, rather that giving a description of the structural failure frequency.

ANNEX D - PROCEDURES FOR UPDATING OF MEASURED QUANTITIES

Updating procedures can be used to derive updated probabilities of failure or characteristics and representative values of basic variables to be used in probability based assessment, partial factor based methods or to compare directly action effects with limit values (cracks, displacements).

D.1 Evaluation of probabilistic measures

Given the result of an investigation, there is a need to update the properties and reliability estimates of the structure. Two different procedures can be distinguished:

1. Direct updating of the structural failure probability.
2. Updating of the individual or multivariate probability distribution of the basic variables.

D.1.1 Direct updating of the failure probability

Direct updating of the structural reliability can formally be carried out using Bayesian theorem:

$$P_f^{''} = P(F \mid I) = \frac{P(F \cap I)}{P(I)} \tag{D.1}$$

where:

F denotes local or global structural failure;

I stands for the information gathered by investigation;

∩ means the intersection of two events;

| means conditional upon.

Is the limit state function is $g(\boldsymbol{x})$, where x is the vector of basic variables and failure F described by the inequality $g(x) < 0$. If the result of an investigation I is an event described by the inequality $H > 0$ then the updated probability of failure may be written as:

$$P_f^{''} = P(g(x) < 0 \mid H > 0) = \frac{P(g(x) < 0 \cap H > 0)}{P(H > 0)} \tag{D.2}$$

D.1.2 Updating of probability distributions

The updating procedure of the individual or the multivariate probability distributions method is given formally by:

$$f_x(X|I) = C \cdot P(I|x) \cdot f_x(X) \tag{D.3}$$

where

$f_X(X|I)$ denotes the updated probability density function of X after updating with information I;

X a basic variable or statistical parameter;

$f_X(X)$ the prior probability density function of x;

$P(I|x)$ the so called likelihood function (likelihood of finding information I for given value x of X) – it may also be written as $L(x|I)$;

C is the normalising factor.

Once the updated distribution for the basic variables $f_X(X|I)$ have been found, the updated failure probability $P(F|I)$ may be determined by performing a probabilistic analysis using common methods of structural reliability for new structures:

$$P(F|I) = \int_{g(x)\leq 0} f_X(X|I)dx \tag{D.4}$$

D.2 Evaluation of characteristic and design values

A practical method for processing new information about load and resistance, obtained i.e. by inspection, proof loading or structural health monitoring is the updating of the characteristic or design values. For updating resistance variables two general approaches can be used [1]:

1. Evaluation based on updated probability distributions
2. Direct evaluation of test results

D.2.1 Evaluation based on updated probability distributions

Once the probability distribution of the basic variable under consideration is updated as described in D.1.2, characteristic and design value can be determined as follows (ISO 13822):

Characteristic values:

For normal distributed variables of resistance parameters the value can be estimated by:

$$x_k = \mu - k\sigma \tag{D.5}$$

and for lognormal distributed variables of resistance parameters the value can be estimated by:

$$x_k = \mu \cdot exp(-k\sigma - 0,5k\sigma^2) \tag{D.6}$$

where

x_k is the updated characteristic value

μ the updated mean value,

σ the updated standard deviation

k coefficient, usually k = 1,64

Design values:

The design value can be determined by:

a) applying the partial safety factor γ_M

$$x_d = x_k / \gamma_M \tag{D.7}$$

or

b) applying the target reliability β

$$x_d = \mu\text{-}\alpha\beta\sigma \quad \text{(normal random variable)} \tag{D.8}$$

$$x_d = \mu \cdot exp(-\alpha\beta\sigma - 0,5\alpha\beta\sigma^2) \quad \text{(log normal random variable)} \tag{D.9}$$

where

x_d is the updated design value

α the probabilistic influence coefficient, usually 0,8 for dominating resistance parameter and 0,8*0,4 = 0,32 for other resistance parameters

σ the sample standard deviation

It may be helpful to consider both methods and to use the most conservative result. The procedure may be applied for loads and geomechanical properties, but usually other distribution types will be more appropriate.

D.2.2 Direct evaluation of test results

Resistance of a structural element or the strength of a material can directly be evaluated from tests and associated with partial factor based safety verification, the characteristic or design values can be obtained by two approaches (ISO 2394):

1. Classical approach
2. Bayesian approach

D.2.2.1 Classical approach for characteristic and design values

The characteristic value x_k is estimated from the test results, taking into account a confidence level of at least 0,75. In the absence of other information, the characteristic value is assumed to be the 0,05 fractile of a normal distribution.

For normal distributed variables of resistance parameters the value can be estimated by:

$$x_k = m - ks \tag{D.10}$$

x_k is the updated characteristic value

m the sample mean value,

s the sample standard deviation

k coefficient, depending on the sample size, the confidence level, the probability value (fractile) and whether the prior standard deviation is known or unknown, (k-values should be taken from table D1 or ISO 12491).

