ISSN 1831-9424 (PDF)
ISSN 1018-5593 (Printed)

European Commission

Standardization of Safety Assessment Procedures across Brittle to Ductile Failure Modes

(SAFEBRICTILE)

Research and Innovation

EUR 28906 EN

EUROPEAN COMMISSION
Directorate-General for Research and Innovation
Directorate D — Industrial Technologies
Unit D.4 — Coal and Steel

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B-1049 Brussels

European Commission

Research Fund for Coal and Steel

Standardization of Safety Assessment Procedures across Brittle to Ductile Failure Modes (SAFEBRICTILE)

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Grant Agreement RFSR-CT-2013-00023
1 July 2013 to 30 June 2016

Final report

Directorate-General for Research and Innovation

2017 EUR 28906 EN

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Luxembourg: Publications Office of the European Union, 2017

Print	ISBN 978-92-79-77000-5	ISSN 1018-5593	doi:10.2777/762072	KI-NA-28-906-EN-C
PDF	ISBN 978-92-79-76999-3	ISSN 1831-9424	doi:10.2777/76892	KI-NA-28-906-EN-N

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1 <u>Final summary</u>

1.1 *Project objectives*

The safety assessment was not consistently considered throughout the many parts of Eurocode 3, mainly due to a lack of guidance and lack of existing databanks containing information on the distribution of the relevant basic variables and steel properties.

Therefore, in SAFEBRICTILE an objective and consistent assessment procedure for the safety assessment of the various failure modes that are relevant for steel structures was developed. The unified procedure resulted in codified procedures for inclusion in the structural Eurocodes and is able to cover modes driven by plasticity, stability and fracture.

A complementary and required task to accomplish this is also carried out within this project and consists of the conceptual development and further maintenance of a European database of steel properties resulting from experimental tests.

In addition, several rules in Eurocode 3 covering the failure modes treated in the project were reassessed in order to fulfil the developed safety assessment procedures.

The results of this project will lead to major competitiveness gains: (1) faster time-cycle in the development of new design procedures able to cope with innovation; (2) increased reliability in the accuracy of new design models; (3) major savings in R&D costs by avoidance of major duplication of work.

The following objectives will be fulfilled within SAFEBRICTILE:

- Development of an objective and consistent assessment procedure for the safety assessment of the various failure modes that are relevant for steel structures. The unified procedure will be able to cover:
  - modes driven by plasticity
  - modes driven by stability
  - modes driven by fracture.

- Development of a more complex procedure in which newly developed design rules are defined to a pre-established safety factor;

- Reassessment of several rules in Eurocode 3 covering the failure modes treated in the project by applying the developed safety assessment procedures in order to check their compliance with the target failure probability.

- Proposal of new or improved design rules for the cases where deviations from the target failure probability were observed.

- Conceptual development and further maintenance of a European database of steel properties resulting from experimental tests.

1.2 *Work package 1*

The first work package focused on the development and elaboration of a procedure for the determination and validation of partial factors for the Eurocode-based (EN 1990, EN 1993) design of steel elements. For this purpose, the existing procedures for the safety assessment of design rules verified by testing, given in EN 1990 Annex D, were reanalysed, adapted and expanded for the purposes and applications of the project. The main, semi-probabilistic reliability approach adopted in EN 1990 was kept as a reference framework for the developed safety assessment procedure. The main clarifications, changes and additions made in the safety assessment procedure developed in the project to the existing regulations in EN 1990, and - in particular - to the assessment procedure for γ_M in EN 1990 – Annex D, are summarized in the following:

i. The reliability differentiation in EN 1990, as it may be applied to steel structures, was explained; the developed recommendation clearly states that the reliability differentiation for steel structures is not typically conducted at the level of resistance factor differentiation (with the exception of fatigue design). Instead, a common target reliability level, expressed by a value of β=3.8, was applied for all safety verifications.

ii. The need for and the scope of the use of experimental data was clarified. The developed guideline to the safety assessment procedure shows how to use the experimental data in the context of the calculation of the error propagation term $V_{r,t}$ and in the averaging of the calculated values of γ_M^* for any given data pool.

iii. Methods for the reduction of the calculated model error parameters *b* and V_δ, i.e. the division of the experimental data into subsets and the method of tail approximation, were explained in the developed guideline.

iv. "Acceptance levels" for deviations between the calculated values of γ_M^* and existing (or desired) "target" values of safety factors γ_{Mx} (γ_{M0}, γ_{M1}, γ_{M2}…) were given and justified on both the basis of non-exceedance probabilities and past practice and experience. The values of "permissible" $\gamma_M^*=\gamma_{M,target}$ associated with certain failure probability multipliers are thereby strongly dependent on the scatter of the resistance. A small scatter of the resistance function implies that using an "incorrect", and too low, value of $\gamma_M^*=\gamma_{M,target}$ may lead to much higher than desired probabilities of non-exceedance of the strength used in design. High scatter – while usually not desirable – has the effect of making a precise choice of γ_M less relevant in terms of failure or non-exceedance probability.

v. The use of numerical experiments, in lieu or in addition to physical tests in the laboratory, was explained in detail in the guideline, and requirements and limits for their application were developed.

vi. The possibility of performing more advanced methods of reliability assessment in compliance with EN 1990, as well as the most important methods of this type, were described. These include Monte Carlo and response surface methods.

vii. The type and content of documentation reports needed for an independent evaluation (for example by code committees) were explained.

Finally, a complete worked example was prepared for the guidelines, which illustrates the main aspects of the proposed procedure.

1.3 *Work package 2*

The aim of Work Package 2 was to collect experimental results within the European research community and industry in a systematic way in order to statistically characterize the basic variables relevant to steel structures.

For that, a database including experimental data and statistical characterization of the basic variables was developed. The platform is accessible from the web site http://www.steelconstruct.com/ to a list of predefined users. It was already presented in conferences, scientific meetings in order to increase its popularity in the structural steel community in Europe. A standard form has been made available for those users willing to join.

The collection of the data from ArcelorMittal has concerned the production steel profiles covering the production of 2013 and 2014. The collected data cover the S235, S355 and S460 steels and steel profiles with flange thickness up to 140mm.

During the project, data was continuously collected from various sources stored in the database. The collected data were mostly coupon tests performed in universities in Europe. These tests serve for independent comparison with the results which were supplied by the steel producers, as the steels tested at the university laboratories are supplied by random producers. The collected data covers for European steel grades S235, S275, S355, S420, S460, S690, S960; API Line Pipe-5L X60, X70, ASTM grades, Chinese Q grades.

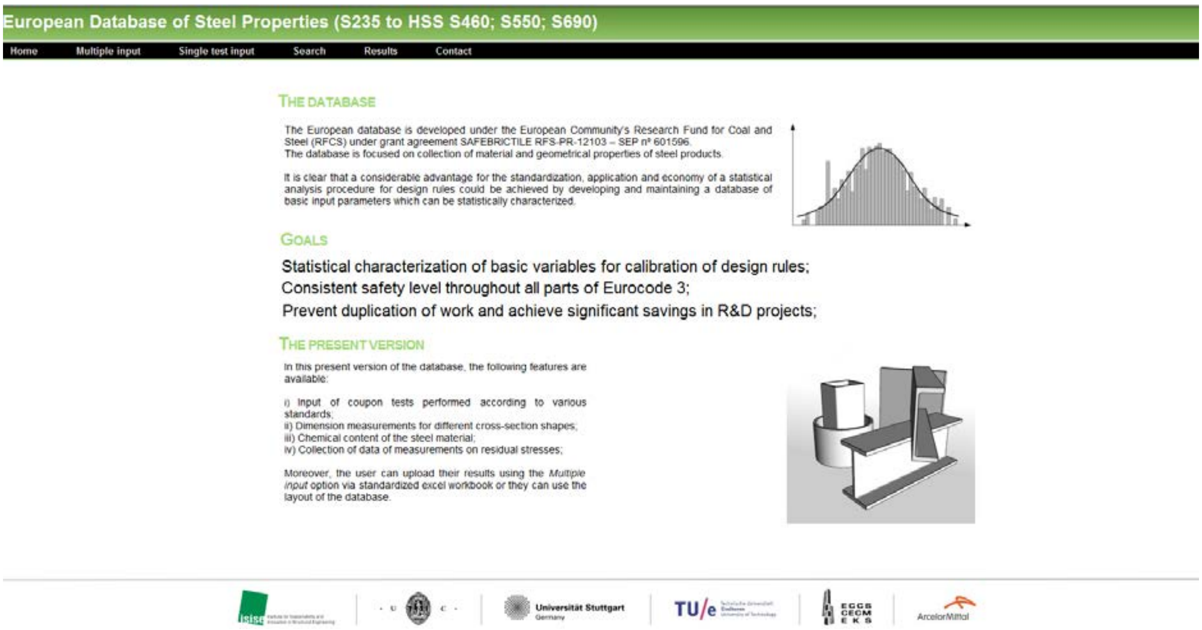


Figure 1.1 Database interface

Data was also collected from the literature. In particular, the two following sources of data were assessed:

- Data collected in Simões da Silva et al (2009) that comprises a large amount of data tested between 1996 and 2007 for steel grades S235, S275, S355, S460 and S690
- Data collected within the framework of the European project OPUS that comprises a large amount of data tested between 2007 and 2010 for steel grades S235, S275, S355, S460

Data collection for geometrical properties of steel H and I profiles was performed among several steel producers in Europe: ArcelorMittal, Dillingen, Salzgitter, Stahlwerk-Thueringen, Tata Steel. The results were supplied only as statistical parameters.

Table 1.1 Recommended distributions for yield and ultimate stresses

Steel	$f_{y,nom}$	$f_{ym}/f_{y,nom}$	**c.o.v.**	$f_{u,nom}$	$f_{u,m}/f_{unom}$	**c.o.v.**
S235	235	1.25	5.5%	360	1.2	4.5%
S355	355	1.2	5%	470	1.125	3.25%
S460	460	1.15	4.5%	540	1.1	3.25%

Table 1.2 Recommended distributions for geometrical dimensions of H and I sections

Dimension	**b**	**h**	t_w	t_f
mean/nom	1	1	1	0.975
c.o.v	0.9%	0.9%	2.5%	2.5%

Based on all collected data, the project consortium proposed recommended statistical distributions for yield and ultimate stresses of steel given in Table 1.1 and for geometrical dimensions of hot-rolled H and I sections in Table 1.2 that reflect current production results. It is noted that these distributions should be applied in accordance to the product standards, namely for each thickness interval.

1.4 *Work package 3*

In this part of the project (Work package 3, WP3), the safety assessment procedure of WP1 was applied to design rules for failure modes driven by plasticity depending on the material strength, using as an example the cross-sectional resistance focussing on combinations of internal forces. In WP3, design rules for the following failure modes were considered:

- moment-shear (M-V) interaction of I-shaped sections;
- net cross-section;
- moment-normal force (M-N) interaction of I-shaped sections, and;
- moment-normal force (M-N) interaction of rectangular hollow sections (RHS).

Except for the net cross-section failure mode, all other failure modes have design rules using the yield stress. The design rule for the net cross-section failure mode uses the tensile strength. However, it turned out that substantial yielding occurs when plates fail on their net cross-section. So all considered cases have failure modes driven by plasticity.

In the safety assessment procedures carried out in WP3, the statistical data on material properties of steel and on geometrical properties of I-shaped and rectangular cross-sections and steel plates, as gathered in WP2, were used. In some cases, also information on statistical data was obtained from literature.

In WP3 an extensive literature survey was carried out on available experimental and numerical test results related to cross-sectional resistance and a survey of current and proposed design rules in codes and in the literature for cross-sectional resistance was made. This was done for the failure modes listed above with an emphasis on combinations of internal forces: M-V and M-N.

A substantial number of 28 experimental reference tests were carried out, investigating the load bearing capacity and safety against yielding of sections under combined bending and shear (M-V) for steel grades up to and including S460. Also extensive associated standard tensile coupon tests were carried out to determine the material properties (stress-strain diagrams) in detail over the cross-section. Compressive material tests were also carried out as well as stub column tests. For net cross-section resistance, 37 experimental reference tests were carried out as well as associated standard tensile coupon tests. For moment-normal force (M-N) interaction of I-shaped sections 10 full scale tests were carried out with associated standard tensile coupon tests.

Finite element models were validated against the test results for moment-shear (M-V) interaction of I-shaped sections, net cross-section and moment-normal force (M-N) interaction of I-shaped sections by carrying out finite element simulations of the performed tests. For moment-normal force (M-N) interaction of rectangular hollow sections (RHS), the finite element model was validated for bending only and for normal force only. By doing further finite element simulations, the validated finite element models were used to form a database of 'numerical test results' against which design rules were statistically evaluated. These finite element simulations were done based on nominal material and geometry properties. Statistical variations on these properties were included in the safety assessment procedure.

For moment-shear interaction of I-shaped sections, it was shown that the current design rule of EN 1993-1-1 is inadequate. The formula for the shear area needs to be adapted and a new design rule for moment-shear interaction needs to be developed. For net cross-section, the research shows that the reduction factor 0.9 in the current design rule can be omitted making the design rule less conservative. Alternatively the partial factor can be relieved. For moment-normal force interaction of I-shaped sections and rectangular hollow sections, a modified design rule was proposed and evaluated, which better describes the moment-shear interaction diagram. The newly proposed design rules for moment-normal force interaction of I-shaped sections and of rectangular hollow sections are such that they have adequate safety with a partial factor of 1.0.

1.5 *Work package 4*

In Work Package 4, the modes driven by stability were assessed. The aim of this part of the project was to contribute towards the revision of EN 1993-1-1 by achieving transparent, simple and straight-forward unified stability verification procedures. For that, focus was firstly given to the application of the safety assessment procedure to the existing stability design rules in EN 1993-1-1 thus assessing the current safety level of uniform members in compression (cl. 6.3.1), bending (cl. 6.3.2), bending and compression (cl. 6.3.3) and the general method for lateral and lateral-torsional buckling of structural components (cl. 6.3.4). Based on the results obtained the stability verifications were extended to non-uniform members.

The safety assessment was performed on the basis of the procedure developed in Work Package 1. The assessment covered various hot-rolled cross-sectional shapes, loading conditions and buckling modes: flexural buckling about minor and major axes and lateral-torsional buckling including members loaded in bending and compression. The assessment revealed some inconsistencies in the buckling curves for flexural buckling about minor axis and steel grade S460. As a result, a modification of these curves was proposed. Regarding lateral-torsional buckling, the following design methods were assessed, namely the General and Special cases from EC3 and the new proposal from *Taras and Greine*r (2010). The latter method was found to give the most accurate estimations. Furthermore, the interaction formula for design of uniform members in bending and compression was also assessed based on a large amount of numerical results. The assessment showed satisfactory results. This assessment, incorporating the changes indicated for the buckling curves relatively to minor axis flexural buckling and steel grade S460 and the recommended statistical characterization of material and geometrical properties of steel and rolled steel profiles, showed adequate safety.

Regarding the general method for lateral and lateral-torsional buckling of structural components (cl. 6.3.4), it was found difficult to give recommendations on the correct application of the method for non-uniform members. It was assessed for large amount of cases. The results revealed that the method not only does not provide clear guidelines on which curve to be considered, but also may lead to a high (and random) spread regarding the level of safety, ranging from a decrease of 46% or an increase of 37% in the resistance capacity when compared to numerical results (GMNIA).

EN 1993-1-1 does not provide specific rules for non-uniform members. In SAFEBRICTILE, stability design rules were developed that address the specific difficulties in the verification of non-uniform members, such as critical location, choice of buckling curve, and cross-section properties.

The Eurocode 3 design rules for columns and beams make use of the Ayrton-Perry equations. In *Marques et al.* (2012) and (2013) it was found practical to have a similar format for the verification of web tapered columns and beams. Within the project, the Ayrton-Perry format for columns and beams was extended to the verification of web-tapered beam-columns loaded with major axis bending moment and axial force by adaptation of the interaction formulae in EN 1993-1-1, by adjusting the interaction factors k_{yy} and k_{zy}. The method was validated with a large number of numerical simulations, covering different cross-sectional shapes and bending moment diagrams.

Finally, as nowadays the daily design process makes extensive use of numerical models based on linear elastic analyses complemented by linear eigenvalue calculations, a method was developed for columns and beams based on discretized cross sectional verifications, which is able to cope with members with various geometries and loading. The method requires first a linear buckling analysis for each member. Then, the critical buckling mode is amplified with the imperfection factors originally deducted for uniform members. Consistency with the existing design rules and mechanical background is maintained.

1.6 *Work package 5*

The main objective of WP5 is to develop a method for statistical validation of design rules for typical failure driven by fracture depending on material strength using as example weld design strength of mixed connections of Mild Carbon Steel (MCS) and High Strength Steel (HSS) as base materials. The detailed specification of the WP5 can be found in Deliverable 5.1

For the dimensioning of welded joints with base and filler metals of different strengths, it is necessary to consider both strengths in the rated equation for consideration of the mixing of different materials. *Rasche*, (2012) has derived such a design rule. It is proposed that the strength of the base metal and the filler metal be weighted with factors. Accordingly, the strength of the base metal is taken into account with a weighting factor of 0.25 (25%) and that of the filler metal with a weighting factor of 0.75 (75%). The new design rule by *Rasche* (2012) is based only on welded connections with the same base metal.

$$\tau_{II,Rd} = \frac{0.25 \cdot f_{u,BM,i} + 0.75 \cdot f_{u,FM,i}}{\sqrt{3} \cdot \beta_{w,FM}}$$

One of the other objectives of the WP is to verify the applicability the modified design resistance by *Rasche*, (2012) for dual-steel connections and, if necessary, recalibrate them. A statistical evaluation is carried out in WP5 to pursue the goal on the basis of newly acquired test results and an adapted statistical evaluation method.

In addition to the earlier research results the new project wants to systematically feed the database with relevant data of welded connections, evaluate lots statistically in dependence on the different parameters such as type of welding (manual, automatic) and shape of weld in order to give at the end rules for mixed connections but also rules for an appropriate safety assessment procedure, where failure modes driven by fracture and tensile material strength play the important role.

1.7 *Work package 6*

Finally, a project management work package ensured effective communication between partners throughout the length of the project, coordinating consortium meetings, preparation of deliverables and other reporting documents.

Although Work packages 3, 4 and 5 came out with proposals of new rules for modes driven by plasticity, stability and fracture, in WP6 these proposals were collected and transferred to a format that can be put into code language.

As a result, a design guideline was prepared in the scope of Task 6.2. The document summarizes the developments done for the design rules treated within the scope of the project in a systematic way. For each design rule in the scope of the failure modes tacked in the project, firstly the possible issues related to the design rule are discussed, furthermore amendments in the existing rules, new rules or the satisfactory status of the design rules were proposed, and finally the background information for these proposals were summarized.

At the end of the project, a workshop was held in a parallel session of the International Colloquium on Stability and Ductility of Steel Structures – SDSS 2016 (31 May 2016). During the workshop, the key results achieved during the project were presented by representatives of each partner institution. The workshop was attended by roughly 40 participants from 20 different countries, among who experts involved in code drafting such as the appropriate TCs of ECCS and the Working Groups of CEN/TC250/SC3, which led to an interesting debate between speakers and attendees. The discussion was further extended outside Europe by prof. Richard Liew from the National University of Singapore, who added his contribution to the workshop, presenting the perspectives of large-scale buildings using Eurocodes.

The workshop presentations were collected in workshop proceedings which are available on the ECCS website.

1.8 *Conclusions*

The main achievements of the SAFEBRICTILE project are in line with the project objectives. Firstly, a detailed and fully tested procedure for the safety assessment of design rules was developed that fully linked each design rule to the target probability of failure of the complete structure, in the framework of EN 1990, Annex D. This detailed procedure is now progressively adopted by researchers around Europe, allowing consistency across research and code developments with respect to their underlying safety. Specific but crucial application aspects such as ensuring homogeneity of safety levels across the range of application of design rules (sub-sets) and acceptance levels for the unavoidable scatter of safety across subsets were addressed, as well as the issue of quality control (documentation) and ease of use (through the supply of detailed examples).

Secondly, the project collected a database of steel properties based on recent tests. Given that many steel properties are nominal properties, the project proposed statistical distributions that correspond to modern steel production in Europe for yield and ultimate stresses and geometric dimensions that are in line with the recommended partial factors γ_{M0}, γ_{M1} and γ_{M2} of Eurocode 3, part 1-1 in terms of satisfying the target failure probabilities.

Finally, the project carried out a comprehensive safety assessment of most design rules in EN 1993-1-1 concerning cross sectional resistance and buckling resistance of members (see sections 4 and 5) and the resistance of fillet welds in EN 1993-1-8 (section 6) that, in some cases, led to the adjustment of some rules and showed that the recommended partial factors of EN 1993-1-1 were acceptable. Additionally, for non-uniform members and mixed fillet welded connections, extended but compatible rules were proposed that present a similar level of safety.

The outcomes of the SAFEBRICTILE project should provide objective and transparent guidance for the ongoing revision of Eurocode 3 and its future maintenance, while contributing to the competitiveness of the European steel sector.

2 Introduction

The SAFEBRICTILE project (Ref. No. RFSR-CT-2013-00023) intended to contribute towards the harmonization of the reliability level of design rules for steel structures covering modes driven by ductility, stability and fracture. There were some inconsistencies regarding the design rules in the various parts of Eurocode 3, which were of major focus during the project, i.e. discontinuity in the safety levels due to the lack of guidance on how to perform safety assessments and to calibrate new design rules, as well as the lack of information on the distribution of the relevant basic variables such as the steel properties.

Therefore, in SAFEBRICTILE, an objective and consistent safety assessment procedure for the various failure modes that are relevant for steel structures was developed. The developed procedure makes use of statistical distributions of the relevant basic variables, which were collected continuously during the project in a database of steel properties. This collection allowed for final recommendations on the statistical distributions of the relevant parameters. The unified procedure, being able to cover failure modes driven by plasticity, stability and fracture, was used to reassess and amend several design rules within the project. The results of this project led to major competitiveness gains: (1) faster time cycle in the development of new design procedures able to cope with innovation; (2) increased reliability in the accuracy of new design models; (3) major savings in R&D costs by avoidance of major duplication of work.

The project objectives identified at the beginning of the project were:

- Development of an objective and consistent assessment procedure for the safety assessment of the various failure modes that are relevant for steel structures. The unified procedure will be able to cover:
    - modes driven by plasticity
    - modes driven by stability
    - modes driven by fracture.
- Development of a more complex procedure in which newly developed design rules are defined to a pre-established safety factor;
- Reassessment of several rules in Eurocode 3 covering the failure modes treated in the project by applying the developed safety assessment procedures in order to check their compliance with the target failure probability.
- Proposal of new or improved design rules for the cases where deviations from the target failure probability were observed.
- Conceptual development and further maintenance of a European database of steel properties resulting from experimental tests.

In the following paragraphs, a summary of the work carried out is presented. The reporting is organized per work packages in order to facilitate the identification of the detailed objectives, as more specific information about the work performed can be found in the project deliverables.

3 Work package 1 – Development of safety assessment procedure

3.1 *Objectives of WP1*

The first work package focused on the development and elaboration of a procedure for the determination and validation of partial factors for the Eurocode-based (EN 1990, EN 1993) design of steel elements. The objectives of WP1 consisted of the following points:

- To develop a semi-probabilistic safety assessment procedure in line with EN 1990;
- To give guidance on the choice and characterization of the relevant variables to be considered in the safety assessment procedure;
- To develop a procedure with pre-established values of the partial factor for a certain design rule.

3.2 *Work undertaken and results obtained*

Introduction

The objectives of WP1 were tackled in two tasks:

- Task 1.1: Development of a safety assessment procedure in line with EN 1990
- Task 1.2: Development of procedures for the assessment of safety with pre-established, target values of the safety factors

The two tasks were strongly connected: in the first Task 1.1, a systematic, harmonized procedure for the safety evaluation of steel structural components was developed, which may be applied to all types of design criteria for steel structures and is in line with the general guidelines of EN 1990. Task 1.1 led to the deliverable D1.1, i.e. a guideline for the harmonized determination of partial factors γ_M for any set of structural steel design rules. In Task 1.2, this procedure was reversed: starting from a given, pre-set value of the partial factor (the "target value"), the proposed design rules were analyzed systematically in order to identify the parameters that may have to be modified in order to achieve the desired level of reliability.

Task 1.1: Development of a safety assessment procedure in line with EN 1990

The safety assessment procedure developed in Task 1.1 represents a steel-specific implementation of the rules given in EN 1990 – Annex D. It provides a method of complying with the reliability requirements detailed above, and is thus an implementation of First Order Reliability Methods and a standardized "split" allocation of reliability components between the actions and resistance side. The flow-chart provided in the guideline (deliverable D1.1) is shown in Figure 6.1.

In many cases, γ_M^* values determined in accordance with the above procedure will be compared with already-existing, codified values of γ_M, e.g. γ_{M0} (=1,00), γ_{M1} (=1,00 or 1,10) or γ_{M2} (=1,25) in EN 1993 and its national annexes. This will particularly be the case when a new design rule is developed for a specific application, and it is desired that this rule may fit in the existing framework of codified values of the partial factor. Obviously, due to the coupling with experimental results and statistical data, an evaluation of γ_M^* will lead to values that are not "precisely" equal to – for example – 1,00, 1,10 or 1,25. Thus, acceptance criteria (i.e. limits of acceptability) must be declared in these cases.

If the codified or desired value of γ_M is termed "$\gamma_{M,target}$" and the calculated value is γ_M^*, the following condition can be written:

$$\gamma_M{}^*/\gamma_{M,target} \overset{!}{\leq} f_a \text{ (acceptance limit)} \tag{3.1}$$

In Task 1.1, the value f_a was quantified on the basis of probabilistic considerations and a proposal was made for an acceptance level of f_a, which was then used consistently throughout the project. A graphical representation of the acceptance value, as a function of the total scatter of the resistance function V_r, is shown in Figure 3.1.

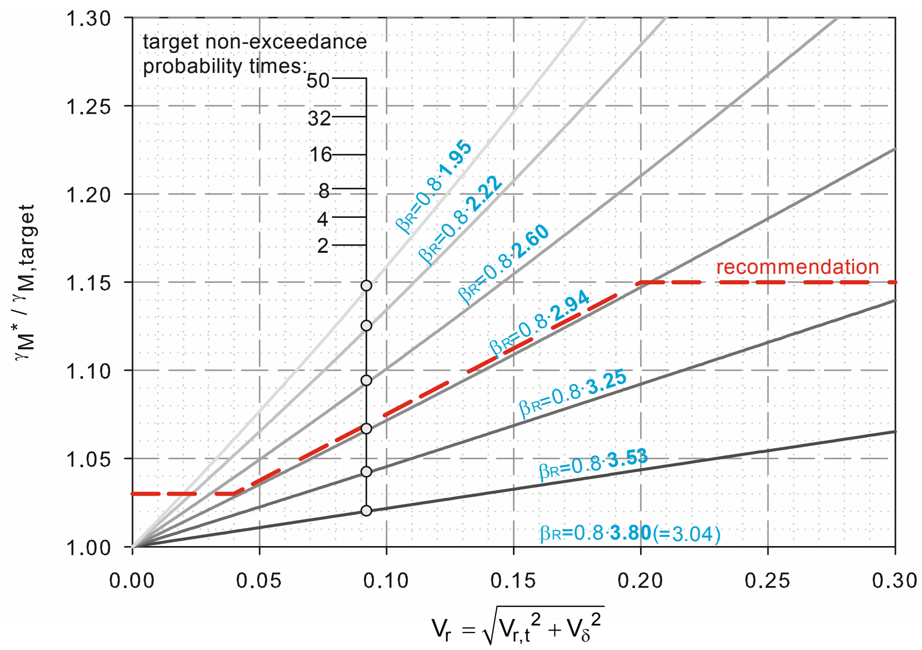


Figure 3.1 Impact of the ratio γM*/γM,target on the implied notional failure probability as a function of the coefficient of variation Vr and recommended values of fa (i.e. the acceptance limit for γM*/γM,target)

Calculation of the error propagation term Vrt and possible simplifications

In Task 1.1, the influence of the scatter of individual basic variables on the scatter of the resistance function of various steel design rules was thoroughly analyzed, Thereby, several alternative ways of calculating the coefficient of variation of the theoretical resistance function, the "error propagation" term V_{rt}, were calculated.

It was particularly shown that, for a number of applications, it may be convenient to separate and visualize the different impact of material and geometric parameters, respectively, on the coefficient of variation $V_{r,t}$. Figure 6.3 shows, as an illustrative example, values of V_{rt} for weak-axis flexural buckling of a HEA 300 compression member, and its split among terms related to the yield strength f_y, the E-modulus E and the cross-sectional geometry (CS), plotted over different nominal slenderness values $\bar{\lambda}_{nom}$.

For example, in Figure 6.3 the values for $V_{rt,fy}$ represent the contribution of the scatter of the yield strength f_y to the scatter of the column resistance, while the values for $V_{rt,CS}$ pertain to the cross-sectional geometric properties of the HEA section: the flange thickness t_f, the web thickness t_w, total depth and width, etc. Finally, $V_{rt,E}$ shows the influence of the scatter of the Young's modulus.

$V_{rt,fy}$ and $V_{rt,E}$ may be further grouped together into a category $V_{r,t,mat}$ for all material strength and stiffness parameters, while $V_{r,t,CS}$ may be grouped with other geometric parameters into a value $V_{r,t,geom}$.

For one, this type of split can help one identify which parameters are truly of relevance and which ones may be omitted, and over which parameter ranges. Additionally, it helps in the decision of how to categorize the "n" experimental results into sub-categories.

Step 1

Resistance function – theoretical resistance $r_{t,i}$

$$r_{t,i} = g_{rt,i}(X_1, X_2 ... X_k)$$

The resistance function is a function of the selected relevant basic variables X_1

Experimental resistance

Experimental values $r_{e,i}$ for each test specimen (these results can be obtained from tests or numerical estimates from advanced simulations)

Basic variables

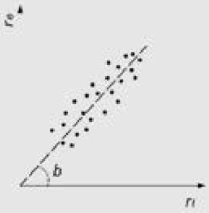
Pre-information is needed regarding the distributions of the basic variables $\mathbf{X_j}$: *mean value* $\mathbf{X_{jm}}$ and *standard deviation* $\boldsymbol{\sigma_j}$

Step 2

Estimate mean vale of correction factor *b*

$$b = \sum_{i=1}^{n} r_{e,i} r_{t,i} \Big/ \sum_{i=1}^{n} r_{t,i}^{2}$$

The factor is a global value for the selected subset. The correction factor can be used as an indicator of the accuracy of the model, i.e. it should not strongly diverge from unity (0.85-1.15)



Estimate CoV of the error:

$$\delta_i = r_{e,i} / b r_{t,i} \qquad \Delta_i = \ln(\delta_i)$$

$$s_\Delta^{\ 2} = \frac{1}{n-1} \sum_{i=1}^{n} \left(\Delta_i - \bar{\Delta}\right)^2$$

$$V_\delta = \sqrt{\exp(s_\Delta^{\ 2}) - 1}$$

The CoV of the model error is a global value for the selected subset and it represents the difference between th model estimate and the experimental estimate.

