

Condition assessment and inspection of steel railway bridges, including stress measurements in riveted, bolted and welded structures

Background document SB3.4



PRIORITY 6

SUSTAINABLE DEVELOPMENT
GLOBAL CHANGE & ECOSYSTEMS
INTEGRATED PROJECT

This report is one of the deliverables from the Integrated Research Project "Sustainable Bridges - Assessment for Future Traffic Demands and Longer Lives" funded by the European Commission within 6th Framework Programme. The Project aims to help European railways to meet increasing transportation demands, which can only be accommodated on the existing railway network by allowing the passage of heavier freight trains and faster passenger trains. This requires that the existing bridges within the network have to be upgraded without causing unnecessary disruption to the carriage of goods and passengers, and without compromising the safety and economy of the railways.

A consortium, consisting of 32 partners drawn from railway bridge owners, consultants, contractors, research institutes and universities, has carried out the Project, which has a gross budget of more than 10 million Euros. The European Commission has provided substantial funding, with the balancing funding has been coming from the Project partners. Skanska Sverige AB has provided the overall co-ordination of the Project, whilst Luleå Technical University has undertaken the scientific leadership.

The Project has developed improved procedures and methods for inspection, testing, monitoring and condition assessment, of railway bridges. Furthermore, it has developed advanced methodologies for assessing the safe carrying capacity of bridges and better engineering solutions for repair and strengthening of bridges that are found to be in need of attention.

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Figure on the front page: Photos of Hohenzollern-bridge in Cologne, Germany, and of some hot rivet details from Hochdonn-bridge north of Hamburg, Germany and Köln Hauptbahnhof, Cologne, Germany.

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1 Introduction

1.1 General remarks

This technical report is prepared on the basis of Contract No. FP6-PLT-001653 between the European Community represented by the Commission of the European Communities and Skanska Teknik AB contractor acting as coordinator of the Consortium.

Bridges are important connection elements in the European railway network. The project "Sustainable bridges" is focusing on European railway bridges. To improve the transport capacity as well as to enhance the residual service life of existing bridges requires extensive and advanced investigation. In work-package WP3 condition assessment and inspection methods will be developed, based on a wide experience on measurements in civil engineering and new research results. A catalogue with maintenance problems compiled among all European railway companies pinpoints the demand of better inspection and diagnostic tools. A considerable amount of information about the actual conditions of all components of a bridge is required.

The objective of this report is related to work package WP3 "Condition Assessment and Inspection", specifically to deliverable SB3.4 "Report on investigation on steel bridges/elements, including stress measurements in riveted, bolted and welded structures". Condition assessment and inspection methods for other bridge types will be the subject of other deliverables within the project.

At RWTH, condition assessment, inspection and monitoring as well as strengthening techniques for in-service steel railway bridges are main subjects of the consulting and research activities since many years. A lot of experience has been gained on applicability and limitations of the mentioned methods. Another main topic of RWTH is the work on and development of the harmonized European design standards (Eurocodes) in particular according steel railway bridges and of the harmonized CEN-standards for steel railway bridge execution and condition assessment. The results and experiences of several previous and ongoing research projects are processed in this report and are the basis for evaluation of guidelines and further developments to be performed in "Sustainable Bridges". Also the European state-of-the-art in condition assessment and inspection of steel railway bridges are presented in this report for "Sustainable Bridges" purposes, showing the efficiency of various condition assessment and measurement strategies which are customized to meet the demands of the railway authorities. For a list of typical damage types of old steel railway bridges refer also to the WP3-Deliverable SB3.3, for further condition assessment methods to Deliverable SB3.2; WP5-Deliverable SB5.2.1 [1] presents guidelines for monitoring of steel railway bridges including the health-monitoring concept.

1.2 Work objectives

The presented report gives guidance for the application of condition assessment and inspection methods of steel railway bridges as a basis for future activities within the "Sustainable Bridges" project.

According to the task in the work package WP3.4 results are presented according to the following objectives:

- Overview of condition assessment and inspection methods for members, bolts, welds, rivets and surface protection
- Guidance for necessity, choice and execution of stress/strain measurement methods of components and structural connection elements

- Criteria for necessity, choice and execution of strengthening or replacement measures of components and connections depending of the grade of deterioration
- Acceptance criteria required for Fitness-for-Service procedures
- Experimental and analytical research to evaluate replacement criteria for hot rivets
- Finally, the practical application of condition assessment for old steel railway bridges, presented for two examples using field measurements

Utilization of the provided results will help to improve the sustainability, maintenance and safeguarding procedures for old in-service steel railway bridges and may allow for an extension of the remaining service life in several cases.

1.3 Summary

The objective of this report is related to work package WP3 “Condition Assessment and Inspection”, deliverable SB3.4 “Report on investigation on steel bridges/elements, including stress measurements in riveted, bolted and welded structures”.

Of particular interest for the application of condition assessment of existing steel railway bridges are the procedures which are given by

- BS 7910 [2]
- SINTAP-Procedure [3]
- Ril 805 of Deutsche Bahn [4]
- EN 1090-2 [5]

Conventional non-destructive testing methods involve an element of subjective judgement and the output of the testing is considered to be an evaluation and not a measurement (even though figures may be reported). The evaluation has two final outcomes: Accepted or not accepted.

Table 1. Outcomes of conventional inspections

Result of inspection	Structure	
	Safe	Unsafe
Accepted	OK	Customer's and society's risk
Rejected	Producer's risk	OK (but unwanted)

ECA - Engineering Critical assessment - is a designation for methods used for the assessment of the acceptability of imperfections. These ECA methods are also recommended for the assessment of steel railway bridges. Application of ECA for assessment of crack growth, corrosion, wear and tear and other deterioration detected during in-service inspection is a well-established practice. In service inspection is facilitated by the fact that deterioration usually results in well-defined imperfections of a single type and often localised (e.g. fatigue cracks). This permits application of special procedures for non-destructive testing, able to give quantitative information on the size of the imperfections. The deterioration may be monitored during a number of inspections in order to follow the growth of cracks, the progress of corrosion, etc.

The ECA-based SINTAP procedure is the product of a three-year Brite-Euram programme involving seventeen partners from nine European countries. The method represents a current European consensus on structural integrity assessment and offers several distinct advantages over current approaches. In the context of a standard on fitness-for-purpose, it is therefore eminently suitable for substitution in place of the brittle fracture and plastic collapse clauses of BS 7910. The report CEN/TR TC121/WG14 [6] provides a detailed comparison of the BS 7910 and SINTAP procedures showing clearly the advantages of the SINTAP procedure.

The final draft of the coming execution standard for steel products EN 1090 “Execution of steel structures and aluminium structures”, prepared by CEN/TC 135, has been released for CEN-enquiry in 2005. Part 2 of this standard “Technical requirements for the execution of steel structures” [5], contains the European state-of-the-art for execution, inspection, testing and corrections of steel railway bridges and comprises detailed acceptance criteria as well as requirements for inspection, maintenance and repair. Therefore, for condition assessment of old steel railway bridges the given acceptance criteria are of particular interest. Within this Technical Report SB3.4 the relevant content of this draft standard is elaborated for “Sustainable Bridges” purposes.

One element of this draft standard is a sequential method for bolt tightening inspection. This method is carried out according to the principles in ISO 8422 “Sequential sampling plans for inspection by attributes” [7], the purpose of which being to give rules based on progressive determination of inspection results. ISO 8422 gives two methods for establishing sequential sampling plans: numerical method and graphic method. The graphic method is applied for bolt tightening inspection.

In the graphic method (see Figure 5) the horizontal axis is the number of bolt assemblies inspected and the vertical axis the number of defective assemblies. The lines on the graph define three zones: the acceptance zone, the refusal zone and the indecision zone. As long as the inspection result is in the indecision zone the inspection is continued until the cumulative plot emerges into either the acceptance zone or the refusal zone.

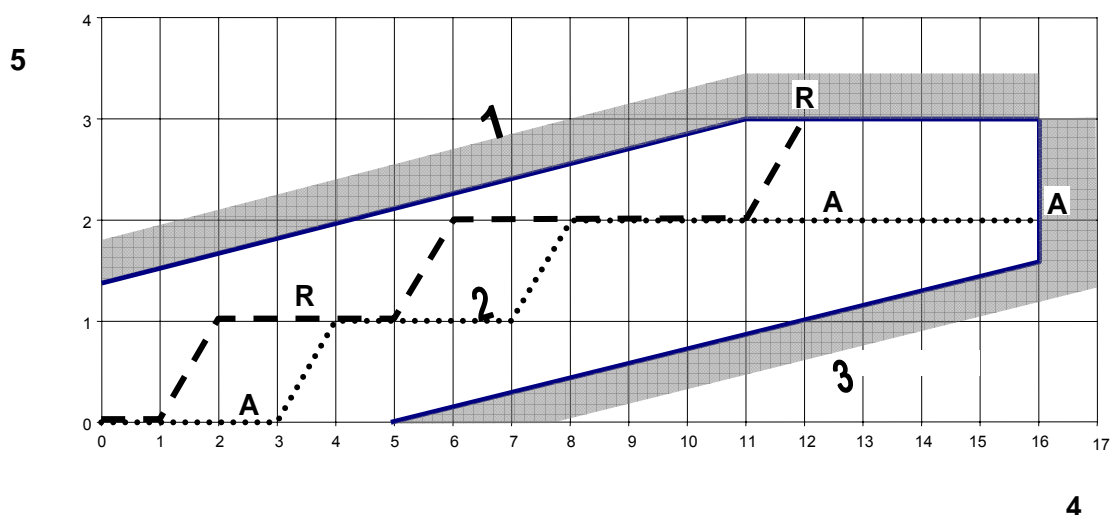


Figure 1 Example of sequential inspection diagram

Also covered by the draft standard prEN 1090-2 is the inspection and repairs of hot rivets in the following way: The number of rivets inspected overall in a structure shall be at least of 5%, with a minimum of 5. Heads of driven rivets shall be visually inspected and shall satisfy the following acceptance criteria (in some cases provisions for detection of non-conformities will not be available):

- The rivet heads shall be centred. The head eccentricity relative to the shank axis shall not exceed $0,15 d_0$ where d_0 is the hole diameter,
- The rivet heads shall be well formed and shall not show cracks or pits,
- The rivets shall be in satisfactory contact with the assembled parts both at the outer surface of the plies and in the hole. No movement or vibration shall be detected when the rivet head is lightly tapped with a hammer.
- A small well-formed and centred lip may be accepted if only a small number of rivets in the group is concerned.
- Outer faces of plies free of indentation by the riveting machine may be specified.

Inspection of satisfactory contact shall be done by lightly ringing the rivet head with a hammer of 0,5 kg. The inspection is carried out in a sequential fashion according to the sequential method for bolt tightening inspection described above to a sufficient number of rivets until either the acceptance or the refusal conditions for the relevant sequential type are met for the relevant criteria.

Replacement criteria are required for defective hot rivets. Defects in riveted connections may originate from fabrication or may be induced during service life by corrosion. Rivets with defects that originate from fabrication usually are not critical, because they have been in service since assembly without any negative effects. On the other hand rivet defects induced by corrosion are of particular concern. Typical fabrication defects of riveted connections are listed.

The influence of rivet deterioration, i.e. rivet head corrosion, on the pre-stress and fatigue effectiveness is investigated. Based on the results of numerical investigation and experimental tests according to the fatigue effectiveness and the rivet head corrosion two limit criteria could be established (ultimate limit of the load-bearing capacity for the rivet head and a serviceability limit for the riveted connection).

The Technical Report SB3.4 contains several stress measurement methods of components and connections of steel railway bridges. As an example, one exceptional method for direct bolt stress measurement is mentioned in the following. This method – rarely used in practical application – is offered by an ultrasonic measurement method, see [8]. This so-called “combined ultrasonic method” was developed by the Fraunhofer-Institute for non-destructive testing in Saarbrücken. Basis of this method is a combined measurement of longitudinal and transversal ultrasonic waves. Although the running distance of both waves varies uniformly with a change in preload, in contrast to that the running time varies differently with a change of bolt tension stresses. In consequence, the bolt strain can be determined directly from a simultaneous measuring of the running time of the longitudinal and transversal waves, even if the initial bolt length is unknown.

According to [9] fatigue assessment has not been carried out in many existing bridges, since fatigue design specifications did not exist at the time of their design and erection. Often when measurements are carried out, they show that stresses are lower than used in assessment. But, in some details, higher values can be measured. Usually these points are not in the main structure, but in secondary elements. The underestimation of stress ranges in some details may result in early fatigue cracking. Fatigue critical details may suffer from fatigue failure as result of secondary effects as bending moments, e.g. in connections. It also results from the interaction between main and secondary system (e.g. cross-beams and lateral wind bracing system, or the influence of the track system on the lateral distribution of the loading).

The objective of measurements of loads or loads effects is to gain information on the real structural system, the static and dynamic loading of the structure in order to reduce the uncertainties associated with the static calculations made in design or made in an assessment. The main areas of possible knowledge improvement can be summarised as follows:

- Verification of the real structural system and system details: type of connection, real bearing conditions, sensitivity to fatigue, etc. The calculation model is to be optimised for recalculation

- Dynamic behaviour of a structure (estimation of dynamic amplification due to traffic and wind)
- Changes in structural response after local damage (e.g. buckling of members after collision)

Within this Technical Report SB3.4 fitness-for-service acceptance criteria are given based on a calculation method for the necessary strengthening measures for members under tension or bending stresses to increase their resistance against crack growth and the required cross section for strengthening of riveted girders.

Finally, two condition assessment examples are presented.

In conclusion, this Technical Report provides the user with a compendium of condition assessment and inspection for steel railway bridges according to the current European state-of-the-art considering the usefulness for practical application. It makes available essential extracts from several recent documents, guidelines and standards, describes novel methods based on recent research results, contains all necessary details to apply the condition assessment and inspection in practice, to choose the adequate inspection tools and to solve maintenance problems, and consequently is a valuable contribution towards the overall objectives of the Sustainable Bridges Project.

2 Condition assessment and inspection methods for steel railway bridges

2.1 Common principles

2.1.1 Introduction

European provisions for assessing imperfections in existing metallic structures like old steel railway bridges are needed to meet the requirements of industry. The technology is being applied by many industries for materials selection, design and fabrication and particularly in-service assessment using existing methods.

This section provides guidance on the selection and application of methods for assessing the significance of imperfections primarily tailored to welded structures and components in steel. Some of the methods may also be applied for other types of metals and for non-welded structures and components. See the publication CEN/TR TC121/WG14, Welding – methods for assessing imperfections in metallic structures [6] for further detail.

Of particular interest for the application of condition assessment of existing steel railway bridges are the procedures which are given by

- BS 7910 [2]
- SINTAP-Procedure [3]
- Ril 805 of Deutsche Bahn [4]
- EN 1090-2 [5]

Experience from the application should, in a few years, provide enhanced technology in the subject and eventually permit standardisation at the European level.

Conventional design procedures involve application of mathematical models such as the theory of elasticity. Actions are described by characteristics such as stress and strain. Resistance described by characteristics such as yield stress and ultimate limit stress. The designer has to assure that the resistance of the structure is adequate, using adequate safety factors, partial coefficients, etc. The mathematical models presuppose a homogenous material.

Many failure modes involve cracks. Failure may originate from a crack and/or failure may propagate (slow or fast) as a crack. Application of the conventional theory of elasticity to a structure with a crack leads to a singularity at the crack tip because the stresses approach infinity. To this should be added that a closer study of the fracture processes shows that inhomogeneities such as grain structure and even the atomic structure may influence the mode of fracture. Conventional design procedures can, for these reasons, not be applied in situations where an analysis of the significance of a crack-like imperfection is necessary and they cannot be applied for an analysis of the propagation of fatigue cracks, creep cracks, stress corrosion cracks, etc.

Alternative methods termed fracture mechanics have been developed in order to model the behaviour of structures containing cracks. Fracture mechanics interpret actions and materials resistance by an alternative set of characteristics such as stress intensity factors, crack tip opening displacement, etc.

Engineering critical assessments use a combination of conventional design procedures and fracture mechanics calculations, depending on the nature of the imperfection and the likely type of failure. General corrosion results for example in a reduction in cross section and may

be analysed by conventional design procedures whereas propagation of fatigue cracks has to be analysed by fracture mechanics methods.

For the purposes of this Technical Report, the following definitions apply, see also [2], [3]:

ECA = Engineering Critical Assessment (ECA) methods for the assessment of the significance of imperfections for the strength and usability of structures

FAD = Failure Assessment Diagram. Combines the analysis of the safety against general yield and fast (brittle) fracture in a single diagram

CDF = Crack Driving Force plot

The following symbols are used to characterise the local stress-strain field around the crack front. They are (usually with indices) used for actions as well as resistance:

K = Stress intensity factor

J = A line or surface integral that encloses the crack front from one crack surface to the other

CTOD = Crack tip opening displacement

2.1.2 Safety considerations

Conventional provisions for acceptance of welded structures

Standards for design and fabrication of welded structures do, as a general rule, include provisions for inspection and testing of the welded joints. The standards usually specify:

1. Acceptance levels for imperfections, normally by reference to a quality level in standards such as EN ISO 5817 [10].
2. Methods for non-destructive testing by reference to the comprehensive system of EN standards for NDT, at least by reference to EN 12062 [11].
3. The amount of testing (100% or examination of only a part of the welds).
4. Procedures for action when non-conformity is detected, typically requirements for repair, re-examination and some supplementary non-destructive testing.
5. Appropriate safety factors.

Conventional non-destructive testing methods involve an element of subjective judgement and the output of the testing is considered to be an evaluation and not a measurement (even though figures may be reported). The evaluation has two final outcomes: Accepted or not accepted.

Table 2. Outcomes of conventional inspections

Result of inspection	Structure	
	Safe	Unsafe
Accepted	OK	Customer's and society's risk
Rejected	Producer's risk	OK (but unwanted)

Experience has shown that the system results in structures characterised by acceptable risks of failure (customer's and society's risk) during lifetime of a structure. The actual risk depends on the nature of the structure and on the failure mode. The acceptable risk for sudden, catastrophic failure may be of the order 10^{-6} or even lower for critical structures. The accept-

able risk of having substantial fatigue cracks prior to expiration of the stipulated life time of the structure may be much higher, for example of the order 10^{-2} .

Engineering Critical assessment (ECA) principles

ECA - Engineering Critical assessment - is a designation for methods used for the assessment of the acceptability of imperfections. ECA methods for the assessment of imperfections have received further support by the EC directive 97/23/EC [12] concerning pressure equipment (PED) which permits such methods as an alternative to conventional methods. These ECA methods are also recommended for the assessment of steel railway bridges.

Assessment of the acceptability involves consideration of:

1. Legal requirements

Legal requirements and/or provisions in the code(s) for the structure in question or contractual requirements may restrict the acceptance. Mandatory acceptance criteria, to be used for fabrication of new structures may e.g. be specified in the code or contract covering the structure.

2. Contractual requirements

The application of ECA methods should be acceptable to the parties concerned in each particular case.

3. Commercial requirements

Costs and market position may influence the benefits or disadvantages of an application

4. Requirements to fabrication.

A key consideration is maintenance of proper quality control.

Application of ECA for new products

Application of ECA as a tool for specification of quality criteria for new structures is feasible in theory but difficult in praxis. ECA is not a panacea for acceptance of bad workmanship.

Application of ECA involves several requirements:

1. Fracture toughness and other relevant materials data for weld metals and base metals have to be determined. This is usually performed as part of the welding procedure qualification. However, strict process control of welding operations is required in order to assure that materials data obtained during procedure testing are truly representative. If not testing of production test plates may be required.
2. The welds have to be inspected by one or more procedures for non-destructive testing able to:
 - Detect all potentially dangerous imperfections.
 - Determine the type of the imperfections, at least to distinguish between planar and non-planar imperfections.
 - Measure imperfection size, position and orientation.
3. All procedures for non-destructive testing have to be validated on representative samples and the inspection uncertainties determined.
4. Safety factors have to be calculated in order to counteract inspection uncertainties and other uncertainties. This may involve application of advanced probabilistic methods.
5. Acceptance criteria, extents of testing and other quality criteria have to be specified.

The common procedure for measurement of imperfections buried in the weld metal is the ultrasonic TOFD technique, standardised in prCEN/TS 14751 [13]. TOFD requires a high quality of the weld metal as such. Porosity and slag inclusions may mask more serious imperfections and lead to a very comprehensive work of analysis. E.g. level B according to e.g. EN ISO 5817 [10] (or even better) may be required for slag and porosity. The requirements as regards scattered imperfections needed for application of TOFD may actually be more restrictive than the conventional acceptance criteria.

The principles for evaluation of a measurement of dimensions are specified in EN ISO 14253-1 [14]. A measured value Y is associated with a measuring uncertainty U . U is usually determined from the measurement standard deviation multiplied by a safety factor. The real value may be any value in the interval $Y \pm U$ (with a confidence determined by the safety factor). For a largest acceptable imperfection A , the acceptance limit consequently becomes:

$$Y + U < A$$

This illustrates the key role of the uncertainty U . ECA is impossible without reliable information on the uncertainties involved in detection, sizing, identification, etc of imperfections. It should be noted that the present version of prCEN/TS 14751 [13] does not include provisions on determination of uncertainties.

Specification of adequate safety factors or partial coefficients may be a problem for many reasons:

1. Standards for design and fabrication of structures and products rarely specify safety factors or partial coefficients for ECA. Safety considerations are covered in an annex to BS 7910 [2], Annex K: Reliability, partial safety factors, number of tests and reserve factors. However, compatibility with the relevant design code shall be assured.
2. It should be noted that each large imperfection represents a possible place for initiation of a fracture. The risk of fracture therefore depends on the number of imperfections. The accumulated risk of overlooking or misjudging the size of at least one large imperfection depends, therefore, also on the number of imperfections. The size of appropriate safety factors may thus depend on the results of the examination. This is one reason why application of probabilistic methods may be necessary.

ECA may, however, be used for new structures in certain cases. One case, where the application may be most useful, is for design of fatigue loaded structures. Fatigue cracks are likely to initiate at the edge of welds in areas characterised by high structural stress concentrations. Finite element analysis + ECA may be the only alternative to full-scale fatigue testing in such cases. Inspection uncertainties are of less importance because the safety mainly depends on visual examination of weld surface quality and in particular the occurrence of undercut in the critical areas. Non-conformity may be removed by grinding. Specification of safety factors is also comparatively simple when growth of fatigue cracks is the dominating failure mode.

Another case involves intentional introduction of "imperfections" in the welds during the design phase. Conventional codes, standards and recommendations often require butt welds to be fully penetrated. However, ECA may document that double-V butt welds with intentional central lack of penetration may be acceptable in many cases, not least when the welds mainly are exposed to bending stresses. Conventional criteria and inspection procedures may then be used for inspection of the quality of the weld metal. A supplementary examination by TOFD permits determination of the height of the "imperfection" with an uncertainty measured in fractions of a millimetre.

Application of ECA for assessment of nominally non-conforming structures

It is well known that the producer's risk may be significant and entailing many repairs, which are unnecessary from a technical point of view. It is tempting to use ECA in order to avoid such repair but this is not without problems:

- a. Deviations in welding parameters between the welding of the test plates, used for determination of fracture toughness, and the actual welds in the structure may result in the fracture toughness of the actual welds being different from the measured values. Some imperfections, in particular cracks, but also lack of penetration, lack of fusion and excessive porosity, may indicate a deviation from the qualified welding procedure. For instance, hydrogen cracks in C-Mn steel welds are caused by a combination of high hardness (low fracture toughness), high concentration of hydrogen and high residual stresses (restraint). Determination of the critical crack size on the basis of fracture toughness values obtained from the procedure tests is likely to be unsafe in such cases.
- b. All the requirements and limitations mentioned above for new structures also apply for assessment of nominally non-conforming welds.
- c. The limitations for application of TOFD procedures mentioned above for new structures also apply for assessment of nominally non-conforming welds. The welds have in general to be of a high quality and the non-conformity limited to the occurrence of a single, well-defined type of imperfection. Central lack of penetration and sidewall lack of fusion are typical examples.
- d. Conventional quality and inspection criteria are efficient mainly because the high costs related to non-conformity induce the manufacturers to maintain a high quality level in production. This reduces occurrence of really dangerous imperfections to a very low level. Structures, as produced, are inherently safe and inspection largely becomes a formality. Any action, which diminishes the relative costs related to non-conformity, is likely to change the balance and eventually result in manufacture of a higher proportion of structures having really dangerous imperfections. This shall be counteracted in order to avoid higher failure rates.

Application of ECA for in-service inspection

Application of fitness-for-purpose assessment for in-service inspection is quite common and it has far fewer complications than the application for new structures. There are two different types of application:

1. Conventional inspection procedures are known to be characterised by quite large uncertainties. A structure may easily hold imperfections, which surpass the acceptance criteria, used during the original non-destructive testing. Such imperfections may (by coincidence) be found during in-service inspection. The owner of the structure may use fitness-for-purpose assessment for documentation of the acceptability of these imperfections.
2. Application of ECA for assessment of crack growth, corrosion, wear and tear and other deterioration detected during in-service inspection is a well-established practice. Provisions have, for several years, been included in a number of codes for pressurised equipment and other structures. In service inspection is facilitated by the fact that deterioration usually results in well-defined imperfections of a single type and often localised (e.g. fatigue cracks). This permits application of special procedures for non-destructive testing, able to give quantitative information on the size of the imperfections. The deterioration may be monitored during a number of inspections in order to follow the growth of cracks, the progress of corrosion, etc.

The following considerations are recommended:

- a. The ECA may require information on fracture toughness and other materials properties. This information should be established during procedure testing. Application for in-service inspection should be taken into consideration at the design stage.
- b. A fingerprint inspection should be performed on the finished structure, prior to use, in order to detect substantial imperfections originating from the fabrication but overlooked or misinterpreted during the outgoing inspection (1. above).

2.2 BS 7910:1999

Two publications from the British Standards Institution, namely: PD 6493:1991 "Guidance on methods for assessing the acceptability of flaws in fusion welded structures" [15] and PD 6539:1994 "Guide to methods for the assessment of the influence of crack growth on the significance of defects in components operating at high temperatures" [16] have been widely used. PD 6493 has been in existence for more than two decades and it has been successfully applied in many countries and industrial sectors.

The two PD publications were superseded (in December 1999) by BS 7910 [2]: "Guide on methods for assessing the acceptability of flaws in metallic structures".

PD 6493 [15] and PD 6539 [16] have been referred to by a number of European design codes such as:

- EN 13445: Unfired pressure vessels [17]
- EN 12952: Water tube boilers [18]
- ENV 1999: Design of aluminium structures [19]
- EN 13480 Metallic industrial piping [20]

All these references will presumably be substituted by a reference to BS 7910 [2] in the final standards. BS 7910 is very comprehensive and it gives an adequate coverage of most aspects.

2.3 SINTAP (Structural integrity assessment procedures for European industry)

2.3.1 Introduction

A comprehensive European project: Structural Integrity Assessment Procedures for European industry (SINTAP), has resulted in a report supplementing the provisions of BS 7910 [2]. The SINTAP report [3] may be used in lieu of the sections in BS 7910 on fracture and plastic collapse. SINTAP was a Brite-Euram project Framework IV part-funded by the European Commission under its Brite-Euram framework, project number BE95-1426. The project had a duration of three years and finished in April 1999. The consortium consisted of seventeen members from nine European countries and comprised a cross-section of industrial, safety assessment, research and academic institutions. The project was initiated in response to a number of issues pertaining to the various structural integrity methods which were reviewed early in the project: It was therefore structured into a number of tasks to address these issues, the overall objective being to derive a unified structural integrity evaluation method applicable across a wide range of European industries and which would be suitable for consideration as a CEN procedure.