Probability	Number of tests, n								
P	3	4	6	8	10	20	30	100	∞
0,10	2,50	2,13	1,86	1,74	1,67	1,53	1,47	1,38	1,28
0,05	3,15	2,68	2,34	2,19	2,10	1,93	1,87	1,76	1,64
0,01	4,40	3,73	3,24	3,04	2,93	2,70	2,61	2,46	2,33

Table D1: Values of k for normal distribution, unknown σ and confidence level = 0,75

Probability	Number of tests, n								
P	3	4	6	8	10	20	30	100	∞
0,10	1,67	1,62	1,56	1,52	1,50	1,43	1,40	1,35	1,28
0,05	2,03	1,98	1,92	1,88	1,86	1,79	1,77	1,71	1,64
0,01	2,72	2,66	2,60	2,56	2,54	2,48	2,45	2,39	2,33

Table D2: Values of k for normal distribution, known σ and confidence level = 0,75

D.2.2.2Bayesian method

In the Bayesian method the characteristic value may be estimated for normal distributed resistance parameters directly from test data.

$$x_k = m - t_{vd} \cdot s\sqrt{\left(1 + \frac{1}{n}\right)} \tag{D.11}$$

where

n number of samples

m mean value

s standard deviation

t_{vd} the coefficient of the Student distribution depend on the distribution form, the sample size, the probability of occurrence and whether the prior standard deviation is known or unknown, (t_{vd} -values should be taken from table D3)

Probability	Degrees of freedom, ν = *n - 1*							
Φ (-β)	1	2	3	5	7	10	20	30	∞
0,10	3,08	1,89	1,64	1,48	1,42	1,37	1,33	1,31	1,28
0,05	6,31	2,92	2,35	2,02	1,89	1,81	1,72	1,70	1,64
0,01	31,8	6,97	4,54	3,37	3,00	2,76	2,53	2,46	2,33
0,005	63,7	9,93	5,84	4,03	2,50	3,17	2,84	2,75	2,58
0,001	318	22,33	10,21	5,89	4,78	4,14	3,55	3,38	3,09

Table D3: Values of t_v

If the standard deviation σ is known (e.g. from the past experience), then $\nu = \infty$ and s should be replaced by σ.

Example 1 *[13]:*

As an example consider a sample of $n = 6$ concrete strength measurements having the mean $m = 37{,}5$ N/mm² and a standard deviation $s = 4{,}7$ N/mm², which is to be used for assessment of the characteristic value of the concrete strength $f_{ck} = x_k$ with $p = 0{,}005$. If no prior information is available, then $n' = \nu' = 0$ and the updating characteristics m'', n'', s'', ν'' equal the sample characteristics m, n, s, ν. The predictive value of x_k then follows from (D.11):

$$x_k = 37{,}5 - 2{,}02 \cdot 4{,}7\sqrt{\left(1 + \frac{1}{6}\right)} = 27{,}5 \quad N/mm^2$$

where the value $t_p = 2{,}02$ is taken from Table D.3 for $p = 0{,}05$ and $\nu = n - 1 = 6 - 1 = 5$.

Use of prior information:

When prior information about the basic variable is available (e.g. from earlier test series) the prior (normal) distribution is given by:

$$f'(\mu,\sigma) = k\sigma^{-(\nu'+\delta\{n'\}+1} exp\left(-\frac{1}{2\sigma^2}\left(\nu'(s)^2 + n'(\mu - m')^2\right)\right) \quad \text{(D.12)}$$

where

$\delta(n') = 0$ for $n' = 0$ (D.13)

$\delta(n') = 1$ for $n' > 0$ (D.14)

$E(\sigma) = s'$ (D.15)

$V(\sigma) = 1/\sqrt{2\nu'}$ (D.16)

$E(\mu) = m'$ (D.17)

$V(\mu) = s'/(m'\sqrt{n'})$ (D.18)

The equation (D.11) for determining the characteristic value changes then to

$$x_k = m'' - t_{\upsilon''} \cdot s'' \sqrt{\left(1 + \frac{1}{n''}\right)} \quad \text{(D.19)}$$

with

$n'' = n' + n$ (D.20)

$\nu'' = \nu' + \nu + \delta(n')$ (D.21)

$n''n'' = n'm' + nm$ (D.22)

$[\nu''(s'')^2 + n''(m'')^2] = [\nu'(s')^2 + n'(m')^2]\ [\nu s^2 + nm^2]$ (D.23)

Example 2 *[13]:*

Consider once more example 1, but assume previous test series have shown that:

m' = 40,1 N/mm², V(m') = 0,5 , s' = 4,4 N/mm² and V(s) = 0,28.

It follows from equation (D16) and (D.18) that:

$$n' = \left(\frac{4,4}{40,1} \frac{1}{0,5} \right) < 1 \qquad \nu' = \left(\frac{1}{2} \frac{1}{0,28^2} \right) \approx 6$$

It is therefore considered that n' = 0 and ν' = 6. Taking into account, that ν = n - 1 = 5,

(D.20) leads to n'' = 6;

(D.21) to ν'' = 11;

(D.22) to m'' = 37,5 N/mm² ;

(D.23) to 11(s'')²+6(37,5)² = 6(4,4)²+0(40,1)² + 5(4,7)²+6(37,5)²

→ s'' = 4,5 N/mm²

It follows finally from equation (D.19), that:

$$x_k = 37,5 - 1,80 \cdot 4,5 \sqrt{\left(1 + \frac{1}{6} \right)} = 28,8 \;\; N/mm^2$$

In the above example the resulting characteristic concrete strength is greater (by about 5%) than the value obtained by prediction method without using prior information. Thus, when previous information is available Bayesian approach may improve the fractile estimate, particularly in the case of a great variance of the investigated variable. However, the origin of prior information should be always critically verified in order to affirm the identical nature of prior and observed data [13].