Step 3 (for each specimen i)

Determine the coefficient of variation V_{rt} – it relates to the basic input variables:

i) In case of simple product function, the following expression is used: $V_{rt} = \sum V_{Xj}^{\ 2}$

ii) In case of more complicated resistance function, V_{rt} can be obtained for each specimen as:

$$V_{rt,i}^{\ 2} = \frac{1}{r_{t,i}\left(\underline{X_m}\right)^2} \sum_{j=1}^{k} \left(\frac{\partial r_{t,i}\left(X_j\right)}{\partial X_j} \sigma_j \right)^2$$

Often, it is complicated to use the analytical expression of the partial derivatives; a numerical estimate can be used instead. It can be determined from:

$$\frac{\partial r_{t,i}}{\partial X_j} \approx \frac{r_{t,i}(X_1,..., X_j + \Delta X_j,....X_k) - r_{t,i}(X_1,..., X_j,....X_k)}{\Delta X_j}$$

where ΔX_j is sufficiently small increment

Step 4 (for each specimen i)

Determine design value of the resistance, for each test and $V_{rt,i}$

$$Q_i = \sqrt{\ln(V_\delta^{\ 2} + V_{rt,i}^{\ 2} + 1)}$$

$$r_{d,i} = b r_{t,i}(\underline{X_m}) \exp\left(-k_{d,\infty} Q_i - 0.5 Q_i^{\ 2}\right)$$

Determine the partial factor, as a mean value from the factors for each specific case:

$$\gamma_{M,i}{}^* = r_{nom,i} \big/ r_{d,i} \longrightarrow \gamma_M{}^* = 1/n \sum \gamma_{M,i}{}^*$$

Figure 3.2 Flow-chart for the test-based statistical evaluation procedure, based on EN 1990 Annex D

On the use of numerical simulations

For a number of applications and failure scenarios, it is convenient and has become common practice to make use – as a complementary or exclusive measure – of *numerical experiments*. Recommendations for the validation and use of numerical experiments are given in deliverable D1.1.

Numerical tests are particularly advantageous or even necessary whenever a large pool of "experimental" results is needed, for example to cover an extensive range of parameters, types of materials and cross-sections, so that a complete test campaign in the laboratory becomes unworkable. Particularly in cases that involve failure mechanisms involving yielding and/or elastic instability, FEM or other comparable numerical models may be used to great effect to simulate a laboratory test on the computer.

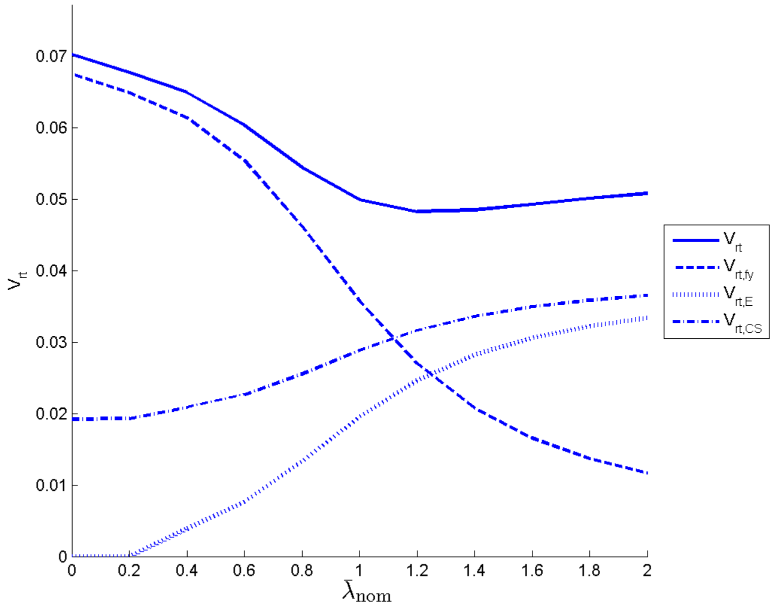


Figure 3.3 Exemplary representation of V_{rt} for weak-axis flexural buckling of a HEA 300 compression member, and its split among terms related to the yield strength f_y, the E-modulus E and cross-sectional geometry (CS).

However, in order to make sure that reliable numerical test results are used in the subsequent reliability analysis, it is strictly necessary to validate the used numerical model against real, physical tests whenever large numerical test campaigns are to be performed with it (however: see the exceptions detailed in D1.1).

In terms of structural reliability, the use of a numerical (instead of a physical) experiment introduces a new degree of model uncertainty into the assessment procedure for the resistance function: in addition to the model error between the theoretical resistance r_t (the design formula to be used) and the experimental results r_e ($=r_{e,phys}$ for "physical test"), an additional model error between the numerical test resistance $r_{e,num}$ and $r_{e,phys}$ is introduced, see Figure 6.4.

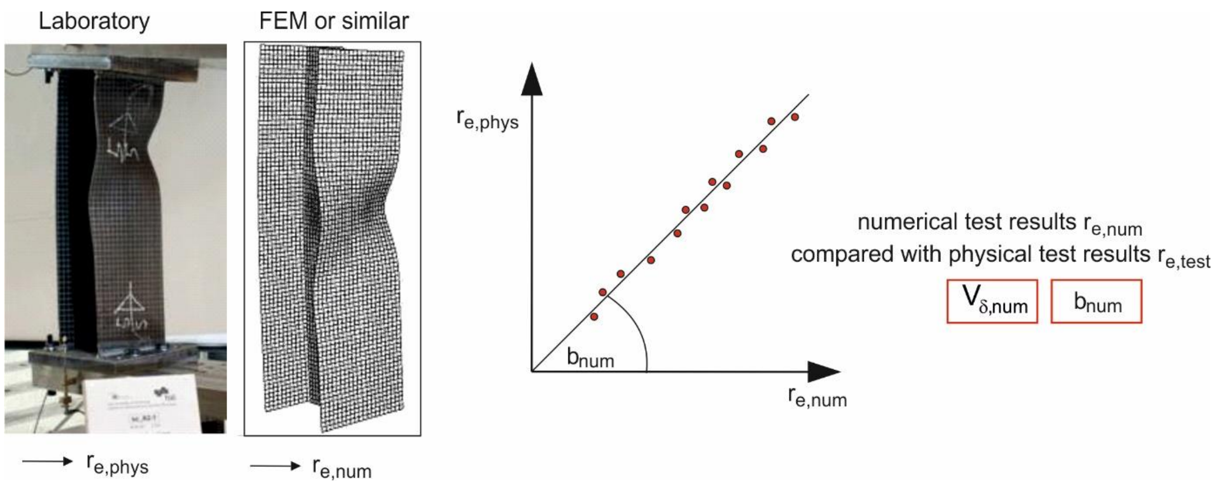


Figure 6.4: Validation of numerical models used to obtain "numerical experiments"

Task 1.2 Development of procedure for the assessment of safety with pre-established target values of the safety factors

In Task 1.2 a procedure is developed to calibrate design rules aiming to reach a target value of the partial factor. This is accomplished based on the EN 1990 procedures detailed and further completed in Task 1.1 (D1.1), applied to GMNIA (Geometrical and material nonlinear analysis with imperfections) models and/or experimental test results obtained in the project.

This approach is essentially a First Order Reliability Method (FORM) based on GMNIA calculations. Similarly, to the procedure developed in Deliverable D1.1, information regarding the scatter band and correlation of the properties of steel members is required. The computation effort for this approach is larger than the direct approach developed in Task 1.1, however, its objective is to achieve a constant value of γ_M.

This procedure was developed and applied to examples in WP4 and WP5. The data collected from the relevant input parameters of the studied problem is used to apply the developed methodology.

The developed procedure has two main objectives: i) to allow for the adjustment of existing design rules to the target safety in the code; ii) to allow for calibration of new design rules to a target partial factor in the code. In both cases, the resulting partial factor shall be constant within the scope of application of the resistance function.

In the scope of the respective work packages, the procedure from D1.1 was applied to various design rules from Eurocode 3. Even though the correct application of the procedure was ensured, in some cases it may lead to safety factors that vary with respect to the subgroups considered (as shown in *Figure 3.4.*); in other cases very high or very low factors were obtained, thus indicating non-homogeneous safety within the design rule and with respect with other design rules. Hence, in order to improve the resistance function, it was considered useful to introduce a factor that modifies the design rule.

There are two possibilities of the factor:

- Introduce a **factor for constant reliability**, which multiplies the final result of the verification;
- Introduce **a factor in the design rule within a relevant parameter**, e.g. imperfection factor α for flexural buckling of columns from EN1993-1-1;

The approach is directly linked to the procedure developed in Deliverable D1.1. It assumes that the constant reliability level of a design rule can be ensured though a factor for constant reliability. By applying the procedure, one calculates the required partial factors by using the distributions of the basic variables and the model variability in order to guarantee the target reliability index for resistances $\alpha_R\beta$ (=3.04 in this case). Hence in order to achieve a constant factor, which accounts for the variability of the basic variables and model uncertainty, additional function/factor needs to be calibrated.

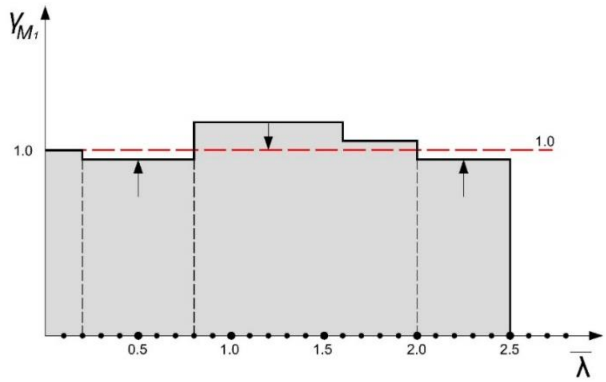


Figure 3.4: Constant reliability

The improvement of the resistance function was done by introducing a change followed by subsequent safety assessment. The judgement of the efficiency of this change is done on the basis of the resulting partial factors. The application sequence of the proposed method is presented in *Figure 3.5*.

- *Step 1:* Estimates of the design resistance – experimental or numerical tests. These are used for comparison with the analytical model;
- *Step 2*: Analytical model which is able to reproduce the design resistance. The calibration of partial factors is done to the existence of a design rule. In the case of existing design rules, those are amended in order to achieve constant reliability. In the case of calibration, a new design rule, the recommendations in Sections 3.3.1 and 3.3.2 from D1.2 can be alternatively applied.

Step 3: Direct assessment – by applying the procedure from D1.1 (

- Figure 3.2)*,* it can be verified, if the partial factor results in uniform value or not. In case they are not, this is the initial point for the assessment.

- *Step 4:* Definition of critical subsets to be addressed – in this step, the critical subsets are defined. They are used in a further step in order to check the relative improvement of the design rule.
- *Step 5:* In this step, a critical evaluation of the results is performed. It serves to introduce a future factor for the further calibration.
- *Step 6:* Choose a factor or a function of constant reliability and repeat the safety assessment;

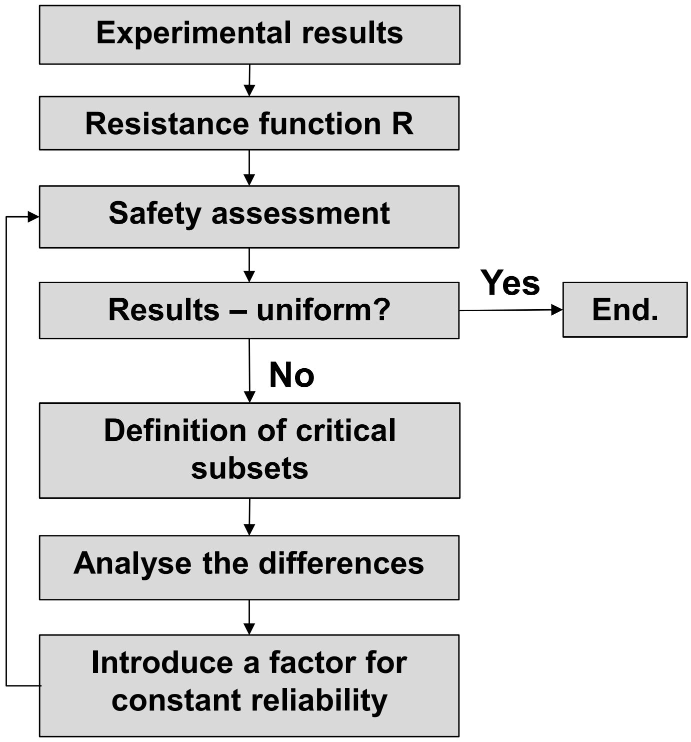


Figure 3.5: Application procedure for achieving pre-established safety factors

As an alternative, an approach based on calibration to the constant reliability curve was suggested for calibration of design rules. The method is suitable for newly developed design rules even prior to having an analytical expression. The process of "building" the curve itself may be laborious since the partial derivatives are evaluated numerically and based on the results from GMNIA.

Alternatively, it was suggested that the quality of the design rule can be based on the reliability index β and therefore a direct comparison of the reliability of the design method. The reliability index obtained from the procedure can be directly compared to the target reliability in EN1990. The advantage of this approach is in the estimation of the model uncertainty, the inconsistencies in the design rule can be spotted in an earlier stage of the assessment and they can be adjusted before applying the FORM.

3.3 *Conclusions*

With this Work Package, a harmonized safety assessment procedure on the basis of EN 1990 was made available, which expands and clarifies the application of EN 1990 Annex D to the assessment of design rules for steel structures. The procedure can be applied across different failure modes, as set out at the project start. The new procedure represents an expansion of the EN 1990 provisions. Therein, the most important points are:

- Methods for the reduction of the calculated model error parameters b and V_δ, i.e. the division of the experimental data into sub-sets and the method of tail approximation.
- "Acceptance levels" for deviations between the calculated values of $\gamma_M{}^*$ and existing (or desired) "target" values of partial factors γ_{Mx} (γ_{M0}, γ_{M1}, γ_{M2}…).
- The use of numerical experiments in lieu or in addition to physical tests in the laboratory, and requirements and limits for their application.
- The type and content of documentation reports needed for an independent evaluation (for example by code committees).

Researchers in the field of steel structures design were thus armed with appropriate tools and reproducible methodologies to assess newly developed design rules with respect to the necessary values of γ_M, respectively to fine-tune the rules to match certain “target” values of the partial factors.

4 Work package 2 – European Database of Steel Properties (S235 to HSS S460; S550; S690)

4.1 *Objectives of WP2*

The objectives of this WP are as follows:

- Collection and treatment of data from physical experiments
- Statistical characterization of the basic variables
- Conceptual development of a platform for the collection and maintenance of the European Database
- Guidelines for standard reporting of test measurements.

4.2 *Work undertaken and results obtained*

Introduction

The variability of the mechanical properties of the building materials plays a primary role in the assessment of structural safety together with the variability of actions that in some case, as for the earthquakes, are hardly predictable. For this reason, the possibility of characterizing real mechanical properties or the use of probabilistic safety "coefficients" consistently with current steel production can help increasing of overall structural safety, and decreasing the uncertainties. In this design context, the characterization of the mechanical properties of steel structural elements was one of main priorities of this work package. In particular, the properties of steel elements are studied in order to accurately define their mechanical behavior and the scatter of their actual properties with respect to the values specified in the production standards and in the structural design codes. Such purposes need the definition of an enriched statistical set of data, as a basis for the future modelling and processing procedures.

The data collection was performed by the work package leader (Arcelor Mittal) from their plants but it was also done by the project partners from the tests performed in their laboratories.

Task 2.1 Collection and treatment of data from physical experiments

The collection of data aimed to attract contributions coming from different industries. In particular, the following steel elements should have been investigated: steel profiles, steel plates and reinforcing bars, characterized by different steel grades.

Unfortunately, apart from *ArcelorMittal*, no other steel producers responded to this call.

ArcelorMittal collected data from the *Differdange* plant (Luxembourg). The steel profiles were rolled in this plant following a thermo-mechanical process. The period, considered for the data collection, covers the production of 2013 and 2014.

The data collected (3587 coupon test results) covers S235, S355 and S460 steels and steel profiles with flange thickness up to 140 mm.

During the project, data was continuously collected from various sources stored in the database. The collected data were mostly coupon tests performed at various universities in Europe. These tests serve as an independent comparison with the results which were supplied by the steel producers, as the steels tested at those university laboratories are supplied by random producers.

- The collected data covers (1760 coupon tests (R_{EH})) for European steel grades S235, S275, S355, S420, S460, S690, S960; API Line Pipe-5L X60, X70, ASTM grades, Chinese Q grades

Data was also collected from the literature. In particular, the two following sources of data were assessed:

- Data collected in Simões da Silva et al (2009) that comprises a large amount of data (7454 coupon test results) tested between 1996 and 2007 for steel grades S235, S275, S355, S460 and S690

Data collected within the framework of the European project OPUS that comprises a large amount of data (25425 coupon test results) tested between 2007 and 2010 for steel grades S235, S275, S355, S460.The comparison between the steel data collected from the different sources revealed that the coefficient of variation for all samples is similar. However, for steel grades S235 and S355, the data supplied by *ArcelorMittal* revealed higher $f_{y,m}/f_{y,nom}$ by about 20% in certain cases S235 and 10% for S355.

Data collection for geometrical properties of steel H and I profiles was performed among several steel producers in Europe: *ArcelorMittal, Dillingen, Salzgitter*, *Stahlwerk-Thueringen, Tata Steel*. The results were supplied only as statistical parameters.

- The database collected 1064 measurement of dimensions for H and I profiles.

Only limited data was collected for member imperfections, residual stresses and out-of-straightness measurements, and therefore they were not statistically characterized. However, as the reference value used in all simulations for the residual stresses and member imperfections is conservative (*Subramanian and White (2017)*), the results might improve from a safety point of view.

Task 2.2 – Statistical characterization of basic variables

The statistical characterization was performed for material and geometrical properties. The results of the characterization were summarized as recommended distributions, given in Table 4.1 and Table 4.2 that reflect current production results. It is noted that these distributions should be applied in accordance to the product standards, namely for each thickness interval.

Table 4.1 Recommended distributions for yield and ultimate stresses

Steel	$f_{y,nom}$	$f_{ym}/f_{y,nom}$	c.o.v.	$f_{u,nom}$	$f_{u,m}/f_{unom}$	c.o.v.
S235	235	1.25	5.5%	360	1.2	4.5%
S355	355	1.2	5%	470	1.125	3.25%
S460	460	1.15	4.5%	540	1.1	3.25%

Table 4.2 Recommended distributions for geometrical dimensions of H and I sections

Dimension	b	h	t_w	t_f
mean/nom	1	1	1	0.975
c.o.v	0.9%	0.9%	2.5%	2.5%

Task 2.3 – Conceptual development of a platform for the collection and maintenance of the European Database of Steel Properties (S235 to HSS S460; S550; S690)

The conceptual development of the platform is hereby presented. The database was developed and currently maintained using the software product FileMaker Pro Advanced 13.0v5. The platform is accessible from the web site http://www.steelconstruct.com/ to a list of predefined users.

The database was already presented in conferences, scientific meetings in order to increase its popularity in the structural steel community in Europe. A standard form has been made available for those users willing to join and it is also available in the aforementioned website.

Currently, the database is hosted at University of Coimbra (UC). A regular data processing is scheduled in order to update the summary of results in the platform. The general work scheme is summarized in *Figure 4.2.* The data is uploaded from the database users to the online platform. The data is statistically characterized in certain interval of time, then the results are updated on the platform, where they can be accessed by the users.

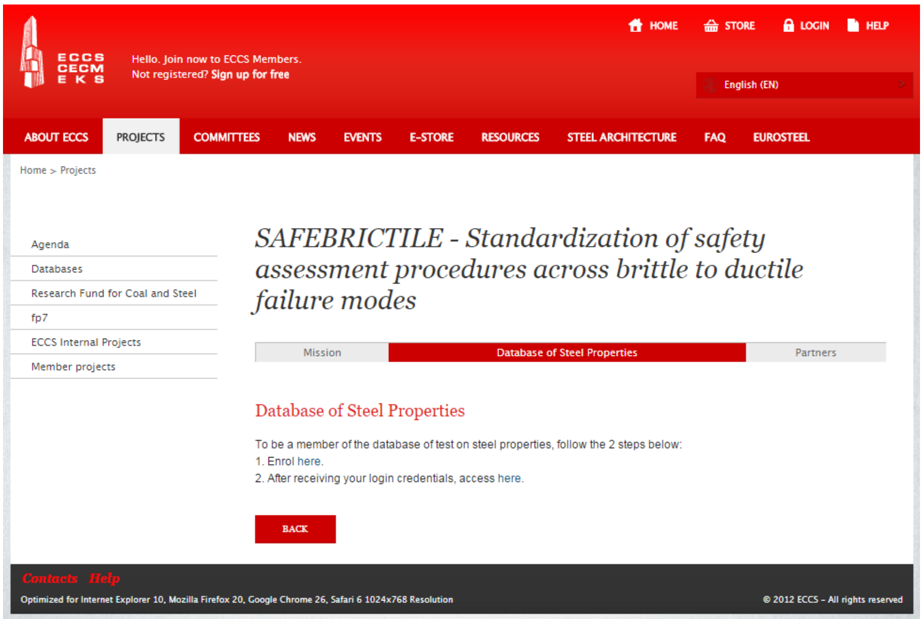


Figure 4.1 http://www.steelconstruct.com/

Figure 4.2 Platform functionality

Structure of the database

Databases are powerful tools to store, manipulate and represent data. They are different from ordinary electronic spreadsheets, which are mostly used to tabulate and calculate data stored in the cells of a table. On the other hand, a database is a collection of data tied together via various relationships which state the organization of the database tables and their fields. The records can be easily divided into subsets with respect to certain criteria.

Therefore, the structure of a database is based on implementation of different tables. These tables may contain different information which can be accessed via predefined relationships. This gives a powerful option for organizing, updating, sorting and searching through data.

There are three main categories of data which are collected in the database: material properties, geometrical data and imperfections. They are related, as the yield stress is dependent on the specimen thickness, the magnitude and shape of residual stresses is dependent on the cross-section type.

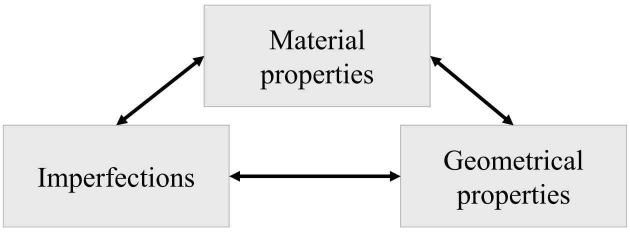


Figure 4.3 Data categories

Collection of data

An essential part of the development and functionality of the database is the collection of data. In this present version of the database, two alternatives are included: i) using the provided interface and specifying each test; or ii) using the pre-defined excel workbook, which can be filled with many tests and submitted at once in the database.

Another important point for the collection is the dissemination of information about it. It was important that it is well advertised in order to attract more contributors. The dissemination of information about the database during the project was focused on steel producers and fabricators; University researchers.

Database interface

The main interface of the database is implemented as shown in *Figure 4.4*. It has a top-bar menu which navigates the users between the different options. A closer look of the menu is better seen on *Figure 4.5*. The *Home* link always returns the users to the initial page of the platform.

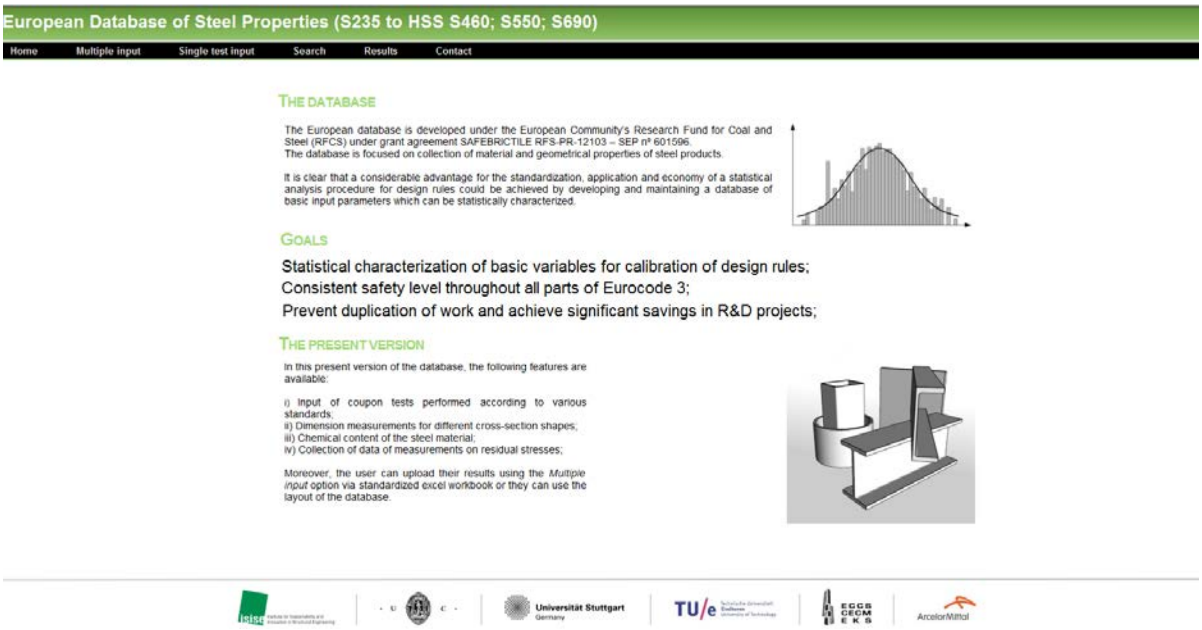


Figure 4.4 Home page

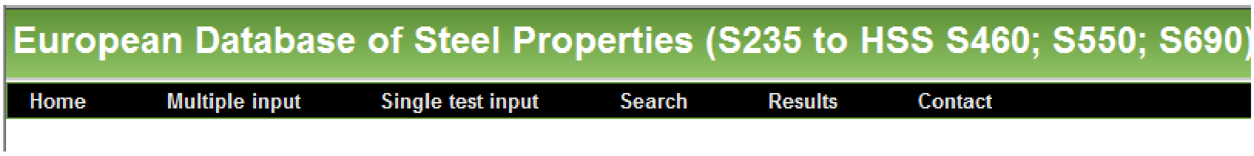


Figure 4.5 Navigation bar

The *Multiple input* is where it is possible to download and upload the standard input workbook. It is also possible to upload supplementary files like pictures, pdf files, etc. which are connected to the respective workbook.

In *Single test input*, special interface is prepared in order to input their test results on screen.

The *Search* option gives opportunity to find statistical parameters for steel properties and geometrical dimensions which are currently on the database and visible to the type of user. In addition, it is possible to see tests on the residual stress distribution of steel profiles. This option show unfiltered data, therefore in the *Results* tab the summary of statistical characterization is presented. The summary is performed in a regular amount of time and therefore it is not updated automatically, when new data is added.

Finally, a *Contact* option is given, in case the users would like to share their opinion and/or if they need any kind of assistance regarding the usage of the database.

Standard reporting formats

Another option of input of data is the standard excel workbook (Figure 4.6). It is prepared in accordance with the database tables in order to facilitate the input of many tests.

The information in the standard input excel workbook is divided into different sheets in order to import the data to the main database. The first sheet of the workbook is informative and gives instructions to the user on how to use it.

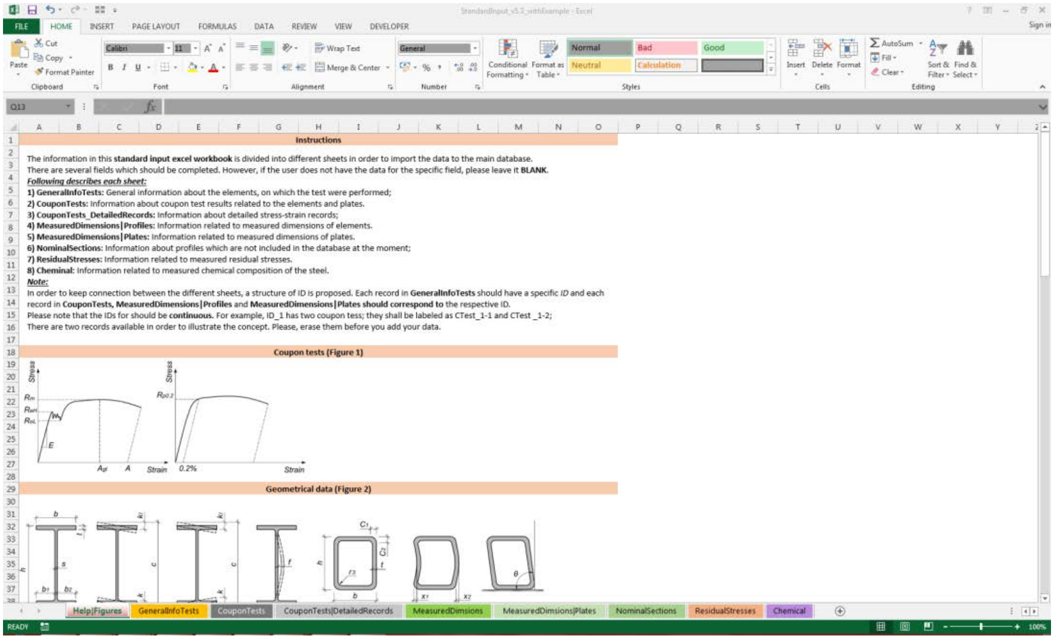


Figure 4.6 The standard excel workbook

4.3 *Conclusions*

In WP2, several actions were taken in order to ensure the desired statistical characterization, namely meetings with steel producers, data collection from the AMBD plants, data collection from the experiments performed in the laboratories of the partners as well as from other European universities. Data was also collected from previous statistical characterizations from the literature which were considered representative.

A platform was developed and presented at different meetings aiming to attract interest from potential data contributors.

Finally, the statistical characterization was based on the results obtained from all these sources, basing the conclusions on more than 28 000 results collected for S235, S355 and S460. The distributions were proposed in a normalized way, as a ratio between the mean and the nominal, thus allowing the adaptation of the distribution with the varying stresses with thickness.

Similar assessment was performed for the geometrical properties of H and I sections. The statistics were based on collected data from physical experiments (1064 measurements); data from various steel producers and data from previous statistical characterizations.

5 Work package 3 – Modes driven by plasticity

5.1 *Objectives of WP3*

The third work package is focused on a evaluation of the current EN 1993-1-1 cross-sectional design rules for assessing the structural safety of steel structures. Particular interest is given to the design rules for failure modes driven by plasticity depending on the material strength. The cross-sectional resistance focusing on the combination of internal forces is especially addressed. The research is mainly focused on I- and H-shaped cross-sections, but also includes other double symmetric cross-sections.