2.3.2 Principles and structure of procedure

The SINTAP procedure is a method for the evaluation of integrity in terms of brittle fracture, ductile tearing and plastic collapse. In its development, other procedures in use within Europe were considered: PD 6493 [15] (now BS 7910 [2]), R6 (from the UK), and the Engineering Treatment Model (ETM, from Germany [21]) and parts incorporated where appropriate. The underlying principles of the SINTAP method are:

1. A hierarchical structure based on quality of available data inputs.
2. Decreasing conservatism with increasing data quality.
3. Detailed guidance on determination of characteristic input values such as toughness.
4. A choice of output within the framework of a so-called Failure Assessment Diagram (FAD) or Crack Driving Force (CDF) plot.
5. Specific methods for allowing for the effect of weld strength mismatch.
6. Guidance on dealing with situations of low constraint and, for components containing fluids, Leak-Before-Break analysis.
7. Compendia of solutions for Stress Intensity Factors, Limit Load Solutions and weld residual stress profiles.

The procedure is arranged in 4 chapters:

In chapter I, the procedure is introduced and its scope outlined. A description of the FAD and CDF approaches is given together with guidance on interpreting results of an analysis and suggestions on the format for reporting an assessment.

Chapter II provides detailed guidance on the treatment of tensile data, determination of characteristic values of fracture toughness, imperfection characterization and stress treatment.

Chapter III provides compendia of stress intensity factor and limit load solutions for a range of common geometries. Residual stress distributions for a variety of welded joint configurations are given. In addition a section on guidance enables the user to determine whether any aspect of an assessment could be refined to enable a reduction in conservatism when an initial analysis has shown a particular case to be unacceptable. This will usually involve enhancement of the quality of the input data.

Finally, chapter IV provides details of methods, which can be used as alternatives to the standard levels described in the main text. These include the most basic 'default' method and also advanced methods, which would only be applied by the specialist user.

2.3.3 Levels of the SINTAP procedure

Table 3. The Different Levels of the SINTAP Procedure

Level	Title	Format of Tensile Data	Format of Toughness Data	Mismatch Allowance?
0	Default	Yield stress only	Estimation of yield/tensile ratio (Y/T) for FAD. Toughness from Charpy energy.	No
1	Basic	Yield stress & UTS only	Estimation of strain hardening exponent from Y/T for FAD. Fracture toughness as equivalent K_{mat} .	No
2	Mismatch	Yield stress & UTS of Parent Plate and weld	Estimation of strain hardening exponent of parent plate and weld metal from Y/T for FAD. Fracture toughness as equivalent	Yes

			lent K_{mat} for relevant zone.	
3	Stress-Strain	Full stress-strain curve of Parent Plate (and weld metal)	FAD determined from measured stress-strain values. Mismatch option based on 'equivalent material' stress-strain curve	Optional
4	Constraint	Full stress-strain curve	Modification of FAD based on T and Q stress approaches.	Possible
5	J-Integral	Full stress- strain curve	Estimation of J-integral as a function of applied loading from numerical analysis.	Optional
6	LBB	Yield stress & UTS only	Application to pressurised components with sub-critical crack growth	No

An objective of the project was to enable the methods to be applied to a full range of industries, a range of assessment levels are offered; the most basic of these levels can be applied with very limited input data although the resulting analysis will be correspondingly conservative. Similarly, in the case of detailed data being available, experienced users can apply the advanced methods and achieve a more accurate result. Levels are grouped into 'Default' (level 0), 'Standard' (Levels 1, 2 and 3) and 'Advanced' (Levels 4, 5 and 6) methods. It is only intended that the Default Level be used when data is severely limited, Level 1 being the preferred starting level.

2.3.4 Description of contents

Chapter I. Description and methodology

Foreword: Provides a brief summary of the structure and objectives of the SINTAP project and the range of applications to which the procedure can be applied.

Nomenclature: A listing of the symbols as used in the main body of the procedure. Symbols for specific application are described with the relevant section.

1. Introduction & Scope: A commentary on the types of application of the procedure and its use in cases pertaining to workmanship criteria. The philosophy of the procedure is introduced and the failure processes covered are listed.

2. Description & Levels of Analysis: Provides a description of the FAD and CDF approaches, the different levels of the procedure and the types of tensile and toughness data, which can be applied.

3. Significance of results: Highlights the importance of data quality and the link between data quality and the level of conservatism of an assessment. The hierarchy of the procedure is described and the application of reserve factors, sensitivity analysis and partial safety factors introduced.

4. Procedures: Gives detailed stepwise descriptions of Levels 1, 2 and 3. Distinction is made between different qualities of tensile data, whether continuous or discontinuous yielding behaviour is present and whether the mismatch options are to be invoked.

Reporting: Provides a standard format for reporting of results.

Chapter II. Inputs and calculations

1. Tensile properties: Detailed treatment of tensile properties, ranging from knowledge of yield strength only through to the full stress-strain curve.

2. Fracture Toughness Data: Procedure for statistical treatment of fracture toughness data, taking account of number of specimens regardless of failure mode. Includes treatment of brittle fracture, ductile tearing and maximum load data and expressions for conversion into equivalent K data from J and CTOD.

3. Imperfection Characterization: Definition of characteristic imperfection size together with guidance on the reliability of NDE methods and imperfection interaction.

4. Primary & Secondary Stress: The treatment of applied and secondary loadings with particular emphasis on effect of Lr regime on weld residual stress significance.

Chapter III. Further details and compendia

1. Guidance on Level Selection: Presents considerations to be made when selecting level of analysis and indicates the cases where benefit is likely to arise from applying the more advanced options. Recommended actions to reduce conservatism depend largely on the region of the FAD in which the initial analysis point falls.

2. SIF & LL Solutions: A fully comprehensive compendium of stress intensity factor (SIF) and limit load (LL) solutions was collated within the project and this stands alone from the main procedure. In this section, approaches for performing calculations, sample solutions and sources of solutions are presented.

3. Residual Stress Profiles: A compendium of weld residual stress profiles for a range of geometries; surface and through-thickness residual stresses are given for longitudinal and transverse orientations. Stresses can be determined from knowledge of the material and weld heat input when known or from polynomial expressions. In the case of limited data knowledge, residual stress level is given as a function of yield stress depending on the imperfection location/orientation and whether the joint has been subjected to a post weld heat treatment.

4. Compendium of Equations: Provides a summary of equations for levels 1, 2 and 3.

Chapter IV. Alternatives and additions to standard methods

1. Default Procedure: Describes the approach to be followed when the tensile and toughness data are known only in terms of yield stress and Charpy impact energy respectively.

2. Ductile Tearing Analysis: Allows for increase in toughness on ductile tearing and can be applied regardless of the level of knowledge of tensile data.

3. Reliability Methods: Provides a description of the two probabilistic methods, which can be applied using the procedure and probabilistic software (Monte-Carlo Simulation (MCS) and First Order Reliability Method (FORM)). In addition, recommendations for partial safety factors that can be applied in order to meet specific target reliability levels are made.

4. Constraint Procedure: Describes the method for calculation of the normalised measure of constraint β , based on both the elastic T stress and the hydrostatic Q stress. The FAD is then modified to take account of this constraint.

5. Leak-Before-Break: Sets out procedures for making an LBB case and recommends methods for each of the steps involved, including guidance on the shape development of part-penetrating imperfections.

6. Prior Overload: Provides guidance on accounting for the effects of prior overload on the mechanical relaxation of residual stress and the effects of warm pre-stress on fracture toughness.

2.4 Comparison of BS 7910 and SINTAP procedures

2.4.1 Differences of BS 7910 and SINTAP

BS 7910 includes detailed methods for assessing brittle fracture, ductile tearing, plastic collapse and fatigue, and in addition guidance for the assessment of corrosion damage and

creep crack growth. SINTAP concentrates on brittle fracture, ductile tearing and plastic collapse. However, in its treatment of these failure modes, the SINTAP procedure is comprehensive and flexible, offering a greater number of options, which are definitively specified. At all levels SINTAP offers improved treatment of tensile and toughness data, particularly with respect to statistical treatment of the latter. Explicit recommendations are made in SINTAP for the treatment of constraint and leak-before-break, while methods for the treatment of weld strength mismatch are an inherent part of the procedure.

In the area of data inputs, improvements have been made in the scope of compendia of information such as stress intensity factors, limit load solutions and weld residual stress profiles. The choice of characteristic values of data inputs based on data quality and perceived risk is also an area where SINTAP offers improvements over BS 7910. The major differences between BS 7910 and the SINTAP procedure are summarised below.

Table 4. Principle Differences Between BS 7910 and SINTAP Procedure

Aspect	BS 7910 [2]	SINTAP [3]
Origins	Largely UK contributions	9 European countries contributed
Failure modes	Brittle Fracture, Plastic Collapse, Ductile Tearing, Fatigue, Corrosion, Creep	Brittle fracture, Plastic Collapse, Ductile Tearing
Structure	Based on failure mode: extensive system of annexes	Based on data quality and hierarchy of procedure; four main chapters
Concept of fracture and collapse treatment	Predominantly FAD based	Option of interpretation as FAD or CDF
Fracture modes	Mainly I but guidance on II and III	Only Mode I
Toughness treatment	K, J or CTOD	FAD defined only in terms of equivalent K_{mat} . J used for CDF. Allowance for ductile tearing can be made. CTOD data converted to equivalent K or J.
Characteristic Input Values	Generalized guidance on toughness treatment (Number of tests, weld testing) and tensile properties.	Specific sections for definition of characteristic values of tensile properties, fracture toughness and imperfection dimensions, including statistical treatment for toughness data.
Probabilistic approaches/safety factors	Guidance on Reliability, Partial Safety Factors and Reserve Factors	As for BS 7910 but additional guidance on probabilistic methods and associated software.
Weld Strength Mismatch	Qualitative guidance as an annex	Inherent part of procedure with specific recommendations.
Constraint Treatment	No specific guidance	Explicit recommendations are given.
Industry specific	Pipeline and Offshore	None

guidance		
Software	Various systems available	Demonstration software available for levels 0-3 inclusive, and for probabilistic analysis at level 1.

Where a specific area is not listed above it can be assumed that the treatment within the two procedures is similar. More fundamentally, the SINTAP procedure is the product of a group, which first reviewed all available methods. It can therefore be considered to be a state of the art methodology based on perceived best practice representing the current European consensus method.

2.4.2 Validation of method

Validation of the individual components of the SINTAP procedure has been well documented by their originators. Within the SINTAP scheme, verification of the hierarchical principles was carried out towards the end of the project by means of a series of exercises comparing actual test data with predictions made using the methods. These showed that in general the correct hierarchy of results was obtained for the various levels and that no un-conservative predictions were made. However, due to the SINTAP scheme being of recent origin, more extensive validation awaits further use of the procedure.

2.4.3 Summary

The SINTAP procedure is the product of a three-year Brite-Euram programme involving seventeen partners from nine European countries. The method represents a current European consensus on structural integrity assessment and offers several distinct advantages over current approaches. In the context of a standard on fitness-for-purpose, it is therefore eminently suitable for substitution in place of the brittle fracture and plastic collapse clauses of BS 7910.

2.5 Assessment methods according to Ril 804 and Ril 805

In Germany requirements for the inspection of bridge structures are covered by the German standard DIN 1076 [22]. In case of structures under the responsibility of the German railway (Deutsche Bahn) the guidelines Ril 804 [23, 24] and Ril 805 [5] apply instead.

Module 804.8001 [23] of Ril 804 defines general rules for the inspection including the regular inspection intervals whereas the module 804.8002 [24] covers the inspection and maintenance of railway bridges in particular. Return frequencies of basic and comprehensive bridge inspections according to Ril 804 are

- 3 years return period: standard inspection (structural safety, bearings, sealing, drainage, cracks, deformation, corrosion etc.)
- 6 years return period: main inspection (foundation, massive components, steel structure, riveted and bolted connections, difficult accessible components, rust grade, material testing etc.)

The German railway (Deutsche Bahn) uses the guidelines Ril 805 for condition assessment of old steel railway bridges. This guideline applies to the assessment of the structural safety for bridges of all types and dimensions that are in service for at least 6 years.

Evaluation can be carried out at 5 different levels of assessment intensity. These different levels account for the individual current condition of each bridge structure. The levels are:

- Level 1: Estimation of structural safety
- Level 2: Approximative determination of structural safety
- Level 3: Assessment of structural safety without consideration of fatigue
- Level 4: Assessment of structural safety with consideration of fatigue
- Level 5: Confirmation of assessment using measurement in addition to level 3 and 4

In general, examination according to level 1 and 2 is sufficient. If results are unsatisfactory or the structure shows significant damages or the design loads are far lower than according to actual loads, respectively, then the structure shall be examined in accordance with the higher levels 3, 4 or, if necessary, 5.

For existing steel railway bridges that are in service for more than 60 years, an assessment of the remaining service life based on the Wöhler concept has to be performed in addition to the structural safety assessment. If necessary, the assessment has to be enhanced by the determination of operating time intervals using fracture mechanics (if components show cracks or if assessment of the remaining service life results in insufficient service life).

Therefore, a stepwise assessment procedure is proposed:

1. Determination of the current structural condition
2. Determination of imposed loads
3. Structural safety assessment
4. Fatigue assessment
5. Assessment of operating time intervals

It is the purpose of the assessment of operating time intervals to prevent from unforeseen fracture within the inspection interval, if fatigue damages remain undetected during inspection. The structure is designated as damage tolerant if time intervals between inspections are smaller than the calculated safe operating time interval.

If the remaining service life according to fatigue assessment is evaluated as lower than 15 years an assessment of operating time intervals according to Ril 805 is mandatory. Assessment of operating time intervals is also mandatory in case of cracks in tension-loaded components. The result of this assessment is decisive for a possible reduction of the 6 years return period of the main inspection according to Ril 804. If components are rated as not damage tolerant due to shortfall of specified limits then immediate measures are required.

For more information about fatigue assessment procedures for riveted, bolted or welded structures by assessment of the remaining service life based on the Wöhler concept and about assessment of operating time intervals according to Ril 805 refer to the WP5-Deliverable SB5.2.1 [1].

2.6 Condition assessment and inspection following the draft standard prEN 1090-2

2.6.1 Introduction

The final draft of the coming execution standard for steel products EN 1090 "Execution of steel structures and aluminium structures", prepared by CEN/TC 135, has been released for CEN-enquiry in 2005. Part 2 of this standard "Technical requirements for the execution of steel structures" [5], contains the European state-of-the-art for execution, inspection, testing and corrections of steel railway bridges and comprises detailed acceptance criteria as well as

requirements for inspection, maintenance and repair. Therefore, for condition assessment of old steel railway bridges the given acceptance criteria are of particular interest.

Within this section the relevant content of this draft standard is elaborated for “Sustainable Bridges” purposes.

2.6.2 General requirements for inspection and testing

All inspection and testing and associated corrections shall be undertaken within the quality requirements set out in the standard prEN 1090-2 to a predetermined plan, documented and included in a quality documentation. If available from the time of bridge erection, documents supplied with constituent products including inspection certificates, test reports, declaration of compliance as relevant shall be checked.

prEN 1090-2 defines Execution Classes (EXC) level 1 to 4 depending on levels of Consequence Classes (CC) from 1 to 3 in accordance with EN 1990:2002 [25] and levels of Production and Service Categories (PS) from 1 to 3. All components for which fatigue assessment is necessary are within level PS3. For railway bridges where consequences of failure are high concerning potential loss of human life, or economic, social or environmental consequences CC3 applies. Consequently, the execution class 4 (EXC 4) applies for steel railway bridges, because in any case they are subject to significant effects of fatigue.

For inspection and testing methods and instruments used shall be selected, as appropriate, from those listed in ISO 7976-1 and -2 [26]. Accuracy shall be assessed in accordance with the relevant part of ISO 8322 [27]. The location and frequency of measurements shall be specified in the inspection plan.

The inspection plan shall include:

- a. the scope of inspection;
- b. acceptance criteria;
- c. actions for dealing with nonconformities, corrections and concessions;
- d. release/rejection procedures.

If inspection results in the identification of nonconformity, the action on such nonconformity shall be as follows:

- a. if practicable, the nonconformity shall be corrected using methods that are in accordance with the specifications of prEN 1090-2 [5] and checked again against acceptance criteria;
- b. if correction is not practicable, modifications to the steel structure may be made to compensate for the nonconformity provided these modifications are in accordance with the specifications.

2.6.3 Inspection of bridge components

Bridge components in general

Essential tolerances relevant for steel bridge structures which are compatible with Eurocode 3 [28] design rules can be found in Annex L of prEN 1090-2 [5]. The choices available in this Annex do not cover all possible situations. It is suggested that, if no particular choices are suitable, the following general criteria may be used:

- a. For welded structures, the following classes according to EN ISO 13920 [29] apply:
 - class C for length and angular dimensions;
 - class G for straightness, flatness and parallelism.
- b. In other cases, apply a general tolerance applicable to any dimension "D" of the maximum of $D/500$ or 5mm.

For further essential tolerances concerning cross-sectional tolerances, tolerances of shells, full contact bearings, baseplates and foundation bolts, see Annex L of prEN 1090-2.

Bridge super-structure

The deviations of erected bridges shall be in accordance with Table 5, unless otherwise specified.

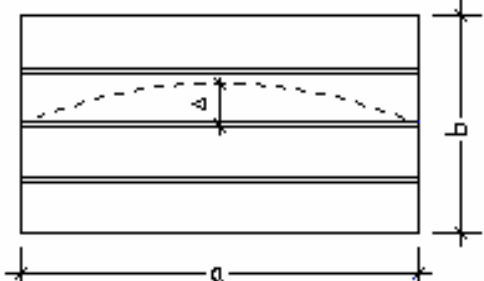

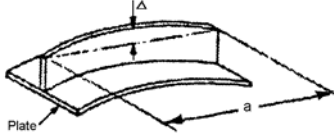
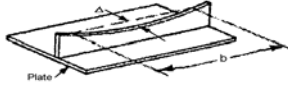
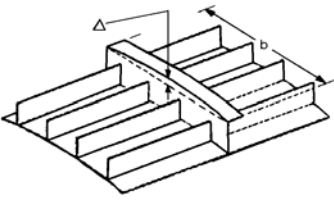
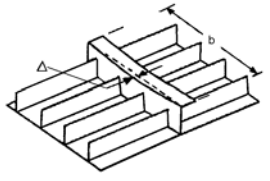
Table 5. Essential erection tolerances of bridges according to prEN 1090-2, Annex L.1

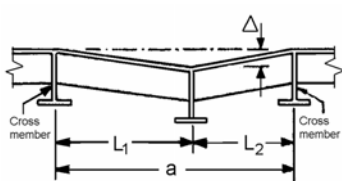
No	Criterion	Parameter	Permitted deviation Δ
1	Span length:	Deviation Δ of distance D between two consecutive supports measured on top of upper flange:	$\Delta = \pm 30 D / 10000$
2	Bridge elevation or plan profile:	Deviation Δ from nominal profile adjusted for as-built levels of supports: D \leq 20 m: D > 20 m:	$ \Delta = D / 1000$ $ \Delta = D / 2000 + 10 \text{ mm} \leq 35 \text{ mm}$
3	Fit-up of orthotropic decks of plate thickness T after erection: Gauge length: L Ste _r . Deviation: Pr	Difference in level at junction: T \leq 10 mm: 10mm < T \leq 70 mm T > 70 mm: Slope at junction: T \leq 10 mm: 10mm < T \leq 70 mm T > 70 mm: Flatness in all directions: T \leq 10 mm: T > 70 mm: General case: Longitudinally: (values for Pr may be interpolated for 10mm < T \leq 70 mm)	Ve = 2 mm Ve = 5 mm Ve = 8 mm Dr = 8 % Dr = 9 % Dr = 10 % Pr = 3 mm over 1 m Pr = 4 mm over 3 m Pr = 5 mm over 5 m Pr = 5 mm over 3 m Pr = 18 mm over 3m
4	Orthotropic deck welding: 	Protrusion Ar of weld above surrounding surface:	Ar = + 1 / - 0 mm

Orthotropic decks

Acceptance criteria for tolerances of stiffened plating and orthotropic decks are given in Table 6.

Table 6. Acceptance criteria for tolerances of stiffened plating and orthotropic decks

 <p>Longitudinal stiffeners in longitudinally stiffened plating</p>		 <p>Transverse stiffeners in longitudinally and transversely stiffened plating</p>	
No	Criterion	Parameter	Permitted deviation Δ
1	Straightness of longitudinal stiffeners in longitudinally stiffened plating:	Deviation Δ perpendicular to the plate: 	$ \Delta = a/400$
2		Deviation Δ parallel to the plate: 	$ \Delta = a/400$
3	Straightness of transverse stiffeners in transversely and longitudinally stiffened plating:	Deviation Δ perpendicular to the plate: 	$ \Delta = a/400$ $ \Delta = b/400$
4		Deviation Δ parallel to the plate: 	$ \Delta = b/400$

5	Levels of cross components in stiffened plating:	Level relative to the adjacent members: 	$ \Delta = a/400$
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Verticality of bridge columns

The deviations of columns shall be in accordance with Tables L-1.5 to –1.7 of prEN 1090-2. The overall inclination shall meet: $|\Delta| = h/300$

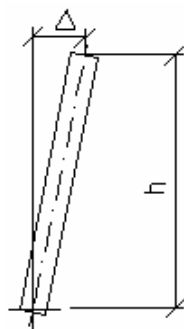


Figure 2 Deviation of columns

Bridge supports

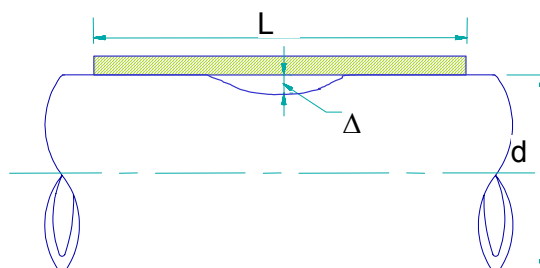
The base level of the bridge support shall be set to within ± 5 mm of its position point. This may be achieved by setting the level at the underside of the bearing, provided that compensation is made for significant thickness variations in the bearing assembly.

The position in plan of the bridge supports shall be set to within ± 5 mm of its position point.

The verticality of the web of main girders at supports shall be within depth/300.

Hollow sections

Damage resulting in local dents in the surface of hollow sections shall be assessed. The method shown in Figure 3 may be used.



Characteristic cross-sectional dimension of section is d
 Straight edge of length $L \geq 2d$ Gap $\Delta \leq$ the larger of $d/100$ or 2 mm

Figure 3 Method of assessment for surface profile and permitted deviation of a dented component

If surface defects revealed during surface preparation or subsequent service life are repaired using methods that are in accordance with prEN 1090-2 [5], the repaired constituent product

may be used provided that it complies with the nominal properties specified for the original constituent product.

Local Hardness and quality of cut surfaces

At locations where processes have been used that are likely to produce local hardness (thermal cutting, shearing punching) their capability may be checked as follows:

- a. Four local hardness tests shall be done on each sample in locations likely to be affected. The tests shall be in accordance with EN ISO 6507 [30];
- b. The worst value measured shall not exceed the values specified in Table 7

Table 7. Permitted maximum hardness values (HV 10)

Product standards	Steel grades	Hardness values
EN 10025-2 to 5 [31]	S235 to S460	380
EN 10210-1 [32], EN 10219-1 [33]		
EN 10149-2 and 3 [34]	S260 to S700	
EN 10025-6 [31]	S460 to S690	450
EN 10025-6 [31]	S890, S960	To be specified

Those values are in accordance with EN ISO 15614-1 [35] applied to steel grades listed in CEN ISO TR 20172 [36].

The quality of cut surfaces defined in accordance with EN ISO 9013 [37] shall be as follows:

- Perpendicularity or angularity tolerance, u, Range 2
- Mean height of the profile, Rz5, Range 2

2.6.4 Inspection of welds

Visual Inspection

All welds shall be visually inspected throughout their entire length for surface imperfections in accordance with EN 970 [38]. The visual inspection shall be performed before any other NDT inspection is carried out. In case one of the following imperfections is detected additional inspection and further measures are necessary as indicated:

- Weld spatter and arc strikes shall be removed;
- Visible imperfections such as cracks, notches, cavities and other not permitted imperfections shall be removed;
- All slag shall be removed from the surface of the weld. Particular attention shall be paid to the junctions between the weld and the parent metal.

The inspection of the shape and surface of welds of welded branch joints using hollow sections shall pay careful attention to the following locations:

- for circular sections: the mid-toe, mid-heel and two mid-flank positions;
- for square or rectangular sections: the four corner positions.

Additional inspection

If surface imperfections are detected, additional surface testing by liquid penetrant testing or magnetic particle inspection shall be carried out on the inspected weld.

For welds the extent of additional NDT is specified in Table 8 where percentages apply to the overall extension of each joint, with the following conditions:

a. Shop welds:

The first 5 joints of each type welded according to the same procedure qualification shall be tested according to the tabulated values. If accepted the extent of additional NDT is reduced to 20% of tabulated values for joints of each type with a minimum of 5% provided acceptable results are maintained.

b. Site welds:

All joints shall be tested according to the tabulated values.

If the inspection exposes weld imperfections in excess of the requirements specified in the acceptance criteria, the frequency of testing shall be increased.

The extent of NDT covers both testing of surface or internal imperfections if applicable. The NDT method shall be selected in accordance with EN 12062 [11]. Generally ultrasonic testing or radiographic testing applies to butt welds and liquid penetrant testing or magnetic particle inspection applies to fillet welds.

The following NDT methods shall be carried out in accordance with the general principles given in EN 12062 and with the requirements of the standard particular to each method.

- a) liquid penetrant testing (EN 571 [39]);
- b) magnetic particle inspection (EN 1290 [40]);
- c) ultrasonic testing (EN 1713 [41], EN 1714 [42]);
- d) radiographic testing (EN 1435 [43]);
- e) eddy current testing (EN 1711 [44]).

The field of application of NDT methods is specified in their relevant standards.

If only partial inspection is necessary, the joints for inspection shall be selected on the basis of Annex C of EN 12062, ensuring that sampling covers the following variables as widely as possible: the joint type, the constituent product grade, the welding equipment and the work of the welders.

If inspection discovers weld imperfections within an inspection length in excess of the requirements specified in the acceptance criteria, additional tests shall be undertaken over two inspection lengths, one on each side of the length including the defect. The guidelines in Annex D of EN 12062 should be followed in deciding further action.

Table 8. Extent of additional NDT

Type of weld	Shop welds	Site welds
Transverse butt welds subjected to tensile stress $U = \sigma / R_{eH}$ $0.75 \leq U$ $0.5 < U < 0.75$	100 % 50 %	100 % 100 %
Transverse fillet welds at end of lap joints and at connection gussets	10 %	10 %
Longitudinal welds and welds to stiffeners	5 %	10 %

with:

σ = nominal stress based on externally applied loads in ULS at the location of the weld

R_{eH} = yield stress of the local parent metal

Longitudinal welds are those made parallel to the component axis. All the others are considered as transversal welds.

Acceptance criteria for welds

Unless otherwise specified, the acceptance criteria for welds shall be as follows, with reference to EN ISO 5817 [10]. Any special requirements on weld geometry and profile shall be taken into account. For guidance on the classification of geometric imperfections in metallic materials concerning fusion welding refer to [45] and for guidance on quality levels for imperfections for electrons and laser beam welded joints refer to [46].

For steel railway bridges the so-called *Quality level B+* applies, which comprises quality level B of EN ISO 5817 supplemented by the additional requirements given in Table 9. The requirements for quality level B+ take into account requirements for welds subject to significant effects of fatigue.