The work package consists of the following successive objectives (corresponding to tasks 3.1 to 3.5):

- carrying out a literature survey on available experimental and numerical test results and on the current and proposed design rules in codes and in the literature with particular interest for the cross-sectional resistance under combined internal forces.
- performing experimental reference tests - investigating the load bearing capacity and safety against yielding of:
  - sections under combined bending and shear for mild and high-strength steel grades;
  - plates with bolts and plates with bolt holes for mild steel;
  - sections under combined bending and normal force for mild steel.
- Validating and making use of a finite element (FE) model by carrying out FE simulations of the performed tests and forming a database of 'test' results for statistical evaluation by doing further finite element simulations.
- Applying the developed statistical procedures to verify the safety of current and proposed design rules for cross-sectional resistance and propose modifications of these rules if necessary.
- Developing comprehensive recommendations for the statistical evaluation of ductile failure modes and for cross-sectional resistance.

5.2 *Work undertaken and results obtained*

Introduction

In Work Package 3, WP3 the following failure modes were extensively considered:

- moment-shear (M-V) interaction of I-shaped sections, cl. 6.2.8 of EN 1993-1-1;
- net cross-section, cl. 6.2.2 and cl. 6.2.3 of EN 1993-1-1;
- moment-normal force (M-N) interaction of I-shaped sections, cl. 6.2.9 of EN 1993-1-1, and;
- moment-normal force (M-N) interaction of rectangular hollow sections (RHS), cl. 6.2.9 of EN 1993-1-1.
-

Implicitly, also the clauses 6.2.4, 6.2.5 and 6.2.6 of EN 1993-1-1 for individual compression, bending moment and shear respectively are covered. For the failure modes considered, 5 tasks were distinguished and performed to a more or lesser extent. First, a literature survey was made as Task 3.1 to get an overview of current and proposed design rules in codes and in the literature for cross-sectional resistance and of available experimental and numerical test results related to cross-sectional resistance. Then as Task 3.2, experimental reference tests were carried out to obtain reliable and detailed test results for the failure modes considered, except the last one. These test results are characterised by failure loads and load-displacement diagrams. The test results are used for Task 3.3, Finite Element modelling. Finite element models are developed for the failure modes considered and these are validated using the test results. For that reason, detailed material properties and geometrical properties were determined as part of Task 3.2, experimental reference tests. Measured values of these properties were used in the validation of the Finite Element models. With the validated Finite Element models, a database of 'numerical test results' was created using nominal material and geometrical parameters, as part of Task 3.3 by performing numerous finite element calculations. Subsequently, Task 3.4 concerns the statistical evaluation of current and newly proposed design rules against these databases of 'numerical test results' making use of the statistical assessment procedure of WP1 and the statistical data for material and geometry of WP2 to obtain the adequate associated partial factors. Task 3.5 consists of writing recommendations for the statistical procedure and the design rules with their partial factors recommended for use in the future version of Eurocode 3, EN 1993-1-1, clause 6.2. Experimental data on geometry and material as obtained in Task 3.2 is added to the European Database of Steel Properties of WP2.

Task 3.1 Literature survey

As a first step, in this Task 3.1, a literature survey of current and proposed design rules in codes and in the literature for cross-sectional resistance is presented in combination with a survey on available experimental and numerical test results related to cross-sectional resistance. More, and more detailed information on 'Task 3.1 Literature survey' is available in Deliverable D3.1.

The **design rules** covering cross-sectional resistance are largely based on mechanics making use of stress distributions associated with elastic or plastic theory. This approach is regarded as the basic approach to develop design rules for cross-sectional resistance. In the case of single internal forces acting on the cross-section this approach is reliable. However, the interaction of multiple internal forces working on a cross-section might be less straight forward. Moreover, these design rules are often not validated by experimental results. Especially members loaded with multiple internal forces and prone to failure driven by plasticity, have increased chance to react differently than predicted by theoretical reasoning and additional research regarding these design rules is necessary.

The scope of the cross-sectional design rules considered is limited to clauses 6.2.3 to 6.2.10 of EN 1993-1-1, excluding clause 6.2.7. Moreover, the survey is aimed at plastic failure modes of the cross-section without interference of stability issues, therefore local buckling is not regarded. This survey only displays results of sections that reached complete yielding of the cross-section.

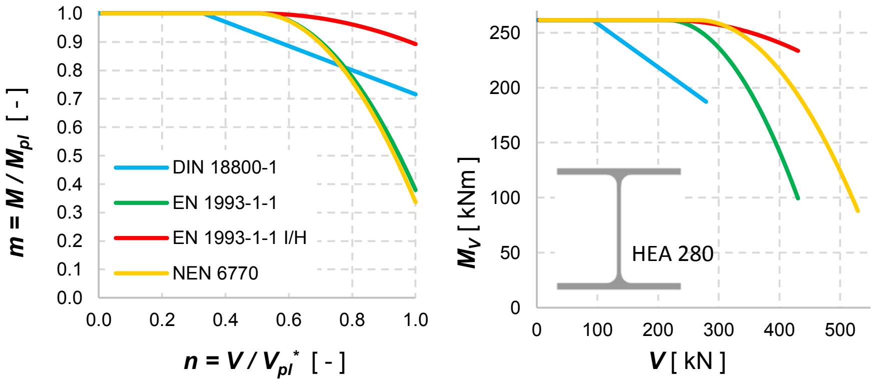


Figure 5.1 $M-V$ interaction design rules of EN 1993-1-1, NEN 6770 and DIN 18800-1

Design rules for **moment-shear (M-V) interaction of I-shaped sections** are mostly neglected in numerical studies and no adaptations of the design rules are presented yet. Investigations on the validity of the present design rules are necessary and might lead to more accurate alternative design rules. An overview of available design rules is shown in Figure 5.1. These design rules make use of different shear areas, Figure 5.2. The design rule of EN 1993-1-1 is under debate since two possible interpretations lead to two different outcomes as can be seen in Figure 5.1. Moreover, in one and the same design rule of EN 1993-1-1, apart from the shear area A_v (Figure 5.2 (middle, left)), also the web area A_w is used, which is confusing.

Figure 5.2 Definition of web and strong axis shear area in rolled I-shaped sections, from left to right: A_w EN 1993-1-1, A_V EN 1993-1-1, A_V DIN 18800, A_V NEN 6770

For **net cross-section** only one design rule is available, namely that of EN 1993-1-1:

$$N_{u,Rd} = \frac{0.9 A_{net} f_u}{\gamma_{M2}} \tag{5.1}$$

For staggered holes:

$$A_{net} = A = t\left(nd_0 - \sum \frac{p_1^2}{4p_2} \right) \tag{5.2}$$

The design rule of Eq. $A_{net} = A = t\left(nd_0 - \sum \frac{p_1^2}{4p_2} \right)$(5.2) is based on the theory of Cochrane (1922). The design rule was later checked by Teh and Clements (2012) for cold rolled sheets, which are very slender plates and therefore not comparable to the plates used in bolted structural connections.

In the past the design rules for **moment-normal force (M-N) interaction** received large interest, mainly because of the relation with the instability phenomenon buckling. This larger interest is still present when regarding the recent reassessments of cross-sectional design rules. New or modified design rules are presented, mainly based on research performed in Germany and France, Figure 5.3.

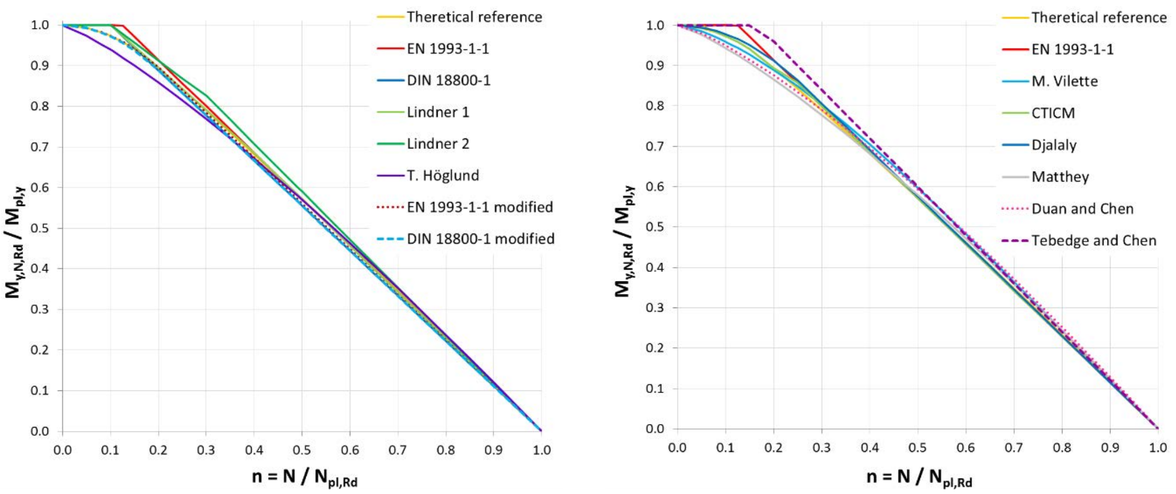


Figure 5.3 Comparison of design rules concerning $M_y - N$ interaction

Most of these design rules are purely based on mechanical behavior, without calibration of the FEM model by experimental tests. Although no calibration is present, the newly developed design rules present a safer and more precise approach of the actual interaction behavior compared to the current cross-sectional design rules in EN 1993-1-1, chapter 6.

Regarding **experimental and numerical test results**, tests describing the single load cases are not elaborately presented in the background documents of EN 1993-1-1 chapter 6. However, if just one internal force is present the theoretical approach based on mechanics is likely to be satisfying. Regarding the load case of axial compression only one result was un-conservative. In the case of bending moment similar results were found: only one of nine results was un-conservative.

The tests on **moment-shear (M-V) interaction of I-shaped sections** resulted in many cases of plastic material behavior. Although many tests qualified, in every test the ultimate moment exceeded the plastic moment. Thereby, these results suggest that the interaction of moment with shear forces does not reduce the moment resistance. The steel grades have increased significantly and therefore new tests are required. On top of that the moment-shear interaction rules described in EN 1993-1-1 are still not complete. Recently the definition of A_v for weak-axis moment-shear interaction was added, though the design rules for weak-axis moment-shear interaction are not yet available.

For **net cross-section,** Epstein and Gulia (1993) performed Finite Element Analyses (FEA) on multiple bolt hole connections in tension to compare the results with the code requirements. The FEA displayed decreasing resistances when transverse distances between bolt holes or transverse edge distances in a simple non-staggered connection became small. The Eurocode does not predict any change in resistance for this. A recent study was performed by Moze, Beg and Lopatic (2007) concerning the net cross-section resistance in High Strength Steel (HSS). Analysis of all specimens which failed on the net cross-section resulted in the conclusion that the design rule for failure of the net cross-section prescribed by the Eurocode was conservative for HSS.

The tests on **moment-normal force (M-N) interaction** resulted in only 7 cases of plastic material behavior relevant to cross-sectional resistance. All these tests had a small normal force compared to the ultimate cross-section resistance, but their moment exceeded the plastic moment.

Task 3.2 Experimental reference tests

Task 3.2 concerns carrying out experimental reference tests. These tests are used in Task 3.3 to validate a finite element model. Full scale experimental reference tests were done for the following failure modes:

- moment-shear (M-V) interaction of I-shaped sections;
- net cross-section;
- moment-normal force (M-N) interaction of I-shaped sections.

Associated to the full scale reference tests, relevant associated standard tensile coupon tests were carried out to determine the material properties (stress-strain diagrams). This information is necessary for validating a finite element model in Task 3.3. Also, for all tests carried out, the actual geometrical dimensions of the specimens were measured for the same purpose. Below a brief overview of the tests carried out is given. More, and more detailed information on 'Task 3.2 Experimental reference tests' is available in Deliverable D3.2.

In case of **moment-shear (M-V) interaction of I-shaped sections** a total number of 28, both strong and weak axis bending tests, were executed over a variety of I-shaped cross-sections used in common building practice (HEA280, IPE360, HEB240 and HEM180). 3-Point bending tests are used for the strong axis tests and the bending dominated weak axis tests, while for the shear dominated tests 5-point bending tests were required to be able to get sufficient shear force in the specimen. The 3-point bending test set-up for bending about the strong axis is shown in Figure 5.4. Measured strains, deflections and angles are also indicated. The measurement set-up varied between simple and extensive for both strong and weak axis tests.

A typical load-displacement diagram for a shear dominated specimen HEA280 in S235 bent about the strong axis with a high shear utilization ratio of 0.83 is shown in Figure 5.5. Also detailed tensile coupon tests were carried out, making it possible to determine the yield stress distribution over the cross-section, Figure 5.6 (left). The flange tips showed increased yield stresses compared to the rest of the flange. In general, the yield stress of the web was higher, especially in the roots, while the lowest yield stress was observed in the flange at the web. For several rolled cross-sections, the material properties were defined in compression at locations corresponding with the tensile coupons, resulting in minor differences between tension and compression coupons. Stub column tests were executed on the HEA240 section in S235 and half the HEA280 section in both S235 and S355. The overall section yield stress was slightly higher than the weighted average based on the individual tensile test coupons across the cross-section.

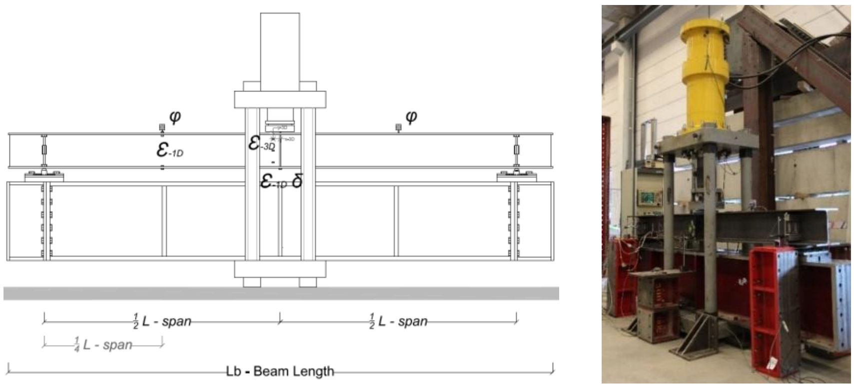


Figure 5.4 3-Point bending test set-up for bending about the strong axis

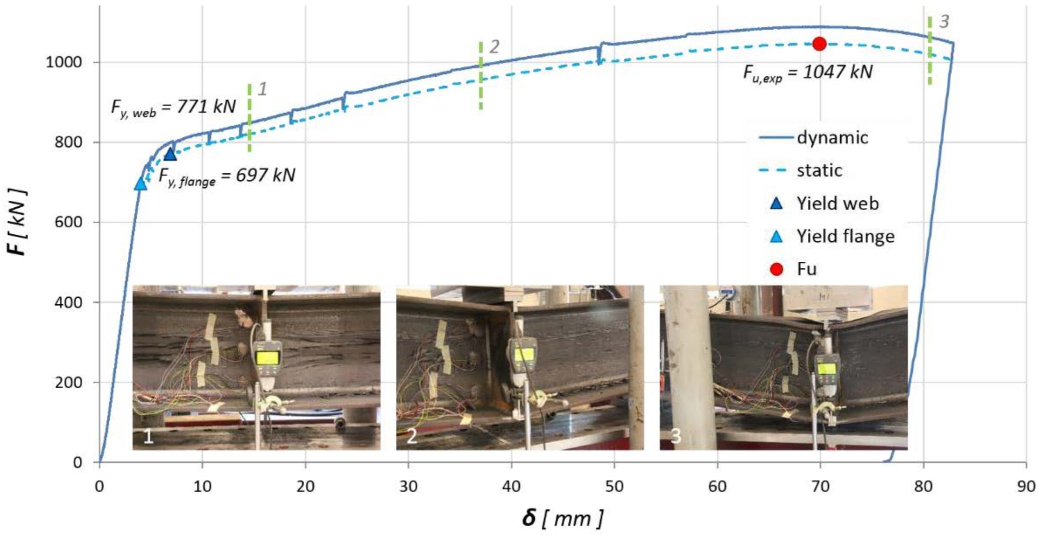


Figure 5.5 Load-displacement diagram test A4a: HEA280 in S235, strong axis with $n_V = 0.83$

The experimental results of HEM180 section are very conservative when compared to the theoretical moment-shear interaction values according to the design rule, while the results of IPE360 sections are un-conservative for the tested shear dominated beams (Figure 5.6 (right)). The test results were compared with the design rules. The plastic shear resistance was not reached in all sections. Therefore, the current EC3 shear area seems to be too large.

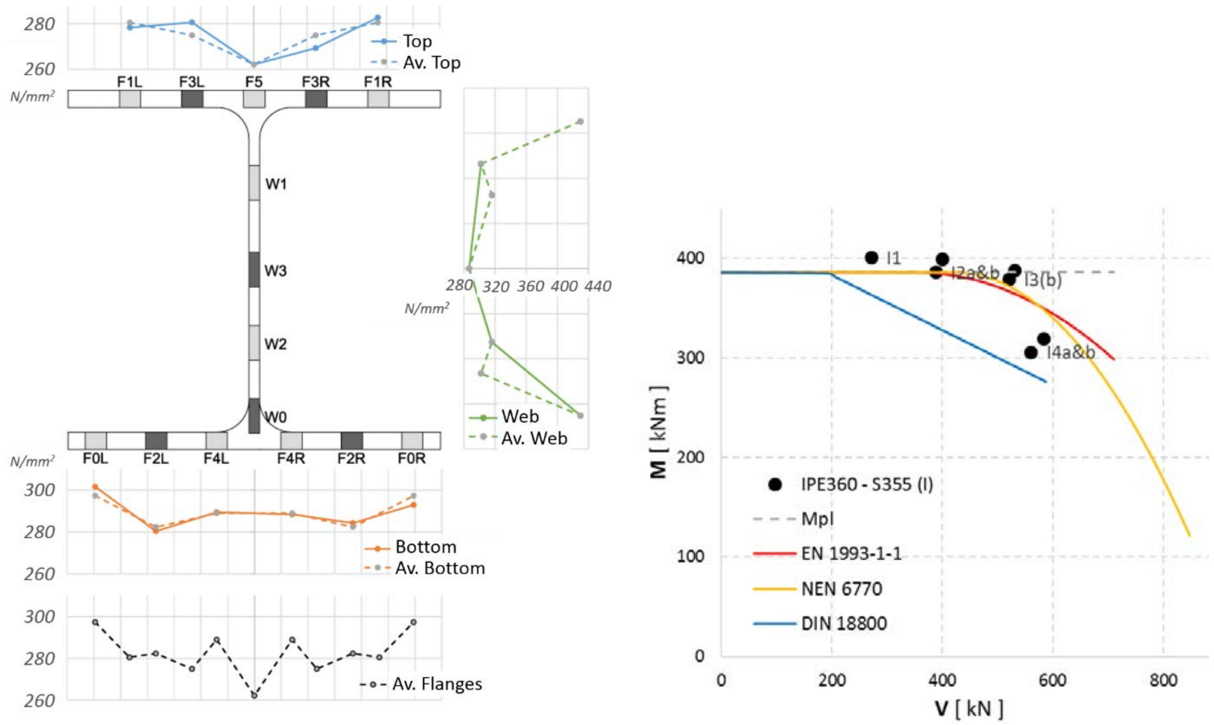


Figure 5.6 Yield stress distribution over HEA280 section in S235 (left), and experimental results of IPE360 (S355) in $M - V$ interaction diagram with design rules (right)

For the **net cross-section** failure mode, 120 mm wide plates of 8 mm thickness in S235 were tested with and without bolts (M16 8.8) and with different number of bolt holes (1 and 2) and different configurations (staggered and non-staggered), Figure 5.7 (left). Typical test results for similar configurations with bolts (B) and without bolts (A) are shown in Figure 5.8 (right).

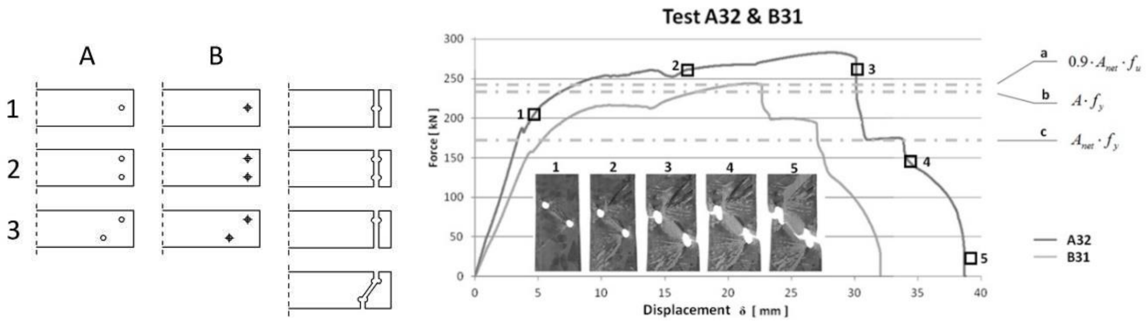


Figure 5.7 Configurations considered for net cross-section tests (left), and typical load-displacement diagrams for staggered configurations with (B) and without (A) bolts (right)

Again the material properties, e.g. Young's modulus E, yield stress f_y and ultimate stress f_u, were determined by tensile coupon tests. The experiments resulted in load-displacement diagrams (Figure 5.8 (right)) with a distinct maximum value and a description of the course of the experiment. All test results exceeded the ultimate resistance according to EN 1993-1-1. The Eurocode correctly does not make a distinction between the connections with and without bolts.

Finally, for **moment-normal force (M-N) interaction of I-shaped sections**, 10 tests were performed on HEA240 (S235) at several utilization ratios for normal force. The test set-up is shown in Figure 5.8 (left). A typical load-displacement diagram is shown in Figure 5.8 (right). Again, detailed tensile coupon tests were carried out to determine the yield stress distribution over the cross-section.

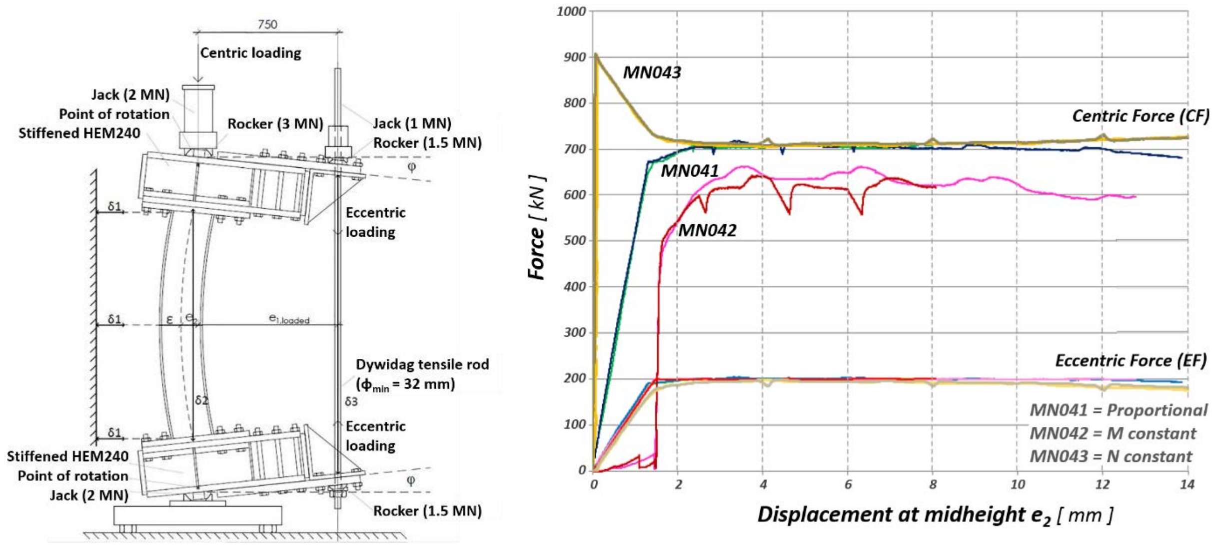


Figure 5.8 M-N interaction: test setup (left), and test results for HEA240, S235, normal force utilization ratio 0.4 (right)

Task 3.3 FE modelling

Numerical models were made in the Finite Element software Abaqus and validated by the experimental test programs. A summary is given below and more, and more detailed information on 'Task 3.3 FE modelling' is available in Deliverable D3.3. In all cases the FE model resembled the experimental test set-up and measured geometry and material properties were used as input.

In case of **moment-shear (M-V) interaction of I-shaped sections** a GMNIA of 3-point bending tests is performed using the Riks arc length method. This resulted in overestimations and some underestimations of the experimental results by maximum 5% in most cases, see Figure 5.9 (centre). The imperfections were added to introduce a perturbation triggering failure of the beam in bending-shear interaction when large parts of the beam were yielding. The required imperfection varied considerably in size: for bending dominated cases up to 0.002b while for most shear dominated cases almost no imperfection was required.

The influence of strain hardening was tested in the numerical models by comparing simulations with bilinear and strain hardening material properties. Like the experiments suggested, strain hardening positively influences the cross-sectional resistance to bending-shear interaction. In bending dominated cases the benefits are minimal. However, in shear dominated cases with compact sections like HEM180, the benefits could increase up to 37%. GMNIA including the strain hardening models of EN 1993-1-5, BSK 99 and NEN 6700 (Figure 5.9 (left)) are compared with GMNIA using stress-strain diagrams measured by tensile coupons. Figure 5.9 (centre) shows the results of an HEM180 section with a span of 1020 mm and imperfection equal to 0.00005*b* to introduce a perturbation. In shear dominated cases the EN 1993-1-5 and the NEN 6700 strain hardening models performed well and similar results were obtained, where the BSK 99 model always resulted in an overestimation. The EN 1993-1-5 strain hardening model was chosen for further simulations, since the yielding plateau length in the NEN 6700 model increases with an increase of yield strength, which is not in agreement with observed material behavior.

The set-up of the numerical model used in the parametric study deviates slightly from the validated model and the simulations within the parametric study are based on numerical results from GMNA. In order to assess the entire scope of the bending-shear interaction design rules, HEA100, IPE100, HEM100, HEA 600, IPE600 and HEM600 sections are regarded in steel grades S235, S355 and S460. In the numerical simulations different beam lengths were used in order to invoke different shear utilization ratios, leading to 180 simulations. Since the emphasis is on shear dominated beams, mainly short beams are of interest, resulting in 55 simulations.

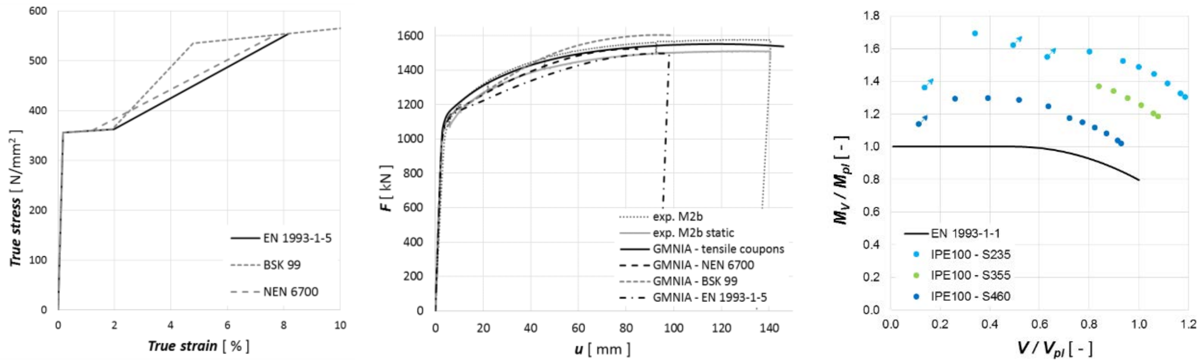


Figure 5.9 Strain hardening models for S355 (left); comparison of strain hardening models for an HEM180 section in S355 (centre); bending-shear interaction for IPE 100 in multiple steel grades (right)

Figure 5.9 (right) displays simulations of IPE100 sections in steel grades S235, S355 and S460. The ratios of the ultimate resistances are not in line with the ratio of the yield strength of the materials. However, the influence of strain hardening – decrease of the f_y/f_u-ratio for higher steel grades – also results in lower relative resistances. Therefore, the numerical results cannot be scaled for the yield stress level to enlarge the database.

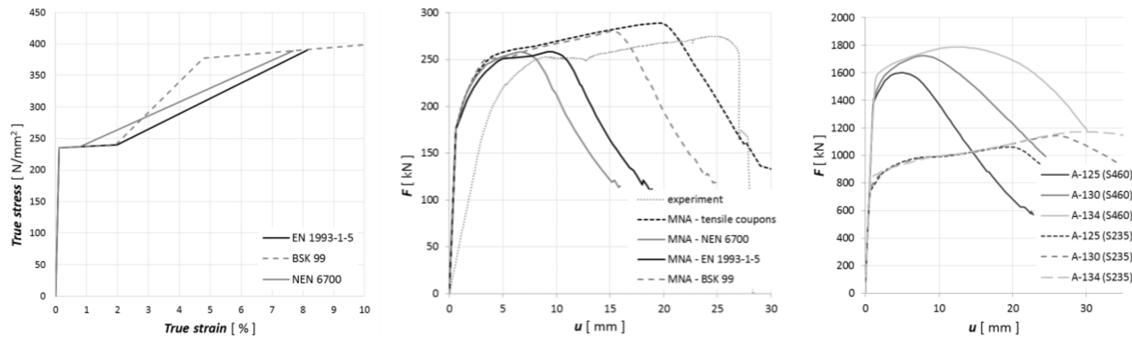


Figure 5.10 Strain hardening models for S235 (left); Comparison of these models and experiments for specimen A25 (plate with 2 aligned bolt holes) (centre); numerical simulations in S235 & S460 (right)

For the **net cross-section** failure mode, a MNA is performed using the Newton-Raphson iteration technique. Continuum linear hexahedral C3D8i elements were used. Typical results are shown in Figure 5.10 (centre) for specimen A25 without bolts. The difference in stiffness is caused by measuring displacements in the experiments over the grips, instead of directly on the specimen.

Strain hardening models are of large importance for the parametric study, therefore the EN 1993-1-5, BSK 99 and NEN 6700 models were compared Figure 5.2 (left). The best result, though still on the conservative side, is obtained with the strain hardening model of BSK 99. However, it was chosen to continue with the strain hardening model of EN 1993-1-5 to be consistent with Eurocode 3, which is ca. 11% on the safe side.

The database consists of 347 simulations in S235 and 40 in S460, with varying plate width *b*, plate thickness *t*, hole diameter d_0, number of bolt holes, pitch *p*, and end distance *e*.

Despite the very different load-displacement diagrams in Figure 5.10 (right), the ratios of the ultimate resistances for S460 and S235 are in line with the ratio of the tensile strength of the materials: $f_{u,S460}/f_{u,S235}$ = 540/360 = 1.5. Therefore, it may theoretically be possible to scale the results for S235 for other steel grades to enlarge the number of simulations in the database.

For **moment-normal force (M-N) interaction of I-shaped sections** a GMNA using the Riks arc length method is performed. Continuum linear hexahedral C3D8R elements were used. The numerical model was validated by means of the experimental test results. A good accuracy was obtained with maximum deviation of 2.8% between static experimental failure load and numerical failure load based on the stub column material model (bi-linear stress-strain curve of the experimental stub column test results), see Figure 5.11 (left and center).