Table 9. Additional requirements for quality level B+

Imperfection designation		Limits for imperfections
undercut (5011)		not permitted
excess weld metal (502)		$h \leq 2 \text{ mm}$
incorrect weld toe (505)	Butt welds	$\alpha \geq 165^\circ$
	Fillet welds	$\alpha \geq 120^\circ$
internal pores (2011 to 2014)	Butt welds	$d \leq 0,1 s$, but max 2 mm
	Fillet welds	$d \leq 0,1 a$, but max 2 mm
solid inclusions (300)	Butt welds	$h \leq 0,1 s$, but max 1 mm and $l \leq s$, but max 10 mm
	Fillet welds	$h \leq 0,1 a$, but max 1 mm and $l \leq a$, but max 10 mm
linear misalignment (507)		$h < 0,05 t$, but max 2 mm
root concavity (515)		not permitted

For acceptance criteria for welds of steel bridges annex C of Eurocode 3, part 2 [47] is also relevant. The acceptance criteria in this annex are equivalent to those given in Table 9.

Repair

Repairs by welding shall be carried out in accordance with qualified welding procedures. Qualification of welding procedures shall be performed in accordance with EN ISO 15610 [48], EN ISO 15611 [49], EN ISO 15612 [50], EN ISO 15613 [51] and the relevant part of EN ISO 15614 [35], as appropriate. Co-ordination personnel shall have comprehensive technical knowledge as specified in EN 719 [52].

Corrected welds shall be checked and shall meet the requirements of the original welds.

In case of repair welding the final NDT of a weld shall generally be carried out not earlier than 16 hours from the time of the completion of the welds to be inspected. This period shall be increased to at least 40 hours if one or more of the following conditions related to cold cracking risk are met:

- a) constituent product thickness above 40 mm thick;
- b) steel grades higher than S355;
- c) high restraint of the weld within the fabricated component;
- d) steels with improved atmospheric corrosion resistance.

Any weld located in a zone where unacceptable distortion has been corrected e.g. by flame straightening shall be inspected again.

Any requirements for grinding and dressing of the surface of completed welds shall be specified.

2.6.5 Inspection of bolted and riveted connections

Inspection of all bolted connections

All connections with non-preloaded and preloaded mechanical fasteners shall be visually checked

- a) for missing bolts;
- b) if the structure is aligned with locally;
- c) differing ply thickness;
- d) bolt protrusion.

Acceptance criteria and action to correct nonconformity shall be in accordance with clause 8 of prEN 1090-2 [5].

If nonconformity is identified due to differing ply thickness that exceeds the following criteria, the connection shall be remade.

Separate components forming part of a common ply shall not differ in thickness by more than 2 mm generally, or 1 mm in preloaded applications, see Figure 4. If steel packing plates are provided to ensure that the difference in thickness does not exceed the above limit, their thickness shall not be less than 2 mm. In case of severe exposure, avoiding cavity corrosion may require closer contact.

The connected components shall be drawn together such that they achieve firm contact. Shims may be used to adjust the fit. For thicker gauge constituent product ($t \geq 4$ mm for plates and sheeting and $t \geq 8$ mm for sections), residual gaps up to 2 mm may be left between contact faces unless if full contact bearing is specified.

Thickness should be fitted so as to limit the number of packing plates to a maximum of three.

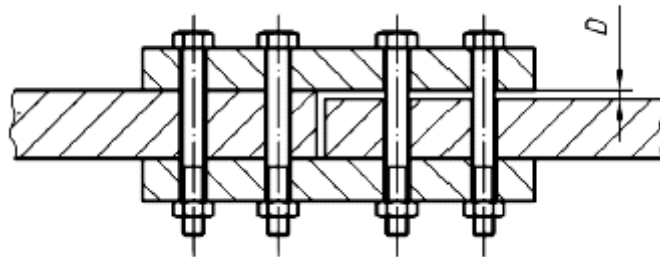


Figure 4 Difference of thickness between components of a common ply

Corrected connections shall be checked again on re-completion.

The bolt shank shall protrude from the face of the nut after tightening. For bolts acting under tension loading, the protrusion shall be not less than one full thread pitch; in the other cases the protrusion may be reduced to the thread run out.

Inspection of preloaded bolted connections

Inspection of installed fasteners shall be undertaken depending on the tightening method used. The locations selected shall be on a random basis ensuring that the sampling covers the following variables as appropriate:

- joint type;
- fastener lot, type and size;
- equipment used and the work of the operatives.

Bolt assemblies installed shall be inspected as follows:

- a) For the purposes of the inspection, a bolt group is defined as bolt assemblies of the same origin in similar connections with the bolt assemblies of the same size and class. The same bolt group may be subject of a number of inspections each covering a subgroup.
- b) The number of bolt assemblies inspected overall in a structure shall be at least 10%, with a minimum of 5
- c) The inspection is carried out in a sequential fashion as described below to a sufficient number of bolt assemblies until either the acceptance or the refusal conditions for the relevant sequential type are met for the relevant criteria. The sequential type B shall be applied, if the connection is subject to significant effects of fatigue. Otherwise sequential type A may be used.
- d) The criteria defining a “defection” of a bolt assembly are specified for each tightening method.
- e) If the inspection leads to a “refusal”, all the bolts in the bolt subgroup shall be checked and corrective actions shall be taken.

The inspection of a bolt assembly shall be carried out by the application of a torque to the nut (or to the bolt head if specified) using a calibrated torque wrench. The objective is to check that the torque value necessary to initiate rotation is at least equal to that required to achieve the specified minimum preloading force. Caution shall be taken to keep the rotation to a strict minimum.

The torque wrench calibration certificates shall be checked to verify the accuracy of $\pm 4\%$ according to EN ISO 6789 [53]. Each wrench shall be checked for accuracy at least once per working day, and in case of pneumatic wrenches, every time hose lengths are changed. Checking shall be carried out after any incident occurring during use (significant impact, fall, overloading).

Hand or power operated wrenches may be used, with the exception of impact wrenches. The inspection torque shall be applied continuously and smoothly.

A bolt which turns by more than 15° by the application of the inspection torque is considered defective for under-tightening ($< 100\%$) and shall be retightened up to 110% of the specified minimum preloading force.

It shall be specified whether a check of over-tightening is required, and state the procedure to be applied.

Sequential method for bolt tightening inspection

The sequential method for bolt tightening inspection is carried out according to the principles in ISO 8422 “Sequential sampling plans for inspection by attributes” [7], the purpose of which being to give rules based on progressive determination of inspection results.

ISO 8422 gives two methods for establishing sequential sampling plans: numerical method and graphic method. The graphic method is applied for bolt tightening inspection.

In the graphic method (see Figure 5) the horizontal axis is the number of bolt assemblies inspected and the vertical axis the number of defective assemblies.

The lines on the graph define three zones: the acceptance zone, the refusal zone and the indecision zone. As long as the inspection result is in the indecision zone the inspection is continued until the cumulative plot emerges into either the acceptance zone or the refusal zone. Two examples are given below.

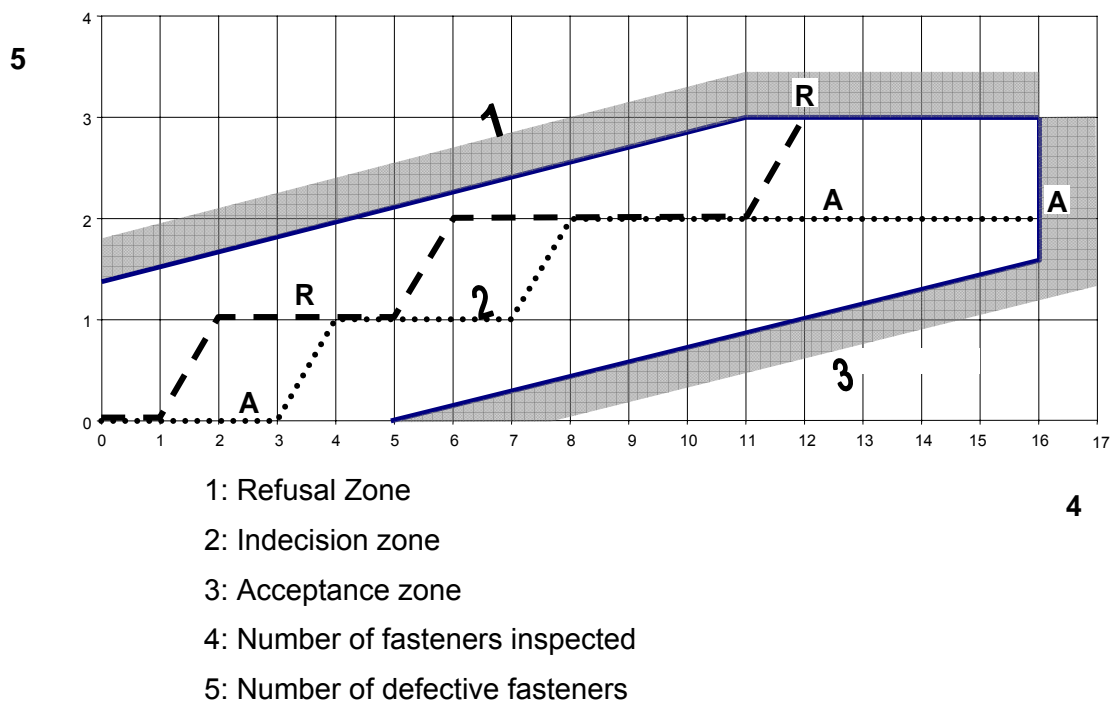


Figure 5 Example of sequential inspection diagram

EXAMPLES

- A The 4th and 8th bolts were found defective. Inspection was continued until crossing the vertical truncation line. The result is acceptance of the bolt tightening operation, subject to corrective actions on the two defective bolts.
- R The 2nd, 6th and 12th bolts were found defective. Exit from the indecision zone is into the refusal zone. The result is negative and the inspection is extended to 100% of the bolt assemblies.

APPLICATION

The following diagrams, sequential type A and sequential type B apply as relevant.

— Sequential type A:

- minimum number of assemblies to be inspected: 5
- maximum number of assemblies to be inspected: 16

— Sequential type B:

- minimum number of assemblies to be inspected: 14
- maximum number of assemblies to be inspected: 40

If the result of inspection when using a sequential type A is negative, the inspection may be enlarged to the sequential type B.

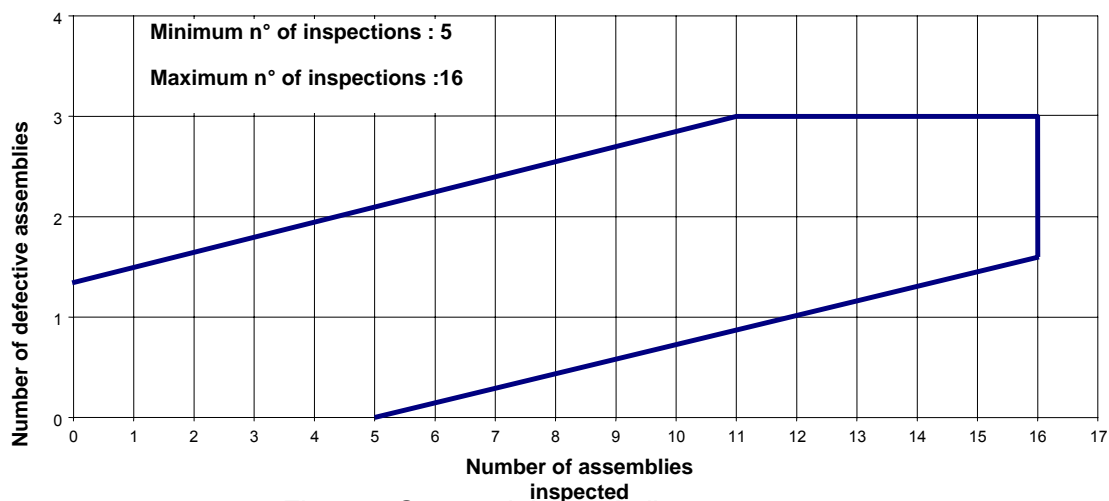


Figure 6 Sequential type A diagram

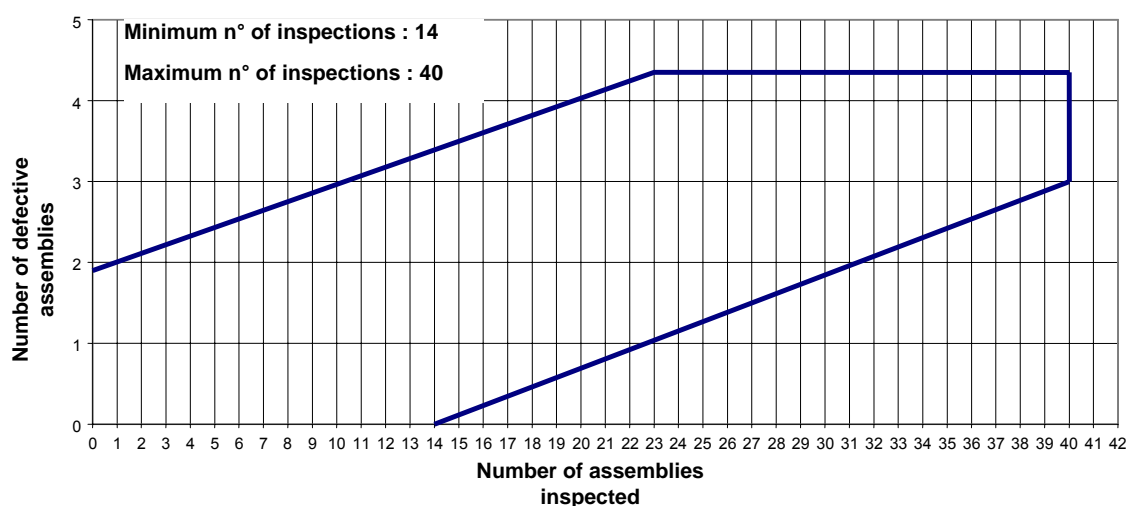


Figure 7 Sequential type B diagram

Inspection of compressible washer-type direct tension indicators

In case compressible washer-type direct tension indicators are used, the indicator gap has to be checked of at least 10% of the fasteners. Due to dimensional tolerances in steelwork and alignment of components, the indicator does not always compress evenly. When checking the gap, the average gap should be measured as shown in Figure 8.

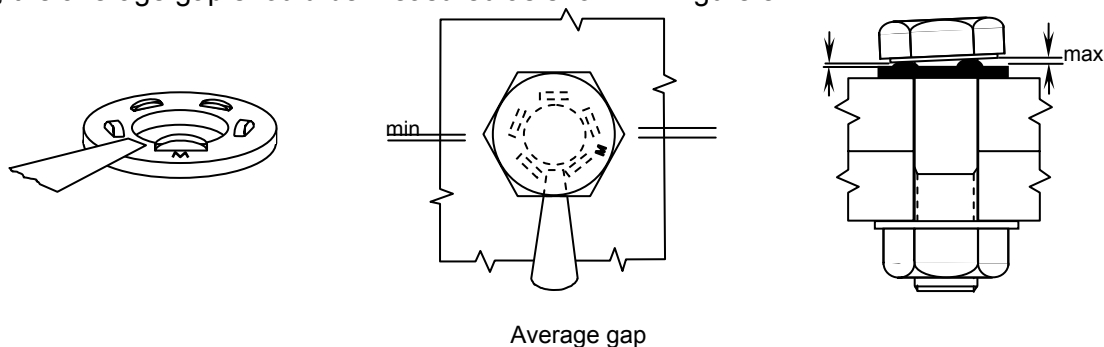


Figure 8 Checking the indicator gap

If the final indicator setting is not within the specified limits, the removal and reinstallation of the non-conforming bolt assembly shall be supervised, and the whole bolt group shall then be inspected.

Inspection and repairs of hot rivets

INSPECTION

The number of rivets inspected overall in a structure shall be at least of 5%, with a minimum of 5. Heads of driven rivets shall be visually inspected and shall satisfy the following acceptance criteria (in some cases provisions for detection of non-conformities will not be available):

- The rivet heads shall be centred. The head eccentricity relative to the shank axis shall not exceed $0,15 d_0$ where d_0 is the hole diameter,
- The rivet heads shall be well formed and shall not show cracks or pits,
- The rivets shall be in satisfactory contact with the assembled parts both at the outer surface of the plies and in the hole. No movement or vibration shall be detected when the rivet head is lightly tapped with a hammer.
- A small well-formed and centred lip may be accepted if only a small number of rivets in the group is concerned.
- Outer faces of plies free of indentation by the riveting machine may be specified.

Inspection of satisfactory contact shall be done by lightly ringing the rivet head with a hammer of 0,5 kg. The inspection is carried out in a sequential fashion according to the sequential method for bolt tightening inspection described above to a sufficient number of rivets until either the acceptance or the refusal conditions for the relevant sequential type are met for the relevant criteria. The sequential type A is applicable.

REPAIRS

If it is necessary to replace a defective rivet, it shall be done while the structure is not loaded. Cutting out shall be done by means of a chisel or by cutting.

After removing a rivet, sides of the rivet hole shall be inspected carefully. In case of cracks, pits, or hole distortion, the hole shall be reamed. If necessary, the replacement rivet shall be of a larger diameter than that removed.

2.6.6 Inspection of corrosion protection

The inspection of the corrosion protection shall be carried out according to Annex K of prEN 1090-2 [5] and in accordance with EN ISO 12944-7 [54]. The acceptance criteria shall meet requirements in ISO 8501-3 [55], EN ISO 8501-1 [56], EN ISO 8503-2 [57] and EN ISO 19840 [58]. Inspection of corrosion protection shall comprise:

- a) Visual inspection of all surfaces, welds and edges throughout their entire length, with regard to:
 - surface cleanliness, assessment in accordance with EN ISO 8501-1
 - coarseness, surface roughness, assessment in accordance with EN ISO 8503-2
 - detailing in accordance with ISO 8501-3
- b) Measurement of the thickness of the protective coating in accordance with EN ISO 19840
- c) Checks that the corrosion protection treatment in all other respects complies with the provisions of EN ISO 12944

- d) Special attention is necessary in case of galvanized components where pickling has been used prior to galvanizing, high performance steels may become susceptible to hydrogen-inducing cracking (see Annex C of EN ISO 1461 [59]).

Examples of inspection measures in conjunction with corrosion protection are given in EN ISO 12944-7 [54].

Nonconforming components shall be retreated, retested and re-inspected afterwards.

In case of repair, the appropriate information should be given in a performance specification, containing the following, as appropriate:

- a) the foreseen life span (see EN ISO 12944-1);
- b) the corrosivity category (see EN ISO 12944-2);
- c) preference or requirement for metal spraying, galvanizing or painting in particular;
- d) exact products and work methods for site applied corrosion protection and repair.
- e) friction surfaces and class of treatment or tests required;
- f) requirements relevant to subsequent decorative coatings;
- g) restrictions on choice of colour for coating products;
- h) special requirements for bimetallic interfaces.

EN ISO 12944-8 gives guidelines for developing specifications for new work and also for maintenance of corrosion protection using paints.

Overpainting and overcoating

If overpainting of zinc coated steel is carried out, the cleaning of the surface requires particular attention. The treatment shall be carried out according to EN ISO 12944-4, -5, -7 [54]. The surface condition of the component shall be checked immediately prior to painting to ensure that it complies with the required specifications, EN ISO 12944-4, EN ISO 8501-1 [56], ISO 8501-3 [55] and EN ISO 8503-2 [57] and the manufacturer's recommendations for the product about to be applied. Surfaces shall be cleaned (removal of dust and grease) and possibly treated with a suitable etch primer or sweepblasting according to EN ISO 12944-4 to surface roughness "fine" in accordance with EN ISO 8503-2. The pre-treatment shall be verified before subsequent overcoating. If two or more coats are to be applied, a different colour shade shall be used for each coat.

Additional edge protection, a stripe coat, extending across approximately 25 mm on both side of the edge shall be applied to a thickness of 40 µm

Work shall not proceed when the surfaces to be coated are wet or the ambient temperature or dew point is below that recommended in the manufacturer's recommendations for the product to be applied. Those data shall imperatively be mentioned in the product data sheet. Against condensation risk a minimum difference of 3 °C is generally considered between surface to be coated temperature and dew point unless otherwise specified in the product data sheet. Painted surfaces shall be protected against the accumulation of water for a period after application as required by the product data sheet.

If pre-coated galvanized steel is supplied with a chromate passivation, a mordant wash or etch primer may be necessary to provide a sound base for a subsequent treatment.

If coated materials are to receive further treatment, the surface preparation shall be appropriate to the surface to be treated. Abrasive cleaning and wire brushing are not appropriate to sound metallic or organically coated components. However, if repairs to coatings are needed, it may be necessary to remove debris or corrosion deposits locally to reveal the basic steel substrate before carrying out the repair.

The thickness of the protective system (Galvanizing and paint) shall be measured according to EN ISO 2808 [60] and evaluated according to EN ISO 19840 [58]: (See EN ISO 12944-5 [54])

Subsequent Welding

If a component to be repaired is subsequently to be welded, the surfaces of the component within 150 mm of the weld coating shall be removed or shall not be coated with materials that will impair the quality of the weld, respectively.

Welds and adjacent parent metal shall not be painted before deslagging, cleaning, checking and acceptance of the weld.

Surface repair by metal spraying

Metal spraying shall be of zinc, aluminium or zinc/aluminium 85/15 alloy and be undertaken in accordance with EN ISO 2063 [61].

Metal sprayed surfaces shall be treated with a suitable sealer before overcoating with paint in accordance with EN ISO 12944-4, -5, -7 [54]. This sealer shall be compatible with the overcoating paint and shall be applied immediately after metal spraying cooling so as to avoid oxidation or moisture trapping.

The thickness of the protective system (Metal spray and paint) shall be verified according to EN ISO 19840 [58].

2.7 Other European and International developments

Methods for assessing imperfections in metallic structures have been published in several standards and recommendations. Research and experience from practical applications contribute to further developments. The following specifications may provide useful information. However, an assessment of the application of the specifications in a European context is outside the scope of the present Technical Report.

- a. American Petroleum Institute API RP 579 [62]: Recommended practice for fitness-for-service. RP 579 is limited to in-service inspection and designed to support API inspection codes for pressure vessels, piping and tankage (API 510/570/653). A new API RP 571 on refining damage mechanisms will provide a link between RP 579 and API RP 580: Risk based inspection.
- b. Validation, Expansion and Standardisation of Procedures for High Temperature Defect Assessment (HIDA) [63] is a four year (January 1996 - December 1999) project (reference BE 1702) and is partly funded by the European Commission under Brite Euram Framework IV. Thirteen partners from 7 European countries form the project consortium. The HIDA project is in particular aimed at addressing the issues such as validating and expanding the database of the existing high temperature crack assessment procedures, developing new methodologies for predicting the behaviour of high temperature components and unifying and refining existing procedures with a view to making recommendations for a European Standard.
- c. SFB 477 Life Cycle Assessment of Structures via Innovative Monitoring [64]. Relevant partial reports (in German) are:
 - A1: „Methoden zur risiko- und schwachstellenorientierten Bewertung und Optimierung von Bauwerküberwachungsmaßnahmen“, Hosser
 - B3: „Lebensdauervorhersage von ermüdungsbeanspruchten Bauwerken durch Monitoring und begleitende Versuche“, Peil

- B4: „Restlebensdauervorhersage für Schweißverbindungen an Stahlkonstruktionen und Maßnahmen zur kontrollierten Nutzungsausweitung“, Wohlfahrt
- C2: „Zustandserfassung und -beurteilung vorgespannter Zugglieder durch Monitoring“, Budelmann, Rostásy

In context with the SFB 477–research the dissertation of Mehdiانpour was prepared [65].

- d. Current condition assessment methods are also covered by the report of DB-SBB [66] as result of a joint work of the German and the Swiss railway. This report contains amongst other subjects a discussion of the DS 803 and DS 805 assessment procedures of the German railway as they have been before 2000. The new issues Ril 804 [23, 24] and Ril 805 [5] partly trace back on this joint report.
- e. European Commission Research COST (European Co-operation in the Field of Scientific and Technical Research) Action 345: Procedures Required for Assessing Highway Structures [67]. Although here the focus is drawn on highway structures there are many similarities and overlapping. Relevant partial reports are:
 - WG1: „Report on the current stock of highway structures in European countries, the cost of their replacement and the annual cost of maintaining, repairing and renewing them”
 - WG2&3: „Methods used in European states to inspect and assess the condition of highway structures”
 - WG4&5: “Numerical techniques for safety and serviceability assessment”
 - WG6: ”Report on remedial measures for highway structures”The final reports are available for download at http://cost345.zag.si/final_reports.htm
- f. By order of the German Ministry of Defence a report “Intelligente Strukturen” was issued in Dec. 2000 concerning condition assessment, damage detection and health monitoring of steel bridges for military purposes [68].
- g. Results of a recent study in U.S. were published [69, 70]. In this paper, time-dependent relationship between the reliability-based analysis results, representing the future trend in bridge evaluation, and the load ratings is investigated for different types of bridges located within an existing bridge network (U.S. National Bridge Inventory database). The comparisons between live load rating factors and reliability indices are made over the life-time of each bridge in the network. The rating–reliability profile and rating–reliability interaction envelope concepts are introduced. Furthermore, the rating–reliability profiles are collectively examined in order to evaluate the time-dependent performance of the overall bridge network.

3 Stress measurement methods of components and connections of steel railway bridges

3.1 General

3.1.1 Introduction

During the design, the assumptions made simplify the real conditions. Therefore, for example, the static system used in design may differ significantly from the real existing structure. Measurements can be used for verification of the expected static system, if the structural safety or fatigue safety cannot be guaranteed by calculation.

According to [9] fatigue assessment has not been carried out in many existing bridges, since fatigue design specifications did not exist at the time of their design and erection. Often when measurements are carried out, they show that stresses are lower than used in assessment. But, in some details, higher values can be measured. Usually these points are not in the main structure, but in secondary elements. The underestimation of stress ranges in some details may result in early fatigue cracking. Fatigue critical details may suffer from fatigue failure as result of secondary effects as bending moments, e.g. in connections. It also results from the interaction between main and secondary system (e.g. cross-beams and lateral wind bracing system, or the influence of the track system on the lateral distribution of the loading).

The objective of measurements of loads or loads effects is to gain information on the real structural system, the static and dynamic loading of the structure in order to reduce the uncertainties associated with the static calculations made in design or made in an assessment. The main areas of possible knowledge improvement can be summarised as follows:

- Verification of the real structural system and system details: type of connection, real bearing conditions, sensitivity to fatigue, etc. The calculation model is to be optimised for recalculation.
- Dynamic behaviour of a structure (estimation of dynamic amplification due to traffic and wind).
- Changes in structural response after local damage (e.g. buckling of members after collision).

The following sections mainly focus on strain measurements but also give some hints on other measurement methods used to gain information on the real structural system, the static and dynamic loading of steel bridges.

3.1.2 Sensors and measurement set-up

To guarantee an acceptable relation between data to be obtained and costs, the measurements have to be planned carefully. The information needed is important for the choice of number and type of sensors. Table 10 lists most commonly physical values and corresponding sensors to measure them.