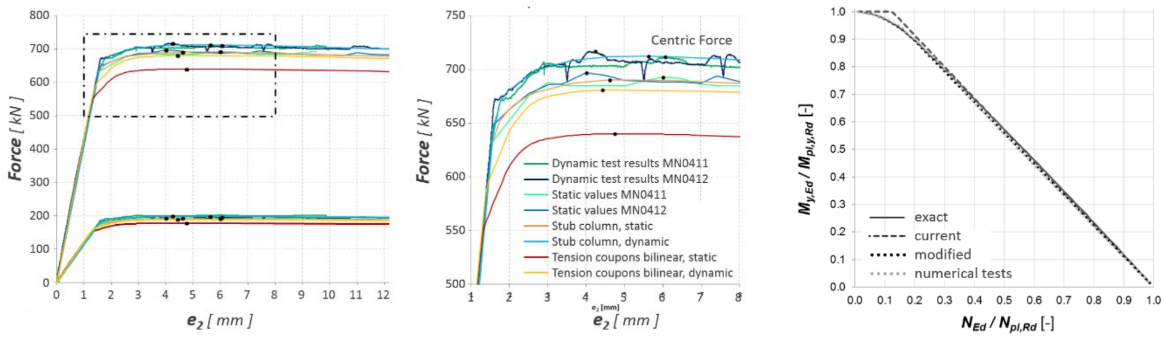


Figure 5.11 Validation of numerical model for HEA240 with $n_N = 0.4$ (left and centre); comparison of the current and the modified design rule with the exact solution for strong axis M-N interaction for HEA240 (right)

The parametric study consisted of 1188 numerical results. For the I-shaped cross-sections HEA240, HEB200, HEM400 and IPE330, simulations are performed with varying utilization ratio n_N with Δn = 0.01, which means 99 numerical test results per section, Figure 5.11 (right) displays these results for HEA240. Finally, different steel grades are regarded, namely steel grade S235, S355 and S460. Rescaling based on yield strength was shown to be accurate. Therefore, all simulations were performed in S235, and the results were scaled for S355 and S460. New simulations were not performed for these steel grades.

In the case of HEA240, HEB200 and IPE330 sections, the numerical test results are similar as the exact solution for small values of n_N, while the EN 1993-1-1 design rule gives an un-conservative prediction. For utilization ratios between roughly 0.4 and 0.8, the numerical test results are more in line with those according to the EN 1993-1-1 design rule. In the case of the HEM400 section a large deviation from the exact solution is observed, particularly for $n_N < 0.5$.

The used beam length limited the cross-sectional resistance. With the used length, only the HEM400 section was able to reach the state of strain hardening because of its stocky flanges. In case of shorter beams, strain hardening does provide in additional strength. However within the relative slenderness range $\lambda_{rel} < 0.2$, where buckling is disregarded, strain hardening is not always beneficial.

Finally, for **moment-normal force (M-N) interaction of rectangular hollow sections (RHS)** a GMNA using the Riks arc length method is performed. Continuum quadratic hexahedral C3D20R elements were used. The model could not be validated by experimental tests (not available). However, the plastic moment resistance M_{pl} and plastic normal force N_{pl} were accurately described by the model.

A length study was performed in order to define the minimum length of the specimens. An adequate stress distribution was already obtained at a beam length L of 1.5 times the section height h, 300 mm in Figure 5.12.

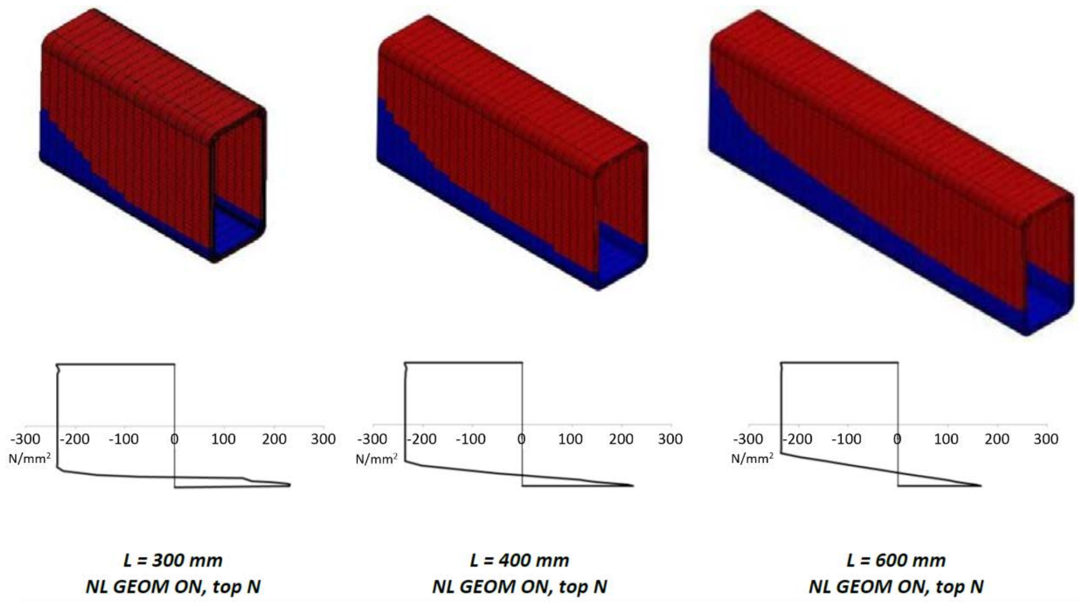


Figure 5.12 Stress distributions and yielding for different lengths (red = yield)

The parametric study consisted of 1584 numerical results. For the RHS 200/100/10, HF RHS 150/100/12, SHS 150/150/12.5, RHS 160/80/8, and RHS 400/200/16 sections, simulations are performed with varying utilization ratio n_N with $\Delta n = 0.01$, which means 99 numerical test results per section, Figure 5.13 displays these results for RHS 200/100/10. Finally, different steel grades are regarded, namely steel grade S235, S355 and S460. Scaling the numerical result for the yield stress level was possible. However, still all simulations were executed.

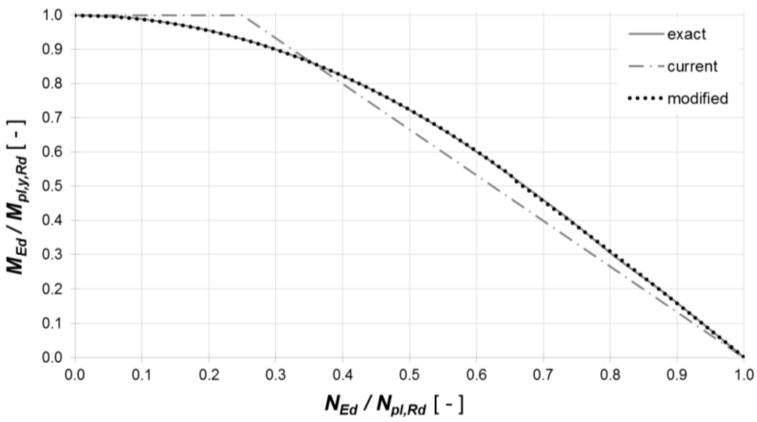


Figure 5.13 Comparison of current and modified design rule with exact solution (strong axis M-N) RHS 200/100/10 (right)

The effect of strain hardening was investigated for various sections with a beam length of $1.5\,h$. An increased bending moment resistance was found for steel grades S235 to S460, see Figure 5.14 (left). However, this increased cross-sectional resistance is not present when the relative slenderness is increased up to $\lambda_{rel} = 0.2$, as in Figure 5.14 (right), which is still in the range where the effects of local buckling may be neglected.

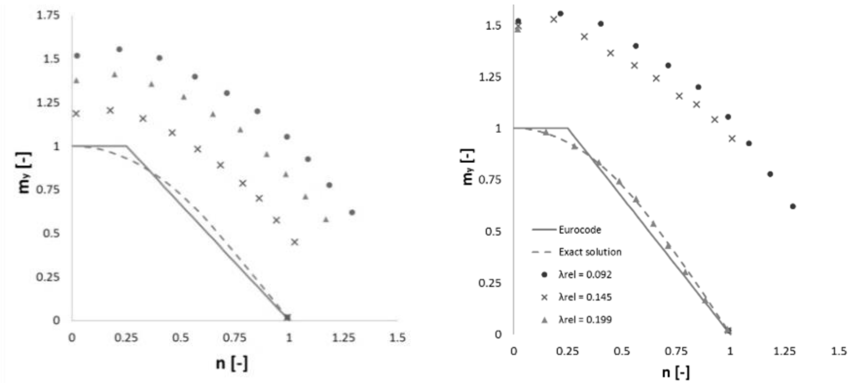


Figure 5.14. Influence of strain hardening for RHS 200/100/10: in multiple steel grades with $\lambda_{rel} = 0.092$ (left); and in S235 with multiple λ_{rel} (right)

Task 3.4 Statistical evaluation

The previously described databases of numerical test results were used to perform statistical assessments of the design rules as given in EN 1993-1-1, using the procedure of Annex D of EN 1990 as further developed in this project in WP1. For the statistical assessment procedure itself, the reader is referred to (Taras et al., 2014 and Simões da Silva et al., 2017) and of course to WP1. Below, a summary of results is presented. More, and more detailed information on 'Task 3.4 Statistical evaluation' is available in Deliverable D3.4.

In case of **moment-shear (M-V) interaction of I-shaped sections** 180 numerical test results were used to perform a statistical assessment. The theoretical resistance r_t (red line in Figure 5.1) is compared with the numerical resistance r_e originating from the database. The theoretical resistance did not comply with the numerical results.

The geometry is defined by the width b, height h, flange thickness t_f, web thickness t_w, and root radius r of the I-shaped section. These parameters are not completely independent, due to standardized section typology. In addition to these geometric parameters, the yield stress f_y is of influence. For the statistical assessment, the distributions of these parameters are required, since the database contains GMNA analyses with nominal values. For values of the used statistical distributions the reader is referred to Deliverable D3.4.

First the entire population is assessed, Table 5.1 presents the results of the statistical evaluation for the different steel grades in the second to fourth column. As expected, the EN 1993-1-1 design rule does not comply with the results from the numerical simulations when the different steel grades are regarded. This is expressed by high values for γ^*_M. The acceptance diagram in Figure 5.15 (left) gives all subsets by means of $\gamma^*_M/\gamma_{M,target}$ to the variation V_r. The dashed line represents the acceptance limit, markers below this line are accepted, markers above need a higher value for $\gamma_{M,target}$. Only for IPE100, HEM100 and 5 other subsets $\gamma_{M,target} = 1.0$ suffices, Figure 5.15 (left). Overall $\gamma_{M,target} = 1.45$ is required.

Table 5.1 Assessment results for $M - V$ interaction in 3 point bending tests

Set	*all simulations*			*simulations $M < M_{pl}$*		
	#	V_r	γ^*_M	#	V_r	γ^*_M
S235	68	0.2571	1.703	16	0.1295	1.333
S355	49	0.1964	1.647	19	0.1180	1.323
S460	63	0.1428	1.355	20	0.1174	1.337
S235-S460	180	0.2224	1.666	55	0.1207	1.307

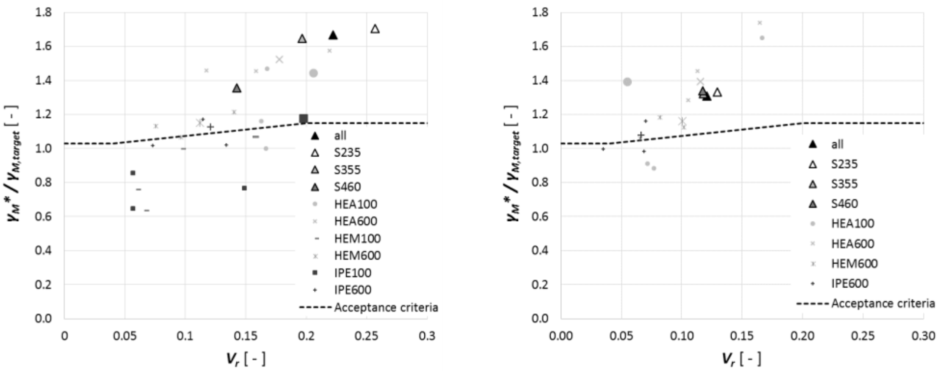


Figure 5.15 Acceptance diagram for partial factor of all simulations (left), and limited group of simulations with $M < M_{pl}$ (right)

Alternatively, only test results with a bending moment resistance lower than M_{pl} are regarded, resulting in a set of 55 numerical simulations. A large number of simulations resulted in higher resistance to bending moment due to strain hardening. If these results are left out, e.g. the complete set of IPE100 and HEM100 sections was excluded. The last three columns of Table 5.1 present the assessment results. When only simulations with low bending resistance are regarded, the scatter decreases, and a few more subsets are acceptable with $\gamma_{M,target} = 1.0$ even though these sets are punished for having a small number of results. For this case $\gamma_{M,target} = 1.2$ is required overall. In general it is concluded that the EN 1993-1-1 design rule for bending moment-shear interaction is not adequate, and a new design rule is required.

In case of **net cross-section** failure 387 numerical test results were used to perform a statistical assessment. The theoretical resistance r_t of a configuration of the database is obtained from Eq. (8.1), when setting $\gamma_{M2} = 1.0$.

Four independent variables determine the net cross-section resistance: plate width b, plate thickness t, hole diameter d_0 and tensile strength f_u. For values of the statistical distributions used in the statistical assessment, the reader is referred to (Snijder et al., 2017). The results for equation (8.1) are shown in Table 5.2, columns 3 and 4 for plates with and without bolts.

Table 5.2 Assessment results for plates with and without bolts for the net cross-section design rule

Considered set		Eq. (8.1)		Eq. (8.1) / 0.9		# tests
d_0	steel grade	V_r	γ^*_{M2}	V_r	γ_{M2}	
18, 22, 26	S235	0.072	1.05	0.072	1.17	347
18,22,26 26	S235 S460	0.075	1.05	0.075	1.17	387

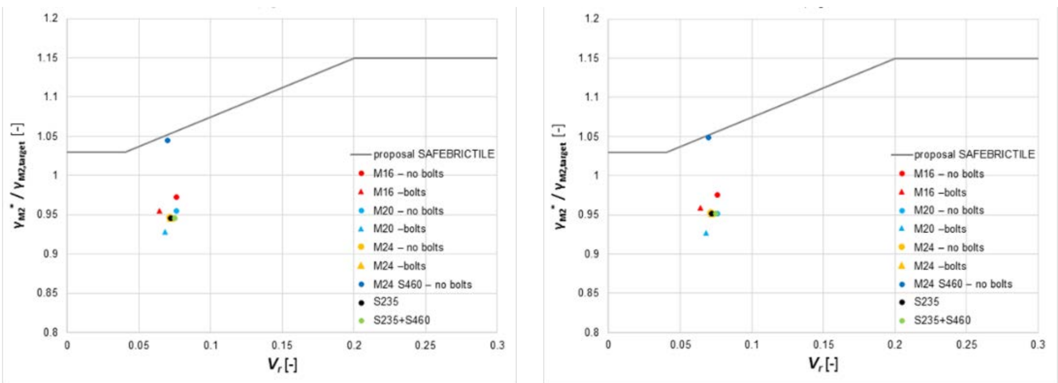


Figure 5.16 Acceptance diagram for partial factor: net-section design rule of *Eq. (8.1)* with $\gamma_{M2,target} = 1.11$ (left); modified net-section design rule (*Eq. (8.1)*/0.9) with $\gamma_{M2,target} = 1.23$ (right)

In Table 5.2, the partial factor is $\gamma^*_{M2} = 1.05$ as an overall value for all considered results. However, it is interesting to also consider subsets, this is done in the acceptance diagram (Simões da Silva et al., 2017) of Figure 5.16. If the target value for the partial factor is chosen to be $\gamma_{M2,target} = 1.11$, then all subsets show acceptable results (Figure 5.16 (left)).

The current value of the partial factor being $\gamma_{M2} = 1.25$, it should be possible to optimize the design rule, by e.g. omitting the factor 0.9 from Eq. (8.1) as suggested by Može and Beg (2014). If the statistical assessment procedure is now repeated for this modified design rule, the results of Table 5.2 columns 5 and 6 are obtained. So overall $\gamma_{M2} = 1.17$ is acceptable. If the target value for the partial factor is taken as $\gamma_{M2,target} = 1.23$, Figure 5.16 (right), all subsets fulfill the acceptance criterion.

For **moment-normal force (M-N) interaction of I-shaped sections** 1188 numerical test results were used to perform a statistical assessment. The theoretical resistance r_t of a configuration of the database is obtained from the design rule in EN 1993-1-1.

The parameters as previously described for $M - V$ interaction are used in the statistical assessment. This assessment resulted in a target value of the partial factor for the entire set of $\gamma_M = 1.01$. This is slightly un-conservative. For steel grade S460, γ^*_M is slightly greater than 1.0. Table 5.3 presents the outcomes in column 2 and 3. The recommended value for the partial factor used in the EN 1993-1-1 design rules for $M - N$ interaction for I-shaped sections is acceptable for all steel grades, as shown in Figure 5.17 (left). However, for some subsets in S460 this is not the case.

Table 5.3 Assessment results for $M - N$ interaction for I-shaped cross-sections

set	*EN 1993-1-1*		*Rombouts*		# tests
	V_r	γ^*_M	V_r	γ^*_M	
S235	0.0648	0.960	0.0643	0.953	396
S355	0.0610	0.988	0.0601	0.979	396
S460	0.0573	1.019	0.0560	1.009	396

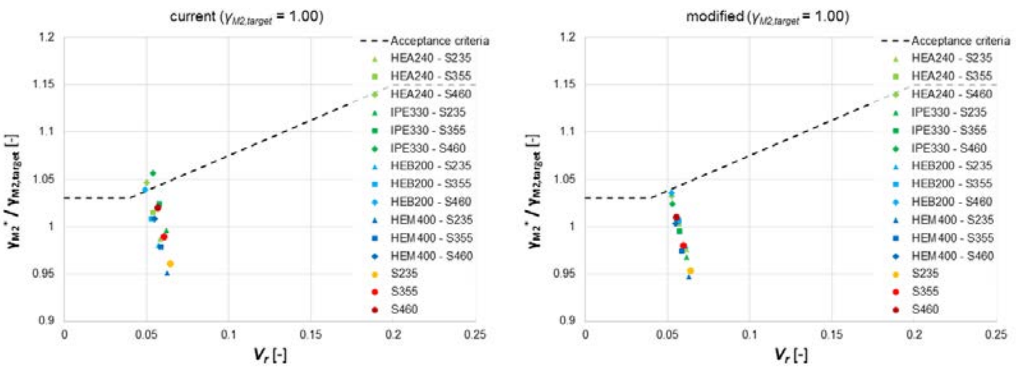


Figure 5.17 Acceptance diagram for partial factor ($\gamma_{M,target} = 1.0$): $M - N$ interaction design rule for I-shaped sections following EN 1993-1-1 (left); proposal Rombouts (2016) (right)

A modified design rule was proposed by Rombouts (2016). This design rule is more accurate, which resulted in lower values for γ^*_M (Table 8.3, column 5). With $\gamma_{M,target} = 1.0$ every subset gives acceptable results, see Figure 5.17 (right) in combination with column 4 and 5 of Table 5.3. In the case of the entire population being considered, the acceptance criterion is fulfilled for $\gamma_{M,target} = 1.0$.

Finally, for **moment-normal force (M-N) interaction of rectangular hollow sections (RHS)** 1485 numerical test results were used to perform a statistical assessment. The theoretical resistance r_t of a configuration of the database is obtained from the design rule in EN 1993-1-1.

Four partly dependent variables determine the resistance to $M - N$ interaction of RHS and SHS: width b, height h, thickness t, and root radius r. In addition the yield stress f_y is of influence. Table 5.4 (left) presents the used statistical distributions, and (right) the results of the assessment.

Table 5.4 Geometrical distributions (left), assessment for $M-N$ interaction for RHS and SHS (right)

parameter	b	h	t
mean / nom.	0.984	1	1
V [%]	4.24	0.39	0.44

set	*EN 1993-1-1*		*Stroetmann,*		# tests
	V_r	γ^*_M	V_r	γ^*_M	
S235	0.100	1.065	0.064	0.979	495
S355	0.099	1.104	0.061	1.006	495
S460	0.099	1.148	0.058	1.039	495
All	0.099	1.103	0.061	1.007	1485

The target values of the partial factor for the entire set is $\gamma_{M,2} = 1.03$. This is on the un-conservative side and for steel grade S460 an even higher value ($\gamma_{M,target} = 1.07$) is required. The recommended value for the partial factor used in the EN 1993-1-1 design rules for $M-N$ interaction for RHS and SHS is acceptable for steel grade S235 only, as shown in Figure 5.18 (left).

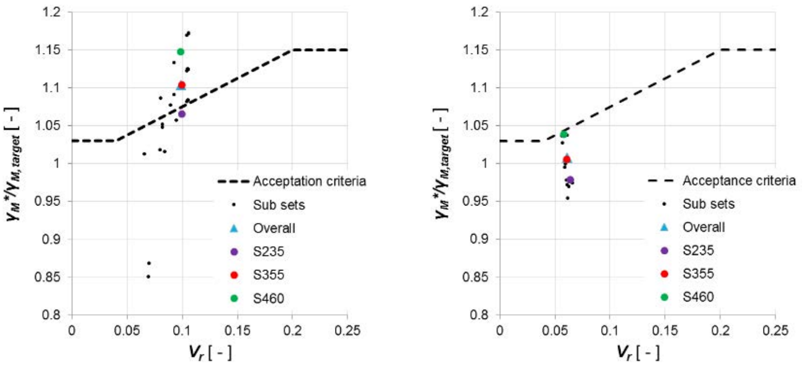


Figure 5.18 Acceptance diagram for partial factor ($\gamma_{M,target} = 1.0$): M-N interaction design rule for RHS and SHS following EN 1993-1-1 (left); proposal Stroetmann and Kindmann (right)

A modified design rule was proposed by Stroetmann and Kindmann (2013). This design rule is more accurate, which resulted in a lower $\gamma_{M,target}$. For the entire set $\gamma_{M,target} = 0.96$ suffices, see Figure 5.18 (right). All subsets fulfil the acceptance criteria for $\gamma_{M,target} = 1.0$.

Task 3.5 Recommendations on statistical evaluation of ductile failure modes and cross-sectional resistance

The statistical assessment procedure developed in WP1 was adequate for failure modes driven by plasticity. A brief summary of the recommendations is presented below. More detailed information on 'Task 3.5 Recommendations on statistical evaluation of ductile failure modes and cross-sectional resistance' is available in Deliverable D3.5.

In case of **moment-shear (M-V) interaction of I-shaped sections** the current EC3 design rule is inadequate. The shear area is overestimated in many cases and a new design rule for M-V interaction is needed.
For **net cross-section** failure, the research shows that the reduction factor 0.9 in the current design rule can be omitted making the design rule less conservative. It is advised to keep the partial factor equal to 1.25, even though a lower value is permitted based on the statistical assessment. Still some additional margin is recommended to take untimely failure, due to imperfections around the bolt hole, into account. Alternatively to omitting the factor 0.9, the partial factor can be relieved.
For **moment-normal force interaction of I-shaped sections** the current design rule was shown to be slightly inaccurate. The partial factor should be increased to 1.01 for S460. Alternatively the design rule proposed by Rombouts (2016) may be used, which is more accurate and therefore results in a partial factor of 1.0 being acceptable.
For **moment-normal force interaction of rectangular hollow sections (RHS)** the current design rule was shown only to be accurate for S235. In case of S355 and S460 an increased partial factor up to 1.07 should be used. A modified design rule was proposed by Kindmann and Stroetmann (2013) which describes the moment-normal force interaction diagram much better. The use of this design rule in combination with the original partial factor (1.0) results in adequate safety.

5.3 *Conclusions and future work*

Four different failure modes were extensively considered in WP3:

- moment-shear (M-V) interaction of I-shaped sections;
- net cross-section;
- moment-normal force (M-N) interaction of I-shaped sections, and;
- moment-normal force (M-N) interaction of rectangular hollow sections (RHS).

The main conclusions can be summarised as follows:

1. The statistical assessment procedure developed in WP1 turned out to be easy to work with for failure modes driven by plasticity. Having acceptance criteria developed in WP1 for a certain target partial factor depending on the coefficient of variation was very helpful to establish an overall partial factor for a specific cross-sectional design rule.
2. Material properties and geometrical data as measured were added to the European Database of Steel Properties of WP2.
3. The yield stress varies considerably over the cross-section of I- and H-shaped cross-sections. Yield stress distributions over the cross-section were measured in detail.
4. Experiments revealed that all considered failure modes in deed were driven by plasticity, the yield stress being the governing material property.
5. The experiments, in combination with numerical analyses, showed that strain-hardening has a substantial contribution to cross-sectional resistance, in cases without normal forces or in short columns ($\lambda_{rel} < 0.15$). However, if normal forces are present and (local) buckling gets influence, the positive effect of strain-hardening disappears.
6. For every failure mode and cross-section type, the influence of strain hardening was investigated. In case of influence (M-V interaction, and net-section failure), the strain hardening model was used to generate 'numerical test results'.
7. Reliable well-documented test results (ultimate loads and load-displacement diagrams) were obtained for moment-shear (M-V) interaction of I-shaped sections, net cross-section and moment-normal force (M-N) interaction of I-shaped sections.
8. Finite element models could be validated against the test results with an accuracy of about 5% in terms of ultimate load.
9. The accuracy of the finite element model is included in the statistical analysis by the term $V_{\delta,num}$, which eventually is taken into account in the coefficient of variation V_r.
10. Extensive and representative databases with 'numerical test results' for the failure modes considered were created. This was done based on nominal material and geometrical properties. Variations in these properties were taken into account in the statistical assessments.
11. Existing and newly proposed design rules were validated against these databases to evaluate the partial factor belonging to the respective design rules.
12. For moment-shear interaction of I-shaped sections, it was shown that the current design rule of EN 1993-1-1 is inadequate. The formula for the shear area needs to be adapted and a new design rule for moment-shear interaction is required.
13. The behaviour of I-shape sections under moment-shear interaction turned out to be complex. A workable definition of the shear area for all I-shaped sections is not obvious and therefore a new design rule requires more research.
14. For net cross-section, the research shows that the reduction factor 0.9 in the current design rule can be omitted making the design rule less conservative. Alternatively the partial factor can be relieved.
15. For moment-normal force interaction of I-shaped sections and rectangular hollow sections, a modified design rule was proposed, which better describes the moment-normal force interaction.
16. The newly proposed design rules for moment-normal force interaction of I-shaped sections and rectangular hollow sections are such that they have adequate safety with a partial factor of 1.0.

Future work consists of the following:

1. Almost the complete cl. 6.2 of EN 1993-1-1 concerning cross-sectional resistance has been covered by WP3. However, some design rules of cl. 6.2 of EN 1993-1-1 were not investigated and they can be considered in a similar way as the considered failure modes:
   - design rules for class 3 and class 4 cross-sectional resistance;
   - the capacity design rule of cl. 6.2.3(3);
   - shear area design rules for other cross-sections than I- and H- sections, cl. 6.2.6(3) of EN 1993-1-1;
   - design rules for torsion, cl. 6.2.7 of EC3-1-1;
   - design rule for combined bi-axial bending and normal force, cl. 6.2.9.1(6) of EC3-1-1;

- design rules for combined bending, shear and axial force, cl. 6.2.10 of EC3-1-1.

2. For moment-shear interaction of I-shaped sections a new design rule needs to be developed as well as a new definition of the shear area.
3. Apart from these design rules, there are also failure modes that are not covered at all by design rules in EN 1993-1-1. As an example it is mentioned that a design rule for bi-axial bending with shear in two directions is missing. Clause 6.2 of EN 1993-1-1 is also not clear about the combination of bi-axial bending, shear in two directions and normal force. Many design rules are available for I- and H-shaped cross-sections only, not for other cross-sections. As an example it is mentioned that a specific moment-shear interaction design rule, comparable to the one of cl. 6.2.8(5) for I-shaped section, is missing for CHS and it is not obvious if cl. 6.2.8 of EN 1993-1-1 applies at all to CHS.
4. Also, it should be mentioned that design rules for new cross-section types, like elliptical hollow section (EHS), are underway. Also these design rules can be investigated in a similar way as the failure modes considered in WP3.
5. To carry out a limited set of experiments for moment-normal force (M-N) interaction of rectangular hollow sections (RHS) for better validation of the numerical model.

6 Work package 4 – Modes driven by stability

6.1 *Objectives of WP4*

The objectives of this WP were:

- Application of the safety assessment procedure to the General Method in EC3-1-1 to a range of non-uniform isolated members and frames;
- Development of a mechanical generalized slenderness model for the any stability phenomena of non-uniform isolated members;
- Safety assessment of the existing and developed rules;
- Contribution towards the revision of EC3-1-1, by achieving transparent, simple and straight-forward unified stability checking procedures.

6.2 *Work undertaken and results obtained*

Introduction

Work package 4 aimed at achieving consistent level of safety throughout verification rules concerning failure modes focusing on stability aspects. Over the course of the project the following work was performed:

1.On the scope of Task 4.1, the validation of the General Method in EC3-1-1, clause 6.3.4 was performed for a wide range of non-uniform members. Similar validation for prismatic members had been carried out in previous studies from the research group (*Simões da Silva* et al, 2010). Additionally, safety assessment was carried out and the partial factor γ_{Rd} was determined.

2. Regarding Task 4.2, it was intended to extend the rules for prismatic members in EC3-1-1 for non-uniform members, it was of major importance to assess the safety of these in a preliminary step and proceed with a proposal of modifications in case this is necessary. This was done for clause 6.3.1 for flexural buckling columns, clause 6.3.2 for lateral-torsional buckling beams and clause 6.3.3 for members under bending and compression.

3.Subsequently, according to the scope of the specific tasks of Task 4.2, a verification procedure for any non-uniform member and consistent with rules for prismatic columns and beams is proposed.

4. Moreover, in Task 4.3, the interaction formula from Eurocode 3 clause 6.3.3, was extended to the verification of web-tapered members.

5. Finally, in Task 4.4, the verification formats were put into code conform guidelines.

Task 4.1 – Validation of the General Method in EC3-1-1 to a wide range of boundary and loading conditions and cross section shapes

In Task 4.1, the General Method (clause 6.3.4 of EC3-1-1) was validated for a wide range of non-uniform members and systems, with higher focus on web-tapered members subject to axial force and/or major axis buckling. The work carried out comprised of the following steps: i) Discussion of the theoretical background of this method; ii) a parametric study based on advanced numerical simulations using GMNIA (geometrically and materially non-linear analysis with imperfections) for a wide range of cross-section shapes and variation along length, buckling lengths, loadings, production processes, restraining conditions, and steel grades; iii) the safety of the General Method against the numerical simulations was assessed.

General Method (GM)

The general method, as given in EN 1993-1-1 in clause 6.3.4 (EN 1993-1-1, 2011) aims at verifying lateral and lateral-torsional buckling of structural components such as: (i) single members, built-up or not, with complex support conditions or not; or (ii) plane frames or sub-frames composed of such members which are subject to compression and/or mono-axial bending in the plane, but which do not contain plastic hinges. Application of the General Method is summarized in Figure 6.1, where $\alpha_{ult,k}$ is the minimum load amplifier of the design loads to reach the characteristic resistance of the most critical cross section of the structural component, considering its in-plane behaviour without taking lateral or lateral-torsional buckling into account however accounting for all effects due to in-plane geometrical deformation and imperfections, global and local, where relevant; χ_{op} is the reduction factor for the non-dimensional slenderness $\bar{\lambda}_{op}$, which should be obtained from either by considering: (i) the minimum value of χ (for lateral buckling, according to clause 6.3.1 of EC3-1-1) or χ_{LT} (for lateral-torsional buckling, according to clause 6.3.2); or (ii) an interpolated value between χ and χ_{LT} (determined as in (i)).

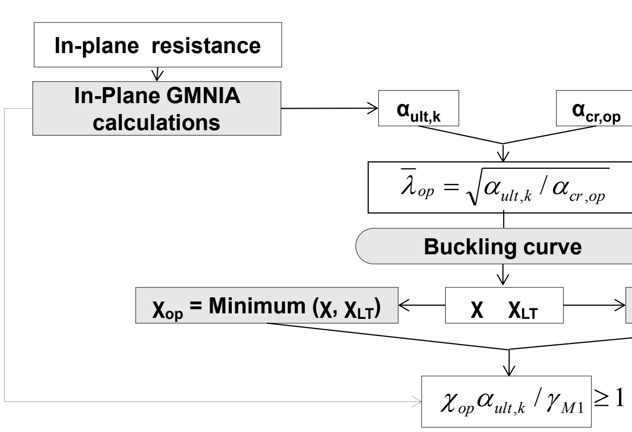
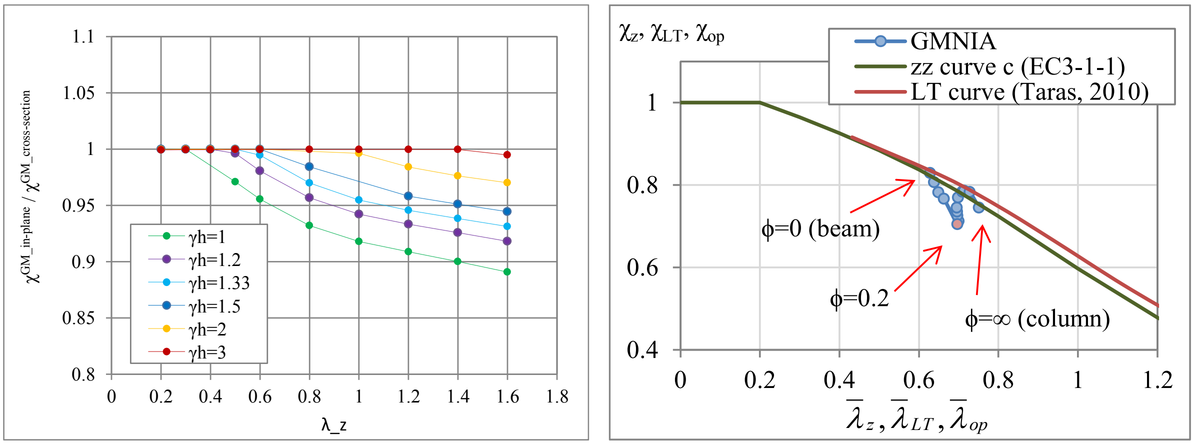


Figure 6.1. Application of the General Method to non-uniform members

Discussion

In previous assessments of the method for simply supported prismatic members (*Simões da Silva et al (2010)*) similar reliability levels as for application of the interaction formulae were found. However, when dealing with non-uniform members, several inconsistencies were noticed. This led to the need of re-evaluating the method and its analytical basis. There are a few aspects, which deserve special attention and are summarized in the following paragraphs. The General Method requires that the in-plane resistance of the member accounting for second order in-plane effects and imperfections is considered as an absolute upper bound of the member resistance ($\alpha_{ult,k}$). This assumption leads to conservative estimates of the ultimate resistance (up to 20% (*Simões da Silva et al*, 2010)). The same trend was reported, see e.g. *Ofner and Greiner* (2005) or *Taras* (2010), if the in-plane effects are of the same magnitude as the out-of-plane effects (for example, RHS sections). Similar conclusions were verified for tapered members in Task 4.1 of SAFEBRICTILE. Figure 6.2a illustrates the relationship between the resistance obtained by the GM when in-plane imperfections are considered or not, $\gamma_h=h_{max}/h_{min}$ is the relationship between the maximum h_{max} and minimum h_{min} height of the member;



a) resistance ratio given by the GM using 2nd order in-plane imperfections or CS resistance for $\alpha_{ult,k}$

b) Example of a member which χ_{op} is lower than both χ_z and χ_{LT}

Figure 6.2. The General method: issues

For the determination of the "global" reduction factor, a minimum or interpolated value between the reduction factors for flexural buckling χ_z and lateral-torsional buckling χ_{LT} should be calculated. ECCS TC8 (2006) recommends only the "minimum" reduction factor option, which is justified due to the fact that a lower estimate of the maximum capacity is set. On the other hand, GMNIA results plotted in a buckling curve representation may fall below the lowest of the column flexural buckling or lateral-torsional buckling curves, see Figure 6.2b. For prismatic members, it was seen in *Simões da Silva et al*. (2010) that the consideration of the minimum between the lateral-torsional reduction factor and the out-of-plane reduction factor results in a discontinuity in the *M-N* interaction curve. In addition, the consideration of the existing buckling curves a_0 to d to tapered beams or columns is incorrect as it may lead to a spread in the safety level, mainly because more than one buckling curve is applicable to the same tapered beam. Hence, an adequate interpolation was required.

For the above mentioned reasons, it was necessary to conduct a study to evaluate the safety of the General Method for non-uniform members and systems.

Parametric study and Methodology

The General Method was validated against a range of non-uniform members and frames, covering: symmetrically web tapered I-section beams, columns and beam-columns; web tapered members with only inferior half of web tapered; tapered flanges; linear and nonlinear tapering of web; members with non-symmetrical restraints along the cross-section; irregular intermediate restraints along the member; members with other support conditions (analysis of sway imperfections); a frame. The parametric study comprised about 2000 numerical results where slenderness, tapering ratio, bending moment distribution, were varied. Safety assessment of the given parametric study was performed. Results of the GM were analyzed against GMNIA results obtained with finite element software Abaqus (2010).

In order to have a common basis, the generalized reduction factors were compared: $\chi^{GMNIA}(x_c^I)=\alpha_b^{GMNIA}/\alpha_{ult,k}^{CS}(x_c^I)$ and $\chi^{GM}(x_c^I)=\alpha_b^{GM}/\alpha_{ult,k}^{CS}(x_c^I)$, in which α_b^x is the resistance multiplier obtained numerically (*x=GMNIA*) or by the General Method (*x=GM*) and $\alpha_{ult,k}^{CS}$ is the cross section resistance multiplier (without accounting for in-plane second order effects). x_c^I is the location along the member where the utilization due to applied (first order forces) is maximum and becomes: $x_{c,N}^I$ for a column; $x_{c,M}^I$ for a beam and $x_{c,MN}^I$ for a beam-column.

For computation of the General Method, all existing buckling curves along the member were considered: for example, if a welded beam presents cross sections where h/b≤2 as well as cross sections where h/b>2, 2 independent resistances for the same beam were computed, respectively one where curve *c* was considered, and other where curve *d* was considered, see Table 6.4 of EC3-1-1. Additionally, for choice of the buckling curve for lateral-torsional buckling, both General Case (GC) and Special Case (SC) for lateral-torsional buckling of beams (clause 6.3.2 of EC3-1-1) were used.

A high CoV (between 6.5% and 13%) was observed for all subsets (e.g. Table 6.1), as well as high percentages of values that exhibit $\chi^{Method}(x_c^I)/\chi^{GMNIA}(x_c^I)$ below 0.9 (even for buckling curve *b*. Same trends were noticed: high CoV in general; buckling curves that correspond to lower imperfection present mean values closer to 1, however high percentage of unsafe results; and buckling curves that correspond to higher imperfection lead to mean values that are significantly lower and in addition to high percentage of conservative cases, such that at times 100% of the cases falling more than 10% in resistance with respect to the numerical results. No clear trend can be defined indicating the unreliability of the method and general spread and inconsistency of results. Similar trends are noticed when evaluating the partial factor γ_{Rd}, whose values higher than 1.05 are highlighted in red.

Table 6.1. Statistical evaluation concerning the ratio $\chi^{Method}(x_c^I)/\chi^{GMNIA}(x_c^I)$ – all results

Sub-set	**Curve**	**n**	**Mean**	**CoV (%)**	**Min.**	**Max.**	**% cases <0.9**	**% cases >1.03**	
All	a (LT)	478	0.95	9.33	0.73	1.17	29.7	16.7	1.20
	b (LT)	1207	0.90	9.47	0.66	1.11	47.3	3.6	1.11
	c (LT and/or zz)	1610	0.86	10.81	0.60	1.07	64.9	1.3	1.07
	d (LT)	372	0.76	12.44	0.54	0.98	91.4	0.0	1.03

In summary, the General Method, which is the current alternative for the stability verification of non-uniform members, not only does not provide clear guidelines of which curve to be considered, but also may lead to a high (and random) spread regarding the level of safety, ranging from a decrease of 46% or an increase of 37% in the resistance capacity when compared to geometrically and materially non-linear analysis with imperfections (GMNIA). Moreover, evaluation of the partial factor γ_{Rd} was seen to fall mostly above 1.05, and consideration of the buckling curves of clause 6.3.2.3 (special case for lateral-torsional buckling) was shown to be inadequate, since γ_{Rd} is found between 1.41 and 1.22. On the other hand, a minimum partial factor of γ_{Rd}=0.91 was found. These values and uncertainties in application of the method clearly show its level of inconsistency.

Task 4.2 – Extension of stability verification non-uniform columns and beams subject to arbitrary loading

Safety assessment of rules for prismatic columns, beams and beam-columns

The aim of Task 4.2 was the extension of the stability verification to non-uniform columns and beams subject to arbitrary loading. In order, to ensure consistency with the existing design rule in Eurocode 3, they were assessed using the achievements of WP1 (the safety assessment procedure) and WP2 (statistical characterization). The safety assessment of the existing rules for steel members was performed covering prismatic columns and beams for the purposes of Task 4.2 and prismatic beam-columns for Task 4.3.

Scope and assumptions

The assessment was based on the procedure reported in Section 3. The assessment was based on the calculated partial factors γ_{M1}*, which were obtained for stability design rules summarized in *Table 6.2.*

For each case, a wide range of I-shaped cross sections covering several buckling curves are analyzed across practical ranges of slenderness.

Table 6.2 Design procedures for safety assessment

Failure mode	Method
Flexural buckling about minor and minor axis	EC3-1-1, clause 6.3.1
Lateral-torsional buckling of beams	EC3-1-1, clause 6.3.2: 6.3.2.2, General case (GC) 6.3.2.3, Special case (SC) 6.3.2.2, General case modified with f factor from clause 6.3.2.3 (GC/f) (Rebelo et al, 2009) Taras (2010) (new Ayrton-Perry analytical formulation, new imperfection factors)
Members under bending and compression	EC3-1-1, clause 6.3.3

Parametric studies

The parametric studies for beams and columns were defined in order to cover all buckling curves, various slenderness ratios and loading. The main parameters are summarized in Table 6.3. Material nonlinearity is incorporated in the model by using elastic-plastic constitutive law with strain-hardening, following the recommendations from ECCS (1978). The value of the yield stress, f_y, is considered either according to the provisions of the product standard EN 10025 (2004), or from Table 3.1 of EC3-1-1. Since Table 3.1 of EC3-1-1 does not account for t>80 mm, for such cases, the same value of f_y as in EN 10025 was considered.

Table 6.3 Parametric studies

<table>
<tr><th colspan="2" rowspan="2">Limits</th><th colspan="2">Number Sections</th><th colspan="2">Slenderness</th><th rowspan="2">Steel</th></tr>
<tr><th>Columns</th><th>Beams</th><th>Columns $\lambda_{y(z)}$</th><th>Beams λ_{LT}</th></tr>
<tr><td rowspan="3">h/b>1.2</td><td>$t_f \leq 40$mm</td><td>13</td><td>12</td><td rowspan="4">0.5; 0.6; 0.7; 0.8; 0.9; 1.0; 1.2; 1.4; 1.5; 1.6; 1.8; 2.0; 2.5</td><td rowspan="4">0.2; 0.4; 0.6; 0.8; 1.0; 1.1; 1.2; 1.3; 1.4; 1.5; 1.6; 1.7; 1.8; 2.0; 2.1</td><td rowspan="4">S235
S355
S460</td></tr>
<tr><td>$40 < t_f \leq 100$</td><td>13</td><td>9</td></tr>
<tr><td>$t_f > 100$</td><td>8</td><td>0</td></tr>
<tr><td>h/b≤1.2</td><td>$40 < t_f \leq 100$</td><td>13</td><td>9</td></tr>
</table>

Geometrical imperfections for columns were modelled using an initial sinusoidal imperfection introduced in the weak or strong axis of the cross-section, with an amplitude $e_0=L/1000$ at mid span; for beams the geometrical imperfections were modelled using an initial imperfection according to the first global buckling mode with an amplitude $e_0=L/1000$. Residual stresses were considered according to the ECCS recommendations (1978). For hot-rolled cross sections, the value of f_y* was considered using $f_{y,235}$=235 MPa. Nevertheless, for comparison, equivalent cases for columns were also included using the nominal value of the yield stress f_y.

Results and discussion

Columns

The parametric study was based on about 7300 cases. Its main purpose was to assess the safety of the code prescriptions of the flexural buckling of columns. The results were analyzed in various subsets. The study revealed that the variation in the relative values of the partial factor is not high, except for steel grade S460 and minor axis of flexural buckling. New imperfection factors were analyzed and the results showed good agreement with the ones obtained for S235 and S355.

Table 6.4 Values of γ_{M1}* obtained using different combinations of basic variables for minor and major axis flexural buckling

Limits		**Axis**	**Annex D (fy)**		**Annex D (fy+CS)**		**Annex D (fy+CS+E)**	
			S235 S355	**S460**	**S235 S355**	**S460**	**S235 S355**	**S460**
h/b>1,2	t_f≤40mm	y-y	0.978	1.006	1.030	1.066	1.079	1.120
		z-z	0.989	1.015	1.055	1.077	1.098	1.118
	40mm<t_f≤100mm	y-y	0.935	0.973	0.973	1.018	1.011	1.063
		z-z	0.925	0.955	0.983	1.011	1.021	1.048
	t_f>100mm	y-y	0.943	0.980	0.978	1.021	1.018	1.067
		z-z	0.954	0.974	1.008	1.032	1.043	1.070
h/b ≤1,2	t_f≤100mm	y-y	0.961	0.998	1.004	1.047	1.046	1.094
		z-z	1.020	1.019	1.079	1.075	1.115	1.109

<u>Beams</u>

The parametric study for beams was performed with respect to the parameters listed in Table 6.3. In addition, five load cases were covered - uniform; triangular and bi-triangular bending moment distribution, as well as concentrated and distributed loading, resulting in more than 3200 numerical simulations. Various subsets have been analysed, in order to assess the relevant parameters. The study confirmed the conservative nature of the General case from EC3-1-1, which was previously reported in *Rebelo et al* (2009). The Special case exhibited the highest values for the partial factor. On the contrary, the General case/f and the newly proposed method by Taras (2010) presented good agreement with the numerical results. The General case was conservative for non-uniform bending moment distributions. Table 6.5 and Table 6.6 summarize the partial factors obtained for different combinations of basic variables and methods.

Table 6.5 Values of γ_{M1}* obtained using different combinations of basic variables for General case and Special case

Limits		**Annex D (fy)**		**Annex D (fy+CS)**		**Annex D (fy+CS+E)**	
		GC	**SC**	**GC**	**SC**	**GC**	**SC**
h/b>1.2	t_f≤40mm	0.987	1.056	1.030	1.128	1.039	1.160
	40mm<t_f≤100mm	0.983	1.032	1.017	1.082	1.020	1.094
	t_f>100mm	-	-	-	-	-	-
h/b≤1.2	t_f≤100mm	0.983	1.010	1.018	1.050	1.018	1.052

Table 6.6 Values of γ_{M1}* obtained using different combinations of basic variables for General case/f and Taras

Limits		Annex D (fy)		Annex D (fy+CS)		Annex D (fy+CS+E)	
		GC/f	**Taras**	**GC/f**	**Taras**	**GC/f**	**Taras**
h/b>1.2	tf≤40mm	1.004	1.003	1.056	1.058	1.072	1.069
	40mm<tf≤100mm	0.978	0.978	1.018	1.027	1.023	1.034
	tf>100mm	-	-	-	-	-	-
h/b≤1.2	tf≤100mm	0.999	1.006	1.038	1.051	1.039	1.054

Beam – columns

The interaction formula from clause 6.3.3 for members in bending and compression was assessed in the scope of the project. The assessment covered more than 11 000 simulations were included in the assessment, covering various cross-section shapes, bending moment distributions, slenderness ranges. The obtained partial factors are summarized in Table 6.7.

Table 6.7 Values of γ_{M1}* obtained using the interaction formula

Limits		Annex D (fy)		Annex D (fy+CS)		Annex D (fy+CS+E)	
		No LTB	**LTB**	**No LTB**	**LTB**	**No LTB**	**LTB**
h/b>1,2	t_f≤40mm	0.979	0.984	1.006	1.005	1.014	1.013
	40mm<t_f≤100mm	-	0.979	-	0.985	-	1.000
	t_f>100mm	-	-	-	-	-	-
h/b≤1,2	t_f≤100mm	0.994	1.020	1.026	1.034	1.034	1.043

Extension of stability verification non-uniform columns and beams subject to arbitrary loading

In the scope of Task 4.2, a verification format applicable to non-uniform columns and beams subject to arbitrary loading was developed. The method is based on the same (Ayrton-Perry) formula as the rules in Eurocode 3 for columns and beams, but in the format of an interaction equation, and not as reduction factor χ calculation as the current design formats. The proposed interaction equation is composed of linear stress utilization that includes: (i) normal stresses due to applied forces; (ii) normal stresses due to second order forces. *Figure 6.3* illustrated the stress utilization in the most compressed section of a flange of a web-tapered column buckling about its major axis.

The utilization ratio is plotted along the longitudinal member axis x/L, representing the length of the column. This interaction between first and second order stresses is consistent with the current Eurocode 3 procedure where the reduction factor χ is also derived based on a linear direct stress criterion. The application of the procedure is based on the use of a Linear Buckling Analysis (LBA), which results in the critical load factor and the relevant buckling mode shape. These are further used to compute the direct stresses due to second order forces. The terms concerning the stress utilization due to second order forces are amplified by the imperfection according to the relevant buckling mode. Therefore consistency is kept with the rules for prismatic members. The adopted verification formats are summarized in Table 6.8.

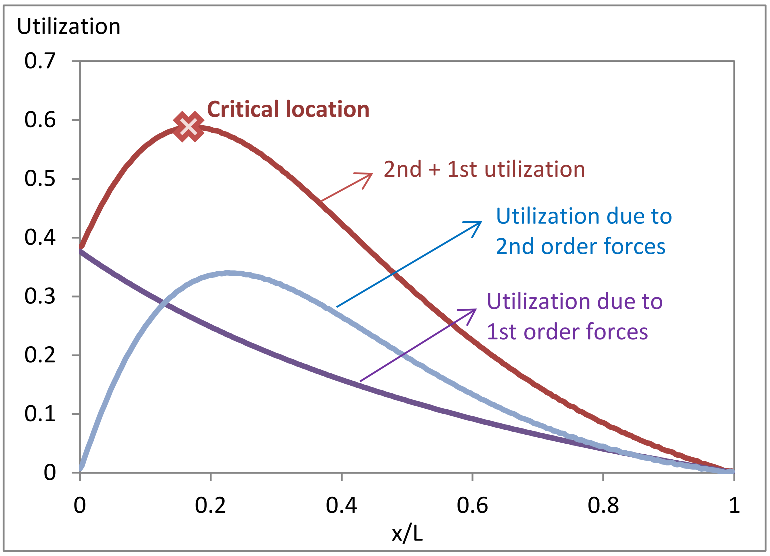


Figure 6.3 Determination of the failure location

There is an inevitable level of approximation in this generalization, since the equation is nonlinear but the value of the imperfection for any level of loading is fixed here, even if the amplitude factor is still considered. In fact, this imperfection is valid when the equation is equal to 1 and not <1: the location of the critical cross section varies with the increase of loading along the member since the relationship between first order and second order terms varies. For instance, in Figure 6.4, the method is applied for a tapered column with taper ratio $\gamma_h=\gamma_b=3$ loaded with a uniformly distributed axial force. The load level is changed and the total utilization ratio is plotted along the column length. For this column the maximum axial force is estimated to N_{max}=582 kN, if a higher force is used in the interaction equation (in this case when N_{max}=658 kN) the utilization ratio becomes higher than unity. It is also observed how the critical position of the member changes with the level of applied force due to the change of the amplification for the second order effects.

Although its generalization (to other buckling modes and their interaction) is still being validated, due to its strong analytical base, it is expected that it will lead to an adequate and consistent level of safety. Moreover, it is much more flexible and general when compared to a procedure that relies on unavoidable calibration for each and possible case of a non-uniform member – either due to a non-standard loading case, or due to an irregular distribution of restraints, etc.

Local effects can be easily introduced in a further step by considering the effective cross sectional properties for verification of the interaction equation at each location.

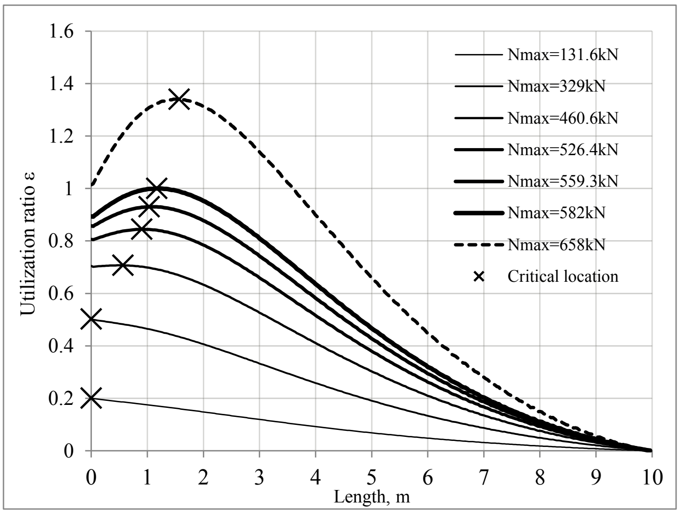


Figure 6.4: Utilization ratio for various load levels

Finally, the relevant second order forces should be chosen by a critical observation of the buckling mode shape and the forces involved. This can be done upon the determination of the critical load amplifier, α_{cr}, with any general finite element software.

An example of the sequence of application of the method for flexural buckling is summarized in Figure 6.5.

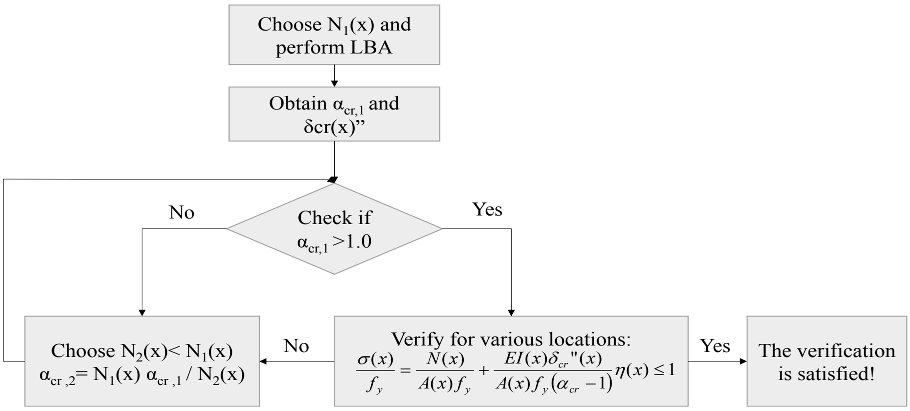


Figure 6.5: Application of the method

Table 6.8 Verification formats for columns and beams

Parameter	Flexural buckling	Lateral-torsional buckling
$\bar{\lambda}(x)$	$\bar{\lambda} = \sqrt{\frac{A(x)f_y}{\alpha_{cr}N(x)}}$	$\bar{\lambda} = \sqrt{\frac{A(x)f_y}{N_{cr,z,eq}(x)}}$
$\eta(x)$	$\alpha(\bar{\lambda}(x)-0.2)\frac{\alpha_{cr}N(x)\lvert\delta_{cr}(x_m)\rvert}{EI(x_m)\lvert\delta_{cr}''(x_m)\rvert}$	$\alpha(\bar{\lambda}(x)-0.2)\frac{\alpha_{cr}M_y(x)\lvert\theta_{cr}(x)\rvert}{EI(x_m)\lvert v_{cr}''(x_m)\rvert}$
$\varepsilon(x)$	$\frac{N(x)}{A(x)f_y} + \frac{EI(x)\delta_{cr}''(x)}{A(x)f_y(\alpha_{cr}-1)}\eta(x) \le 1$	$\frac{M_y(x)}{W_y(x)f_y} + \frac{EI_z(x)\eta(x)}{A(x)f_y(\alpha_{cr}-1)}\left(v''_{cr} + \left(\frac{h}{2}\theta''_{cr} + \theta'_{cr}h'\right)\frac{v''_{cr}}{\theta''_{cr}}\frac{N_{cr,z,eq}}{M_{cr}}\right) \le 1$

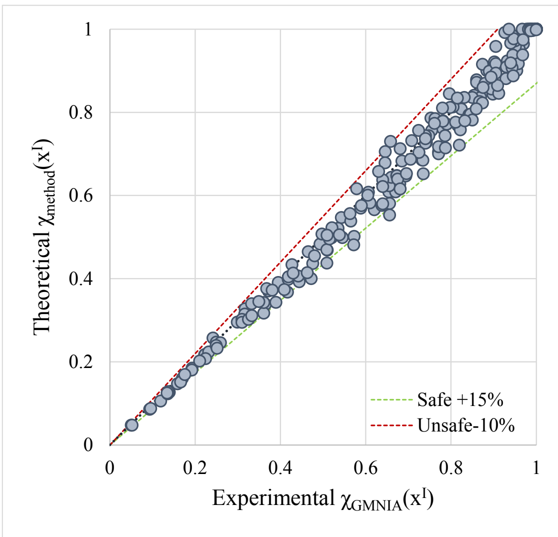


Figure 6.6 Scatter plot: columns and beams

The method was verified for cases coving various bending moment distributions, support conditions, partial restraints, and different boundary conditions for different tapering ratios. The scatterplot is shown in Figure 6.6 for columns and beams together. In all cases the method predicts a resistance level between 80% and 110% of the GMNIA resistance.

Task 4.3 – Generalized slenderness procedure for verification of non-uniform beam-columns

In Task 4.3, the objective was to develop a procedure for the verification of non-uniform beam-columns. In EC3-1-1, the safety verification of non-uniform may be performed by the General Method, which application was shown not to be reliable in section 0. On the other hand, the interaction formulae in EC3-1-1 were specifically calibrated for stability verification of prismatic members. Ayrton-Perry based proposals for the stability verification of web-tapered columns and beams, in line with the Eurocode principles for the stability verification of prismatic members, have shown to lead to a substantial increase of accuracy and to provide mechanical consistency relatively to application of the General Method. Such methodologies may be further applied to the existing interaction formulae.

It is the purpose in Task 4.3 a verification procedure for the stability verification of web-tapered beam-columns under in-plane loading by adaptation of the interaction formulae in EC3-1-1, validated through extensive FEM numerical simulations covering several combinations of bending moment about strong axis, M_y, and axial force, N, and levels of taper.

A main problem in the adaptation of the interaction formula to non-uniform beam-columns relates to the correct location to take into consideration in the given interaction formulae. For the case of prismatic beam-columns, this location is always the location of maximum bending moment utilization as the axial force is constant; however, for tapered beam-columns, it may not be the case. Nevertheless, according to the definitions of utilizations for tapered beams and columns, the quantities $n_y=N_{Ed}/(\chi_y N_{Rk})$, $n_z=N_{Ed}/(\chi_z N_{Rk})$ or $m_y=M_{y,Ed}/(\chi_{LT} M_{y,Rk})$ are constant along the member length and, as a result, it is irrelevant which location is chosen and is recommended here (for simplicity) the consideration of the first order failure location of the axial force acting alone ($x_{c,N}^I$) for the utilization term regarding axial force; and the first order failure location of the bending moment acting alone ($x_{c,M}^I$) for the utilization term regarding the bending moment. Then the interaction equations can be rewritten as follows:

$$\frac{N_{Ed}(x_{c,N}^I)}{\chi_y(x_{c,N}^I)\,N_{Rk}(x_{c,N}^I)/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}(x_{c,M}^I)}{\chi_{LT}(x_{c,M}^I)M_{y,Rk}(x_{c,M}^I)/\gamma_{M1}} \leq 1.0 \tag{6.1}$$

$$\frac{N_{Ed}(x_{c,N}^I)}{\chi_z(x_{c,N}^I)\,N_{Rk}(x_{c,N}^I)/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}(x_{c,M}^I)}{\chi_{LT}(x_{c,M}^I)M_{y,Rk}(x_{c,M}^I)/\gamma_{M1}} \leq 1.0 \tag{6.2}$$

The interaction factors given in Annex B (Method 2) of EC3-1-1 were adapted to the tapered beam-column case. Method 2 is considered for a straightforward application/adaptation of the interaction formulae to the case of tapered beam-columns and further validation. Because I-sections are susceptible to torsional deformations, according to Method 2, the interaction factors to be considered are summarized in *Table 6.9* – the only difference between these factors and the ones present in EC3-1-1 is the properties of the cross section at the respective location to be considered – which, for a prismatic member are constant.