Table 10. Physical values and commonly used corresponding sensors [9]

No.	Physical value	Sensors
1	Strain, stress, transverse load distribution	Strain gauges Fibre optic sensors (Bragg sensors, SOFO, ...) Mechanical strain devices
2	Position of the neutral axis	Strain gauges
3	Rotation	Inclinometer
4	Horizontal or transverse deflection, displacements, stiffness	Geodesic instruments, Laser, Inductive position encoder
5	Settlement of supports	Hydrostatic levelling system, Geodesic instruments
6	Dynamic response, vibration, damping, natural frequencies	Accelerometers, strain gauges
7	Static and In-service loading	weigh-in-motion (WIM) system Strain gauges calibration using defined traffic load

Strain gauges are the most precise system to get local strains needed in the fatigue assessment of fatigue critical details. For evaluating dynamic effects on the stress distribution of the structure strain gauges are the only system recommended. Preliminary assessment and previous experience, resulting from full-scale testing and fatigue failure analysis of already detected damages shall be analysed. The choice of cross sections to be assessed are based on this analysis.

3.1.3 Structural behaviour

Measurements can explain differences between model and real behaviour, differences between static and dynamic behaviour and therefore help improving the modelling, for example in the following cases [71]:

- Unintended composite behaviour
- Contributions to strength from non-structural elements, such as parapets
- Unintended partial end fixity at abutments
- Catenary tension forces due to “frozen” expansion joints or rigid end supports
- Longitudinal distribution of moment, unintended continuity at intermediate supports
- Direct transfer of load through the deck to supports in truss bridges
- Improper modelling of transverse load distribution
- Chord continuity and stiffness of joints in truss bridges, partial end fixity in connections

Depending on the element in which strain measurements are taken to understand its structural behaviour, according to [9] the following rules can help in reducing the number of sensors to the minimum needed:

- If the neutral axis in the main structure shifted due to contribution of the secondary load carrying system or an unintended composite behaviour, strain gauges, applied only to both flanges in the middle of a girder span, measure reliably the position of the neutral axis.
- To evaluate longitudinal distribution of forces, the recommended locations for the strain gauges are the middle and the quarter points of the span.
- To gain information about secondary bending effects or partial end fixity, strain gauges have to be applied only in cross-section near the members support or connection.
- For structural systems such as orthotropic decks carrying load in two directions it is advisable to use fewer multi-axial strain gauges (rosettes) instead of many uni-axial strain gauges.

Strain measurements are the most precise and common method to gain information on the real structural behaviour.

3.1.4 Permanent loads

The partial safety factor values used in assessment calculations can be lowered down using measurements. A measurement campaign should be carried out to gain information on real geometry value affecting dead and permanent loads such as depth of concrete deck, of surfacing, etc. This will give information on real dimensions and variability in these dimensions. Typically, 10 measurements per parameter (like the concrete slab thickness) shall be taken using a non-destructive instrument or a destructive method.

Strain measurements can not be used to gain information on dead or permanent loads.

3.1.5 Variable loads

In addition, the traffic loads or density change during the life of every bridge and in order to compute the remaining fatigue life, one must evaluate past, present and future traffic on the bridge. Therefore it is crucial to get an accurate estimation of the load and load effect distributions on bridges for fatigue issues. Any overestimation or crude assumption will highly penalise steel structures or bridges sensitive to fatigue, i.e. light structures. The same holds true for wind actions.

Variable loads can be measured as follows:

- By counting methods for traffic (using humans or video image treatment)
- By instrumenting some elements, usually the ones identified as being critical, with strain measurement systems (strain gauges, optic fibers, ...)
- By installing on the road weigh-in-motion (WIM) systems, either temporary or permanent, for measuring traffic volume and weighs.

The second is the only possibility to have action effects in the structure, at a particular location. When measuring traffic one must still rely on proper modelling of the structure to derive action effects representing realistic behaviour.

One particular measurement issue is the differentiation between static and dynamic effects.

Dynamic amplification measurements: In the past some methods have been developed and used for calculation of stress influence lines on the basis of measured values. Special software was generated. The German railways e.g. use an adapted procedure from Braune [72]

to get the influence line for her special load model from the engine BR 232 based on measured data.

Dynamic factors are usually the conservative upper limit of all dynamic influences, which in reality will not be reached. Stress-time curves, determined during the measurement of static traffic load from the vehicles in certain positions and during passage with the maximum allowed speed, give different values for main, cross and roadway beams. Speeds between 10 and 50 km/h do not result in significant different dynamic behaviour. The dynamic factor is a proportional component. Equal absolute dynamic values are lower percentage for high loads than for smaller vehicles. Experimental determining of dynamic factors should be based on measurements with the maximum allowed load on the evaluated bridge.

Weigh in motion systems (WIM): Continuous or sequential collection of WIM data on various types of road help to improve the knowledge - on longer periods of time - on operating traffic loads in different countries and sites. WIM data collected together with measurements of traffic load effects in existing bridges, can in addition give valuable information on dynamic effects on different bridge types and bridge elements. WIM data, combined with computer simulations, will reduce the uncertainties on the load side in fatigue assessment/design.

These considerations motivate the need for accurate WIM data, measured over long time periods, or at least periodically along the bridge lifetime. If no WIM data are available for a given structure, it is still possible to use other traffic patterns (records) for fatigue assessments, but they should be carefully chosen in the existing databases as representative of the real traffic conditions. Particular attention should be paid to the axle and vehicle load distributions and to the traffic volume. For local effects, the vehicle silhouettes and axle groups have also to be considered. Even if traffic measurements are carried out on an existing bridge, the fatigue assessment should take into account the whole past history and possible changes in the traffic pattern.

If traffic data are available for a given bridge for fatigue calculation, they still must be used with caution. For local effects, such as some details in orthotropic steel decks, the traffic measurements may be considered lane by lane, because the transverse influence length is very limited, and shorter than the lane width. But for global effects, such as those linked to the bending moment of a span (e.g. details of steel main girders), the traffic data must be collected simultaneously on all the traffic lanes, and the time history recorded, in order to provide the whole load configuration on the bridge deck at any time.

Another important question to be investigated before using traffic data for fatigue assessment is the representativity of the traffic sample. Most of the detailed (vehicle by vehicle) traffic records are limited to a few days or to a few weeks because of the limited memory size of the WIM systems. When calculating bridge lifetimes up to 50 to 200 years (or more), a strong hypothesis is made about the stationary nature of the traffic process. Even if the long-term changes are unpredictable, and thus not considered in these studies, the short traffic measurement period(s) must nevertheless be representative of longer ones. Measurements periods shorter than a week should in principle be avoided, because of the weekly periodicity and week-end effect, and the choice of the week of record within the year should be done carefully.

The use of traffic samples for fatigue verification requires a simulation programme, with a clear description explaining all the internal assumptions, as well as an users' manual. Such a software requires both the traffic data and the influence lines or surfaces of the bridge load effects to be considered, and a few more pieces of information given by the user (e.g. sampling rate). Then it computes in real time the stress range histograms (minima and maxima, level crossings and 'rainflow'), as well as the fatigue damage and lifetime for any given fatigue classes.

Summarising this section strain measurements can be used:

- to carry out dynamic amplification measurements and
- to gain information on local variable load effects on critical steel members.

It should be kept in mind that often the dynamic factors given in various design codes and the information on local variable load effects taken from statically models are good enough for an assessment of a steel bridge.

3.2 Basic information on strain measurements

3.2.1 General

The following basic information on strain measurements and related figures are taken out of the web-page <http://www.ni.com/>.

3.2.2 The Strain Gauge

While there are several methods of measuring strain, the most common is with a strain gauge, a device whose electrical resistance varies in proportion to the amount of strain in the device. The most widely used gauge is the bonded metallic strain gauge.

The metallic strain gauge consists of a very fine wire or, more commonly, metallic foil arranged in a grid pattern. The grid pattern maximizes the amount of metallic wire or foil subject to strain in the parallel direction, see Figure 9. The cross sectional area of the grid is minimized to reduce the effect of shear strain and Poisson Strain. The grid is bonded to a thin backing, called the carrier, which is attached directly to the test specimen. Therefore, the strain experienced by the test specimen is transferred directly to the strain gauge, which responds with a linear change in electrical resistance. Strain gauges are available commercially with nominal resistance values from 30 to 3000 Ω , with 120, 350, and 1000 Ω being the most common values.

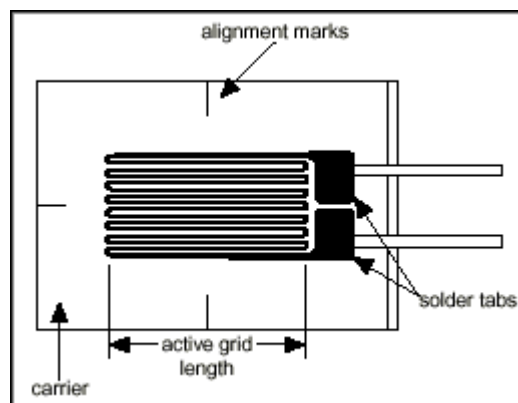


Figure 9 Bonded Metallic Strain Gauge

It is very important that the strain gauge be properly mounted onto the test specimen or onto a steel member so that the strain is accurately transferred from the test specimen, through the adhesive and strain gauge backing, to the foil itself.

A fundamental parameter of the strain gauge is its sensitivity to strain, expressed quantitatively as the gauge factor (GF). Gauge factor is defined as the ratio of fractional change in electrical resistance to the fractional change in length (strain):

$$GF = \frac{\Delta R/R}{\Delta L/L} = \frac{\Delta R/R}{\epsilon}$$

The Gauge Factor for metallic strain gauges is typically around 2.

3.2.3 Strain Gauge Measurement

In practice, the strain measurements rarely involve quantities larger than a few milli-strain ($\epsilon \times 10^{-3}$). Therefore, to measure the strain requires accurate measurement of very small changes in resistance. For example, suppose a test specimen undergoes a strain of $500 \mu\epsilon$. A strain gauge with a gauge factor of 2 will exhibit a change in electrical resistance of only $2 \times (500 \times 10^{-6}) = 0.1\%$. For a 120Ω gauge, this is a change of only 0.12Ω .

To measure such small changes in resistance, strain gauges are almost always used in a bridge configuration with a voltage excitation source. The general Wheatstone bridge, illustrated below, consists of four resistive arms with an excitation voltage, V_{EX} , that is applied across the bridge.

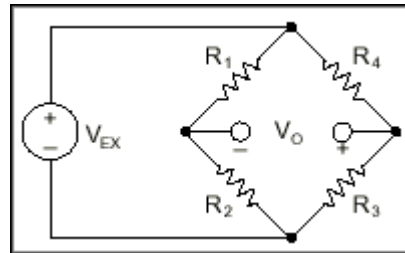


Figure 10 Wheatstone Bridge

The output voltage of the bridge, V_O , will be equal to:

$$V_o = \left[\frac{R_3}{R_3 + R_4} - \frac{R_2}{R_1 + R_2} \right] \times V_{EX}$$

From this equation, it is apparent that when $R_1/R_2 = R_4/R_3$, the voltage output V_O will be zero. Under these conditions, the bridge is said to be balanced. Any change in resistance in any arm of the bridge will result in a nonzero output voltage.

Therefore, if we replace R_4 in Figure 10 with an active strain gauge, any changes in the strain gauge resistance will unbalance the bridge and produce a nonzero output voltage. If the nominal resistance of the strain gauge is designated as R_G , then the strain-induced change in resistance, ΔR , can be expressed as $\Delta R = R_G \cdot GF \cdot \epsilon$. Assuming that $R_1 = R_2$ and $R_3 = R_G$, the bridge Eq. above can be rewritten to express V_O/V_{EX} as a function of strain, see Figure 11. Note the presence of the $1/(1+GF \cdot \epsilon/2)$ term that indicates the non-linearity of the quarter-bridge output with respect to strain.

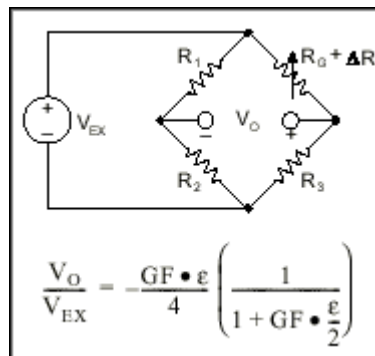


Figure 11 Quarter-Bridge Circuit

Ideally, we would like the resistance of the strain gauge to change only in response to applied strain. However, strain gauge material, as well as the specimen material to which the

gauge is applied, will also respond to changes in temperature. Strain gauge manufacturers attempt to minimize sensitivity to temperature by processing the gauge material to compensate for the thermal expansion of the specimen material for which the gauge is intended. While compensated gauges reduce the thermal sensitivity, they do not totally remove it.

By using two strain gauges in the bridge, the effect of temperature can be further minimized. For example, Figure 12 illustrates a strain gauge configuration where one gauge is active ($R_G + \Delta R$), and a second gauge is placed transverse to the applied strain. Therefore, the strain has little effect on the second gauge, called the dummy gauge. However, any changes in temperature will affect both gauges in the same way. Because the temperature changes are identical in the two gauges, the ratio of their resistance does not change, the voltage V_O does not change, and the effects of the temperature change are minimized.

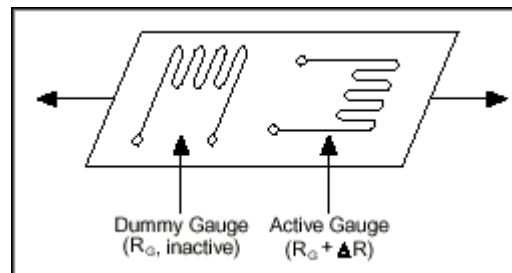


Figure 12 Use of Dummy Gauge to Eliminate Temperature Effects

The sensitivity of the bridge to strain can be doubled by making both gauges active in a half-bridge configuration. For example, Figure 13 illustrates a bending beam application with one bridge mounted in tension ($R_G + \Delta R$) and the other mounted in compression ($R_G - \Delta R$). This half-bridge configuration, whose circuit diagram is also illustrated in Figure 13, yields an output voltage that is linear and approximately doubles the output of the quarter-bridge circuit.

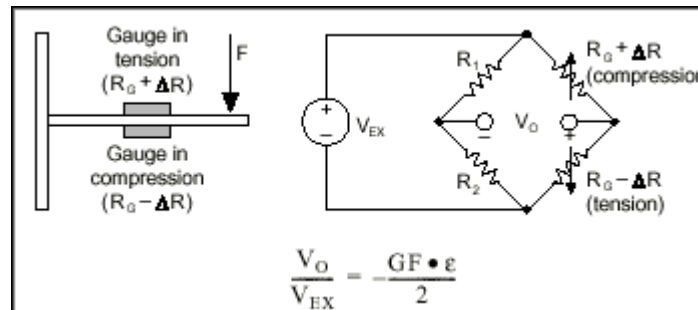


Figure 13 Half-Bridge Circuit

Finally, you can further increase the sensitivity of the circuit by making all four of the arms of the bridge active strain gauges in a full-bridge configuration. The full-bridge circuit is shown in Figure 14.

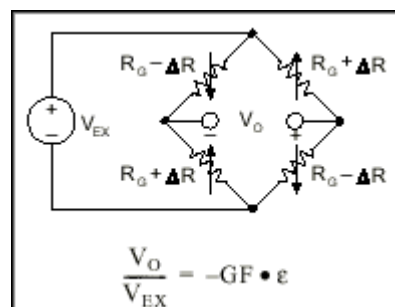


Figure 14 Full-Bridge Circuit

The equations given here for the Wheatstone bridge circuits assume an initially balanced bridge that generates zero output when no strain is applied. In practice however, resistance tolerances and strain induced by gauge application will generate some initial offset voltage. This initial offset voltage is typically handled in two ways. First, you can use a special offset-nulling, or balancing, circuit to adjust the resistance in the bridge to rebalance the bridge to zero output. Alternatively, you can measure the initial unstrained output of the circuit and compensate in software.

The equations given above for quarter, half, and full-bridge strain gauge configurations assume that the lead wire resistance is negligible. While ignoring the lead resistances may be beneficial to understanding the basics of strain gauge measurements, doing so in practice can be a major source of error. For example, consider the 2-wire connection of a strain gauge shown in Figure 15 a). Suppose each lead wire connected to the strain gauge is 15 m long with lead resistance R_L equal to $1\ \Omega$. Therefore, the lead resistance adds $2\ \Omega$ of resistance to that arm of the bridge. Besides adding an offset error, the lead resistance also desensitises the output of the bridge.

You can compensate for this error by measuring the lead resistance R_L and accounting for it in the strain calculations. However, a more difficult problem arises from changes in the lead resistance due to temperature fluctuations. Given typical temperature coefficients for copper wire, a slight change in temperature can generate a measurement error of several $\mu\epsilon$.

Using a 3-wire connection can eliminate the effects of variable lead wire resistance because the lead resistances affect adjacent legs of the bridge. As seen in Figure 15 b), changes in lead wire resistance, R_2 , do not change the ratio of the bridge legs R_3 and R_G . Therefore, any changes in resistance due to temperature cancel each other.

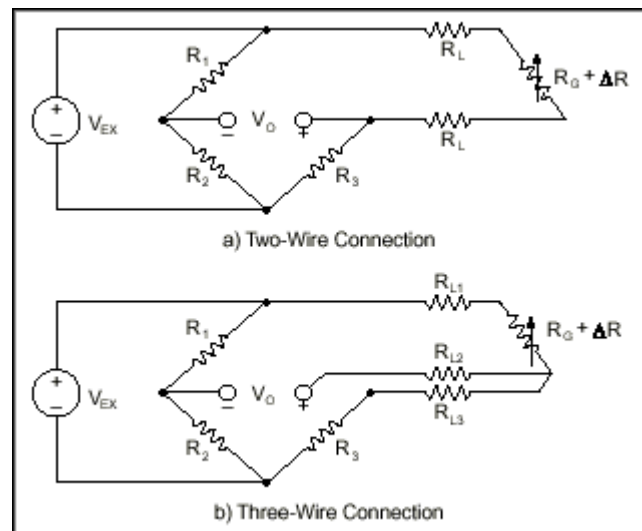


Figure 15 a) 2-Wire and b) 3-Wire Connections of Quarter-Bridge Circuit

3.2.4 Signal Conditioning for Strain Gauges

Strain gauge measurement involves sensing extremely small changes in resistance. Therefore, proper selection and use of the bridge, signal conditioning, wiring, and data acquisition components are required for reliable measurements. To ensure accurate strain measurements, it is important to consider the following:

- Bridge completion
- Excitation
- Remote sensing
- Amplification

- Filtering
- Offset
- Shunt calibration

Bridge Completion – Unless you are using a full-bridge strain gauge sensor with four active gauges, you will need to complete the bridge with reference resistors. Therefore, strain gauge signal conditioners typically provide half-bridge completion networks consisting of high-precision reference resistors. Figure 16 shows the wiring of a half-bridge strain gauge circuit to a conditioner with completion resistors R_1 and R_2 .

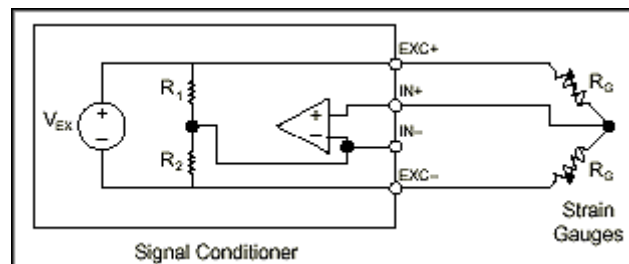


Figure 16 Connection of Half-Bridge Strain Gauge Circuit

Excitation – Strain gauge signal conditioners typically provide a constant voltage source to power the bridge. While there is no standard voltage level that is recognized industry wide, excitation voltage levels of around 3 and 10 V are common. While a higher excitation voltage generates a proportionately higher output voltage, the higher voltage can also cause larger errors because of self-heating.

Remote Sensing – If the strain gauge circuit is located a distance away from the signal conditioner and excitation source, a possible source of error is voltage drop caused by resistance in the wires connecting the excitation voltage to the bridge. Therefore, some signal conditioners include a feature called remote sensing to compensate for this error. Remote sense wires are connected to the point where the excitation voltage wires connect to the bridge circuit. The extra sense wires serve to regulate the excitation supply through negative feedback amplifiers to compensate for lead losses and deliver the needed voltage at the bridge.

Amplification – The output of strain gauges and bridges is relatively small. In practice, most strain gauge bridges and strain-based transducers will output less than 10 mV/V (10 mV of output per volt of excitation voltage). With 10 V excitation, the output signal will be 100 mV. Therefore, strain gauge signal conditioners usually include amplifiers to boost the signal level to increase measurement resolution and improve signal-to-noise ratios.

Filtering – Strain gauges are often located in electrically noisy environments. It is therefore essential to be able to eliminate noise that can couple to strain gauges. Low-pass filters, when used in conjunction with strain gauges, can remove high-frequency noise prevalent in most environmental settings.

Offset Nulling – When a bridge is installed, it is very unlikely that the bridge will output exactly zero volts when no strain is applied. Slight variations in resistance among the bridge arms and lead resistance will generate some nonzero initial offset voltage. Offset nulling can be performed by either hardware or software:

1. **Software Compensation** – With this method, you take an initial measurement before strain input is applied, and use this offset to compensate subsequent measurements. This method is simple, fast, and requires no manual adjustments. The disadvantage of the software compensation method is that the offset of the bridge is not removed. If the offset is large enough, it limits the amplifier gain you can apply to the output voltage, thus limiting the dynamic range of the measurement.

2. **Offset-Nulling Circuit** – The second balancing method uses an adjustable resistance, a potentiometer, to physically adjust the output of the bridge to zero. By varying the resistance of potentiometer, you can control the level of the bridge output and set the initial output to zero volts.

Shunt Calibration – The normal procedure to verify the output of a strain gauge measurement system relative to some predetermined mechanical input or strain is called shunt calibration. Shunt calibration involves simulating the input of strain by changing the resistance of an arm in the bridge by some known amount. This is accomplished by shunting, or connecting, a large resistor of known value across one arm of the bridge, creating a known ΔR . The output of the bridge can then be measured and compared to the expected voltage value. The results are used to correct span errors in the entire measurement path, or to simply verify general operation to gain confidence in the set-up.

3.2.5 DAQ Systems for Strain Gauge Measurements

General

Further information on DAQ systems, on their features for strain gauges and on recommended starter kit for strain gauge DAQ systems can be found at the internet, e.g. on the web-page <http://www.ni.com/>.

Using SCXI with Strain Gauges

SCXI is a signal conditioning system for PC-based instrumentation applications. An SCXI system consists of a shielded chassis that houses a combination of signal conditioning input and output modules, which perform a variety of signal conditioning functions. You can connect many different types of sensors, including strain gauges, directly to SCXI modules. The SCXI system operates as a front-end signal conditioning system for PC plug-in data acquisition (DAQ) devices (PCI and PCMCIA) or PXI DAQ modules.

Using SCC with Strain Gauge Measurements

SCC provides portable, modular signal conditioning for DAQ systems. SCC can condition a variety of analog I/O and digital I/O signals. SCC DAQ systems include a shielded carrier, SCC modules, a cable, and a DAQ device. Figure 18 below illustrates a carrier with SCC modules.



Figure 17 SCXI Signal Conditioning System



Figure 18 Shielded carrier with SCC Modules

3.3 Rivets

The major part of the old steel railway bridges in Europe are riveted structures. Other steel construction methods like welding and bolting were only used since 1960 in bridge structures when active shielding gases – in particular for the welding process using covered rod electrodes – and appropriate high-strength fit bolts became available. Typical examples of an old riveted steel bridge, nowadays exposed to extreme railway traffic, are the “Hohenzollern”-Bridge in Cologne, rebuilt and in service since 1948, see Figure 19, and the “Hochdonn”-Bridge crossing over the “Nord-Ostsee-Kanal”, see Figure 20.



Figure 19 Typical old riveted steel railway bridge (Hohenzollernbrücke in Cologne)



Figure 20 Typical old riveted steel railway bridge (Hochdonnbrücke north of Hamburg)

For rivets only few product standards exist throughout Europe. In Germany the product standard DIN 101 is available [73], making reference of the related material standard DIN 17111 [74]. DIN 101 contains acceptance criteria and tolerance limits for dimensions, permitted error of coaxiality and rectangularity as well as some test procedures to check material requirements such as Vickers-hardness and head impact toughness.

Current research on the remaining load-bearing capacity, rivet pre-stress and the influence corrosive deterioration is presented in this report, see section 4.1.5 and [75].

3.4 Bolts

In general, non-preloaded bolts do not require any measurement for condition assessment, visual inspection is sufficient. On the other hand, for preloaded bolts it is an essential requirement to prevent a loss of preload to maintain full load-bearing capacity of the connection, which can not be ascertained with visual inspection solely. Unfortunately, for detection of a potential loss of preload an adequate method for the direct measurement of the current preload is presently not available.

Restricted to preloaded bolts tightened using the torque control method, one possible solution, adequate in most practical cases, is to apply a specific torque moment and inspect the further rotation angle of the nut, as described in section 2.6.5. A bolt which turns by more than 15° due to application of the inspection torque is then considered defective for under-tightening and shall be retightened up to 110 % of the specified minimum preloading force.

One exceptional method for direct stress measurement – rarely used in practical application – is offered by an ultrasonic measurement method, see [8]. This so-called “combined ultrasonic method” was developed in 1987 by the Fraunhofer-Institute for non-destructive testing in Saarbrücken.

Basis of this method is a combined measurement of longitudinal and transversal ultrasonic waves. Although the running distance of both waves varies uniformly with a change in preload, in contrast to that the running time varies differently with a change of bolt tension stresses. In consequence, the bolt strain can be determined directly from a simultaneous measuring of the running time of the longitudinal and transversal waves, even if the initial bolt length is unknown.

For the elastic bolt strain in longitudinal direction ε the following Eq. is derived analytically:

$$\varepsilon = \frac{t_L - t_T Q}{t_T K_T Q - t_L K_L}$$

where: t_L = running time of the longitudinal wave

t_T = running time of the transversal wave

$$Q = \sqrt{\frac{\mu}{\lambda + 2\mu}} = \sqrt{\frac{1 - 2\nu}{2(1 - \nu)}} = \text{material constant}$$

$$K_L = \left[2 + \frac{\mu + 2m}{\lambda + 2\mu} + \frac{\mu\lambda}{2(\lambda + \mu)(\lambda + 2\mu)} \left(1 + \frac{2l}{\lambda} \right) \right] = \text{longitudinal material constant}$$

$$K_T = \left[2 + \frac{\lambda n}{8\mu(\lambda + \mu)} + \frac{m}{2(\lambda + \mu)} \right] = \text{transversal material constant}$$

and: μ, λ = Lamé modules ($\mu = G$ = shear modulus)

ν = Poisson ratio

E = Elastic modulus = $\mu(3\lambda + 2\mu) / (\lambda + \mu)$

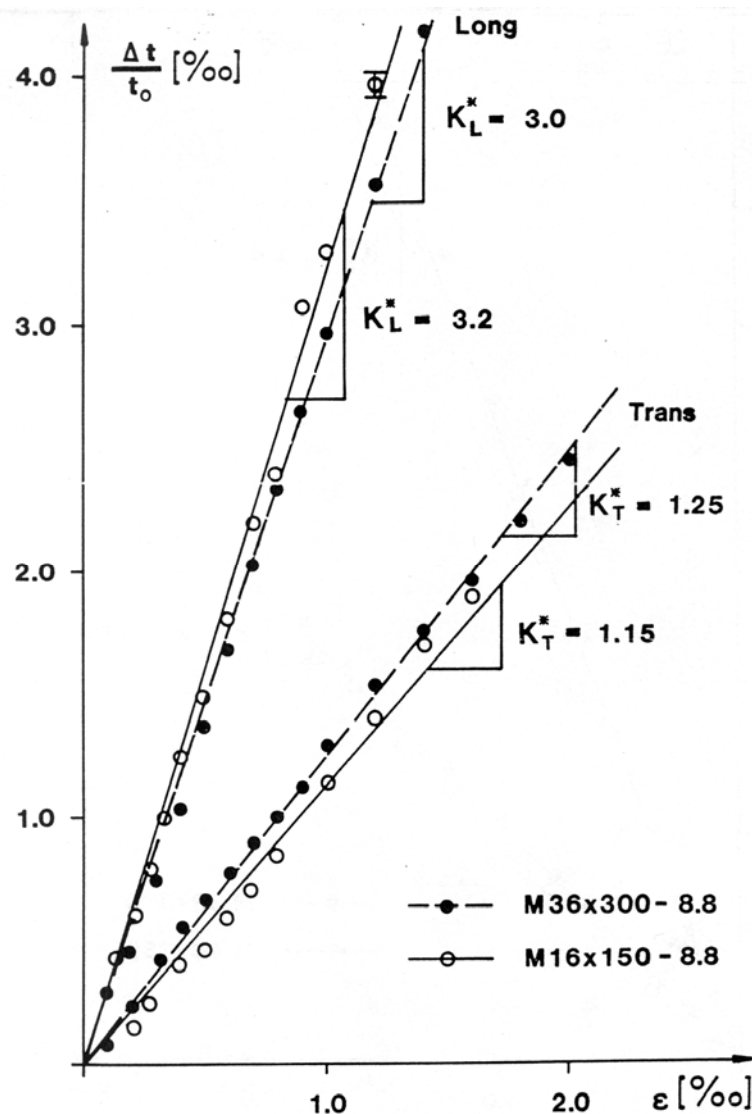
l, m, n = 3rd order elastic constants

The relevant material properties for some typical bolt materials are summed up in Table 11.

The characteristic material constant parameters $K_L = 1 - K_L^*$ and $K_T = 1 - K_T^*$ can also be evaluated experimentally. For two particular cases the results are illustrated in Figure 21.

Table 11. Elastic constants of different metals

Material	λ GPa	μ GPa	l GPa	m GPa	n GPa	K_L	K_T	Q
Stahl St 42	110	81	- 48	-503	-652	-1,390	+0,1038	0,5457
22 NiMoCr 37	109	82	-196	-520	-657	-1,732	+0,0672	0,5481
24CrMo5V	112	82	-440	-600	-670	-2,639	-0,1360	0,5451
30CrMoNiV51	109	83	-357	-574	-670	-2,348	-0,0676	0,5494

Figure 21 Relative difference of the running time $(t - t_0) / t_0$ of a longitudinal and a transversal ultrasonic wave as a function of bolt strain

A comparison of bolt strains evaluated from combined ultrasonic method and from conventional strain gauge measurement for two different types of bolts M36x300, 8.8 and M16x150,

8.8 is shown in Figure 22, together with the bisecting line, proving the good agreement of results.

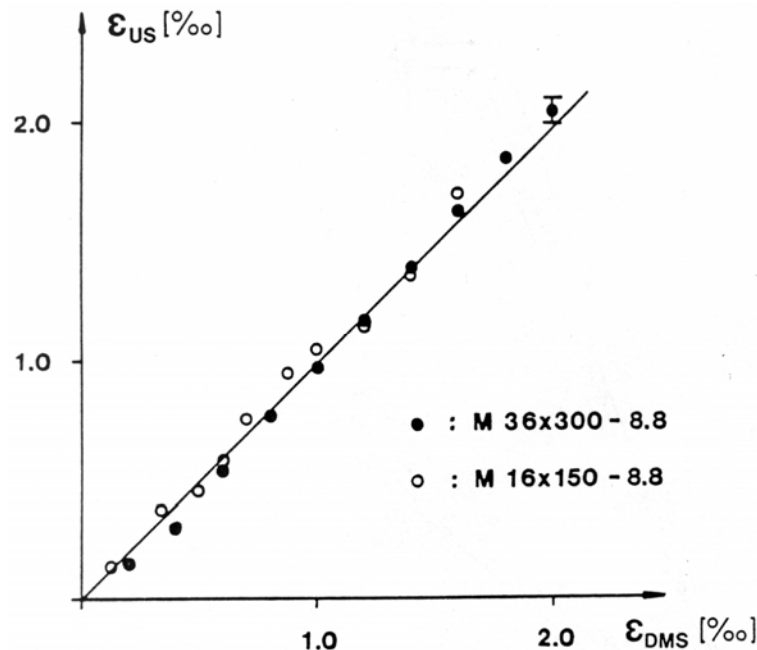


Figure 22 Comparison of bolt strains evaluated from combined ultrasonic method and from conventional strain gauge measurement

Note, that the combined ultrasonic method is restricted to bolt stresses in the elastic range and planar bolt heads only.

Within the scope of the investigation to establish this method the influence of material properties, temperature and torsional effects on the measurements were evaluated and correction terms were proposed. Also the ultrasonic converter unit for sending and receiving of the longitudinal and transversal waves was optimized and miniaturized for easy handling. For more detailed information on this method see [8].

3.5 Welds

Measurement methods and acceptance criteria for NDT of weld defects are comprehensively described in section 2.6.4. More detailed information can be found in the current IIW-Recommendations on the repair of fatigue loaded welded structures [76] which were issued by the IIW Joint Working Group XIII-XV in 2004.

Therefore, this report does not go more into detail.

3.6 Surface treatment

For condition assessment of the surface treatment several methods are available. Usually only the remaining thickness of the surface coating for corrosion protection is of particular interest. This can be a metallic coating like a zinc-coating or a non-metallic coating like paint, see section 2.6.6 for more information on the state-of-the-art of condition assessment and acceptance criteria of surface treatment.

Thickness gauges like fully electronic thickness measuring units in robust compact version exist for all metal base materials. Beside ultrasonic mainly magnet-inductive measuring devices are used for thickness metering of non-ferromagnetic coatings and paint on ferromagnetic steel and non-iron base materials. For steel base material the commonly utilized principle makes use of the magnet-inductive and eddy current method to the fast and exact non-

destructive measuring of coating thicknesses. The coating surface is touched with a sensor. It is good enough to have an uncoated base material sample for the calibration of the measuring device. The principle of the method is the replacement of the coating by a defined air gap produced between the sensor and the base material sample.

4 Criteria for replacement of components and connections of steel railway bridges

4.1 Fitness-for-Service acceptance criteria

4.1.1 General

Fitness-for-service measures are required for old steel railway bridges, if damages or other deterioration defects are detected during inspection which reduce the load-bearing capacity, fatigue resistance or the serviceability of the structure. For securing a sufficient remaining service life, it is necessary to define criteria for necessity, choice and execution of strengthening measures for components and connections of steel railway bridges. Note, that a survey of current application and future requirements for European Fitness-for-Service technology is currently undertaken within the European Fitness-for-service Network [77].

Section 4 focuses on fitness-for-service for all types of members loaded by cyclic tension and bending stresses regardless of members having bolted, riveted or welded connections. Due to the substantial role of hot rivets in old steel railway bridges also specific replacement criteria for hot rivets affected by corrosive surface defects are evaluated.

To carry out fitness-for-service assessments on old steel railway bridges more and more fracture mechanics calculations were used. In the field of civil engineering, such methods were introduced in the nineties of the last century, e.g. see [78], [79]. Nowadays these methods are developed in a way that also some design guides are based on it, e.g. see Ril 805 of the German Railway [5] or SBB-Weisung [80] of the Swiss Railway. All of these fracture mechanics based assessment methods have a common ground; it based on the assumption of cracks in the assessed steel member. In most cases of fitness-for-service assessments cracks were assumed notionally at a critical location of a steel member. In some rare cases cracks were found at a steel structure by inspections. In such cases the real dimension of the cracks were used in the assessment.

If the visible initial crack length a_0 is assumed or measured and if the critical crack length a_{crit} is determined, one can calculate the maximum permissible number of load cycles for a steel member under a certain fatigue load. This maximum permissible number of load cycles defines the period in which a crack growth under fatigue loads starting with an initial crack length a_0 to a critical length a_{crit} . For that the maximum permissible number of load cycles gives a hint on the residual service life of the structure / member and on its robustness.

After the calculation of maximum permissible number of load cycles two cases can occur:

1. The maximum permissible number of load cycles is higher than the number of load cycles occurring between two inspections.
2. The maximum permissible number of load cycles is lower than the number of load cycles occurring between two inspections.

In the first case the structure / member has a sufficient robustness against crack initiation and crack growth. However, in the second case the robustness is insufficient and either the inspection interval must be decreased or the assessed structure / member has to be strengthened.

The following sections describe the determination of the maximum permissible number of load cycles (section 4.1.2), describe the calculation of eventually necessary strengthening measures for members under tension or bending stresses to increase its resistance against crack growth (section 4.1.3) and describe the calculation of eventually necessary strengthening measures to increase the bearing capacity (section 4.1.4) according to [78].

4.1.2 Determination of the maximum permissible number of load cycles

The most common formula in fracture mechanics calculations for the determination of the maximum permissible number of load cycles is the so-called Paris-equation [81].

$$\frac{da}{dN} = C \cdot \Delta K^m \quad (1)$$

with: $\Delta K = \Delta \sigma \cdot \sqrt{\pi \cdot a} \cdot Y(a, T)$

C, m = material constants

According to Hensen [78] for normal old steels the material constants C and m can be taken as $C = 4 \cdot 10^{-13}$ and $m = 3$ to calculate the crack growth conservatively using the Paris-equation.

An integration of Eq.(1) leads to a formula for the determination of the number of load cycles N in relation to the crack length a .

$$N = \int_{a_0}^{a_{crit}} \frac{da}{C \cdot \Delta K^m} \quad (2)$$

The integration of Eq.(2) can be expressed by a sum function as follows:

$$N = \sum_{i=0}^{i=crit} \frac{a_{i+1} - a_i}{C \cdot \left\{ \Delta \sigma \cdot \sqrt{\pi} \cdot \left[\sqrt{a_{i+1}} \cdot Y(a_{i+1}, T) - \sqrt{a_i} \cdot Y(a_i, T) \right] \right\}^m} \quad (3)$$

The Eq.(3) can be well used for programming.

Figure 23 shows the result of a crack growth calculation. The maximum permissible number of load cycles N can be determined by subtracting the number of load cycles N_0 related to the initial crack length a_0 from the number of load cycles N_{crit} related to the critical crack length a_{crit} . Figure 23 shows this approach, too.

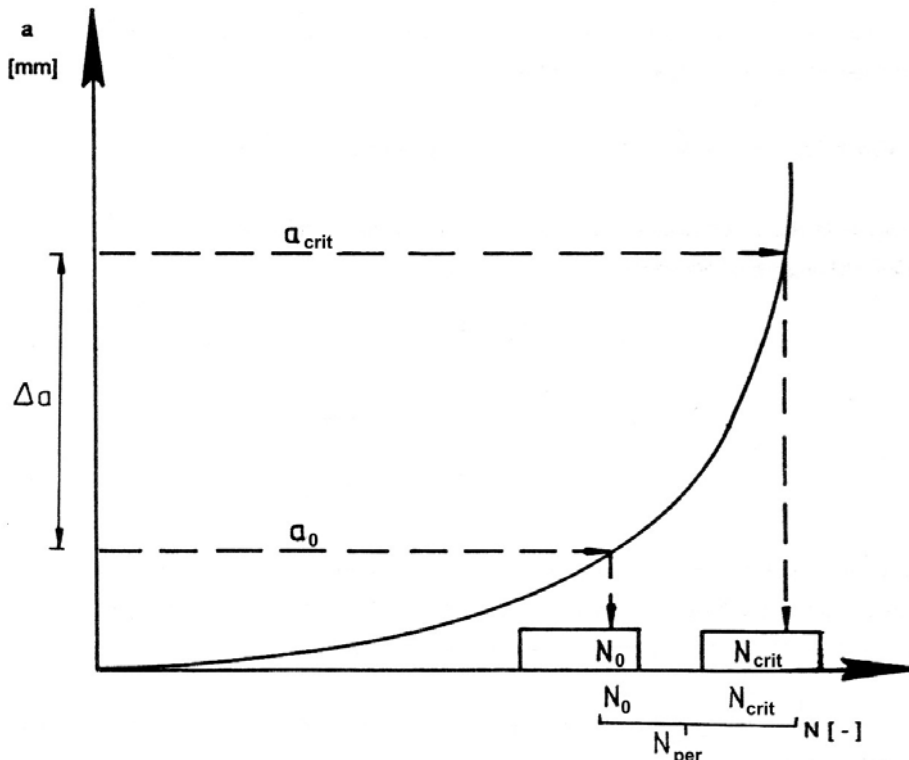


Figure 23 Result of a crack growth calculation according to Eq.(1)

To simplify the fracture mechanics calculation Hensen [78] has derived tabulated values for the number of load cycles N based on:

- Crack growth calculations using Eq.(1) and material constant $C = 4 \cdot 10^{-13}$ and $m = 3$
- Three different geometric models (plates under cyclic tension loads with through cracks on each side, with a through crack only on one side and with a trough crack in the middle of the plate)
- Nine different cyclic stress level $\Delta\sigma$ (10, 15, 20, 25, 30, 40, 50, 60, 80 N/mm²)
- Modified geometrical correction function $Y(a,T)$ derived from a function according to Tada [82] – for the derivation of the modified function see [78], section 3.

Figure 24 shows exemplarily the tabulated values of N resulting from crack growth calculations on a plate with through cracks on both sides under a cyclic tension load $\Delta\sigma = 20 \text{ N/mm}^2$ for various plate widths T .

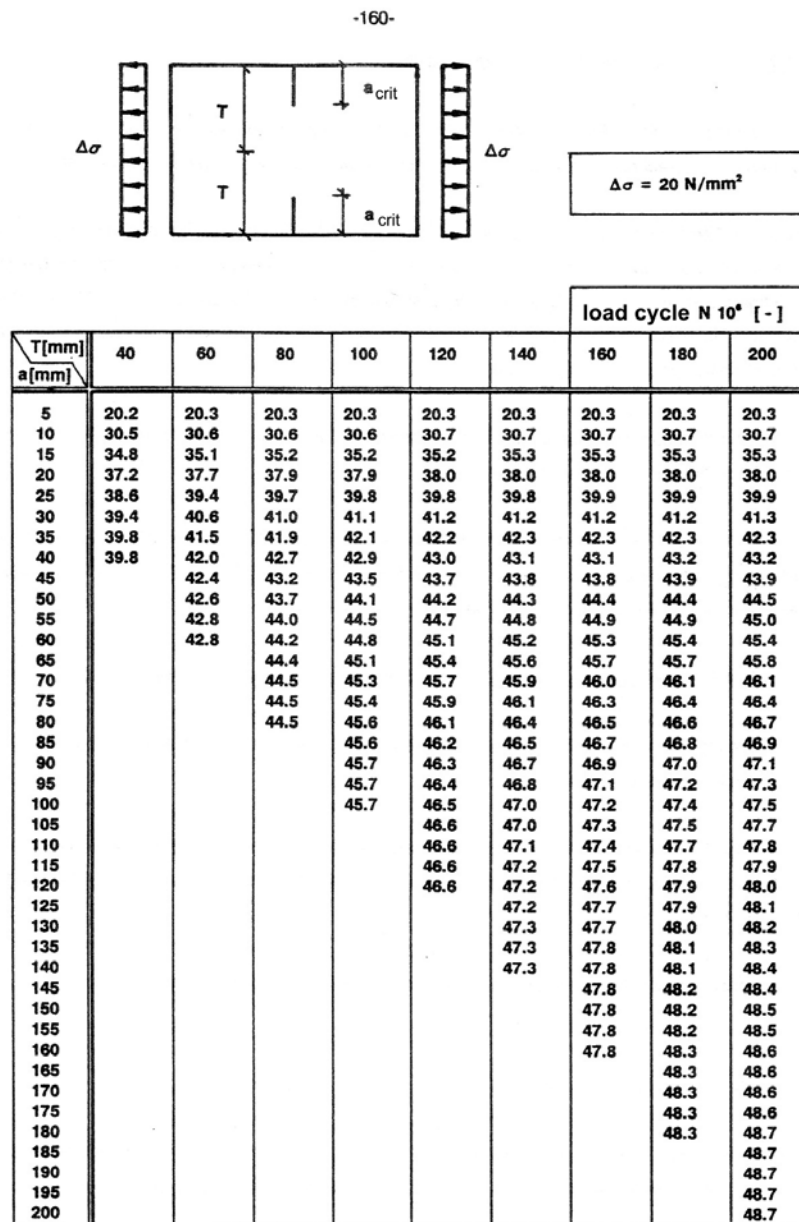


Figure 24 Example for tabulated values of N resulting from crack growth calculations on a plate with through cracks on both sides under a cyclic tension load $\Delta\sigma = 20 \text{ N/mm}^2$ for various plate widths T according to Hensen [78], annex A4, page 3

For more tabulated values see [78], annex A4 to A6. Linear interpolation can be used in applying the tables in these annexes.

For the determination of the maximum permissible number of load cycles N_{per} on the basis of tabulated values the following principally approach can be used:

1. Determination of the relevant crack configuration in relation to the relevant structural detail using [78], annex I
2. Determination of the relevant cyclic stress level $\Delta\sigma$
3. Choice of the relevant table ([78], annex A4 to A6) related to the results of 1. and 2.
4. Estimation of the plate width of the geometric model in relation to the relevant structural detail using [78], annex I
5. Assuming / measuring initial crack length a_0
6. Determination of the critical crack length a_{crit} using tabulated values ([78], annex A1 to A3) related to:
 - the fracture toughness expressed as J_c at low temperatures,
 - the relevant crack configuration, see also point 1.
 - the ratio of the maximum stress σ taken out of static calculation to the yield strength σ_F of the material
 - the dimension of the geometrical model, see also point 4.
7. Determination of the number of load cycles N_0 related to the initial crack length a_0 and the number of load cycles N_{crit} related to the critical crack length a_{crit} using the relevant table, see also point 3.
8. The subtraction of the number of load cycles N_0 from the number of load cycles N_{crit} leads to the maximum permissible number of load cycles N_{per} .

$$N_{per} = N_{crit}(a_{crit}) - N_0(a_0) \quad (4)$$

As mentioned also in section 4.1.1 two cases can be occur:

1. The maximum permissible number of load cycles N_{per} is higher than the number of load cycles N_{insp} occurring between two inspections.
2. The maximum permissible number of load cycles N_{per} is lower than the number of load cycles N_{insp} occurring between two inspections.

In the first case ($N_{per} \geq N_{insp}$) the structure / member has a sufficient robustness against crack initiation and crack growth. But in the second case ($N_{per} < N_{insp}$) the robustness is insufficient and either the inspection interval must be decreased or the assessed structure / member has to be strengthened.

The required kind and dimension of the strengthening measure is related

- to the geometry of the structure / member,
- to the kind of load and
- to the size of the load.

The following section shows the derivation of formula for the calculation of the required strengthening.

4.1.3 Calculation of necessary strengthening measures for members under tension or bending stresses to increase their resistance against crack growth

In case that the maximum permissible number of load cycles N_{per} is lower than the number of load cycles N_{insp} occurring between two inspections the robustness is insufficient and e.g. the inspection interval must be decreased. This possibility leads often to a high financial effort, because the bridge owner often do not have enough staff to inspect a bridge more often than normally and therefore the owner has to assign external experts. Due to this fact, bridge owner often are interested in strengthening measures to increase the robustness of a structure and therefore also their resistance against crack growth.

Due to the difficulties in strengthening of old riveted structures without replacement of load carrying rivets in connections, the cross section of the strengthening should be not bigger than necessary.

The strengthening has to satisfy two boundary conditions:

1. The cross section of the strengthening has to be as large as the cyclic stress range was decreased in a way that the maximum permissible number of load cycles N_{per} becomes higher than the number of load cycles N_{insp} (N_{act}).
2. The cross section of the strengthening has to be as large as necessary for an accidental load case, where a part or the completely old cross section fails brittle. Normally the failure of the completely old cross section was conservatively assumed and due to that, the cross section of the strengthening has to carry the full maximum load.

The dimension of the strengthening cross section has to meet the requirements of these both boundary conditions. The first one is related to cyclic loading on a structure and the second one is related to an ultimate limit state in an accidental load case.

This section deals with the effects of cyclic loading and the required strengthening cross section.

As mentioned in section 4.1.2, too, the crack growth under an actual stress range $\Delta\sigma_{act}$ can be described according to Paris [81] as follows:

$$N_{act} = \sum_{i=0}^{i=crit} \frac{a_{i+1} - a_i}{C \cdot \left\{ \Delta\sigma_{act} \cdot \sqrt{\pi} \cdot \left[\sqrt{a_{i+1}} \cdot Y(a_{i+1}, T) - \sqrt{a_i} \cdot Y(a_i, T) \right] \right\}^m} \quad (5)$$

In the case that the maximum permissible number of load cycles N_{per} is lower than the number of load cycles N_{insp} occurring between two inspections, the stress range has to be reduced to increase N_{per} and to meet the equilibrium $N_{per} \geq N_{insp}$. The reduced required stress range is called $\Delta\sigma_{req}$ and therefore the Eq.(5) is changed to:

$$N_{req} = \sum_{i=0}^{i=crit} \frac{a_{i+1} - a_i}{C \cdot \left\{ \Delta\sigma_{req} \cdot \sqrt{\pi} \cdot \left[\sqrt{a_{i+1}} \cdot Y(a_{i+1}, T) - \sqrt{a_i} \cdot Y(a_i, T) \right] \right\}^m} \quad (6)$$

To derive a relationship between the actual and required stress range $\Delta\sigma_{act}$ and $\Delta\sigma_{req}$ to the maximum permissible number of load cycles N_{per} and the number of load cycles N_{insp} (also called N_{act}) occurring between two inspections, the ratio of Eq.(5) and (6) has to be made.

$$\frac{N_{act}}{N_{req}} = \frac{\sum_{i=0}^{i=crit} \frac{a_{i+1} - a_i}{C \cdot \left\{ \Delta\sigma_{act} \cdot \sqrt{\pi} \cdot \left[\sqrt{a_{i+1}} \cdot Y(a_{i+1}, T) - \sqrt{a_i} \cdot Y(a_i, T) \right] \right\}^m}}{\sum_{i=0}^{i=crit} \frac{a_{i+1} - a_i}{C \cdot \left\{ \Delta\sigma_{req} \cdot \sqrt{\pi} \cdot \left[\sqrt{a_{i+1}} \cdot Y(a_{i+1}, T) - \sqrt{a_i} \cdot Y(a_i, T) \right] \right\}^m}} \quad (7)$$

After mathematical transformation the following relationship can be found:

$$\Delta\sigma_{req} = \Delta\sigma_{act} \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \quad (8)$$

To explain the denotation of Eq.(8) it can be read from the more common point of view of fatigue design using S/N-curves instead of reading it from the fracture mechanical point of view. Figure 25 shows the relationship of the number of load cycles to a non-snapped off (linear) S/N-curve.

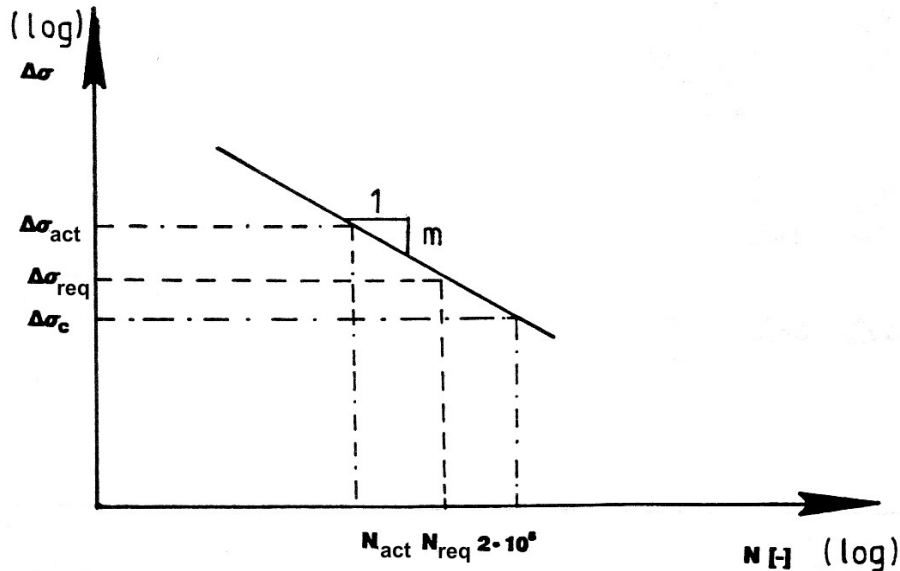


Figure 25 Number of load cycles in relation to a non-snapped off (linear) S/N-curve

An S/N-curve is defined by a characteristic point ($\Delta\sigma_c$ and $N = 2 \cdot 10^6$) as well as a characteristic slope m . The end of the theoretically fatigue life is defined by reaching the S/N-curve.

This means for a certain actual situation:

$$N_{act} = \frac{\Delta\sigma_c^m \cdot 2 \cdot 10^6}{\Delta\sigma_{act}^m} \quad (9)$$

The Eq.(9) can be transferred to a required number of load cycles, too.

$$N_{req} = \frac{\Delta\sigma_c^m \cdot 2 \cdot 10^6}{\Delta\sigma_{req}^m} \quad (10)$$

To derive a relationship between the stress ranges $\Delta\sigma_{act}$ and $\Delta\sigma_{req}$ to the number of load cycles N_{per} and N_{req} , the ratio of Eq.(9) and (10) has to be made.

$$\frac{N_{act}}{N_{req}} = \frac{\frac{\Delta\sigma_c^m \cdot 2 \cdot 10^6}{\Delta\sigma_{act}^m}}{\frac{\Delta\sigma_c^m \cdot 2 \cdot 10^6}{\Delta\sigma_{req}^m}} \quad (11)$$

After mathematical transformation again the following relationship can be found:

$$\Delta\sigma_{req} = \Delta\sigma_{act} \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \quad (12)$$

On the basis of Eq.(12) equations are derived to calculate the required cross section of strengthening of members under tension load and under bending load, see the following two subsections.

Calculation of required cross section of strengthening of members under tension load

Members of truss girders mainly show normal stresses. For members under tension load the decisive stress range $\Delta\sigma$ due to traffic loads is:

$$\Delta\sigma = \frac{\Delta F_t}{A_{net}} \quad (13)$$

where ΔF_t is the algebraic difference between the two extremes of a particular tension force cycle derived from a traffic load history and A_{net} is the net cross section.

The cross section of the strengthening has to be as large as the cyclic stress range $\Delta\sigma$ was decreased in a way that the maximum permissible number of load cycles N_{per} becomes higher than the number of load cycles N_{insp} (N_{act}) between two inspections. With the assumption that the traffic load on a structure was not decreased, a reduction of $\Delta\sigma$ can only be achieved by extension of the cross section (strengthening). Applying Eq.(13) on the actual state and on the state after strengthening leads to the following formulas:

Actual state

$$\Delta\sigma_{act} = \frac{\Delta F_{t,act}}{A_{net,act}} \quad (14)$$

State after strengthening

$$\Delta\sigma_{req} = \frac{\Delta F_{t,act}}{A_{net,req}} \quad (15)$$

The replacement of $\Delta\sigma_{act}$ and $\Delta\sigma_{req}$ in Eq.(12) by the equations (14) and (15) leads to the following equation:

$$\frac{\Delta F_{t,act}}{A_{net,req}} = \frac{\Delta F_{t,act}}{A_{net,act}} \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \quad (16)$$

After mathematical transformation, an equation can be derived to calculate the required cross section of the strengthening ΔA_{req} :

$$\begin{aligned} A_{net,req} &= A_{net,act} \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \\ \text{with} \\ \Delta A_{req} &= A_{net,req} - A_{net,act} \\ \Rightarrow \Delta A_{req} + A_{net,act} &= A_{net,act} \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \\ \Leftrightarrow \Delta A_{req} &= A_{net,act} \cdot \left(\sqrt[m]{\frac{N_{act}}{N_{req}}} - 1 \right) \end{aligned} \quad (17)$$

For steels found in old bridges, the material constant m can be conservatively defined as 3. The actual cross section $A_{net,act}$, which can fail if fatigue cracks occur, was taken conserva-

tively as the actual cross section or a part of it. Due to the low probability of simultaneous failure of all members of a cross section, normally only a part of a cross section was assumed as fault, see table in Figure 26. To cover also the low probability of the failure of the whole cross section an additionally safety check has to carry out, which is described in the following subsection.