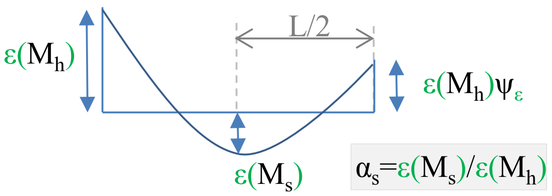
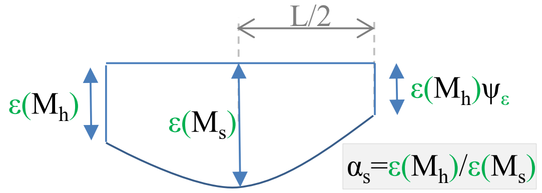
Table 6.9 *Interaction factors for web-tapered beam-columns according to Method 2*

k_{yy}	$C_{my}\times\left(1+\underbrace{\left(\bar{\lambda}_y(x_{c,N}^{II})-0.2\right)}_{\leq 0.8\ \geq 0}\frac{N_{Ed}(x_{c,N}^I)}{\chi_y(x_{c,N}^I)\,N_{Rk}(x_{c,N}^I)/\gamma_{M1}}\right)$
k_{zy}	$1-\frac{\overbrace{0.1\bar{\lambda}_z(x_{c,N}^{II})}^{\leq 0.1}}{C_{m,LT}-0.25}\frac{N_{Ed}(x_{c,N}^1)}{\chi_z(x_{c,N}^1)\,N_{Rk}(x_{c,N}^1)/\gamma_{M1}}$ *for* $\bar{\lambda}_z(x_{c,N}^{II})<0.4$: $0.6+\bar{\lambda}_z(x_{c,N}^{II})\leq 1-\frac{0.1\bar{\lambda}_z(x_{c,N}^{II})}{C_{m,LT}-0.25}\frac{N_{Ed}(x_{c,N}^1)}{\chi_z(x_{c,N}^1)\,N_{Rk}(x_{c,N}^1)/\gamma_{M1}}$

Finally, regarding the equivalent uniform moment factors $C_{m,y}$ and $C_{m,LT}$, Table B.3 of EC3-1-1 can be adopted provided that the diagram to be considered is the bending moment first order utilization diagram instead of the bending moment diagram itself, see Table 6.9.

In a tapered beam subject to a linear bending moment distribution, the diagram of the utilization can be fairly well compared to the diagram of a prismatic beam subject both to uniformly distributed loading and end moments. The C_m factor may be obtained from the respective C_m factor due to that diagram.

Table 6.10 Adaptation of the equivalent uniform moment factors C_m for prismatic members

Moment **utilization** diagram	Range		C_{my} and C_{mLT}
	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi_\varepsilon \leq 1$	$0.2 + 0.8\,\alpha_s \geq 0.4$
	$-1 \leq \alpha_s < 0$	$0 \leq \psi_\varepsilon \leq 1$	$0.1 - 0.8\,\alpha_s \geq 0.4$
		$-1 \leq \psi_\varepsilon < 0$	$0.1(1-\psi_\varepsilon) - 0.8\,\alpha_s \geq 0.4$
	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi_\varepsilon \leq 1$	$0.95 + 0.05\,\alpha_h$
	$-1 \leq \alpha_s < 0$	$0 \leq \psi_\varepsilon \leq 1$	$0.95 + 0.05\,\alpha_h$
		$-1 \leq \psi_\varepsilon < 0$	$0.95 + 0.05\,\alpha_h(1+2\psi_\varepsilon)$

In *Figure 6.7*, to have a common basis, the generalized reduction factors are compared: $\chi_{ov}^{GMNIA}=\alpha_b^{GMNIA}/\alpha_{ult,k}$ and $\chi_{ov}^{interaction}=\alpha_b^{interaction}/\alpha_{ult,k}$, in which α_b is the resistance multiplier obtained numerically or by the interaction approach and $\alpha_{ult,k}$ is the cross section resistance multiplier. It should be mentioned that the General Method would lead to a scatter or results between 50% up to 120%, depending on the buckling curve.

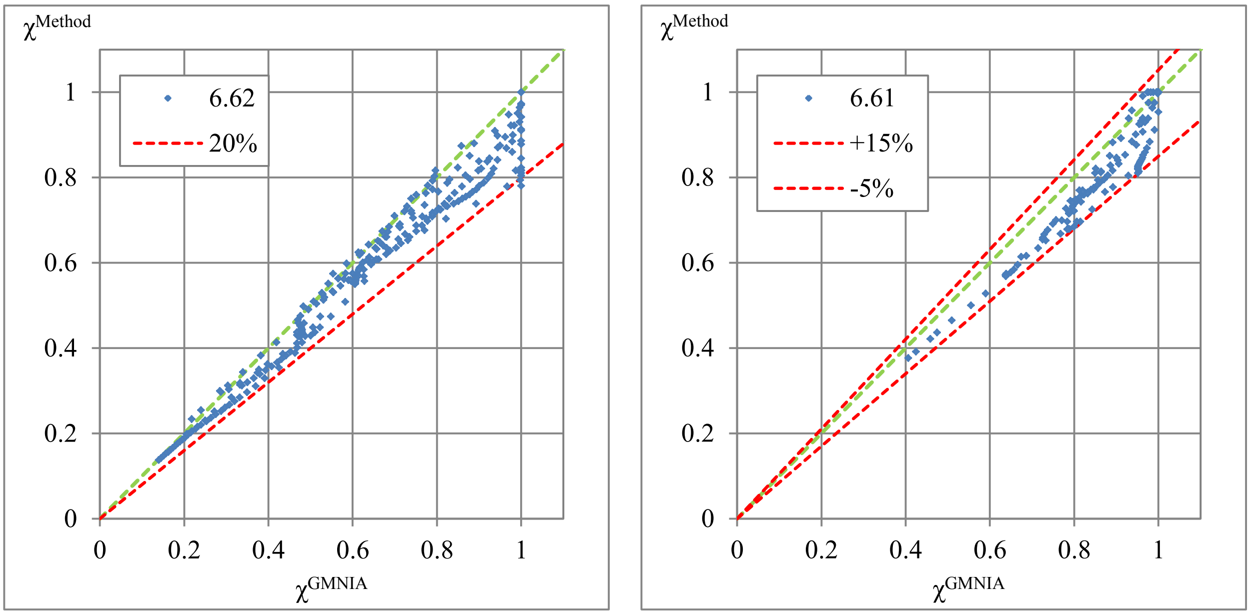
Finally, statistical indicators are also shown in *Table 6.11* and indicate good average and low CoV for all sets when considering the proposed formulation.

In summary, it can be concluded that a straightforward adaptation of the interaction formulae in EC3-1-1 always gives safe results, and the conservatism of this methodology was not shown to be greater than 20%.

Table 6.11: Statistical evaluation concerning the ratio $\chi_{ov}^{Method}/\chi_{ov}^{GMNIA}$ for the interaction formulae

Methodology	**n**	**Mean**	**St. Dev.**	**CoV (%)**	**Min.**	**Max.**	**% cases >1.1**	**% cases <0.97**
6.62	273	1.08	0.053	5.66	0.93	1.28	26.7	2.2
6.61	140	1.08	0.049	5.22	0.97	1.18	26.4	0.0
6.61 or 6.62 S_i	105	1.11	0.050	5.54	0.94	1.23	47	1.9

The stability verification of tapered beam-columns was discussed. The interaction formulae in EC3-1-1 for prismatic members were adapted for tapered members, validated through extensive FEM numerical simulations covering several combinations of bending moment about strong axis, M_y, and axial force, N, and levels of taper. For the time being, a parametric study of 520 beam-columns with or without intermediate lateral restraints indicated that the interaction approach leads to results that are mostly on the safe side. Maximum differences of 20% relatively to the numerical results were achieved for any of the possible failure modes of the beam-column.



(a) Unrestrained beam-columns (b) Continuously restrained beam-columns

Figure 6.7 *Results given by the interaction approach*

6.3 *Conclusions*

The work done within Work package 4 aimed at achieving consistent level of safety throughout verification rules concerning failure modes focusing on stability aspects. The work covered:

- Clause 6.3.1: Uniform members in compression
- Clause 6.3.2: Uniform members in bending
- Clause 6.3.3: Uniform members in bending and compression
- Clause 6.3.4: General method for lateral and lateral-torsional buckling of structural components.

The following main conclusions are drawn:

1 The buckling curves for S460 (minor axis flexural buckling) did not exhibit the same level of safety as all other cases in terms of partial factors calculated using the procedure of WP1 with the distributions from WP2. It was recommended to amend the imperfection factors for flexural buckling minor axis and steel grade S460.

2 The methods for lateral-torsional buckling of beams were also assessed. Revealing some conservative results for the general case, unsafe results for the special case and good agreement between numerical and theoretical estimates for the modified general case (GC/f) and the new design rule by Taras;

3 The interaction formula for the verification of members under bending and compression resulted was assessed and considered to comply with the safety.

4 The general method from clause 6.3.4 was applied to a large number of non-uniform cases. The results exhibited high scatter without any trend which could be adopted for the improvement of the method.

5 A new method for the verification of non-uniform columns and beams was proposed. It is applied as an interaction equation for various locations along the member length. It combines the first and second order forces in order to calculate the most unfavourable location along the member length. The advantage of the method relates to the fact that it does not need to consider an equivalent simply-supported member and it is therefore able to deal with a larger set of situations.

6 A new method for the verification of non-uniform beam-columns was developed. It was based on the developed rules by *Marques et al*. (2012) and (2013). It was validated for a large number of numerical simulations showing very good agreement for the studied cases.

7 <u>Work package 5 – Modes driven by fracture</u>

7.1 *Overview*

The work within **W**ork **P**ackage 5 (**WP5**) is realized at the University of Stuttgart. The Work Package is titled "**Modes driven by fracture**" and it is divided into five tasks, listed as follows:

- Task 5.1 Tests on base and weld materials
- Task 5.2 Tests on weld details
- Task 5.3 Numerical investigations
- Task 5.4 Interpretation and evaluation of results
- Task 5.5 Development of recommendations

7.2 *Objectives of WP5*

The main objective of WP5 is to develop a method for statistical validation of design rules for typical failure driven by fracture depending on material strength using as example weld design strength of mixed connections of Mild Carbon Steel (MCS) and High Strength Steel (HSS) as base materials. The detailed specification of the WP5 could be obtained in Deliverable 5.1

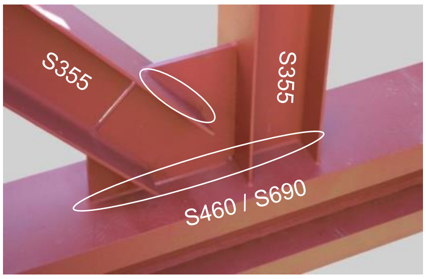


Figure 7.1 mixed connections between Mild Carbon Steel and High Strength Steel.

Especially for the particular situation of joining high strength steel and mild carbon steel elements used in dual-steel structural configurations, the present design rules, e.g. in *EN 1993-1-8,* (2005) are in many cases inadequate as they always require that the weld filler metal should at least match the higher grade (matching or overmatching) of the base metals in terms of strength and toughness. The rules for connections having parent metals of different strengths are insufficient due to the fact that the recent rules and safety margins according to *EN 1993-1-8,* (2005) are developed traditionally for standard steels and then transferred to high strength steels.

Investigations focused on automatically welded specimens of steel S690 indicate a much clearer picture on the influences. It became clear that the filler metal played an eminent role, so that *Rasche*, (2012) suggests a new design strength in dependence on both the filler metal and the base metal strength. This proposal has meanwhile been accepted by TC250/SC3, see Amendment in *Document N2168*, (2015), and will be implemented into the new version of EN 1993-1-8 which in future will also cover Steels of higher strength than 460 N/mm^2.

For the dimensioning of welded joints with base and filler metals of different strengths, it is necessary to consider both strengths in the rated equation for consideration of the mixing of different materials. *Rasche*, (2012) has derived such a design rule. It is proposed that the strength of the base metal and the filler metal be weighted with factors. Accordingly, the strength of the base metal is taken into account with a weighting factor of 0.25 (25%) and that of the filler metal with a weighting factor of 0.75 (75%). The new design rule by *Rasche*, (2012) is based only on welded connections with the same base metal.

One of the objective of the WP is to verify the applicability the modified design resistance by *Rasche*, (2012) for dual-steel connections and, if necessary, recalibrate them. A statistical evaluation is carried out in WP5 to pursue the goal on the basis of newly acquired test results and an adapted statistical evaluation method.

7.3 *Work undertaken and results obtained*

Introduction

Within WP5 an experimental program is conducted and focused on the load carrying capacity and safety against fracture of welded dual-steel connections. The typical failure is a "fracture failure" depending on the tensile strength of the various materials. And typically statistical evaluation of test results plays the decisive role, whereas numerical investigations only support the characterisation of influences and served as test preparation in this case.

The main objective of the WP is to develop recommendations for statistical evaluation of failure modes driven by fracture based on the experimental testing for design and give more detailed rules of welded dual-steel connections composed of two different steel grades: Mild Carbon Steel and High Strength Steel and a range of different filler metals.

On the basis of these results a comprehensive database for the statistical evaluation of failure modes driven by fracture, especially for welded connections tested for fracture has been set up.

In addition to the earlier research results the new project wants to systematically feed the database with relevant data of welded connections, evaluate lots statistically in dependence on the different parameters such as type of welding (manual, automatic) and shape of weld in order to give at the end rules for mixed connections but also rules for an appropriate safety assessment procedure, where failure modes driven by fracture and tensile material strength play the important role.

Task 5.1: Tests on base and weld materials

One purpose of tensile tests has been to supply standard data on material properties in order to be able to compare them without any ambiguity. It is thus important to comply with the corresponding code for the particular tensile testing according to *EN ISO 6892-1*, (2009). It is also required to conform to the notations recommended by it in order to avoid confusion.

The main focus of the tensile tests presented here was to find out material properties such as yield strength (R_{eH} / f_y respectively $R_{p0.2}$ / f_y), tensile strength (R_m / f_u), plastic extension at maximum force (A_g), total extension at maximum force (A_{gt}), elongation after fracture (A), total extension at fracture (A_t) and ultimate tensile load (F_{max}). Another reason for tensile tests was to check and verify independently data from material certificates delivered by the steel producers.

The material testing programme at the University of Stuttgart included material tensile tests on **B**ase Metal (BM) and on Filler Metal (FM)

In the frame of the experimental investigations on mixed connections, 4 different steel grades (S355J2+N, S460ML, S500MC, S690QL) were used in combination with variable filler metals (G42, G46, G55, G69, G89). In

Table 7.1 and

As a general comment to the material properties it may be noted that the results for base metal comply with the provisions whereas the filler metals especially G69 and G89 do not fully fulfil the requirements.

Table 7.2 the detailed characteristics related to the static yield strength R_{eH} / $R_{p0.2}$, and the tensile strength R_m are set out in comparison with the code provisions. The given experimental value is a mean value of 3 tensile tests.

Table 7.1 Test results from tensile tests on base metal

Mechanical properties			**S355J2+N** 1.0577 *EN 10025-2: 2005-04 Table 7*	**S460ML** **1.**8838 *EN 10025-4: 2005-04 Table 5*	**S500MC** **1.**0976 *EN 10149-2: 2013-12 Table 2*	**S690QL** **1.**8928 *EN 10025-6: 2009-08 Table 5*
R_{eH} / $R_{p0.2}$	*[N/mm²]*	nom.	355	460	500	690
		exp.	425.0	512.0	609	824.3
R_m	*[N/mm²]*	nom.	470 - 630	540 - 720	550 - 700	770 - 940
		exp.	545.7	575.0	669	873.7
A	*[%]*	nom.	22	17	14	14
		exp.	26.92	21.28	17.32	14.75

As a general comment to the material properties it may be noted that the results for base metal comply with the provisions whereas the filler metals especially G69 and G89 do not fully fulfil the requirements.

Table 7.2 Test results from tensile tests on filler metal

Mechanical properties			G42 3Si1 *EN ISO 14341:2011-04 Table 1A*	G46 4Si1 *EN ISO 14341:2011-04 Table 1A*	G55 Mn3Ni1Mo *EN ISO 16834:2012-08 Table 1A*	G69 Mn4Ni1.5CrMo *EN ISO 16834:2012-08 Table 1A*	G89 Mn4Ni2CrMo *EN ISO 16834:2012-08 Table 1A*
R_{eH} / $R_{p0.2}$	[N/mm²]	nom.	420	460	550	690	890
		exp.	492.0	519.0	570,7	646.3	729.3
R_m	[N/mm²]	nom.	500 - 640	530 - 680	640 - 820	770 - 940	940 - 1180
		exp.	590.3	606.3	673.3	774.0	939.3
A	[%]	nom.	20	20	18	17	15
		exp.	24.37	28.10	23.93	20.84	18.75

Task 5.2: Tests on weld details

Tests on LAP Connection

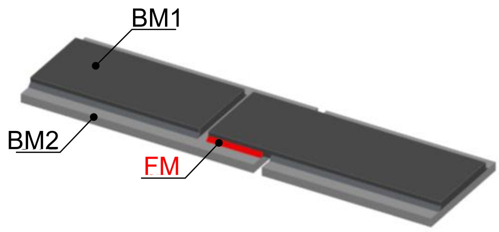


Figure 7.2 Test specimen for LAP-connection

The experimental program on longitudinal fillet welds consists of 18 reference tests (connections made of the same base material, see Table 7.3) and 36 tests on mixed connections (specimens welded single layer made of two different steel grades, see Table 7.4).

The reference tests were intended to supply a better comparability between welded connections made of one steel grade according to *Rasche*, (2012) and welded connections made of two different steel grades, namely mild carbon steel and high strength steel.

Table 7.3 Reference tests (18 specimens)

Base Metal (BM) BM 1 (top)	 BM 2 (bottom)	Filler Metal (FM)	Welding speed v_s [cm/min]	Welding process	Number n [-]
S355J2+N	S355J2+N	G42	min vs/ max vs	automatic	1
		G46	min vs/ max vs	automatic	1
		G69	min vs/ max vs	automatic	1
S460ML	S460ML	G46	min vs/ max vs	automatic	1
		G69	min vs/ max vs	automatic	1
		G89	min vs/ max vs	automatic	1
S690QL	S690QL	G46	min vs/ max vs	automatic	1
		G69	min vs/ max vs	automatic	1
		G89	min vs/ max vs	automatic	1

The varying parameters, which defined the number of weld details for longitudinal fillet welds to be tested, are: base material, filler metal, welding speed and welding process. As a key feature, one series of specimens was produced automatically welded and another series manually welded with more "natural" flaws so that the necessary different statistical treatment could be developed.

Figure 7.2 shows a specimen with a LAP Connection consisting of Base Metal 1, Base Metal 2 and Filler Metal. Base Metal 1 (BM1) - either S355 or S460 - is located on top of Base Metal 2 (S690). A filler metal (FM) joins the two plates of the connection. The detailed experimental programme including the designation of the welded test specimens can be taken from Table 7.3 for the reference test and from Table 7.4 for mixed connections.

Table 7.4 Test on mixed connection (36 specimens)

Base Metal (BM) BM 1 (top)	Base Metal (BM) BM 2 (bottom)	Filler Metal (FM)	Welding speed v_s [cm/min]	Welding process	Number n [-]
S355J2+N	S690QL	G46	min vs	manual automatic	2 1
			max vs	manual automatic	2 1
		G69	min vs	manual automatic	2 1
			max vs	manual automatic	2 1
		G89	min vs	manual automatic	2 1
			max vs	manual automatic	2 1
S460ML	S690QL	G46	min vs	manual automatic	2 1
			max vs	manual automatic	2 1
		G69	min vs	manual automatic	2 1
			max vs	manual automatic	2 1
		G89	min vs	manual automatic	2 1
			max vs	manual automatic	2 1

Test set-up:

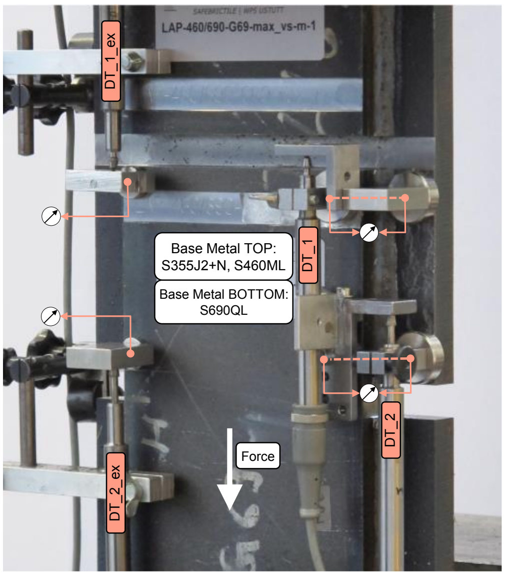


Figure 7.3 Test set-up for LAP-connections

Under displacement control, all the specimens had been subjected to an axial force which was applied by jaws via a hydraulic actuator (SCHENCK) with a maximum capacity in tension of 1000 kN. The load was increased up to fracture.

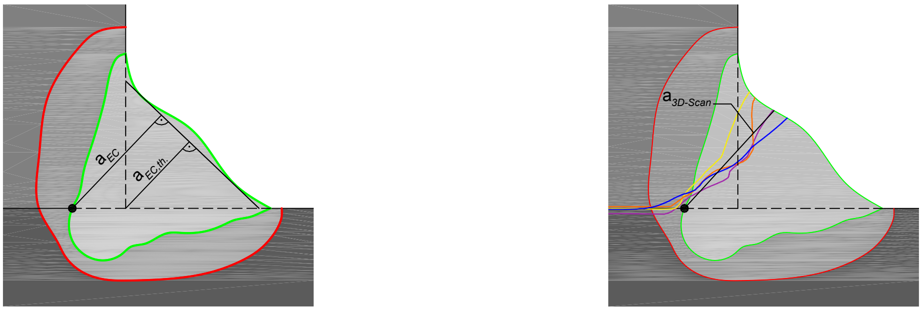
Four displacement transducers (DT) were fixed to the specimens, see Figure 7.3. With the help of the displacement transducers DT_1_ex and DT_2_ex both the absolute displacements were recorded at the beginning and the end of the weld. The measurement of the relative displacement was recorded at the beginning of the weld with the displacement transducer DT_1 and at the end of the weld with DT_2. In general, the weld length was 100 mm.

According to *EN 1993-1-8,* (2005), the shear stress resulting from tension force on the specimen in the weld at the time of the maximum stress is determined by the ratio of the maximum measured test force F_{max} and the associated effective fillet weld fracture area.

After testing and recording the force, three different methods were accomplished to determine the throat thickness of the weld needed to determine the fillet weld fracture area.

- Method 1: a_{EC} (with penetration), see Figure 7.4 a)
- Method 2: $a_{EC,th.}$ (without penetration), see Figure 7.4 a)

- Method 3: $a_{3D-Scan}$ (fracture area), see Figure 7.4 b)



(a) According to Eurocode (b) Reverse engineering (3D-Scan)

Figure 7.4 Determination methods of throat thickness

To achieve an utmost accuracy, an optical measurement system was used. The process of digitalization of the area is called *"reverse engineering"*. That means, the surface of the cracked weld was reconstructed with a 3D scanning system reproducing more than 5 Mio. surface points. The 3D model was transferred to a 2D area with the help of CAD and was passed into the macro section of each test specimen, see Figure 7.4 b).

From further investigations it has been shown that the digitized fracture area $A_{3D\text{-}Scan}$, determined by throat thickness $a_{3D-Scan}$ (Method 3), can be used best as the effective fillet weld surface for the determination of the experimental maximum shear stress $\tau_{II,max}$.

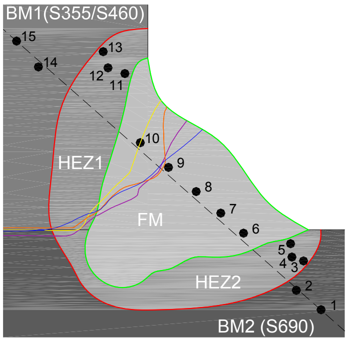


Figure 7.5 Inserted fracture lines

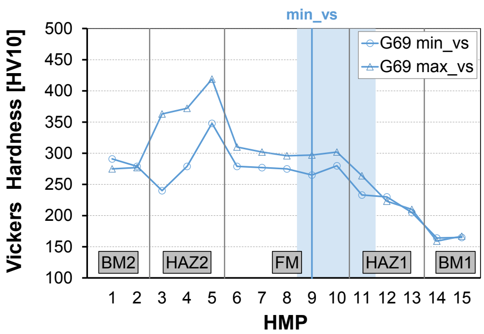


Figure 7.6 Hardness measurement of the connection S355/S690-G69-a

Combining the information taken from the reverse engineering and the hardness measurement points (HMP) it is possible to identify the zone (range of hardness measurement points) where the fracture took place, shown typically in Figure 7.5 and Figure 7.6. By plotting the ranges of the fracture zone a tendency towards Base Metal 1 (>45° fracture angle) can be observed. This leads to the assumption that the influence of the weaker base metal is more pronounced.

Tests on K-Connection

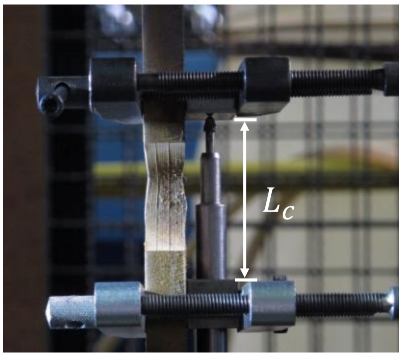


Figure 7.7 Test set-up K-connection

Beside the fillet weld connections (LAP connection) for a small group of specimens butt welds were tested in order to find out the influence of the amount of filler metal and type of weld for fillet welds.

The experimental tests (see Figure 7.7) on K-connections were carried out at the MPA University of Stuttgart. Under displacement control, all the specimens had been subjected to an axial force which was applied by jaws via a hydraulic actuator. The load was increased up to fracture. The detailed experimental could be taken from Table 7.5.

Table 7.5 Experimental program on butt welds (K-connection)

Base Metal (BM) BM 1	BM 2	Filler Metal (FM)	Welding process	Number n [-]
S355J2+N	S690QL	G46	automatic	4
		G69	automatic	4
		G89	automatic	4
S460ML	S690QL	G46	automatic	4
		G69	automatic	4
		G89	automatic	4
S500MC	S690QL	G46	automatic	4
		G55	automatic	4
		G69	automatic	4
		G89	automatic	4

Especially for the combination of S500 and S690 under- and overmatching was investigated. It has evaluated if under matching might need for a new consideration related to the design and also looking at the ductile or non-ductile behaviour of such sections. As a result, it can be clearly recognized in Figure 7.8 Experimental results K-connection that the load carrying capacity is nearly constant for the different configurations while differing the filler metal. Therefore, it can be concluded that the load carrying capacity is much more influenced by the weaker base metal as by the filler metal.

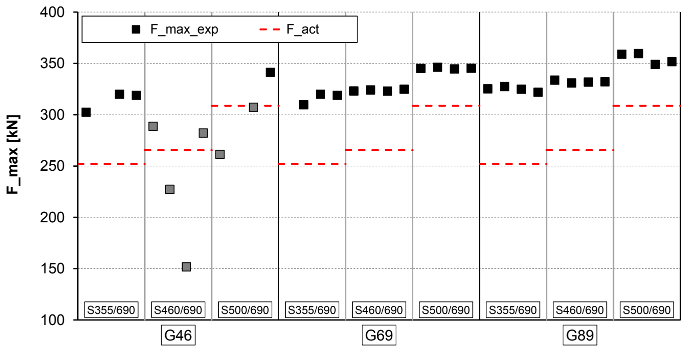


Figure 7.8 Experimental results K-connection

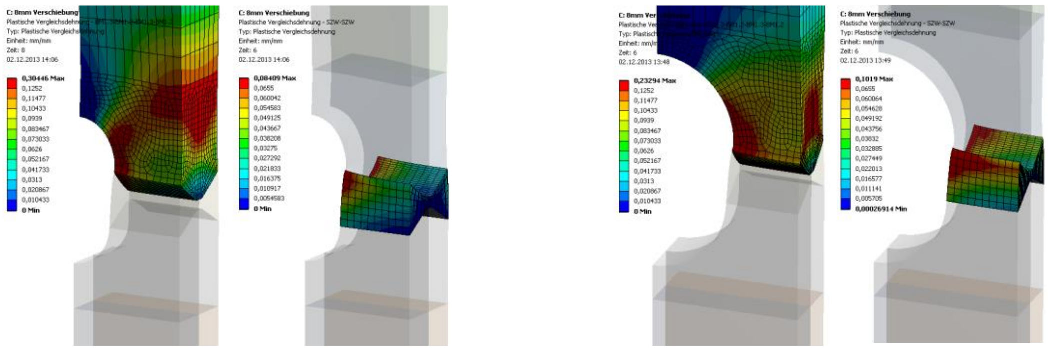
Task 5.3: Numerical investigations

In a first step the material model was calibrated based on results from tensile tests on steel samples and weld material. ANSYS requires the material properties input to be in the form of points on a true stress-strain curve. The multi-linear approximated stress-strain curve was therefore transformed to true stress and true strain. Steel components were modelled based on true stress-true strain material data available from uniaxial tests with the same nominal mechanical properties as those used in the tests. The extraction of the ANSYS material input data was done according to the procedure of *Ling*, (1996) and *Bridgman*, (1952).

As initially planned, one of the main objectives of butt weld specimens was to investigate the influence of the amount of filler metal and the nature and type of fracture: brittle or ductile. Taking into account that two different base metals are connected by in general overmatching welds it is obvious that the specimen subjected to pure tension will fail at the weaker base metal connected.

Plastic strains are concentrated at the weaker side, in that case the base metal 1. Aiming to provoke as much as possible plastic strains and failure in the weld, it has been necessary to optimize the geometry.

The most practicable solution in order to control the development and the location of the plastic strains was to mill out the specimens in conformity with the geometry of tensile test coupon. This led to a more concentrated strain distribution within the milled-out section due to a smaller cross-section.



a) Mill-out section: Quarter circle (r = 7.5mm) b) Mill-out section: Quarter circle (r = 15mm)

Figure 7.9 Optimizing geometry (showing half model) by numerical simulations

First numerical simulations with a quarter circle as milled-out section showed a clear tendency towards a stronger relationship between non-milled to milled width of the specimen, see Figure 7.9. Furthermore, a larger radius leads to higher concentration of the plastic strains.

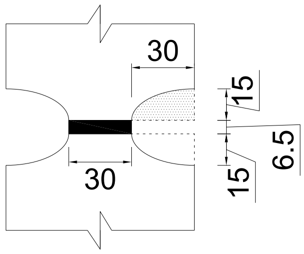


Figure 7.10 Final milled-out section

Finally, a milled-out section of a quarter ellipse (see Figure 7.10) showed the most appropriate relationship of stress/ strain distribution between weaker base metal and filler metal. As a result, it was fixed to mill the member after welding and then cut the whole member into small single specimens. At the experimental test the requested area of the necking could be observed. Because of this the desired effect of the concentration of plastic strain in the weld metal could be generated.

Task 5.4 Interpretation and evaluation of results for LAP Connection

Overview

The first step of the statistical evaluation is the development of a design model in the form of a theoretical resistance function r_t, which includes all relevant basic variables that have an influence on the load carrying capacity of the fillet weld connection.