The following Figure 26 summarises the equation for calculation of the required cross section of the strengthening ΔA_{req} and gives some hints, which parts of cross sections, typically found in old bridges, has to be assumed as members with a high risk of failure.

$$\Delta A_{req} = A_{net,act} \cdot \left(\sqrt[3]{\frac{N_{act}}{N_{req}}} - 1 \right)$$

with

N_{act} = maximum permissible number of load cycles, determined by a fracture mechanics calculation (see N_{per} in section 4.1.2) using a certain stress range calculated with the actual cross section

N_{req} = number of load cycles with a certain stress range between two inspections

$A_{net,act}$ = cross section, which can fail if fatigue cracks occur, see table below

















whole cross section	$A_{net,act}$
	
	
	
	
	
	
	
	

Figure 26 Eq. for calculation of the required cross section of the strengthening ΔA_{req} of members under cyclic tension load

Calculation of required cross section for strengthening of riveted girders under bending moment (plate girder)

Riveted plate girders are normally built up in a way that its have several flange lamellae in areas with a high bending moment load, whereas its have often only two rolled angle profiles in areas with a low bending moment load.

Plate girders built up with two rolled angle profiles and several flange lamellae often have a sufficient robustness against crack initiation. This can be proofed by a worst-case calculation with the assumption that one member of the flange with tension stresses fails and the stresses in the remaining members of this flange are lower than the permissible stresses. In such worst-case calculation no use of partial safety factors are required.

In the case, that the calculation shows stresses in the remaining part of the flange which are higher than the permissible stresses strengthening of the cross section is required. In the following the necessary strengthening of a plate girder with flanges only built up with two rolled angle profiles is derived. But the derived equations apply analogously to plate girders built up with two rolled angle profiles and several flange lamellae.

The stresses of a plate girder due to traffic can be determine as follows:

$$\Delta\sigma = \frac{\Delta M}{W} \quad (18)$$

with

ΔM = algebraic difference between the two extremes of a particular bending moment cycle derived from a traffic load history

W = section modulus

Applying Eq.(18) on the actual state and on the state after strengthening leads to the following formulas:

Actual state

$$\Delta\sigma_{act} = \frac{\Delta M_{act}}{W_{act}} \quad (19)$$

State after strengthening

$$\Delta\sigma_{req} = \frac{\Delta M_{act}}{W_{req}} \quad (20)$$

Both equations show that the stress range depends on the section modulus.

The Eq. for calculation of the section modulus of an uncracked plate girder built up with two rolled angle profiles and a web plate and the Eq. for calculation of the section modulus of such cross section strengthened with an additional flange lamella is given in Figure 27. For calculation of the section modulus of the strengthened cross section, it was assumed that the neutral axis is placed in the middle of the web. Due to that the moment of inertia is not exactly specified, but this inaccuracy can be neglected.

$$I_{act} = \frac{h^3 \cdot t}{12} + A_{angle} \cdot 4 \cdot \left(\frac{h}{2} - e \right)^2$$

$$\Rightarrow W_{act} = \frac{h^2 \cdot t}{6} + A_{angle} \cdot \left(2h - 8e + \frac{8 \cdot e^2}{h} \right)$$

$$I_{req} = \frac{h^3 \cdot t}{12} + A_{angle} \cdot 4 \cdot \left(\frac{h}{2} - e \right)^2 + A_{lamella} \cdot \left(\frac{h}{2} \right)^2$$

$$\Rightarrow W_{act} = \frac{h^2 \cdot t}{6} + A_{angle} \cdot \left(2h - 8e + \frac{8 \cdot e^2}{h} \right) + A_{lamella} \cdot \left(\frac{h}{2} \right)$$

Figure 27 Equation for calculation of the section modulus of an uncracked plate girder built up with two rolled angle profiles and a web plate and for calculation of the section modulus of such cross section strengthened with an additional flange lamella

The distance e between the neutral axes of rolled angle profiles to the outer edge of its flanges varies with the type of profile. A sufficient estimation of e for various types of rolled angle profiles is given in Figure 28.

$$B < C \Rightarrow e = 0,27 \cdot B$$

$$B = C \Rightarrow e = 0,30 \cdot B$$

$$B > C \Rightarrow e = 0,37 \cdot B$$

Figure 28 Estimation of the distance e between the neutral axes of rolled angle profiles to the outer edge of its flanges for various types of profile

The replacement of $\Delta\sigma_{act}$ and $\Delta\sigma_{req}$ in Eq.(12) by the equations (19) and (20) leads to the following equation:

$$\frac{\Delta M_{act}}{W_{req}} = \frac{\Delta M_{act}}{W_{act}} \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \quad (21)$$

With the replacement of W_{req} and W_{act} by the equations given in Figure 27 and after mathematically transformation an Eq. can be derived to calculate the required cross section $A_{lamella}$ of the strengthening (additional flange lamella):

$$\begin{aligned}
W_{req} &= W_{act} \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \\
\Leftrightarrow \frac{h^2 \cdot t}{6} + A_{angle} \cdot \left(2 \cdot h - 8 \cdot e + \frac{8 \cdot e^2}{h} \right) + A_{lamella} \cdot \frac{h}{2} &= \left[\frac{h^2 \cdot t}{6} + A_{web} \cdot \left(2 \cdot h - 8 \cdot e + \frac{8 \cdot e^2}{h} \right) \right] \cdot \sqrt[m]{\frac{N_{act}}{N_{req}}} \\
\Leftrightarrow A_{lamella} \cdot \frac{h}{2} &= \frac{h^2 \cdot t}{6} \cdot \left(\sqrt[m]{\frac{N_{act}}{N_{req}}} - 1 \right) + A_{angle} \cdot \left(2 \cdot h - 8 \cdot e + \frac{8 \cdot e^2}{h} \right) \cdot \left(\sqrt[m]{\frac{N_{act}}{N_{req}}} - 1 \right) \\
\Leftrightarrow A_{lamella} &= \left(\sqrt[m]{\frac{N_{act}}{N_{req}}} - 1 \right) \cdot \left[\frac{h \cdot t}{3} + 2 \cdot A_{angle} \cdot \left(2 - \frac{8 \cdot e}{h} + \frac{8 \cdot e^2}{h^2} \right) \right] \quad (22)
\end{aligned}$$

For steels found in old bridges the material constant m can be conservatively defined as 3. Figure 29 summarises the equation for calculation of the required cross section $A_{lamella}$ to strengthen a plate girder under cyclic bending load, to increase the maximum permissible number of load cycles.

$$A_{lamella} = \left(\sqrt[3]{\frac{N_{act}}{N_{req}}} - 1 \right) \cdot \left[\frac{h \cdot t}{3} + 2 \cdot A_{angle} \cdot \left(2 - \frac{8 \cdot e}{h} + \frac{8 \cdot e^2}{h^2} \right) \right]$$

with

N_{act} = maximum permissible number of load cycles, determined by a fracture mechanics calculation (see N_{per} in section 4.1.2) using a certain stress range calculated with the actual cross section

N_{req} = number of load cycles with a certain stress range between two inspections

A_{angle} = cross section of the rolled angle profiles

e = distance between the neutral axis of rolled angle profiles to the outer edge of one of its flanges

$$B < C \quad e = 0,27 \cdot B$$

$$B = C \quad e = 0,30 \cdot B$$

$$B > C \quad e = 0,37 \cdot B$$

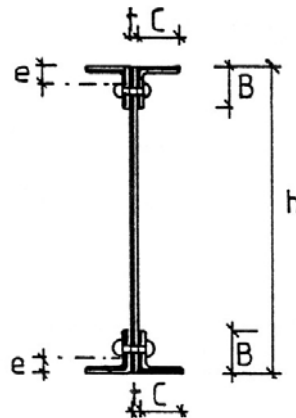


Figure 29 Calculation of the required cross section $A_{lamella}$ to strengthen a plate girder under cyclic bending load, to increase the maximum permissible number of load cycles

4.1.4 Calculation of necessary strengthening measures to increase the resistance

In the above mentioned subsections of 4.1.3 equations were derived for:

- calculation of the required cross section ΔA_{req} of the strengthening of members under cyclic tension load, see Eq.(17)
- calculation of the required cross section $A_{lamella}$ to strengthen a plate girder under cyclic bending load, see Eq.(22)

These strengthening cross sections lead to a decrease of the stress range $\Delta\sigma$ and therefore to a higher number of permissible load cycles equal to number of load cycles between two inspections.

As mentioned in section 4.1.3 the dimension of a strengthening cross section has to meet an additional requirement.

The cross section of the strengthening has to be as large as necessary for an accidental load case, where a part or the completely old cross section fails brittle. Normally the failure of the completely old cross section was conservatively assumed and due to that, the cross section of the strengthening has to carry the full maximum load in the ultimate limit state (dead loads, traffic loads, wind loads, temperature effects, break loads and constraint forces). For such a worst-case scenario, it is maintainable to design the strengthening using the yield strength of the material with partial safety factors equal 1.0. This can be explained as follows:

- low probability of simultaneous failure of all members of the old cross section
- accidental (rare) design situation

For members under tension load the above mentioned additional requirement can be met by a strengthening cross section $\Delta A_{req,wc}$ calculated with:

$$\Delta A_{req,wc} = \frac{\max F}{f_y} \quad (23)$$

with

$\max F$ = maximum force in the ultimate limit state [N]

f_y = yield strength [N/mm²] of the material used for the strengthening

For members under bending load the above mentioned additional requirement can be met by a section modulus that fulfil the following equilibrium:

$$W_{req,wc} = \frac{\max M}{f_y} \quad (24)$$

In the case of a plate girder built up with two rolled angle profiles and a web plate the worst-case scenario is defined as failure of both angle profiles of the flange under tension load. Now it has to be checked whether the section modulus of plate girder, strengthened by an additional flange lamella with a cross section according to Figure 26, is larger or smaller than the section modulus according to Eq.(24). In the case that the section modulus of the strengthened plate girder with failed tension flange is larger than $W_{req,wc}$ the second requirement on the strengthening is fulfilled. If not, the cross section of the additional lamella has to be increased until the section modulus of the strengthened plate girder with failed tension flange is equal to $W_{req,wc}$.

The following both figures combine all equations to fulfil the requirements in section 0 and 0 and the one given in this section. A strengthening of cross section calculated according to Figure 30 and Figure 31 provides:

1. a sufficiently safe service time between two inspections and
2. additionally a sufficient bearing capacity in the worst case of failure of a part of the old cross section.

The required strengthening cross section should be connected to the old cross section using bolts. Beside the calculation of a sufficient strengthening cross section, it has to be checked whether the bolted connection has a sufficient bearing capacity in the worst case of partially cross section failure, too.

$$\Delta A_{req} = \max \left\{ \begin{array}{l} A_{net,act} \cdot \left(\sqrt[3]{\frac{N_{act}}{N_{req}}} - 1 \right) \\ \frac{\max F}{f_y} \end{array} \right.$$

with

N_{act} = maximum permissible number of load cycles, determined by a fracture mechanics calculation (see N_{per} in section 4.1.2) using a certain stress range calculated with the actual cross section

N_{req} = number of load cycles with a certain stress range between two inspections

$A_{net,act}$ = cross section, which can fail if fatigue cracks occur, see table below

$\max F$ = maximum force in the ultimate limit state [N]

f_y = yield strength [N/mm²] of the material used for the strengthening

















whole cross section	$A_{net,act}$
	
	
	
	
	
	
	
	

Figure 30 Required strengthening cross section for members under tension load

$$A_{lamella} = \left(\sqrt[3]{\frac{N_{act}}{N_{req}}} - 1 \right) \cdot \left[\frac{h \cdot t}{3} + 2 \cdot A_{angle} \cdot \left(2 - \frac{8 \cdot e}{h} + \frac{8 \cdot e^2}{h^2} \right) \right]$$

$$\text{and } W_{req,wc} \geq \frac{\max M}{f_y}$$

with

N_{act} = maximum permissible number of load cycles, determined by a fracture mechanics calculation (see N_{per} in section 4.1.2) using a certain stress range calculated with the actual cross section

N_{req} = number of load cycles with a certain stress range between two inspections

A_{angle} = cross section of the rolled angle profiles

e = distance between the neutral axis of rolled angle profiles to the outer edge of one of its flanges

$$B < C \quad e = 0,27 \cdot B$$

$$B = C \quad e = 0,30 \cdot B$$

$$B > C \quad e = 0,37 \cdot B$$

$W_{req,wc}$ = section modulus of the strengthened plate girder with failed tension flange (worst case)

$\max M$ = maximum bending moment in the ultimate limit state [kNcm]
 f_y = yield strength [kN/cm²] of the material used for the strengthening

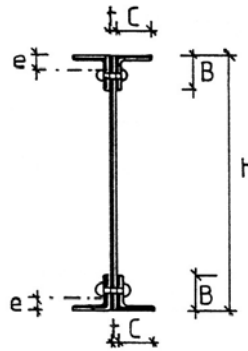


Figure 31 Required strengthening cross section for members under bending load

4.1.5 Extension of Master Curve concept

The basic Master Curve (MC) method for analysis of brittle fracture test results is intended for macroscopically homogeneous ferritic steels only. In reality, the steels in question are seldom macroscopically fully homogeneous. The steels fracture toughness may depend on the specimen location in the sample. For example, thick plates and forgings may have very different fracture toughness at plate center and close to surface. Inhomogeneity may be deterministic or random (or a mixture of both) in nature. Deterministic inhomogeneity can be accounted for, provided that the specimen extraction histories are known and enough specimens are tested. Random inhomogeneity is much more difficult to handle. The structural integrity assessment procedure SINTAP [3], see section 2.3, contains a lower tail modification of the MC analysis. This enables conservative lower bound type fracture toughness estimates also for inhomogeneous materials. The problem is that the SINTAP method does not provide information of the tougher material. Therefore, a probabilistic description of the complete material is not possible. Wallin et. al. [83] recently introduced a new comparatively simple extension of the MC for inhomogeneities governed by two separate MC distributions. The extension is shown to be extremely efficient in describing e.g. weld heat-affected zone (HAZ) data. In addition, a simple method for the analysis of random inhomogeneous material consisting of mixed data is presented. The method is also applicable for data sets including several different materials. It is recommended to apply this proposed extension of master curve concept also for future assessment of old steel railway bridges.

4.2 Replacement criteria for hot rivets

4.2.1 Structural characteristics of riveted connections using hot rivets

Riveted connections are pre-stressed connections. The rivet is tensioned during cooling whereas the plies are set under compression through the rivet head. The stress conditions are illustrated by Figure 32.

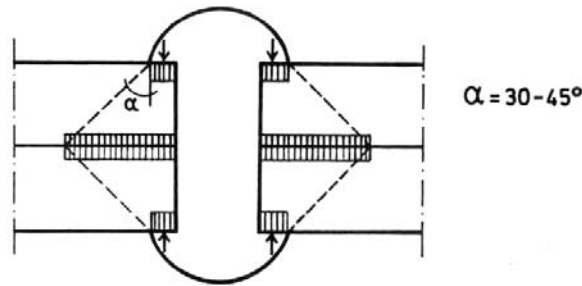


Figure 32 Stresses in connections with hot rivets

For riveted connections two different load-bearing mechanisms are considered:

- Load-bearing by friction forces: The friction coefficient in riveted connections is usually taken as $\mu = 0,25$, see e.g. SIA 161 [84]. If the shear forces exceed the restraining friction force slip occurs in the connection until the rivet shank achieves firm contact with the hole
- Load-bearing by bearing forces: Bearing forces occur if the rivet shank is in firm contact with the hole. Peak stresses are induced which seriously concern the fatigue behaviour of the connection

Figure 33 shows the stress distributions in both cases.

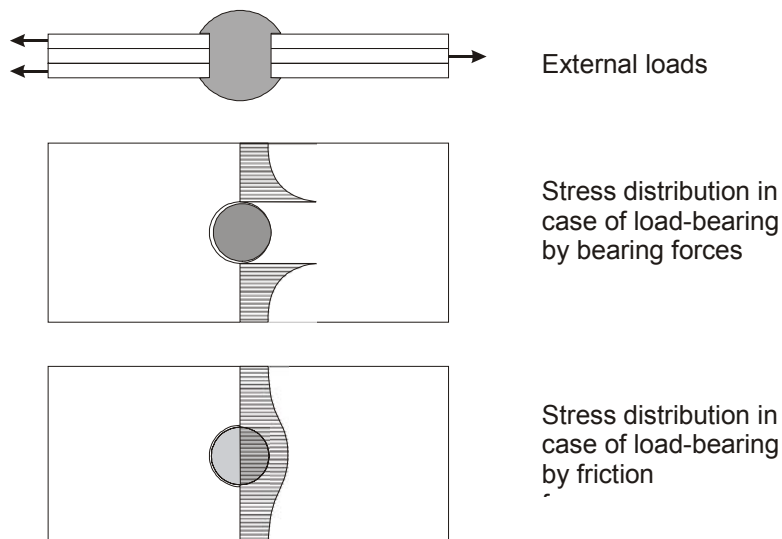


Figure 33 Stress distribution in plies of riveted connections

Another distinction is drawn between load-bearing rivets and tack rivets:

- Load-bearing rivets are used in structural connections
- Tack rivets are used to transfer clamp forces if several lamellar plates need to be set up. The use of tack rivets is required for corrosion protection purposes because they enable to prevent the ingress of water into a gaping. They also are able to prevent local buckling in compressed members in particular cases.

4.2.2 Defects in riveted connections

Defects in riveted connections may originate from fabrication or may be induced during service life by corrosion. Rivets with defects that originate from fabrication usually are not critical, because they have been in service since assembly without any negative effects. On the other hand rivet defects induced by corrosion are of particular concern, e.g. see Figure 34.

Typical fabrication defects of riveted connections are listed in Table 12.

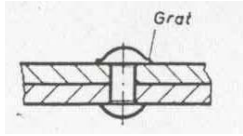
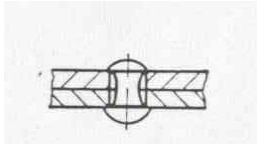
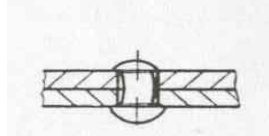
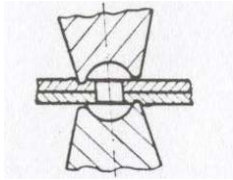
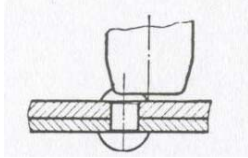
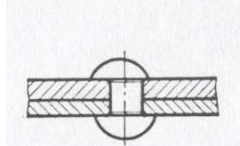
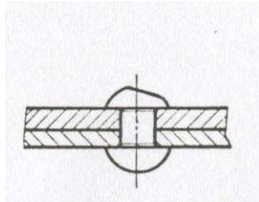
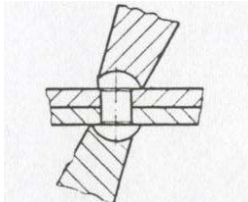
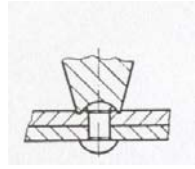
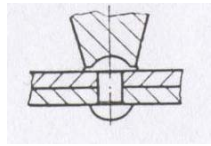
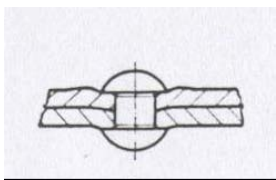
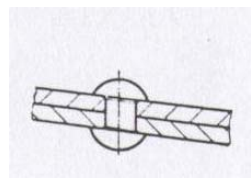
In particular, the defects no. 5, 7 and 8 are relevant, because these defects can easily be mistaken as corrosion defects.

Note, that appropriate tolerances for the dimensions, permitted error of coaxiality and rectangularity and material properties of hot rivets can be taken from DIN 101 [73].



Figure 34 Examples of corrosion of riveted connections

Table 12. Overview of rivet defects

Defect 1: Rivet shank too long 	Defect 2: Closing head too small, Rivet shank too thin 	Defect 3: Closing head too small or offset, Rivet shank without contact 
Defect 4: Plies in skew position 	Defect 5: Eccentric application of rivet header or rivet punch 	Defect 6: Offset rivet heads due to „corrections“ 
Defect 7: Rivet punch put on in skew position 	Defect 8: Rivet header put on in skew position 	Defect 9: Insufficient head form, damage by rivet header 
Defect 10: Rivet header too small, peripheral rim at closing head 	Defect 11: Bulged-out closing head 	Defect 12: Rivet punch and closing head in skew position 

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

- Loss of pre-stress
- Constitutional change of riveted connection
- Loss of position permanence
- Gaping of plies followed by stress corrosion cracking

The bandwidth of rivet pre-stress ranges from 20 to 220 N/mm², at an average value of about 100 N/mm². Due to several parameters influencing the pre-stress, e.g. the clamp length, rivet diameter, material and fabrication method, a reliable calculative method for the determination of the rivet pre-stress is not available.

Note, that for bolts long-term measurements prove the loss of preload forces in preloaded bolted connections, too [85]. The amount of bolt preload loss is particularly significant for hot-galvanized structures with additional surface treatment, e.g. for friction-grip purposes.

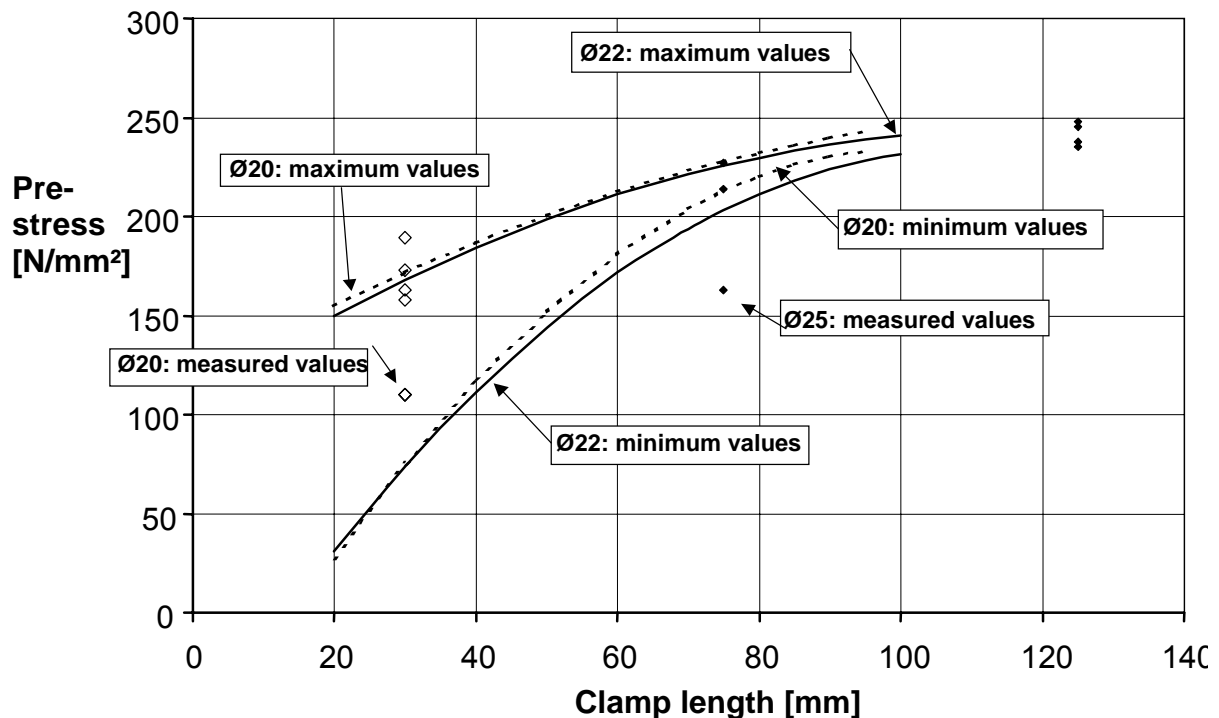


Figure 35 Rivet pre-stress depending on clamp length and diameter for rivet material St44 according to [75]

Numerical investigation of the influence of rivet deterioration on the pre-stress

To clarify the influence of the rivet head deterioration on the pre-stress calculations have been performed with varying dimensional reductions of the rivet head. For this purpose a finite element volume model of the rivets was established.

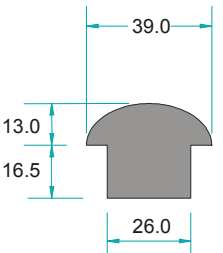
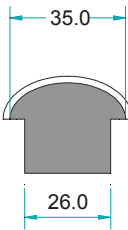
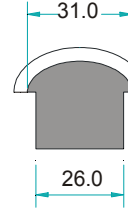
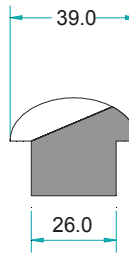
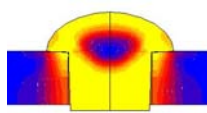
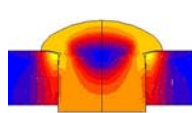
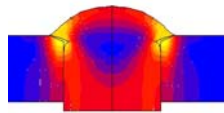
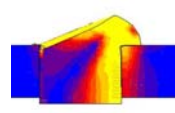
Aim of this investigation was to determine the limit stresses, which can be transferred between deteriorated rivets and plies. The simulated reductions of rivet heads are tabulated in Table 13 exemplary for a rivet diameter of Ø26.

As result of this calculation the imposed tension stress is compared with the extension of rivet shank, see Figure 36. The outcome of this is:

- **Damage type 0:** The rivet is able to bear pre-stresses up to the yield stress.
- **Damage type 1:** A pre-stress up to 170 N/mm² can be achieved. A reduction of shank strain does not occur within the simulated area of deformation.
- **Damage type 2:** At a stress level of 118 N/mm² a stress reduction occurs associated with a reduction of shank strain – the rivet head deforms and slides also into the rivet hole. Although in consequence a stabilisation of the actual condition takes place, i.e. the rivet becomes fixed and gets capable to achieve once again stresses of up to 119 N/mm² at increased loads, this first sliding is rated as a failure of the rivet head.

- Damage type 3: Having a skew rivet head a pre-stress of only 101 N/mm² can be achieved. In case of further increase of loads continued sliding and subsidence of the rivet head occurs.

Table 13. Stress and deformation images for investigated rivet head damage types

Damage type 0	Damage type 1	Damage type 2	Damage type 3
			
without any deterioration	uniformly distributed corrosion of 2mm (32 Vol.-%)	uniformly distributed corrosion of 4mm (56 Vol.-%)	skew corrosion or deformation at an angle of 22,5°
			

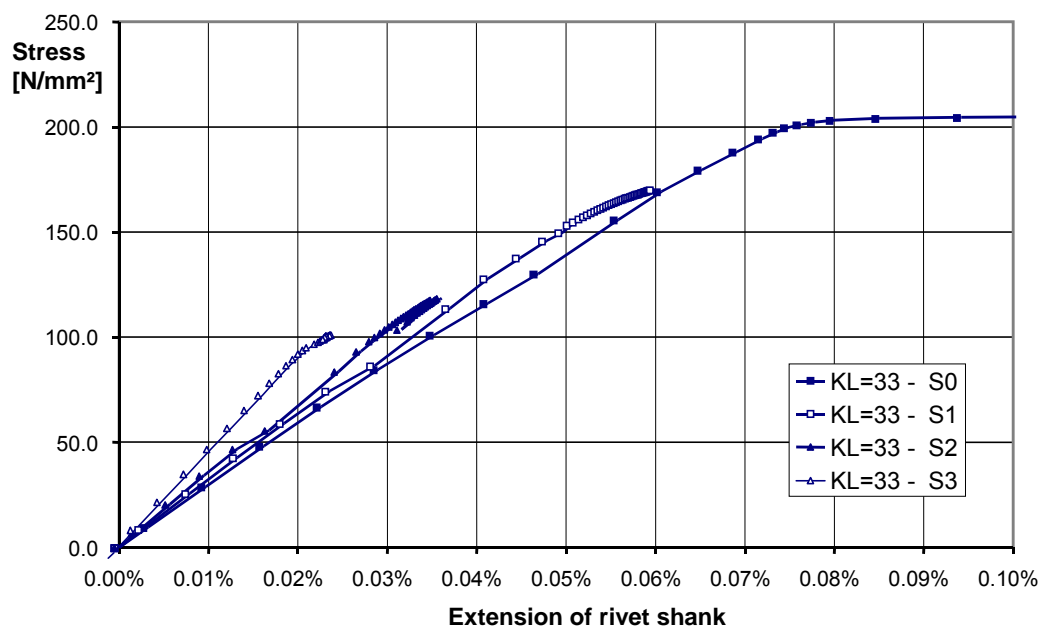


Figure 36 Results of calculation

Conclusion:

For the investigated cases with uniformly distributed corrosion (2 mm or 4 mm, respectively) the rivet is capable to bear an average value of pre-stress of 100 N/mm² (A deterioration of 4 mm means a loss of volume of the rivet head of 56%). Hence they may be classified as uncritical. Furthermore, a skew-shaped corrosion turns out to be more critical than a uni-

formly distributed one. Supposed that rivet pre-stresses of 100 N/mm^2 represent the normal case for undeteriorated rivet heads, then the investigated damage types with uniformly distributed corrosion are uncritical with regard to the load-bearing capacity and the remaining service life of a bridge.

Experimental investigation of fatigue effectiveness

A trial test has been carried out to verify that rivet pre-stresses of 100 N/mm^2 provide sufficient fatigue resistance under real-world conditions for a steel railway bridge structure. Figure 37 shows the test set-up and a rivet prepared for stress measurements in detail.

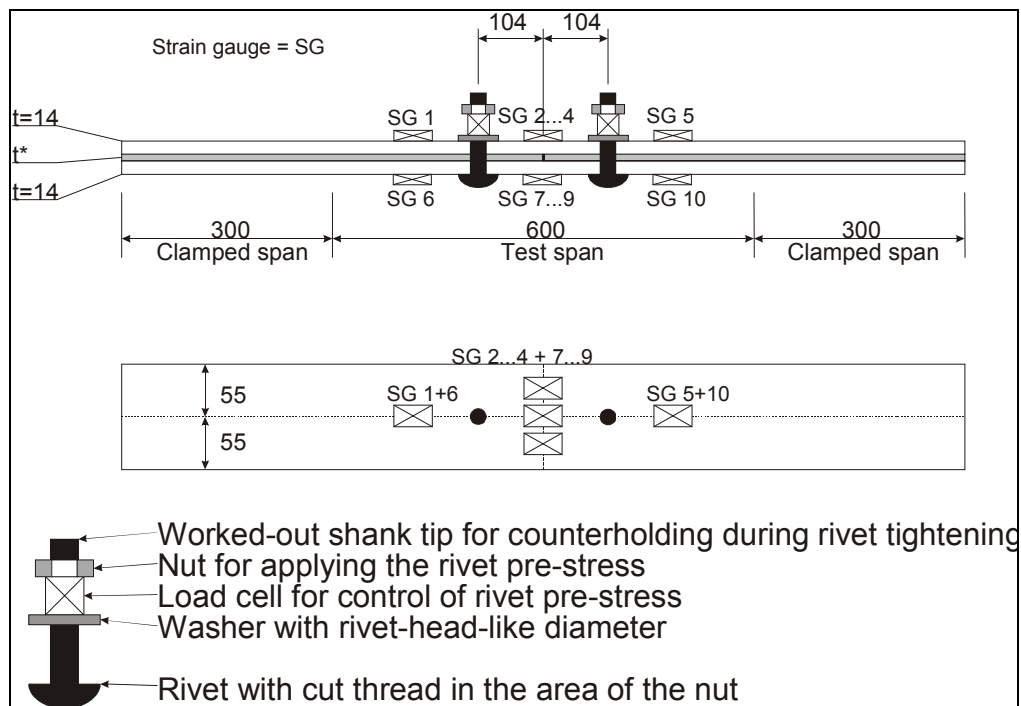


Figure 37 Test set-up

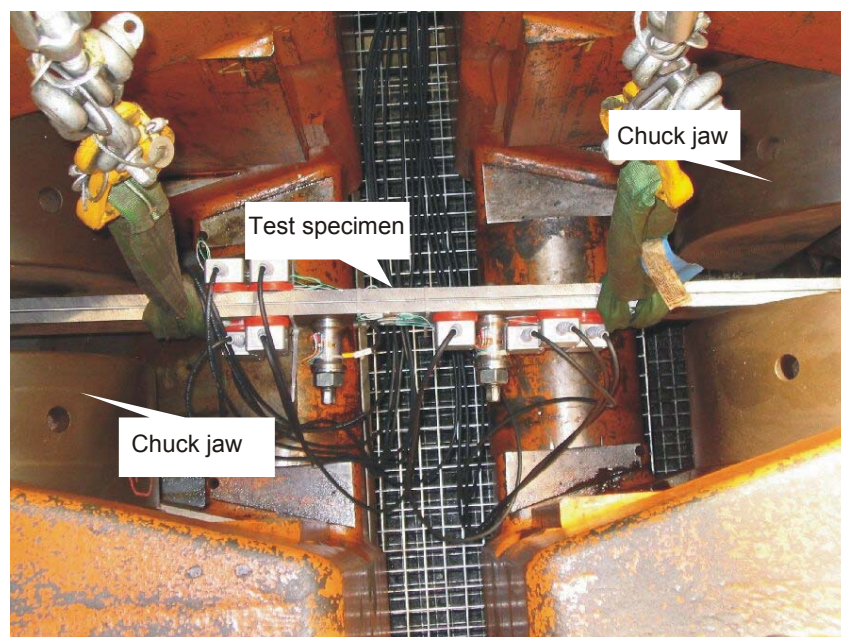


Figure 38 Implemented test specimen

Test result:

By this experiment it proves true that a rivet pre-stress of 100 N/mm^2 is uncritical for the remaining service life of riveted structures. Within the experiment the rivet pre-stress was 80 N/mm^2 , whereas the component was loaded by a stress variation range of 120 N/mm^2 (1,5 times the fatigue strength). After 2 million load cycles no fatigue damage could be observed.

Experimental investigation of rivet head corrosion

The experiments concerning the corrosion of the rivet heads were used to determine the permissible rate of corrosion at the rivet heads, to provide for a sufficient load-bearing capacity of the rivet head, i.e. for the required minimum pre-stress of 100 N/mm^2 .

Therefore, the rivets were assembled and tightened up to 100 N/mm^2 . The pre-stress was measured using a load cell. Then the rivet head was slowly machined layer by layer using a milling cutter such that no warming of the rivets could occur. The machining of the rivet head has been performed parallel to the plies and on the other hand with an inclination angle of 30° to the plies, see Figure 39. Figure 40 shows the test set-up for both directions of machining.

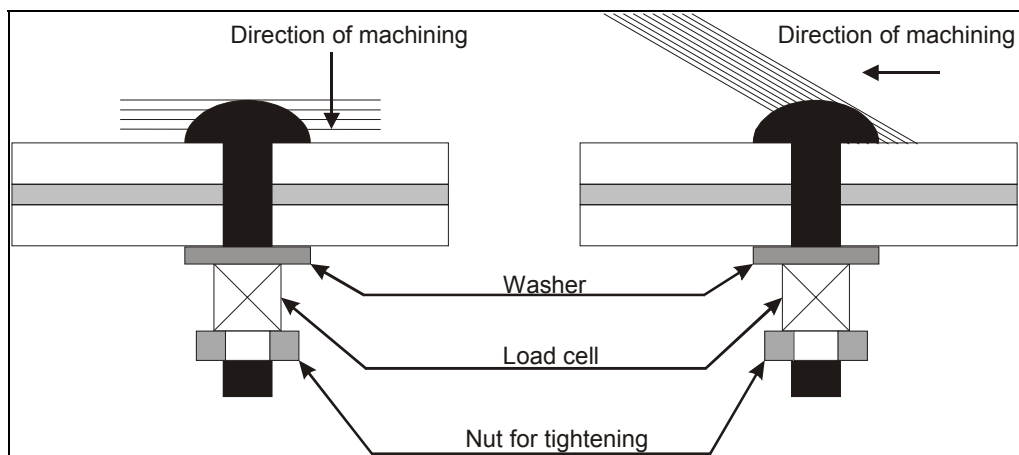


Figure 39 Schematic test set-up

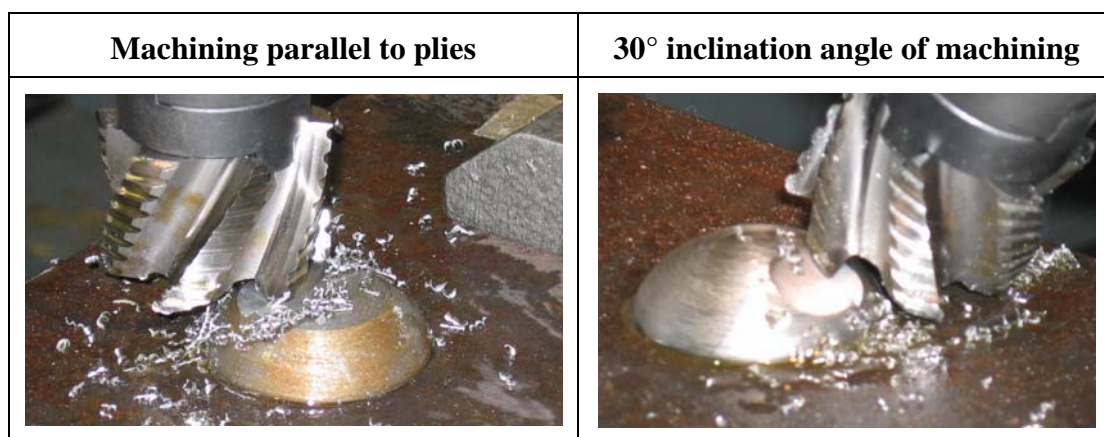


Figure 40 Machining of rivet heads

The next two figures (Figure 41 and Figure 42) show the characteristics of the rivet forces during the milling process for the current rivet head degradation.

Test result:

In general, at minor corrosion rates the corrosion-induced decrease of rivet pre-stress is not caused by yielding of material, but by the reduction of stiffness of the rivet head. For an increase of corrosion, yielding occurs only locally limited combined with a transfer of loads into lower stressed areas.

A reduction of pre-stress down to 90% of the original value is proposed as an appropriate limiting criterion for the permissible loss of pre-stress.

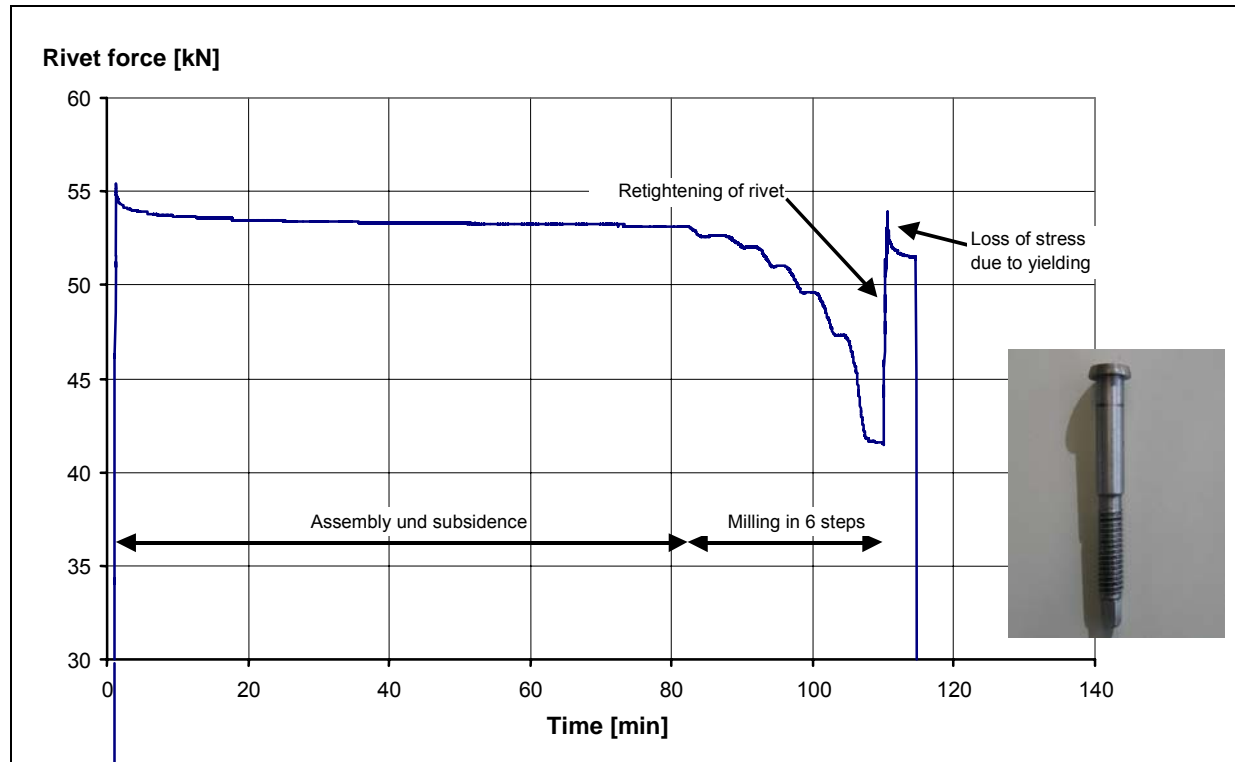


Figure 41 Characteristic of the rivet force for milling parallel to plies

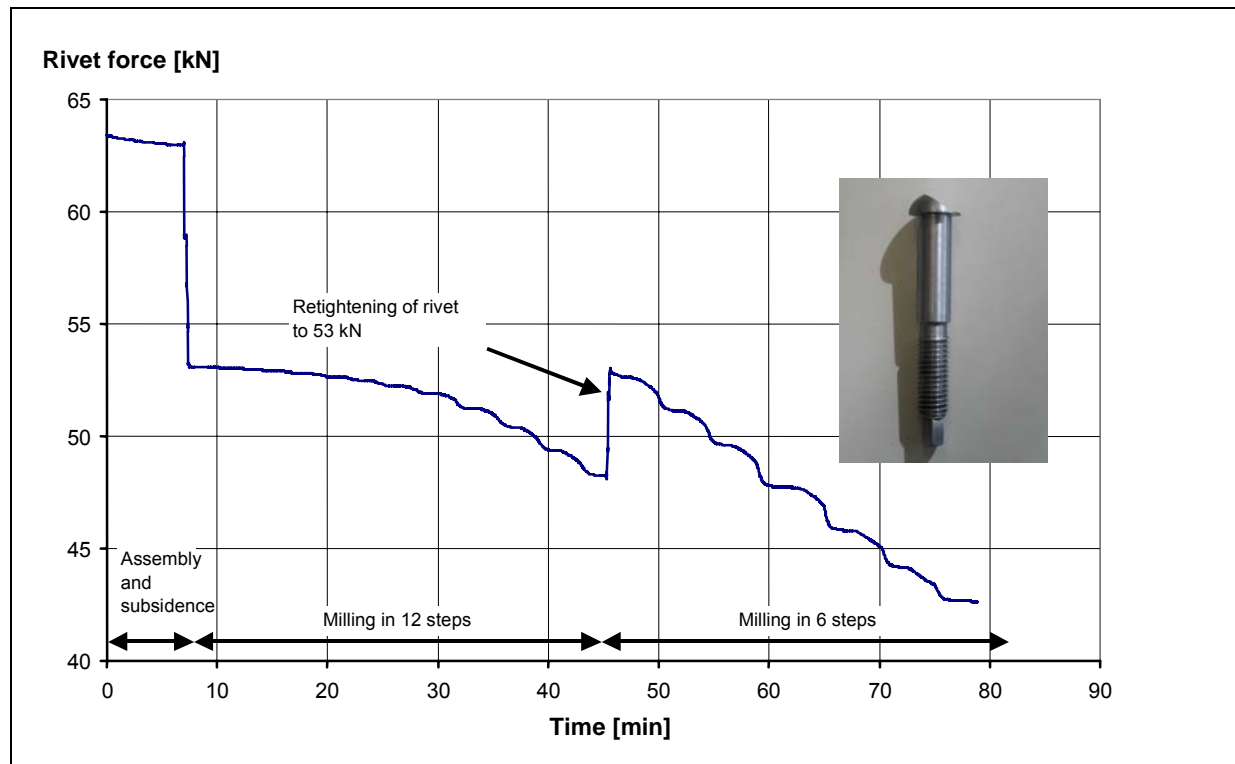


Figure 42 Characteristic of the rivet force for milling using an inclination angle of 30°

The following Figure 43 shows an evaluation of the test, where the corrosion rate is depicted in relation with a loss of pre-stress of 10%.

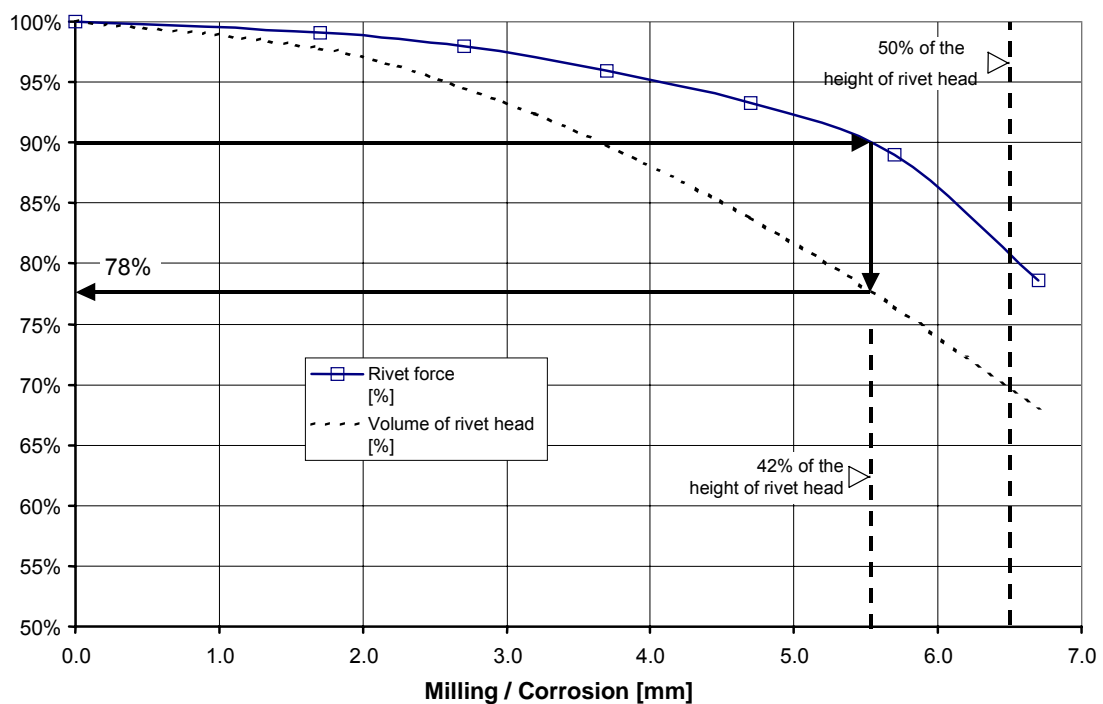


Figure 43 Relationship between rivet force and corrosion / milling for rivet head degradation parallel to plies

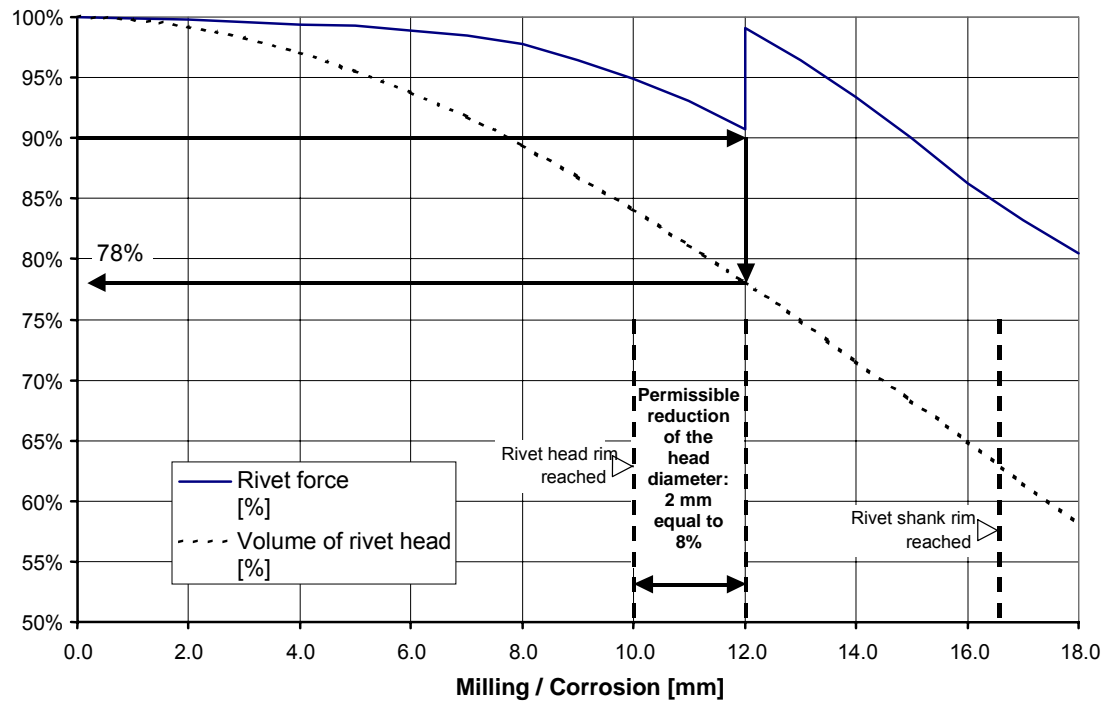


Figure 44 Relationship between rivet force and corrosion / milling for rivet head degradation with an inclination angle of 30°

4.2.3 Summary

Based on the results of numerical investigation and experimental tests according the fatigue effectiveness and the rivet head corrosion two limit criteria could be established (ultimate limit of the load-bearing capacity for the rivet head and a serviceability limit for the riveted connection). More details of this investigation can be taken from [75]. A similar investigation on the fatigue behaviour of riveted bridge girders is available, see [86].

5 Condition assessment examples

5.1 General

Two examples of condition assessment processes based on field measurements are shortly described.

The first example comprises strain measurements carried out in the frame of an expertise on the residual fatigue life of a railway bridge. The aim of the measurements was to gain information about realistic fatigue loads and compare them with the loads used in the fatigue assessment. The expertise was prepared by the authors of the present report.

The second example describe the measurements carried out in the frame of statically recalculation of an old railway bridge. The aim of these measurements was to gain information about the real structural behaviour under horizontal loads. The information were used to refined the model used for the recalculation. All information on the second example were according to [87]

Recent publications on further exemplary cases focusing on the assessment and strengthening of old steel railway bridges are available, see e.g. [88], [89].

5.2 1st Example: Strain measurements on a railway bridge to gain information about realistic fatigue loads

5.2.1 Introduction

In the frame of a change of the utilisation of areas under a couple of old steel railway bridges the question comes up, how long the old steel bridges and therefore also the areas under these bridges can be used. For that an expertise on the residual fatigue life of the bridges was made. Additionally to the calculation for this assessment also strain measurements were carried out to proof the assumptions and results of the fatigue life assessment. It was suspected that the assumed present fatigue load was too high as well as the real structural behaviour was more docile as assumed due to neglected additional load carrying capacities of secondary structures.

The investigated steel bridges were single span bridges each with two riveted main girders and a span length of 8,35 m. The I-shaped cross section was build up by steel plates and rolled L-profiles, see Figure 45.

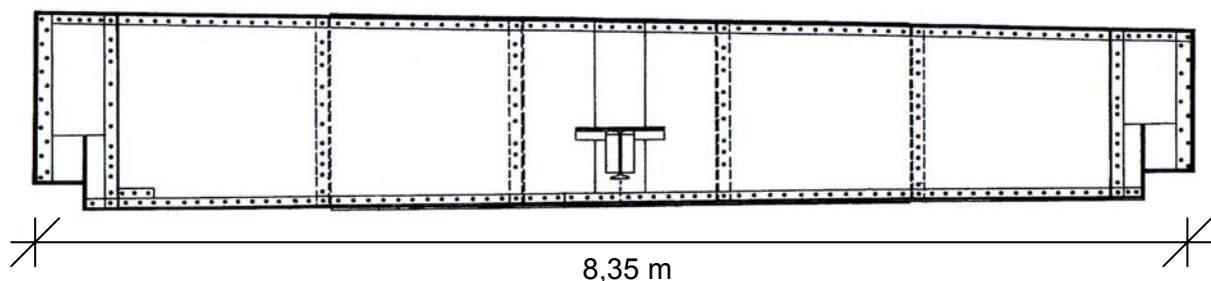


Figure 45 Front view of the investigated main steel girder

5.2.2 Choice of representative cross section, measuring point, representative load

After a representative bridge was chosen two measuring cross section were selected nearly midspan of the bridge, see Figure 46. The second measuring cross section served as a reference with the first one. At the first measuring cross section 18 strain gauges (see Figure 47) and at the reference cross section 7 strain gauges (see Figure 48) were applied. All strain gauges were connected to a PC plug-in data acquisition (DAQ) device, see Figure 49.