For the design of welded joints with base and filler metals of different strengths, it is necessary to consider both strengths, the base and the filler metal strength in the design equation, in order to consider the effect of “mixing” different materials. *Rasche*, (2012) has derived a modified design resistance in her doctoral thesis, which is now also accepted by TC250/SC3 as an amendment for the future *EN 1993-1-8,* (2005). The proposal is weighting the strength of the base metal and of the filler metal by different factors. Accordingly, the strength of the base metal $f_{u,BM}$ is taken into account with a weighting factor of 0.25 (25%) and of the filler metal $f_{u,FM}$ with a weighting factor of 0.75 (75%). However, this proposal has been developed only on the basis of tests with both base metals being of the same type. The hypothesis is that this modified design rule may be applied also for mixed connections. On the basis of the modified design resistance by *Rasche*, (2012), new correlation coefficients were determined as a function of the strength of the filler metal, see Table 7.6. In order to identify that these new correlation coefficients are dependent on the respective filler metal, these coefficients are provided with the index "FM".

Table 7.6 Correlation coefficient $\beta_{w,FM}$ depending on the filler metal according to *Rasche*, (2012)

Filler metal	$\boldsymbol{\beta_{w,FM}}$
G42 / E42	0.89
G46 / E46 / T46	0.85
G69 / T69	1.09
G89	1.19

For the statistical evaluation, on one hand the standardized procedure according to *EN 1990 Annex D*, (2002) (Procedure 1) for the calibration of a resistance model has been applied for the evaluation of the experimental results. On the other hand, the procedure according to *EN 1990 Annex D*, (2002) adapted especially for the evaluation of longitudinal fillet welds by *Kleiner* (2016) (Procedure 2), is presented. This specially adapted statistical evaluation method especially taken into account the important variable of the fracture surface which is not part of the resistance function. The adapted statistical procedure has been adjusted to the effect to determine the statistical parameters for each test specimen individual. The comparison of both procedure are shown in Table 7.7. The adjusted variables for the procedure 2 are blue marked at Table 7.7.

On the basis of both methods, the experimentally determined weld load carrying capacities can be examined with regard to theoretically determined values. The complete procedure of the statistical evaluation is part of the doctoral thesis of *Kleiner*, (2016).

The statistical evaluation pursues the goal to verify and, if it is necessary, recalibrate the modified design resistance by *Rasche*, (2012) on the basis of newly acquired test results and the adapted statistical evaluation method by *Kleiner*, (2016). The evaluation focused on reviewing the applicability of the modified design resistance according to *Rasche*, (2012) for mixed connections. More specifically, the efficiency for the design of mixed connections was investigated when applying the modified design resistance according to *Rasche*, (2012) by using the lower strength of the base metals to determine the design resistance in addition to the filler metal strength. Therefore, correlation coefficients $\beta_{w,FM}{}^{Kl}$ were determined for mixed connections and compared to the correlation coefficients $\beta_{w,FM}{}^{Ra}$ for the modified design resistance by *Rasche*, (2012), see Table 7.6.

Table 7.7 Differentiation of the determination of the coefficient of variation V_{rt}^2 of the base variable X_j and of the standard deviation Q of the resistance function r_t

<table>
<tr><th>Procedure 1
according to EN 1990 Annex D, (2002)
- P1 -</th><th>Procedure 2
adapted procedure according to EN 1990 Annex D
by Kleiner, (2016)
- P2 -</th></tr>
<tr><td colspan="2">Variation coefficient V_r of the value of the resistance function r_t
The following approximation can be used for small values of V_δ^2 and V_{rt}^2
$V_r^2 = V_\delta^2 + V_{rt}^2$
Variation coefficient V_{rt} of the basic variables X_j
Error Propagation of the distribution of the basic variable of the resistance function</td></tr>
<tr><td>$V_{rt}{}^2 = \sum_{j=1}^{k} V_{X_j}{}^2$</td><td>(For each experiment i)
$V_{rt.i}{}^2 = \frac{1}{r_{t,i}(\underline{X}_{j,m})^2}\cdot\sum_{j=1}^{k}\left(\frac{\delta r_{t,i}(X_j)}{\delta X_j}\cdot\sigma_j\right)^2$
Note: Partial derivative for procedure 2 (shear stresses) see equations (11.8) and (11.9)
(Globally for the group under consideration)I
$V_{rt}{}^2 = \sum_{i=1}^{n}\left(\frac{1}{r_{t,i(\underline{X}_{j,m})^2}}\cdot\sum_{j=1}^{k}\left(\frac{\delta r_{t,i}(X_j)}{\delta X_j}\cdot\sigma_j\right)^2\right)/n + V_{X_j}{}^2$</td></tr>
<tr><td colspan="2">Determination of the standard deviations</td></tr>
<tr><td>$Q_{rt} = \sqrt{ln(V_{rt}{}^2 + 1)}$</td><td>(For each experiment i)
$Q_{rt,i} = \sqrt{ln(V_{rt,i}{}^2 + V_{X_j}{}^2 + 1)}$</td></tr>
<tr><td colspan="2">Standard deviation Q_δ of the deviation value δ
$Q_\delta = \sqrt{ln(V_\delta{}^2 + 1)}$</td></tr>
<tr><td colspan="2">Standard deviation Q_δ of the total resistance function r_t</td></tr>
<tr><td>$Q = \sqrt{ln(V_\delta{}^2 + V_{rt}{}^2 + 1)}$</td><td>(For each experiment i)
$Q_i = \sqrt{ln(V_\delta{}^2 + V_{rt,i}{}^2 + V_{X_j}{}^2 + 1)}$
Note: A global variation coefficient of a relevant basic variable X_j can be estimated</td></tr>
</table>

For the statistical evaluations according to *EN 1990 Annex D*, (2002), the following parameters of basic variables given in Table 7.8 are required: The mean value $\bar{x}$, the standard deviation σ and the coefficient of variation CoV. Within the framework of the statistical evaluation method according to *EN 1990 Annex D*, (2002), the variation coefficient CoV is also expressed as V_x.

Table 7.8 shows the analytical equations for the determination of the experimental shear stress $r_{e,i}$ and the theoretical shear stress $r_{t,i}$, which were needed for the statistical evaluation according to *EN 1990 Annex D*, (2002) and the adapted statistical procedure according to *Kleiner*, (2016). The tensile strength of the base metal $f_{u,BM}$ and the tensile strength of the filler metal $f_{u,FM}$ are listed as basic variables for the determination of the theoretical resistance of the shear stress and may be taken from Table 7.1 and Table 7.2.

The modified design resistance by *Rasche*, (2012) developed on test results of welded connections with base metal of steel grade S355, S460, S690 and S700 and filler metal G / T / E46, G / T69 and G89. Due to this the modified design resistance applicable for welded connection with base metal S355 up to S700 and filler metal with a strength of 460 N/mm^2 up to 890 N/mm^2.

Table 7.8 Experimental and theoretical resistance for the statistical evaluation

Experimental resistance for specimen i	**Theoretical resistance determined using the measured parameters X for each specimen i**	**Basic variables**
$r_{e,i} = \tau_{II,max} = \frac{F_{max}}{A_{frac}}$	$r_{t,i} = \tau_{II,Rd} = \frac{0.25 \cdot f_{u,BM,i} + 0.75 \cdot f_{u,FM,i}}{\sqrt{3} \cdot \beta_{w,FM}}$	$f_{u,BM}$ $f_{u,FM}$ A_{frac}

Statistical evaluation of basic variables

Since the present material tests taken from the test specimens are not sufficiently statistically representative in the context of the experimental investigations, the literature *Da Silva*, (2009), *HILONG*, (2016) has been relied on for a statement of the basic totality. The statistical parameters given in Table 7.9 correspond to the basic totality of the respective steel grades. In addition to the mean value $\bar{x}$, the standard deviation *σ*, the variation coefficient CoV and the number of experiments *n*, the minimum values of the tensile strengths from the corresponding product standard are given. It can be clearly seen that the variation coefficient is nearly constant for all base metals (between 2.9% and 3.7%). For the statistical evaluation, a variation coefficient CoV of the tensile strength of the base metal $f_{u,BM}$ of $V_{f_{u,BM}} = 0.04$ is accordingly applied.

Table 7.9 Statistical parameters of the tensile strength of the base metal $f_{u,BM}$

Statistical parameters		**S355** *EN 10025-2:2005-04* *Table 7* 470 - 630 [N/mm²] *da Silva et al.* (2009) $\boldsymbol{X_j}$	**S460** *EN 10025-4:2005-04* *Table 5* 540 - 720 [N/mm²] *da Silva et al.* (2009) $\boldsymbol{X_j}$	**S690** *EN 10025-6:2009-08* *Table 5* 770 - 940 [N/mm²] *HILONG*, (2016) $\boldsymbol{X_j}$	**S700** *EN 10149-2:2013-12* *Table 2* 750 - 950 [N/mm²] *HILONG*, (2016) $\boldsymbol{X_j}$
$\bar{x}$	[N/mm²]	533.44	632.73	846.20	840.56
σ	[N/mm²]	16.53	23.18	29.73	24.15
CoV	[%]	3.10	3.66	3.51	2.87
n	[-]	1972	672	425	14

For the basic variable of the strength of filler metal *Kleiner*, (2016) has collected a large number of tensile tests of different filler metals for a database. Table 7.10 shows the statistical parameters of the basic totality for each filler metal (X_j) according to the database of *Kleiner*, (2016). In comparison, the parameter X_i referring to a sample of the filler metal (solid wires) with a size of 3 is taken from the tests in WP5. It should be pointed out that both, the values of the solid as well as the flux cored wire have been evaluated jointly, since the minimum values of the tensile strength required according to the product standards are the same. For the statistical evaluation, the coefficient of variation CoV for tensile strength of the filler metal by the database of *Kleiner*, (2016) is determined as $V_{f_{u,FM}} = 0.06$. For the statistical evaluation of the load carrying capacity the results of the evaluation of the database of *Kleiner*, (2016) were used because there is a lower scatter than for the results of the 3 tensile tests within the WP.

Table 7.10 Statistical parameters of the tensile strength of the filler metal $f_{u,FM}$

Statistical parameters		G46 / T46 DIN EN ISO 14341: (G46) DIN EN ISO 17632: (T46) 530 - 680 [N/mm²]		G69 / T69 DIN EN ISO 16834: (G69) DIN EN ISO 18276: (T69) 770 - 940 [N/mm²]		G89 / T89 DIN EN ISO 16834: (G89) DIN EN ISO 18276: (T89) 940 - 1180 [N/mm²]	
		Kleiner, (2016) $\boldsymbol{X_j}$	SAFEBRICTILE X_i	*Kleiner*, (2016) $\boldsymbol{X_j}$	SAFEBRICTILE X_i	*Kleiner*, (2016) $\boldsymbol{X_j}$	SAFEBRICTILE X_i
$\bar{x}$	[N/mm²]	610.00	606	818.50	774.00	1001.5	939.33
σ	[N/mm²]	32.77	7.64	40.98	6.56	57.06	6.03
CoV	[%]	5.37	1.26	5.01	0.85	5.70	0.64
n	[-]	124	3	48	3	60	3

Statistical evaluation to determine correlation coefficients for mixed connections

In Figure 7.11 the flowchart for the statistical evaluation is presented. Different values of arc energy *E* and cooling time $t_{8/5}$ are resulting to varying welding speeds v_s. This generates a natural scattering which is close to a standard weld fabrication. Further data groups for the statistic evaluation are manually and automatically welded longitudinal fillet welds, for both mismatch connections and mixed connections. The former are characterized in that the same base metals and varying filler metals are combined. Mixed compounds, on the other hand, are defined as a material combination of different base metals and also varying filler metals. The calculation of a longitudinal fillet weld connection depending on the used filler metal allows considering the test results with base metals of varying strength properties. The steel grades S355, S460, S690 and S700 are considered as base metals.

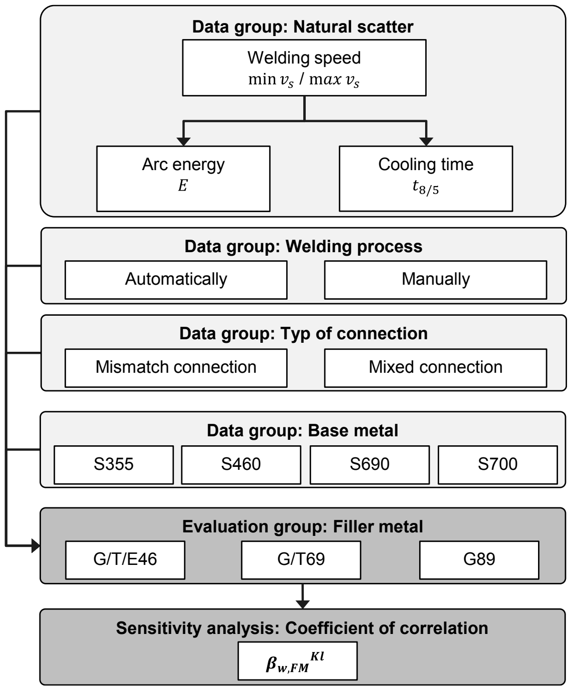


Figure 7.11 Flowchart of the statistical evaluation

The sensitivity analysis to determine the correlation coefficient $\beta_{w,FM}{}^{Kl}$ was carried out for various options (1.1, 1.2, 2, 3 and 4 in Table 7.11 and Table 7.13 and Table 7.15). Option 1 is divided into two separate evaluations using only the test results of the WP. In option 1.1 the theoretical resistance $r_{t,i}$ of each individual experiment is determined with the correlation coefficient $\beta_{w,FM}{}^{Ra}$ (see Table 7.6) derived from *Rasche*, (2012). Using the partial factor it is also verified to what extent the required safety level of $\gamma_{M,target} = 1.25$ has been achieved. The aim of the option 1.2, option 2, option 3 and option 4 is to derive a new correlation coefficient $\beta_{w,FM}{}^{Kl}$ (according to *Kleiner*, (2016)) for the considered filler metal group. Accordingly, the theoretical resistance $r_{t,i}$ is calculated for each experiment without an (input) correlation coefficient $\beta_{w,FM}$. Both option 1.1 and option 1.2 include the variation coefficients of the basis variable statistically determined in the above paragraph.

Option 2 is based on the experimental results and the variation coefficients of the basic variables according to the doctoral thesis by *Rasche*, (2012). In option 3 and option 4, the experiments of the WP5 and the dissertation by *Rasche*, (2012) are combined and evaluated using the variation coefficients according to *Kleiner*, (2016) (option 3) and *Rasche*, (2012) (option 4).

For option 3 all test results of the database of *Rasche*, (2012) and the test results of the WP5 are considered. For the test results a smaller scatter could be observed. This leads to better i.e. smaller variation coefficients for the statistical evaluation. Therefore, option 3 is used for the verification of the applicability of the modified design resistance according to *Rasche*, (2012) for mixed connections in Table 7.12, Table 7.14 and Table 7.16.

The conditions of the different options are explained below within the presentation of the results. Each option has been evaluated with the standardized procedure according to *EN 1990 Annex D*, (2002) (Procedure 1) and the modified statistical procedure (Procedure 2), see Table 7.7. This permits a direct comparison between both statistical procedures.

For each option, the number *n* of evaluated experiments and the considered variation coefficients of the basis variables are listed. Furthermore, the statistical results of the two procedures (Procedure 1 and Procedure 2 see Table 7.7) are given. These include the mean value correction *b*, the variation coefficient V_δ for the measure of variation δ, the resulting variation coefficient V_r, the partial safety factor $\gamma_M{}^*$, and the corresponding correlation coefficient $\beta_{w,FM}{}^{Kl}$.

Results of the statistical evaluation of filler metal with tensile strength $f_y = 460\ N/mm^2$

Table 7.11 summarizes the results of the statistical evaluation of filler metal with strength $f_y = 460\ N/mm^2$.

Under the application of option 1.1, are reviewed what extent the required safety level of $\gamma_{M,target} = 1.25$ has been achieved. The option 1.1 shows in Table 7.6 that the statistical evaluation of the test results from WP5 with the correlation coefficient $\beta_{w,FM}{}^{Ra} = 0.85$ according to *Rasche*, (2012). This results in a partial safety factor $\gamma_M{}^*$ = 1.3 higher than $\gamma_{M,target} = 1.25$. Hence the distance between the nominal resistance $r_{t,nom}$ and the design value r_d is slightly larger than the required safety. Since the resulting nominal resistance is too high, the distance is not conservative. Figure 7.12 shows schematically the two cases: $\gamma_M{}^* < \gamma_{M,target}$ and $\gamma_M{}^* > \gamma_{M,target}$. The aim is to adjust the nominal resistance $r_{t,nom}$ by using the correlation coefficient $\beta_{w,FM}{}^{Kl}$, so that the required safety distance $\gamma_{M,target}$ is reached.

Table 7.11 Statistical evaluation for filler metal group G/T/E46

							Procedure 1 (P1): According to Annex *D*					Procedure 2 (P2): Modified evaluation procedure obtained for each specimen				
Options	**Number** n	$V_{f_{u,BM}}$	$V_{f_{u,FM}}$	$V_{A_{frac}}$	$x \cdot f_{u,BM}$	$y \cdot f_{u,FM}$	b	V_δ	V_r	$\gamma_M{}^*$	$\beta_{w,FM}{}^{Kl}$	b	V_δ	V_r	$\gamma_M{}^*$	$\beta_{w,FM}{}^{Kl}$
		[%]	[%]	[%]	[-]	[-]	[-]	[%]	[-]	[-]	[-]	[-]	[%]	[-]	[-]	[-]
1.1	18	4	6	11	0.25	0.75	1.10	7.88	0.153	1.32	0.90	1.10	7.88	0.141	1.27	0.86
1.2	18	4	6	11	0.25	0.75	1.30	7.88	0.153	1.12	0.90	1.30	7.88	0.141	1.08	0.86
2	66	7	7	10	0.25	0.75	1.33	7.48	0.159	1.08	0.86	1.33	7.48	0.131	0.99	0.79
3	88	4	6	11	0.25	0.75	1.32	7.82	0.153	1.06	0.85	1.32	7.82	0.141	1.02	**0.82**
4	88	7	7	10	0.25	0.75	1.32	7.82	0.161	1.09	0.87	1.32	7.82	0.133	1.00	0.80

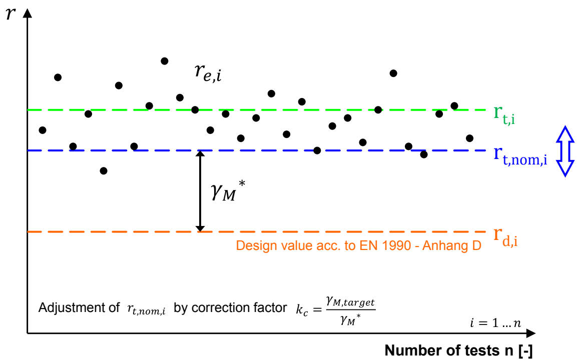


Figure 7.12 Interpretation of safety factor γ_M*

For Option 1.2, no (input) correlation coefficient $\beta_{w,FM}$ was used for the determination of the theoretical resistance $r_{t,i}$. Therefore, a larger mean value correction *b* is obtained compared to Option 1.1, but the same correlation coefficients $\beta_{w,FM}{}^{Kl}$ for Procedure P1 and P2 are obtained as well. Option 2 has to be evaluated for the same conditions as in the doctoral thesis of *Rasche*, (2012). Hence, only 66 mismatch connections were used. As a result, the statistical evaluation is determined under the same conditions as in *Rasche*, (2012). Procedure 1 provides a correlation coefficient $\beta_{w,FM}{}^{Kl} = 0.86$ nearly identical to the value $\beta_{w,FM}{}^{Ra} = 0.85$ (see Table 7.6) determined by *Rasche*, (2012). Option 3 summarizes all test results and a smaller correlation coefficient of $\beta_{w,FM}{}^{Kl} = 0.82$ for Procedure 2 arises.

Table 7.12 Comparison of shear stress for connections with filler metal G/T/E46

Option	S460 N/NL; M/ML					S690 Q/QL/QL1					S700 MC				
	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$
	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]
3	289.4	289.4	±0.0	299.9	+3.7	320.6	320.6	±0.0	332.3	+3.7	317.9	317.9	±0.0	329.5	+3.7

Table 7.12 shows design load carrying capacities $\tau_{II,Rd}{}^{Ra}$ for longitudinal fillet weld using a filler metal with a nominal strength $f_y = 460\,N/mm^2$ according to *Rasche*, (2012). By comparison, both the absolute values of the design load carrying capacities with the determined correlation coefficients $\beta_{w,FM}{}^{Kl}$ according to Table 7.11 for option 3 procedure 2 and the percentage difference are given.

Based on Option 3, a more efficient approach is possible for the design of longitudinal fillet weld connections by using a correlation coefficient of $\beta_{w,FM}{}^{Kl} = 0.82$. Compared to the previous approach the load carrying capacity can be increased by 3.7%. Option 4 increases the load carrying capacity only for Procedure 2, provided that the variation coefficients of the basis variables according to *Rasche*, (2012) are used.

Figure 7.13 shows the test results compared to the previous design load carrying capacities according *Rasche*, (2012) and the design resistance developed with a modified correlation coefficient in Option 3 procedure 2 of $\beta_{w,FM}{}^{Kl} = 0.82$ according to *Kleiner*, (2016). In addition, the nominal resistance $r_{t,nom}$ is shown to illustrate the required safety distance of $\gamma_{M2} = 1.25$.

The increase in load carrying capacity by 3.7% is visible. Consequently, a correlation coefficient of $\beta_{w,FM}{}^{Kl} = 0.82$ may be proposed as a result of the statistical evaluation of longitudinal fillet weld connections with filler metal with strength $f_y = 460\,N/mm^2$.

Results of the statistical evaluation of filler metal with tensile strength $f_y = 690\,N/mm^2$

Table 7.13 summarizes the results of the statistical evaluation of filler metal with strength $f_y = 690\,N/mm^2$.

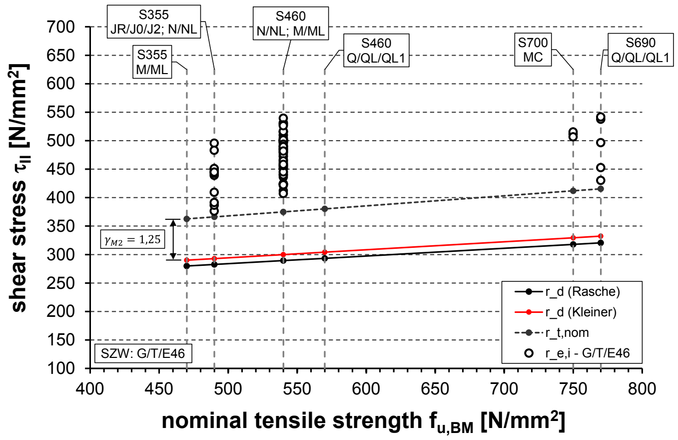


Figure 7.13 Comparison of modified design resistance according to *Rasche*, (2012) and *Kleiner*, (2016) for filler metal G/T/E46

Table 7.13 statistical evaluation for filler metal group G/T69

Options	Number n	$V_{f_{u,BM}}$	$V_{f_{u,FM}}$	$V_{A_{frac}}$	$x \cdot f_{u,BM}$	$y \cdot f_{u,FM}$	Procedure 1 (P1): According to Annex *D*					Procedure 2 (P2): Modified evaluation procedure obtained for each specimen				
							b	V_δ	V_r	$\gamma_M{}^*$	$\beta_{w,FM}{}^{Kl}$	b	V_δ	V_r	$\gamma_M{}^*$	$\beta_{w,FM}{}^{Kl}$
		[%]	[%]	[%]	[-]	[-]	[-]	[%]	[-]	[-]	[-]	[-]	[%]	[-]	[-]	[-]
1.1	18	4	6	11	0.25	0.75	1.26	8.88	0.159	1.26	1.10	1.26	8.88	0.147	1.21	1.06
1.2	18	4	6	11	0.25	0.75	1.16	8.88	0.159	1.37	1.10	1.16	8.88	0.147	1.32	1.06
2	55	7	7	10	0.25	0.75	1.09	7.45	0.151	1.38	1.10	1.09	7.45	0.138	1.32	1.06
3	77	4	6	11	0.25	0.75	1.10	8.81	0.154	1.37	1.10	1.10	8.81	0.142	1.32	**1.06**
4	77	7	7	10	0.25	0.75	1.10	8.81	0.162	1.41	1.13	1.10	8.81	0.134	1.29	1.03

It should be noted that Option 2 was evaluated under the same conditions as by *Rasche*, (2012). Accordingly, no mixed connections were considered. Option 1.1, in which the weighting factors according to *Rasche*, (2012) were applied to the test results of WP5, shows for Pprocedure 1 that a partial safety factor $\gamma_M{}^* = 1.26$ is sufficiently accurate. Procedure 2 even allows the possibility of an optimization of the design load carrying capacity using a partial safety factor of $\gamma_M{}^* = 1.21$. Option 1.2 provides the same correlation coefficients $\beta_{w,FM}{}^{Kl}$ for a different mean value correction *b* but identical scatterings V_δ and V_r. Option 3 summarizes all available test results. Option 4 was evaluated using the variation coefficients of the basis variable according to *Rasche*, (2012). At a large scattering V_δ shows significant deviations to the correlation coefficient of $\beta_{w,FM}{}^{Ra} = 1.09$ (see Table 7.6), derived from *Rasche*, (2012). Comparing the resulting variation coefficients V_r of Option 2, Procedure 1 with procedure 2 in Option 3 shows that a lower scatter ($V_r = 0.142$) exists. This offers the possibility of a more efficient design of the longitudinal fillet weld by an average of 2.8% (see Table 7.14).

Figure 7.14 shows all test results using the filler metals G / T69, the design values according to *Rasche*, (2012) and the design resistance developed with a modified correlation coefficient in Option 3 Procedure 2 of $\beta_{w,FM}{}^{Kl} = 1.06$ according to *Kleiner*, (2016) as a function of the nominal tensile strength of the base metals. The nominal resistance $r_{t,nom}$ represents the required safety distance of $\gamma_{M2} = 1.25$.

Table 7.14 comparison of shear stress for connections with filler metal G/T69

Option	S460 N/NL; M/ML					S690 Q/QL/QL1					S700 MC				
	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$
	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]
3	301.9	299.2	- 0.9	310.5	**+2.8**	326.3	323.3	-0.9	355.5	**+2.8**	324.2	321.2	-0.9	333.3	**+2.8**

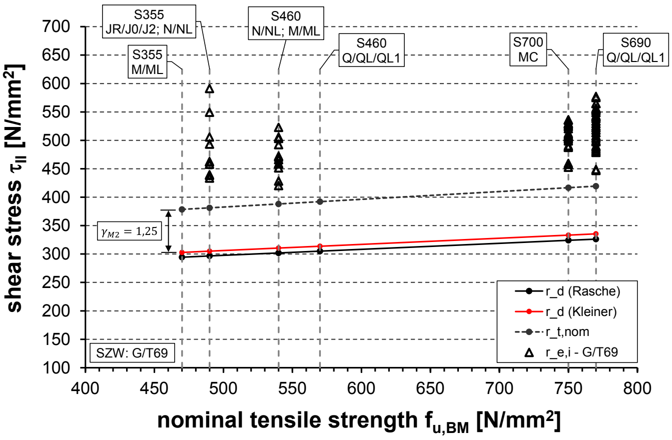


Figure 7.14 Comparison of modified design resistance according to *Rasche*, (2012) and *Kleiner*, (2016) for filler metal G/T69

The optimization of the limit shear stress or rather of the load carrying capacity by 2.8% can be noticed. Consequently, a correlation coefficient of $\beta_{w,FM}{}^{Kl} = 1.06$ may be proposed as a result of the statistical evaluation of the longitudinal fillet weld connections with filler metal with strength $f_y = 690\ N/mm^2$.

Results of the statistical evaluation for filler metal with tensile strength $f_y = 890\ N/mm^2$

Table 7.15 gives an overview of the results of the statistical evaluation of the filler metal with strength $f_y = 890\ N/mm^2$.

Comparing the test results from WP5 (Option 1.2) with the results of the doctoral thesis of *Rasche*, (2012) (Option 2) a significant higher variation coefficient V_δ are shown, which indicates that the resistance function for this group of test results is not sufficient. In Option 1.1, however, the partial safety factors $\gamma_M{}^* = 1.34$ for Procedure 1 and $\gamma_M{}^* = 1.31$ for Procedure 2 indicate that the considered group of experimental results in an excessively large nominal resistance $r_{t,nom}$. In this case the design value r_d would also be too high and the load carrying capacity would be on the non-conservative side. Procedure 1 of Option 2 comes closest to the statistical evaluation of *Rasche*, (2012). However, a more efficient correlation coefficient of $\beta_{w,FM}{}^{Kl} = 1.11$ results compared with the correlation coefficient of $\beta_{w,FM}{}^{Ra} = 1.19$ (see Table 7.6). Option 2 Procedure 2 would allow an even higher design value for a larger correlation coefficient of $\beta_{w,FM}{}^{Kl} = 1.07$ for filler metal with strength $f_y = 890\ N/mm^2$ under consideration.

Table 7.15 Statistical evaluation for filler metal group G89

							Procedure 1 (P1): According to Annex *D*					**Procedure 2 (P2): Modified evaluation procedure obtained for each specimen**				
Options	**Number** n	$V_{f_{u,BM}}$	$V_{f_{u,FM}}$	$V_{A_{frac}}$	$x \cdot f_{u,BM}$	$y \cdot f_{u,FM}$	b	V_δ	V_r	$\gamma_M{}^*$	$\beta_{w,FM}{}^{Kl}$	b	V_δ	V_r	$\gamma_M{}^*$	$\beta_{w,FM}{}^{Kl}$
		[%]	[%]	[%]	[-]	[-]	[-]	[%]	[-]	[-]	[-]	[-]	[%]	[-]	[-]	[-]
1.1	16	4	6	11	0.25	0.75	1.23	10.02	0.165	1.34	1.28	1.23	10.02	0.156	1.31	1.25
1.2	16	4	6	11	0.25	0.75	1.03	10.02	0.165	1.60	1.28	1.03	10.02	0.156	1.56	1.25
2	10	7	7	10	0.25	0.75	1.04	3.53	0.136	1.39	1.11	1.04	3.53	0.124	1.34	1.07
3	26	4	6	11	0.25	0.75	1.03	8.07	0.154	1.48	1.19	1.03	8.07	0.144	1.44	**1.15**
4	26	7	7	10	0.25	0.75	1.03	8.07	0.162	1.52	1.22	1.03	8.07	0.137	1.41	1.13

Option 3 summarizes all test results with the filler metal G89 and, by using a variation coefficients of $V_\delta = 8.07\%$, results in a scattering which is more than twice as large as in the test results considered in Option 2. The large scattering of the test results effects the design value or rather the correlation coefficient $\beta_{w,FM}{}^{Kl}$. However, in an evaluation of the test results based on Procedure 2, using a correlation coefficient of $\beta_{w,FM}{}^{Kl} = 1.15$, an average increase in the load carrying capacity of 3.5% can be achieved according to Table 7.16. Furthermore, Procedure 2 in Option 4 shows the advantage of achieving an increase in the carrying capacity by using a correlation coefficient of $\beta_{w,FM}{}^{Kl} = 1.13$ with the rather conservative variation coefficients of the basis variables according to *Rasche*, (2012).