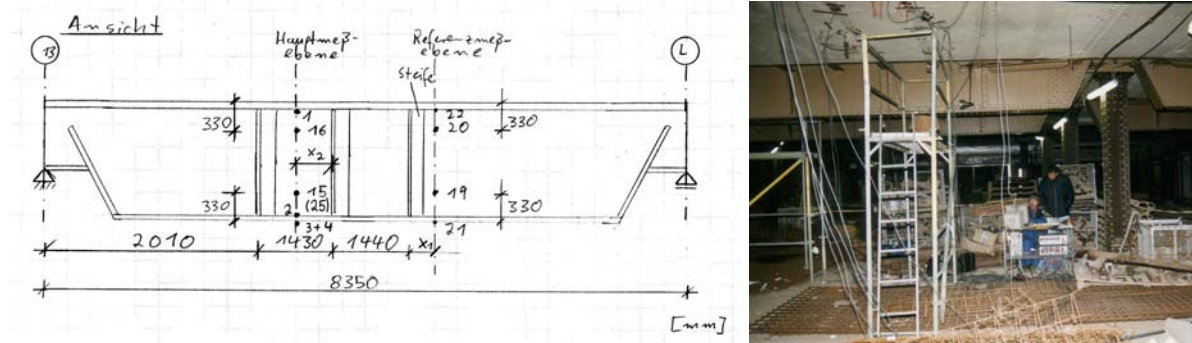


Figure 46 View of the main steel girder with the main and the reference measuring cross section

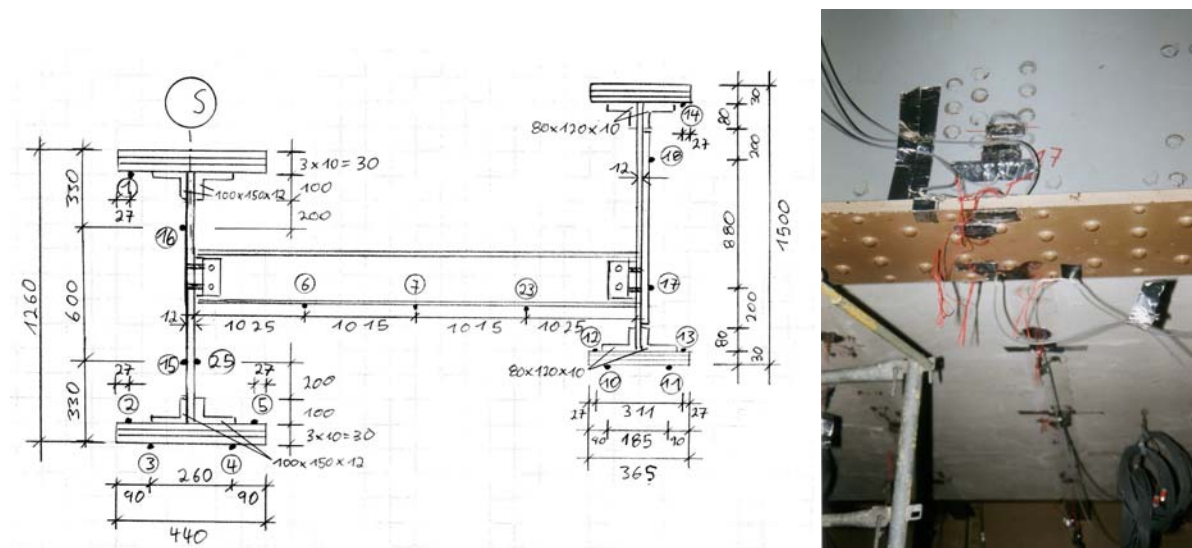


Figure 47 Main measuring cross section

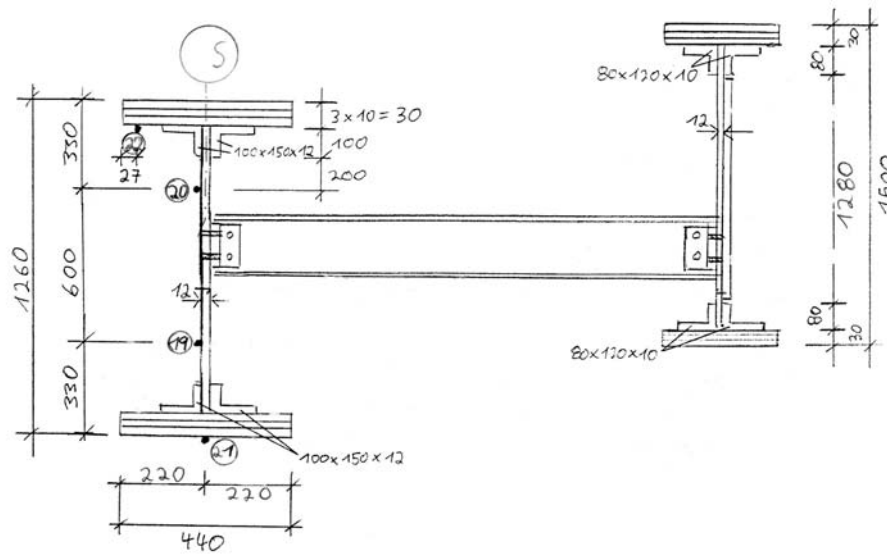


Figure 48 Reference measuring cross section



Figure 49 PC plug-in data acquisition device

5.2.3 Measurements

The measuring was carried out at 7°C steel surface temperature. The loads were caused by original trains of the regular traffic. For each measurement the important information of the passing train were recorded. A list of trains, passing time, train type and the locomotive type is given in Table 14.

Table 14. List of measurement number (1st column), measurement time (2nd column), train type (4th column) and locomotive type (6th column)

Messung					
Messung	Uhrzeit	Gleis	Zugtyp	Name	Loktyp
1	13:42	6	EC	GL61342	103
2	13:50	6	IC	GL61350	120
3	14:39	6	ICE	GL61439	ICE
4	15:09	6 c	RE	GL61509	110
5	15:22	6 c	RE	GL61522	215 Diesel
6	15:42	6	EC	GL61542	101
7	15:50	6	IC Einfahrt	GL615501	101 hinten
8	15:50	6	IC Ausfahrt	GL615502	101 hinten
9	15:50	6 c	RE	GL61609	110

The measurement configuration was:

Sample rate 5 Hz

Measurement length 30 to 60 sec.

The data were stored online at a PC.

5.2.4 Analysis of the measured data and conclusion

Figure 50 shows a measured strain-time-history. This history can read as follows. Between 2.5 and 7.5 sec. after the measurement starts the locomotive had passed the measurement cross section. Between 10 and 12.5 sec. the 1st wagon had passed and after 16 sec. the 2nd wagon had reached the measurement cross section. Due to the fact that the train had stopped with the middle of the 2nd wagon exactly at the measurement cross section the strains did not get back to zero till the end of the measurement.

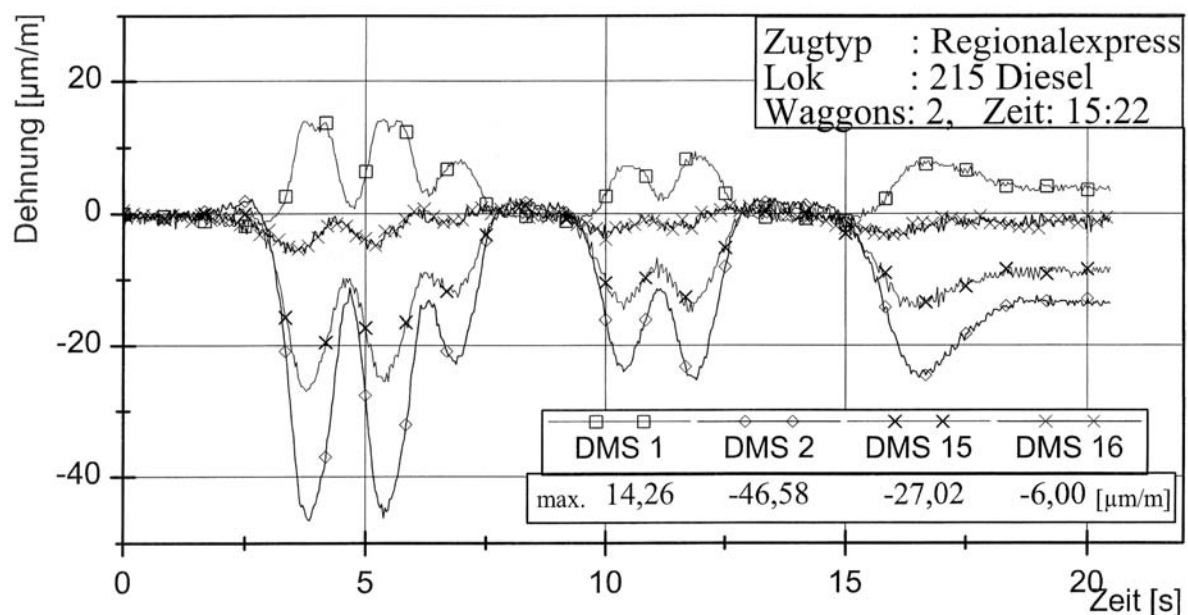


Figure 50 Example for a measured strain(Dehnung)-time(Zeit)-history

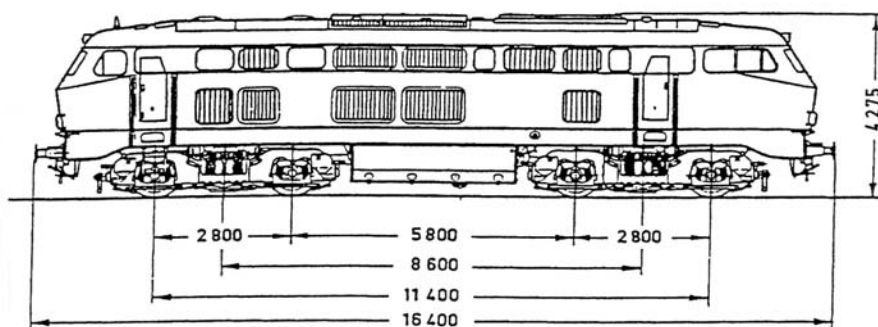
Each strain-time-history was evaluated in a way that the maximum strain was converted to a maximum stress using the assumption of fully elastic behaviour of the structure and therefore validity of the formula $\sigma = E/\epsilon$, see Table 15.

Table 15. Transfer of measured maximum strains (maximale Dehnung, 4th column) into stresses (Schwingbreite, 5th column)

Zugtyp: RE		Loktyp: 215 D		Name: R1522-2	
	Dehnung	Spannungsausschlag	maximale Dehnung	Schwingbreite	
	[$\mu\text{m/m}$]	[N/mm^2]	[$\mu\text{m/m}$]	[N/mm^2]	
DMS 3	43,06	9,0426	48	10,08	
DMS 4	31,1	6,531	33	6,93	
DMS 5	31,14	6,5394	31,14	6,5394	
DMS 25	25,35	5,3235	27	5,67	

For a comparison of the measured values with calculated stresses a calculation was made with the following assumptions:

- Use of the same static model and boundary conditions for the main girder as for the determination of the tension stresses in the context of the static calculations in [90].
- A distribution 25% - 50 % - 25 % of the axial load on three sleeper according to DS 804 [91] was not taken into account.
- Use of the real loads of the actual train passages (only maximum load of the heavy locomotives, see Figure 51) with its load points in the most unfavourable position regarding the placement of the strain gauges.
- Any safety elements or dynamic factors were neglected.
- The tension stresses of the transversal beam were calculated for different degrees of restraint at its supports.



BR 215, 218
mittelschwere
Personen- und
Güterzuglok
Baujahr 1968

Baureihe	Achsfolge	Stundenleistung	Anfahrzugkraft	v_{\max}	Rad-satzlast	Länge ü. Puffer	Dienstgewicht
		[kW]	[kN]	[km/h]	[t]	[mm]	[t]
218	B' B'	1581	165	140	20,3	16400	78,0
232	Co' Co'	2200	348	120	20,5	20820	122,4
290	B' B'	809	192	80	20,5	14320	79,5
333	B	176	59	45	12,5	7830	24,3
360	C	478	118	60	15,9	10450	48,0

Figure 51 Example for a sketch and data sheet with important data of some of the measured locomotives (axial loads (Radsatzlast), 6th column)

At the end the measured values were compared with the calculated one, see Figure 52.

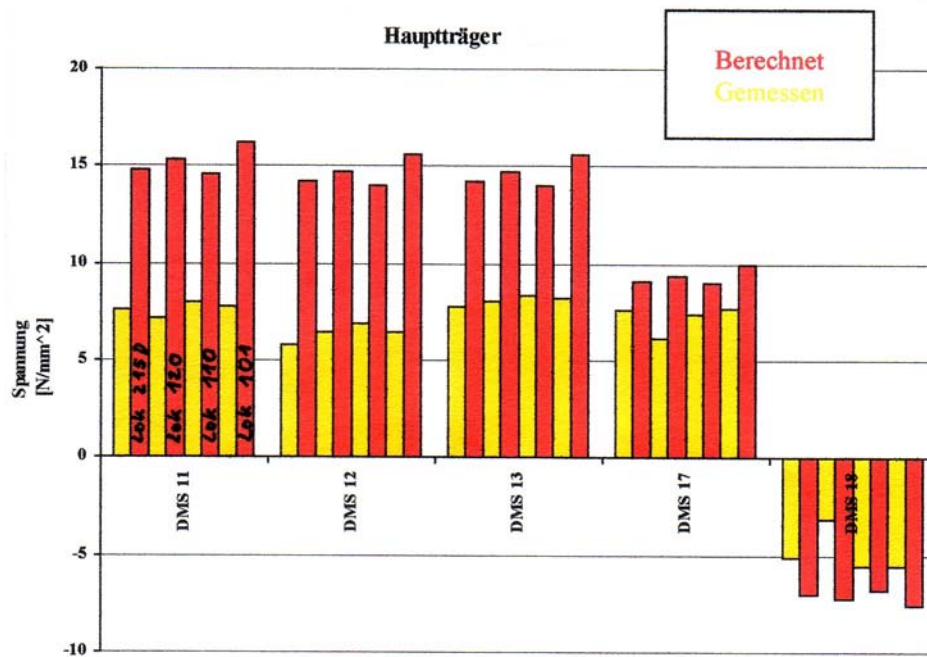


Figure 52 Example for the comparison of calculated stresses (Berechnet, red columns) with the measured stresses (Gemessen, yellow columns) for a main girder (Hauptträger)

All comparisons show that the calculated stresses were lower than the measured one. In some cases more than 50% lower. With regard to the aim of the measurements a more exact investigation of the causes of the lower measured stresses was not carried out.

As the main result of the measurement it could be stated that both the assumed fatigue load model and the used structural model of the statically calculation are safe sided and the results of the calculation are conservative.

5.3 2nd Example: Measurements to gain information about the real structural behaviour under horizontal loads

5.3.1 General

The size of horizontal forces due to braking and traction of trains is important especially for the design of the framework piers and of the foundations of the Rendsburg railway bridge because of the great height over terrain of the carriageway. Considering the continuous rails and the ability to transfer horizontal forces via longitudinally movable frictional bearings a spreading of the loads over several piers is possible. This is beneficial especially for the localised traction forces. In [87] the theoretical modelling of the non-linear system and the experimental approval by a braking test on the ramp bridges is described. The following sections will summarise the important information of [87] regarding the measurements during the braking test.

5.3.2 Description of the structure

The Rendsburg railway bridge span over the Kiel Canal near Rendsburg. It was built during the first extension of the Kiel Canal 1912 to 1913 and consist of a structure with hover ferry directly over the canal and 105 bridges supported on truss pier, see Figure 53.

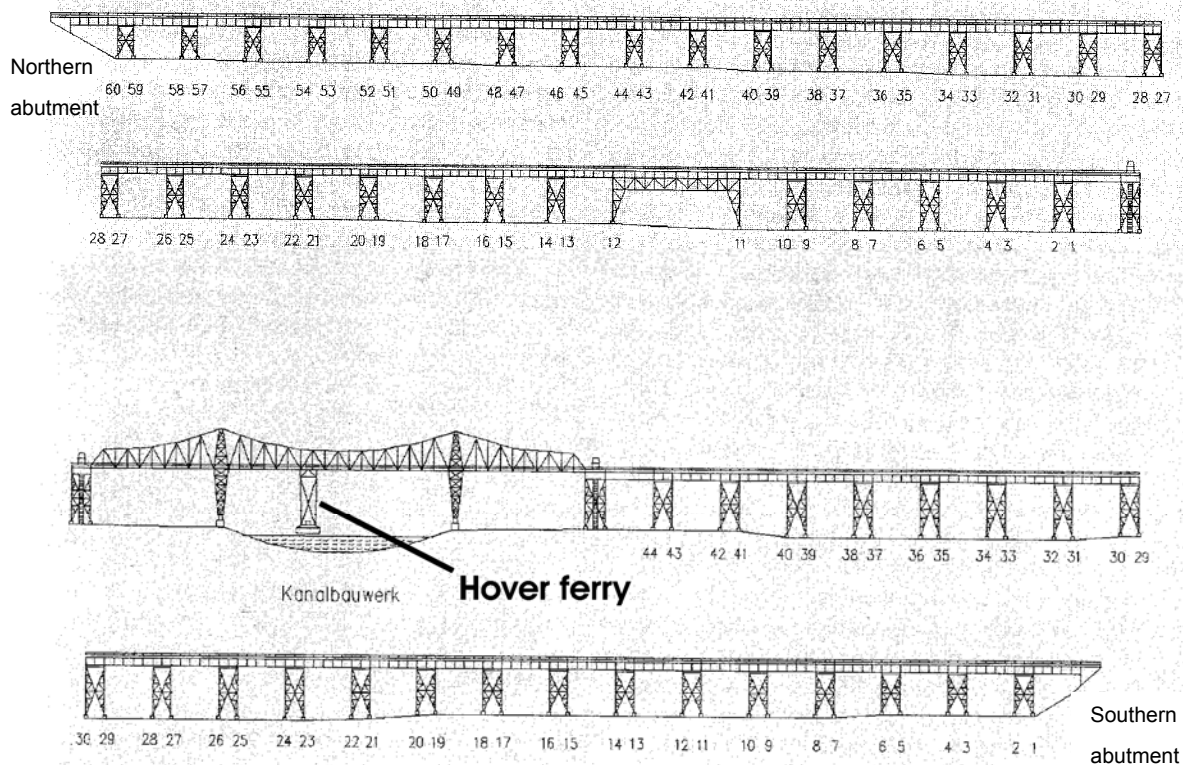


Figure 53 Overview of the structures of the Rendsburg railway bridge

The bridge over the canal has a headroom of 42 m for the shipping traffic. The railway bridge is a strait of the railway service between Germany and Scandinavia.

5.3.3 Formulation of the problem

After 90 years in service and with regard to the increased traffic loads a recalculation of the Rendsburg railway bridge was carried out. This first calculation based on safe sided assumptions on both loads and structural model. It showed that some parts of the bridge had to strengthen or had to rebuild.

Another result of the calculation was that the size of horizontal forces due to braking and traction of trains are important especially for the design of the framework piers and of the foundations because of the great height over terrain of the carriageway. Considering the continuous rails and the ability to transfer horizontal forces via longitudinally movable frictional bearings a distribution of the horizontal loads over several piers seem to be possible, which was not considered in the first calculation.

With regard to the high cost for strengthening or rebuilding and the unjustifiable restriction of the railway traffic during construction it was decided to carry out measurements on the structural behaviour of the bridge under horizontal forces. The aim was to refine the model and therefore to minimise the necessary strengthening measures.

5.3.4 Measurements

The braking test was carried out by the bridge measuring group of the German railway company (DB AG) located in Magdeburg. Due to the complexity of the structure and its load transfer behaviour the measurement was carried out in two steps. In the first step the applied test points were focussed on the area around the pier 28 in the southern ramp bridge [92]. In May 2001 passages of a defined train were carried out, consisting of three locomotives BR 232 (3 x120t).

The first measurement served the reconnaissance of possible deviations in the real load carrying behaviour of the pier in comparison to the model behaviour used in the previous calculations. Furthermore by an effortful equipping of all tracks in the measured cross section it should be checked that the measured tension stresses in tracks do not result from other effects than the applied braking power. On basis of the results of the first measurement in July 2001 the so called big braking test was carried out with optimised test point application in the area of pier 20 to 36 as well as at southern abutment [93].

The big braking test was focused on the following main topics:

- Measuring of strains using strain gauges at the diagonal bracings of the longitudinal sides of the piers 22 to 34 as a criterion for horizontal loading and reaction at the support of each pier.
- Measuring of the tension stresses in the tracks in the area of the truss piers 20 to 36 (Based on the results of the first measurement strain gauges were applied only at one track.)
- Measuring of the horizontal movements of tops of the piers 25, 27 and 29 using laser gauges.
- Measuring of movements of the longitudinally movable frictional bearings of the superstructure in the area of pier 28
- Measuring of strains in the tracks as well as movements of bearings at the southern abutment.

To apply defined vertical and horizontal loads a train was used consisting of eight locomotives BR 232 - 6 x BR 140 – BR 232 on one track and additional two locomotives BR 232 on the adjacent track, see Figure 54.



Figure 54 Train for braking test at the canal bridge

As speed at beginning of the braking 30 km/h was specified. The registered maximum braking power could be determined about the measured braking deceleration. The braking deceleration is not constant over time because of subjective behaviour of the train drivers. However for evaluation of the measurements only the maximum value at the time of the braking jerk is of interest. This value can easily be taken from the measured braking deceleration-time-histories, for example see Figure 55. A detailed evaluation of the measurements can be found in [94]. In the following only some essential results are summarized.

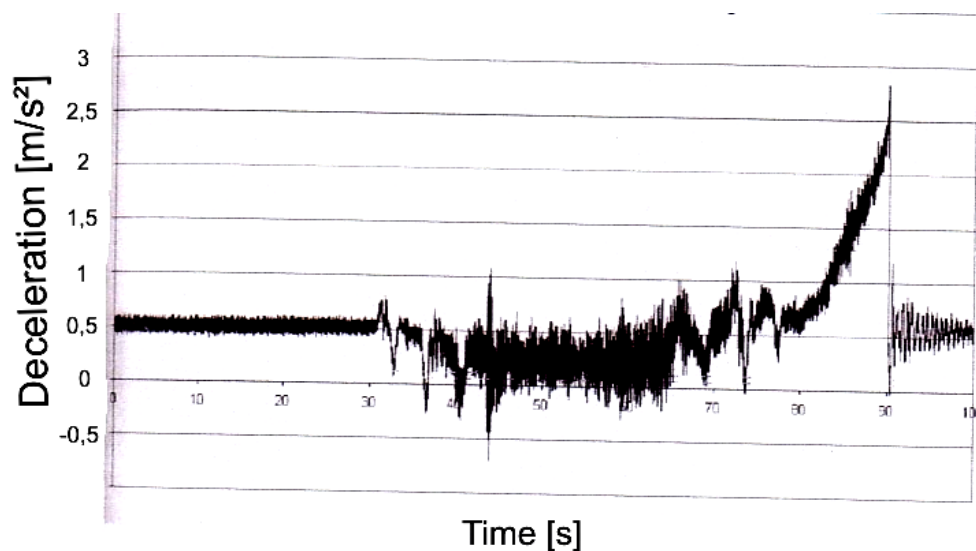


Figure 55 Deceleration-time-curve (measured at the locomotive)

5.3.5 Analysis of the measured data and conclusions

The strains at the diagonal bracings of the longitudinal sides of the piers 22 to 34 were measured by applying strain gauges at every four corners of each diagonal. The measured strains were transferred in stresses as described in section 5.2.4. The mean value of the stresses at the four corners of each diagonal was taken as portion of normal stress of the measured member. Due to the variation of driving direction, variation in driving separately on each track and parallel on both tracks and variation of the braking point the measurements covered all possible situation of braking on the bridge. Figure 56 shows exemplarily a stress-time-history measured at the diagonal bracings of the longitudinal sides of the piers 22 to 34, braking test drive 13 (direction Flensburg, stop at pier 14).

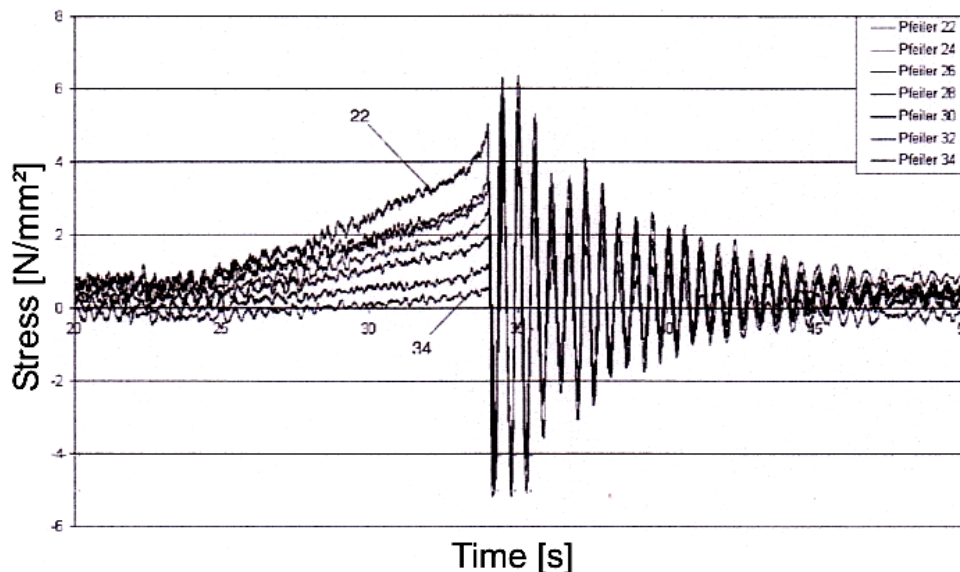


Figure 56 Stress-time-history measured at the diagonal bracings of the longitudinal sides of the piers 22 to 34, braking test drive 13 (direction Flensburg, stop at pier 14)

One can see the relationship of the measured stress to the distance between braking point and location of the strain gauges (strain gauges at pier 22 were located near the braking point of the measurement compared to the gauges at pier 34). Additionally Figure 56 shows the decay of the horizontal oscillation of the bridge after the stop of the braking 34 sec after the measurement was started. All piers oscillate in-phase. It could be concluded that all investigated piers participated in the horizontal load transfer and therefore a distribution of horizontal loads on more than one part of the Rendsburg bridge is acceptable. This conclusion was used to refine the model for the static calculation. The calculation carried out with the refined spatial non-linear model showed a sufficient accordance with the measured values, see Figure 57.

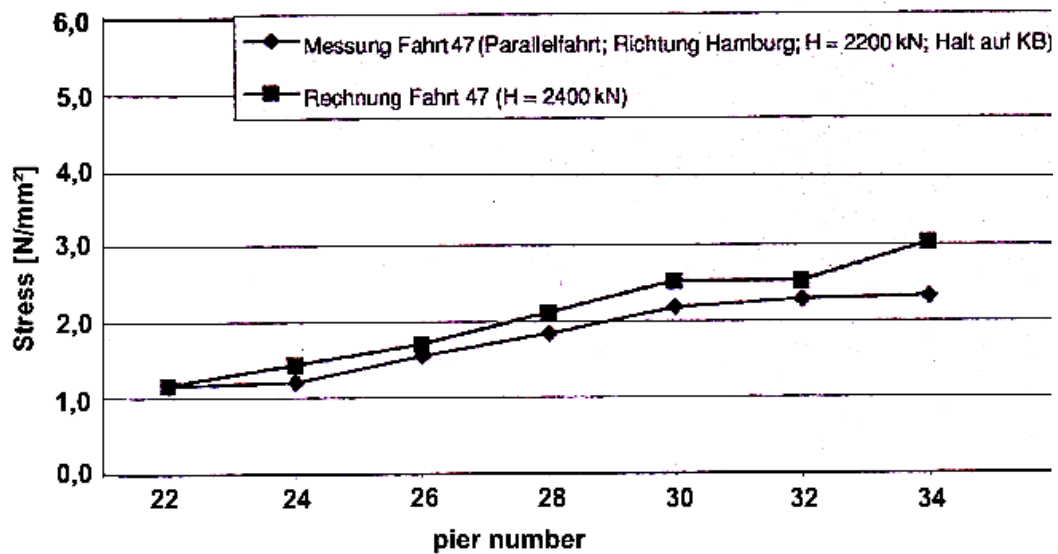


Figure 57 Comparison of calculated and measured stresses in the diagonal bracings of the longitudinal sides for test drive 47 (stop at the canal bridge)

The new calculation using the refined model led to a reduction of highly stressed members of the piers (partial reduction of nearly 20%) and for that to a significant reduction of the necessary strengthening measures.

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