Table 7.16 comparison of shear stress for connections with filler metal G89

Options	S460 N/NL; M/ML					S690 Q/QL/QL1					S700 MC				
	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$	$\tau_{\parallel,Rd}{}^{Ra}$	$\tau_{\parallel,Rd}(P1)$	$\Delta\tau_{\parallel,Rd}(P1)$	$\tau_{\parallel,Rd}(P2)$	$\Delta\tau_{\parallel,Rd}(P2)$
	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[%]
3	326,0	326,0	±0,0	337,4	**+3,5**	348,4	348,4	±0,0	360,5	**+3,5**	346,4	346,4	±0,0	358,5	**+3,5**

Figure 7.15 shows all test results with the filler metal with strength $f_y = 890\ N/mm^2$, the design values according to *Rasche*, (2012) and the design resistance developed with a modified correlation coefficient in option 3 procedure 2 of $\beta_{w,FM}{}^{Kl} = 1.15$ according to *Kleiner*, (2016) as a function of the nominal tensile strength of the base metals.

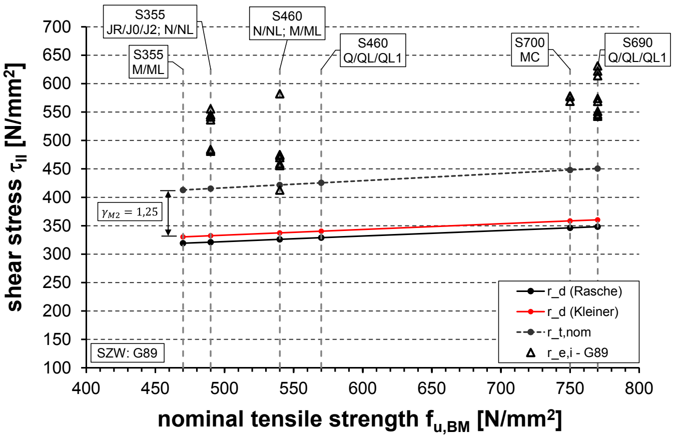


Figure 7.15 Comparison of modified design resistance according to *Rasche*, (2012) and *Kleiner*, (2016) for filler metal G89

In addition to the partially large scattering of the test results, the increase in the load carrying capacity of the longitudinal fillet weld connection with increasing nominal strength of the base metal is clearly visible. Consequently, as a result of the statistical evaluation of the longitudinal fillet weld connections with g filler metals of the strength $f_y = 890\ N/mm^2$, a correlation coefficient of $\beta_{w,FM}{}^{Kl} = 1.15$ may be proposed.

Task 5.5 Development of recommendations

One of the aim of the WP was to develop a method for statistical validation of design rules for typical failure driven by fracture depending on material strength using as example design of fillet welds of mixed connections. First of all, the application of the procedure proved that the method developed by *Rasche*, (2012) depending mainly on the filler metal may also be applied for mixed connections of mild steel and high strength steel, if the weaker base metal is considered in the formula (7.1). The results of the statistical evaluation has shown that the modified statistical method (Procedure 2) according to *Kleiner*, (2016) leads to slightly better correlation coefficients $\beta_{w,FM}{}^{Kl}$ (see Table 7.11, Table 7.13, Table 7.15) than the standardized procedure according to *EN 1990 Annex D*, (2002) (Procedure 1). On the basis of the slightly better correlation coefficients $\beta_{w,FM}{}^{Kl}$ better design resistances could be achieved for longitudinal fillet welds using the modified design resistance function (7.1) according to *Rasche*, (2012).

$$\tau_{II,Rd} = \frac{0{,}25 \cdot f_{u,BM} + 0{,}75 \cdot f_{u,FM}}{\sqrt{3} \cdot \beta_{w,FM}{}^{Kl} \cdot \gamma_{M2}} \quad (7.1)$$

In addition, the applicability of the modified design resistance according to *Rasche*, (2012) using the weaker base metal for mixed connections has been proven. The comparison of the correlation coefficients $\beta_{w,FM}{}^{Ra}$ of *Rasche*, (2012) and the correlation coefficients $\beta_{w,FM}{}^{Kl}$ of *Kleiner*, (2016) in Table 7.17 shows that there is a good accordance between the coefficients. The correlation coefficients for mixed connections $\beta_{w,FM}{}^{Kl}$ are slightly better than the coefficients of *Rasche*, (2012).

Table 7.17 Comparison of the correlation coefficients $\beta_{w,FM}{}^{Kl}$ according to *Kleiner*, (2016) and $\beta_{w,FM}{}^{Ra}$ according to *Rasche*, (2012)

Filler metal	$\boldsymbol{\beta_{w,FM}{}^{Kl}}$	$\boldsymbol{\beta_{w,FM}{}^{Ra}}$
G46 / E46 / T46	0.82	0.85
G69 / T69	1.06	1.09
G89	1.15	1.19

The comparison of the determined design resistances of mixed connections with the different correlation coefficients in Figure 7.13, Figure 7.14 and Figure 7.15 shows an effective improvement up to 3,7% when using the correlation coefficients of $\beta_{w,FM}{}^{Kl}$. This difference of in maximum 3.7% is of course acceptable with regard to efficiency. Therefore, the applicability of the modified design resistance according to *Rasche*, (2012) for mixed connections is confirmed without any modification.

7.4 *Conclusions*

After finishing the experimental program on butt welds (K-connection) and fillet welds (LAP-connection) macro examination specimens were worked out and hardness measurements were accomplished. A clear dependency between the welding speed and the hardness could be observed. Furthermore, differently to what was expected, a quite ductile behavior for all filler metal strength could be observed.

After the tests, the throat thickness was measured by three different methods. The method "reverse engineering", where the fracture area was scanned by an optical system, turned out to be the most detailed one for further investigations concerning the load carrying capacity. Another benefit of this procedure was the possibility to identify in which zone of the weld the fracture took place.

By studying the load carrying capacity a clear dependency on the material strength f_u could be observed. Investigations focused on automatically welded specimens indicate a much clearer view on the influences. It became clear that the filler metal plays an eminent role, so that the assumption suggested by *Rasche*, (2012) have been validated.

The statistical evaluation according to the developed method for statistical validation of design rules for typical failure driven by fracture led to improved results compared to the standardized procedure according to *EN 1990 Annex D*, (2002).

Because of the higher number and the lower scatter of the test results could be determined better coefficients of variations $V_{f_{u,BM}}$, $V_{f_{u,FM}}$ and $V_{A_{frac}}$ for the statistical evaluation in Task 5.4. Better coefficients for the basic variables lead to better variation coefficients V_r for the resistance function. This would allow that slightly better correlation coefficients $\beta_{w,FM}{}^{Kl}$ for the adjusted statistical procedure which led up to 3.7% better design resistances than the correlation coefficients according to *Rasche*, (2012). The applicability of the modified design resistance according to *Rasche*, (2012) for mixed connections could be confirmed due to the fact that the results determined with the correlation coefficient by *Rasche*, (2012) are slightly on the "safe side".

8 Work package 6 – Design guidance, project management and dissemination

8.1 *Objectives of WP6*

The objectives of this Work Package were as follows:

- To manage and coordinate the project and maintain adequate lines of communication between all the partners involved in the project in order to achieve the project objectives within the time and budget allocated
- To prepare the semester, mid-term and final project reports and financial statements.
- To collect the outcome e.g. proposals of new rules and transfer it to a format that can be put into code language
- To prepare a workshop to be held at the end of the project to publicise the developed safety assessment procedures.

8.2 *Work undertaken and results obtained*

The work in WP6 was divided into 3 Tasks:

- Task 6.1 – Project Coordination – a continuous task related to the project management;
- Task 6.2 – Design Guidance Report which summarizes the outcomes from the safety assessment;
- Task 6.3 – Workshop – it was organized in the last semester of the project in order disseminate the project results.

The work undertaken in Tasks 6.1 to 6.3 is described the following sections.

Task 6.1 – Project coordination

Project meetings were organized every six months by one of the project partners:

- July 2013 – Coimbra, Portugal;
- December 2013 – Brussels, Belgium;
- July 2014 – Eindhoven, The Netherlands;
- December 2014 – Esh sur Alzette, Luxembourg;
- July 2015 – Stuttgart, Germany;
- December 2015 – Graz, Austria;
- April 2016 – Coimbra, Portugal;
- May 2016 – Workshop, Timisoara, Romania;
- December 2016 – Stuttgart, Germany (Figure 8.1);

The meetings allowed for better communication between the project partners. During each meeting a work plan for the next period was prepared. The project consortium had active web page on the ECCS website (*https://www.steelconstruct.com/site/*).

All reporting documents were prepared in due time coordinated by UC with the active collaboration of all partners.

Figure 8.1: Lab visit during the last SAFEBRICTILE meeting in Stuttgart, Germany

Task 6.2 – Design guidance report

In this task it was aimed to give a clear guideline for the assessment and development of design rules in steel structures as the outcome of WP1 being applied and further developed by Work packages 3, 4 and 5.

Although Work packages 3, 4 and 5 came out with real proposals of new rules for modes driven by plasticity, stability and fracture, in WP6 these proposals were collected and transferred to a format that can be put into code language.

As a result, a design guideline was prepared in the scope of Task 6.2. The document summarizes the developments done for the design rules treated within the scope of the project in a systematic way. For each design rule in the scope of the failure modes tackled in the project, firstly the possible issues related to the design rule are discussed, furthermore amendments in the existing rules, new rules or the satisfactory status of the design rules were proposed, and finally the background information for these proposals were summarized.

The design guidance report is Deliverable D6.2.

Task 6.3 – Workshop

A workshop was prepared in order to disseminate project results to a broader audience. It was held within a parallel session of the International Colloquium on Stability and Ductility of Steel Structures – SDSS 2016 in Timisoara, Romania (www.ct.upt.ro/sdss2016) on 31 May 2016. The workshop program is summarized in Table 8.1.

During the workshop, the key results achieved during the project were presented by representatives from each partner institution. The workshop was attended by roughly 40 participants from 20 different countries, among who experts involved in code drafting such as the appropriate TCs of ECCS and evolution groups of CEN/TC250/SC3, which led to an interesting debate between speakers and attendees. The discussion was further extended outside Europe by prof. Richard Liew from the National University of Singapore, who added his contribution to the workshop, presenting the perspectives of large-scale buildings using Eurocodes.

Figure 8.2: Photo during the workshop

Table 8.1: Workshop programme

15h00 -15h30	30 min.	**Overview of SAFEBRICTILE project** (*Luís Simões da Silva, University of Coimbra, Portugal*)
15h30 - 16h00	30 min.	**Guideline for the safety assessment of design rules for steel structures in line with EN 1990** (*Andreas Taras, ECCS, Belgium*)
16h00 - 16h30	30 min.	**Conceptual development of a platform for the collection and maintenance of a European Database** (Nicoleta Popa, *ArcelorMittal, Luxembourg*)
16h30 - 17h00	30 min.	**Design of large-scale steel buildings using Eurocodes - an international perspective** (Richard Liew, University of Singapore, Singapore)
17h00 - 17h30	**30 min.**	**Coffee break**
17h30 - 17h55	25 min.	**New design methods 1 – Modes driven by plasticity** (H.H. Snijder, *Eindhoven University of Technology, The Netherlands*)
17h55 - 18h20	25 min.	**New design methods 2 – Modes driven by stability** (Trayana Tankova, *University of Coimbra, Portugal*)
18h20 - 18h45	25 min.	**New design methods 3 – Modes driven by fracture** (Jennifer Spiegler, *University of Stuttgart, Germany*)
18h45 - 19h00	15 min.	**Safety assessment of the EC3 design rules** (*Luís Simões da Silva, University of Coimbra, Portugal*)

A collection of the workshop presentations was prepared. It is available on the ECCS webpage (*https://www.steelconstruct.com/site/*) for free download.

9 Main conclusions of the project

The research work, carried out in the context of this RFCS contract RFSR-CT-2013-00023 SAFEBRICTILE, had as a main objective the harmonization of the reliability level of design rules for steel structures covering modes driven by ductility, stability and fracture.

The work was distributed into six work packages. In a first step the safety assessment procedure was developed in WP1. A harmonized safety assessment procedure on the basis of EN 1990 was made available, which expands and clarifies the application of EN 1990 Annex D to the assessment of design rules for steel structures across different failure modes. The following key contributions are highlighted:

- Methods for the reduction of the calculated model error parameters b and V_{δ}, i.e. the division of the experimental data into sub-sets and the method of tail approximation.
- "Acceptance levels" for deviations between the calculated values of γ_M^* and existing (or desired) "target" values of partial factors γ_{Mx} (γ_{M0}, γ_{M1}, γ_{M2}…).
- The use of numerical experiments in lieu or in addition to physical tests in the laboratory, and requirements and limits for their application.
- The type and content of documentation reports needed for an independent evaluation (for example by code committees).
- A methodology to achieve target values of partial factors for existing and new design rules.

In WP2, it was aimed to collect sufficient amount of data in order to obtain realistic statistical distributions of the basic variables relevant to steel structures. Data was collected from the AMBD plants, data collection from the experiments performed in the laboratories of the partners as well as from other European universities. Data was also collected from previous statistical characterizations from the literature which were considered representative. A database was developed and is available that contains all collected data. Finally, the statistical characterization was based on the results obtained from all these sources, basing the conclusions on more than 28 000 results collected for S235, S355 and S460 as well as for the geometrical properties of H and I sections.

In WP3, the modes driven my plasticity were investigated. The assessment covered:
- moment-shear (M-V) interaction of I-shaped sections;
- net cross-section;
- moment-normal force (M-N) interaction of I-shaped sections, and;
- moment-normal force (M-N) interaction of rectangular hollow sections (RHS).

The following key findings are highlighted:
- Experiments revealed that all considered failure modes in deed were driven by plasticity, the yield stress being the governing material property.
- The experiments, in combination with numerical analyses, showed that strain-hardening has a substantial contribution to cross-sectional resistance, in cases without normal forces or in short columns ($\lambda_{rel} < 0.15$). However, if normal forces are present and (local) buckling gets influence, the positive effect of strain-hardening disappears.
- Existing and newly proposed design rules were validated against these databases to evaluate the partial factor belonging to the respective design rules.
- For moment-shear interaction of I-shaped sections, it was shown that the current design rule of EN 1993-1-1 is inadequate. The formula for the shear area needs to be adapted and a new design rule for moment-shear interaction is required.
- For net cross-section, the research shows that the reduction factor 0.9 in the current design rule can be omitted making the design rule less conservative. Alternatively, the partial factor can be relieved.
- For moment-normal force interaction of I-shaped sections and rectangular hollow sections, a modified design rule was proposed, which better describes the moment-normal force interaction.
- The newly proposed design rules for moment-normal force interaction of I-shaped sections and rectangular hollow sections are such that they have adequate safety with a partial factor of 1.0.

In WP4 focus was given to the stability failure modes. The aim of this part of the project was to contribute towards achieving transparent, simple and straight-forward unified stability verification procedures. For that, focus was firstly given to the application of the safety assessment procedure to the existing stability design rules in EN 1993-1-1 thus assessing the current safety level of:

- Uniform members in compression;
- Uniform members in bending;
- Uniform members in bending and compression
- General method for lateral and lateral-torsional buckling of structural components

Based on the results obtained the stability verifications were extended to non-uniform members. The following key contributions are highlighted:

- The buckling curves for S460 (minor axis flexural buckling) did not exhibit the same level of safety as all other cases in terms of partial factors calculated using the procedure of WP1 with the distributions from WP2. It was recommended to amend the imperfection factors for flexural buckling minor axis and steel grade S460.
- The methods for lateral-torsional buckling of beams were also assessed. Revealing some conservative results for the general case, unsafe results for the special case and good agreement between numerical and theoretical estimates for the modified general case (GC/f) and the new design rule by Taras;
- The interaction formula for the verification of members under bending and compression resulted was assessed and considered to comply with the safety.
- The general method from clause 6.3.4 was applied to a large number of non-uniform cases. The results exhibited high scatter without any trend which could be adopted for the improvement of the method.
- A new method for the verification of non-uniform columns and beams was proposed. It is applied as an interaction equation for various locations along the member length. It combines the first and second order forces in order to calculate the most unfavourable location along the member length. The advantage of the method relates to the fact that it does not need to consider an equivalent simply-supported member and it is therefore able to deal with a larger set of situations.
- A new method for the verification of non-uniform beam-columns was developed. It was based on the developed rules by Marques et al. (2012) and (2013). It was validated for a large number of numerical simulations showing very good agreement for the studied cases.

In WP5, the modes driven by fracture were studied. One of the main objectives was to develop recommendations for the statistical evaluation of failure modes driven by fracture, based on experimental testing for design, exemplified for fillet welded connections. As a second main objective was giving more detailed design rules for welded dual-steel connections composed of two different steel grades, Mild Carbon Steel and High Strength Steel and a range of different filler metals. The following key findings are highlighted:

- A clear dependency between the welding speed and the hardness could be observed. Furthermore, differently to what was expected, a quite ductile behavior for all filler metal strength could be observed.
- By studying the load carrying capacity a clear dependency on the material strength f_u could be observed. It became clear that the filler metal plays an eminent role, so that the assumption suggested by Rasche, (2012) have been validated.
- The statistical evaluation according to the developed method for statistical validation of design rules for typical failure driven by fracture led to improved results compared to the standardized procedure according to EN 1990 Annex D, (2002).
- Because of the higher number and the lower scatter of the test results could be determined better coefficients of variations $V_{f_{u,BM}}$, $V_{f_{u,FM}}$ and $V_{A_{frac}}$ for the statistical evaluation in Task 5.4. Better coefficients for the basic variables lead to better variation coefficients V_r for the resistance function. This would allow that slightly better correlation coefficients $\beta_{w,FM}{}^{KI}$ for the adjusted statistical procedure which led up to 3.7% better design resistances than the correlation coefficients according to Rasche, (2012). The applicability of the modified design resistance according to Rasche, (2012) for mixed connections could be confirmed due to the fact that the results determined with the correlation coefficient by Rasche, (2012) are slightly on the "safe side".

10 Exploitation and impact of the research results

10.1 *Publications resultant from Safebrictile*

At the moment of submission of the draft Final Report, the following publications reflect the work done within the project.

Conference papers:

- Dekker, R.W.A., Snijder, H.H., Maljaars, J. (2017). *Bending-shear interaction of steel I-shaped steel sections – Statistical investigation*, the 8th European Conference on steel and composite structures, Eurosteel 2017, 13-15 September 2017, Copenhagen, Denmark. (submitted)
- Simões da Silva, L., Tankova, T, Marques, L., Kuhlmann, U., Kleiner, A., Spriegler, J., Snijder, H.H., Dekker, R.W.A., Taras, A., Popa, N., (2017). *Safety Assessment across Modes Driven by Plasticity, Stability and Fracture*, Eurosteel 2017, 13-15 September 2017, Copenhagen, Denmark. (submitted)
- Snijder, H.H., Dekker, R.W.A., Teeuwen, P. (2017). *Net cross-section failure of steel plates at bolt holes - numerical work and statistical assessment of design rules*, the 8th European Conference on steel and composite structures, Eurosteel 2017, 13-15 September 2017, Copenhagen, Denmark. (submitted)
- Dekker, R.W.A., Snijder, H.H., Maljaars, J. (2016). *Numerical investigation into strong axis bending-shear interaction in rolled I-shaped steel sections*, The International Colloquium on Stability and Ductility of Steel Structures, 30 May- 1 June 2016, Timisoara, Romania.
- Kuhlmann, U., Spiegler, J., Kleiner, A., (2016) "*Tragfähigkeit von Msichverbindungen normal- und höherfester Stähle im Stahlbau*", DVS Congress 2016, pp 210-215, Leipzig (in German).
- Spiegler, J., Kleiner, A., Kuhlmann, U., (2016) "*Innovative High Strength Steel Construction by Mixed Connections*", 19th IABSE Congress, Stockholm.
- Tankova, T., Marques, L., Simões da Silva, (2016). *Towards a general methodology for the stability design of steel members*, Proc. International Conference on Steel and Aluminium Structure, Hong Kong, China, 7-9 December
- Tankova, T., Marques, L., Simões da Silva, (2015), *Development of a new methodology for the stability design of steel members*, Proc. of X Congresso de Construção Metálica e Mista, 26-27 November, Coimbra, Portugal
- Dekker, R.W.A., Snijder, H.H., Maljaars, J. (2015). *Experimental Study into Bending-Shear Interaction of Rolled I-shaped Sections. In M. Heinisuo & J. Mäkinen. (Eds.)*, *T*he 13th Nordic Steel Construction Conference (NSCC-2015), 23-25 September 2015, Tampere*, Finland,* pp. 115-116 (and full 10 pages paper on USB-stick). Tampere: Tampere University of Technology, Department of Civil Engineering.
- Kuhlmann, U., Kleiner, A., Schmidt-Rasche, C., (2015) "*Tragfähigkeit wirtschaftlicher Schweißverbindungen von höherfesten Baustählen*", 12. Stahl-Symposium, Ehingen (in German).
- Simões da Silva, L., Marques, L., Tankova, T.,(2015), *On the safety of stability design rules for steel members*, Proc. Eight International Conference on Advances in Steel Structures, Lisbon, Portugal, July 22-24, 2015
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10.2 *Online database platform*

The database is available at https://www.steelconstruct.com/site/.

10.3 *Workshop*

Project workshop was held in order to disseminate the project results to a broader audience.

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Acronyms and Abbreviations

BM	Base metal
CHS	Circular hollow section
CoV	Coefficient of Variation
DT	Displacement Transducer
EHS	Elliptical hollow section
FM	Filler metal
FEM	Finite Element Method
GC	General case, clause 6.3.2.2 of Eurocode 3
GC/f	General case divided by factor f from clause 6.3.2.3 of Eurocode 3
GM	General method, clause 6.3.4 of Eurocode 3
GMNIA	Geometrically and materially non-linear analysis with imperfections
HMP	Hardness Measurement Point
HSS	High strength steel
MCS	Mild carbon steel
MNA	Materially Non-linear Analysis
RHS	Rectangular hollow section
SC	Special case, clause 6.3.2.3 of Eurocode 3
WP	Work Package

Notations

Lowercase Latin letters

a0, a, b, c, d	Class indexes for buckling curves according to EC3-1-1
a	Throat thickness
$a_{3D-Scan}$	Derived throat thickness from the 3D-model of the fillet weld's fracture
a_{EC}	Throat thickness with deep penetration
a_γ	Auxiliary term to the taper ratio for application of LTB proposed methodology
b	Correction factor
b	Width of the section
d_0	Bolt hole diameter
e	End/edge distance
e_0	Maximum amplitude of a member imperfection
h	Cross section height
h_{max}	Maximum cross section height
h_{min}	Minimum cross section height
$h_{xc^{II},lim}$	Cross section height at $x_{c,lim}^{II}$
f_y	Yield strength
f_u	Ultimate tensile strength
$f_{u,act}$	Actual ultimate tensile strength
$f_{u,BM}$	Ultimate tensile strength of base metal
$f_{u,FM}$	Ultimate tensile strength of filler metal
k_{yy}, k_{zy}, k_{yz}, k_{zz}	Interaction factors dependent of the phenomena of instability and plasticity involved
l_{eff}	Weld length
m	utilization ratio for the bending moment
n	utilization ratio, used for shear force as well as normal force
n	Number of cases
p_1	pitch distance in the bearing direction
p_2	pitch distance transverse to the bearing direction
$r_{e,i}$	Theoretical resistance
$r_{t,i}$	Experimental resistance
$r_{t,nom}$	Nominal resistance
r_d	Design value of resistance
t	Thickness of plate or RHS section
t_f	Thickness of the flange
t_w	Thickness of the web
$t_{8/5}$	Cooling time
u	Displacement of the specimen
v_s	Welding speed
$\bar{x}_{a_{EC}}$	Mean value for a_{EC}

$x_{c,lim}^{II}$	Second order failure cross section for a high slenderness level
$x_{c,N}^{j}$, $x_{c,M}^{j}$, $x_{c,MN}^{j}$	Denomination of the failure cross section (to differentiate from the type of loading it refers to): N – do to axial force only; M – due to bending moment only; MN – due to the combined action of bending moment and axial force
x_c^I	First order failure cross section
x_c^{II}	Second order failure cross section
x_{min}	Location corresponding to the smallest cross section
x-x	Axis along the member
y-y	Cross section axis parallel to the flanges
z-z	Cross section axis perpendicular to the flanges

Uppercase Latin letters

A	Area of a cross-section
A_g	Plastic extension at maximum force
A_{gt}	Total extension at maximum force
A_{min}	Cross section area of the smallest cross section in of a tapered member
A_{net}	Net area of a cross-section
A_t	Total extension at fracture
A_{frac}	Fracture area
A_V	Shear area
A_w	Web area
$CoV_{A_{frac}}$	Coefficient of Variation of A_{frac}
$CoV_{a_{3D-Scan}}$	Coefficient of Variation of $a_{3D-Scan}$
C_m	Equivalent moment factor according to clause 6.3.3
E	Young's modulus, modulus of Elasticity
F_{max}	Maximum measured test force
L	Member length
M	Value of the bending resistance
M_{pl}	Plastic design resistance to bending about one principal axis of a cross-section
M_V	Reduced design value of the resistance to bending moments about one principal axis making allowance for the presence of a shear force
$M_{b,Rd}$	Design buckling resistance moment
M_{Ed}	Design bending moment
$M_{f,Rd}$	Cross section resistance to bending considering the area of the flanges only
$M_{pl,y,Rd}$	Design value of the plastic resistance to bending moments about y-y axis
M_y	Bending moments, y-y axis
$M_{y,Ed}$	Design bending moment, y-y axis
N	Design value of the normal force
$N_{cr,z}$	Elastic critical force for out-of-plane buckling
N_{Ed}	Design normal force
N_{pl}	Plastic resistance to normal force at a given cross section
$N_{pl,Rd}$	Design plastic resistance to normal forces of the gross cross section

$N_{u,Rd}$	Ultimate design resistance to net-section failure
V	Design value of the shear force
V_{pl}	Design plastic resistance of a cross-section subjected to a shear force
$V_{A_{frac}}$	Variation coefficient of the fracture area A_{frac}
$V_{f_{u,BM}}$	Variation coefficient of $f_{u,BM}$
$V_{f_{u,FM}}$	Variation coefficient of $f_{u,FM}$
V_{δ}	Variation coefficient of δ
V_r	Resulting variation coefficient

Lowercase Greek letters

α	Angle of taper
α, α_{EC3}	Imperfection factor according to EC3-1-1
$\alpha_b^{(Method)}$	Load multiplier which leads to the resistance for a given method
α_{cr}	Load multiplier which leads to the elastic critical resistance
$\alpha_{cr,op}$	Minimum amplifier for the in-plane design loads to reach the elastic critical resistance with regard to lateral or lateral-torsional buckling
α_{pl}^M	Load amplifier defined with respect to the plastic cross section bending *Moment*
α_{pl}^N	Load amplifier defined with respect to the plastic cross section axial force
$\alpha_{ult,k}$	Minimum load amplifier of the design loads to reach the characteristic resistance of the most critical cross section
$\beta_{w,FM}$	Correlation coefficient depending on the strength of filler metal
γ_M^*	Partial factor resulting from a statistical evaluation
$\gamma_{M,target}$	Target value for the partial factor, required safety distance
γ_{M0}	Partial factor for resistance of cross-sections (here $\gamma_{M0} = 1.0$)
γ_{M2}	Partial factor for resistance of cross-sections in tension to fracture (here: $\gamma_{M2} = 1.25$)
δ_0	General displacement of the imperfect shape
δ_{cr}	General displacement of the critical mode
ε	Utilization ratio at a given cross section
ε_M^I	Utilization ratio regarding first order bending moment M
ε_M^{II}	Utilization ratio regarding the second order bending moment
ε_N	Utilization ratio regarding the axial force N
η	Generalized imperfection
λ_{rel}	Relative slenderness
$\bar{\lambda}_{op}$	Global non-dimensional slenderness of a structural component for out-of-plane buckling according to the general method of clause 6.3.4
$\bar{\lambda}$	Non-dimensional slenderness
$\bar{\lambda}(x)$	Non-dimensional slenderness at a given position
$\bar{\lambda}_y$	Non-dimensional slenderness for flexural buckling, y-y axis
$\bar{\lambda}_z$	Non-dimensional slenderness for flexural buckling, z-z axis
$\bar{\lambda}_{LT}$	Non-dimensional slenderness for lateral-torsional buckling

$\bar{\lambda}_{LT,0}$	Plateau length of the lateral torsional buckling curves for rolled sections
$\bar{\lambda}_0$	Plateau relative slenderness
φ	Over-strength factor
ϕ	Ratio between αplM and αplN
φ_y, φ_z, φ_{LT}	Over-strength factor for in-plane buckling, out-of-plane buckling, lateral-torsional buckling
χ	Reduction factor
χ_{LT}	Reduction factor to lateral-torsional buckling
χ_{num}	Reduction factor (numerical)
χ_{op}	Reduction factor for the non-dimensional slenderness $\bar{\lambda}_{op}$
χ_y	Reduction factor due to flexural buckling, y-y axis
χ_z	Reduction factor due to flexural buckling, z-z axis
ψ	Ratio between the maximum and minimum bending moment, for a linear bending moment distribution
ψ_{lim}	Auxiliary term for application of LTB proposed methodology
$\sigma_{\Delta a}$	Standard deviation of Δa_i

Uppercase Greek letters

Δ_m	Deformation at maximum force
Δ_u	Ultimate deformation
Δa_i	Difference between $a_{3D-Scan}$ and a_{EC}
Φ	cumulative distribution function (CDF) for the standard normal distribution

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KI-NA-28-906-EN-N

The European design rules for steel structures in EN1993 cover various failure modes and were developed using different methodologies with respect to accuracy and safety. Consequently, the safety level of these rules is neither homogeneous throughout their field of application, nor across the various design rules.

EN 1990 – Annex D offers a procedure for the reliability assessment of resistance functions. However, its application is not straightforward in many cases concerning steel design rules and several additional assumptions are necessary to ensure that a target probability failure is achieved. Moreover, the design rules are calibrated covering certain variability of the relevant parameters, such as material properties, geometric properties and imperfections. It is, therefore, essential to appropriately characterize the statistical distributions of the basic variables in order to properly apply the safety assessment procedure. The research work, carried out in the context of contract RFSR-CT-2013-00023 SAFEBRICTILE, had as a main objective the harmonization of the reliability level of design rules for steel structures covering modes driven by ductility, stability and fracture.

This report gives an overview of the reliability assessment of the Eurocode 3 design rules carried out within the project related to cross-sectional resistance, buckling resistance of columns, beams and beam-columns and weld resistance; it presents the adjustments to these design rules proposed whenever it was necessary; it also establishes recommendations on the statistical distributions of the relevant basic variables for steel structures, compiled in a "European database of steel properties" that was also developed within the project.

Publications Office

ISBN 978-92-79-76999-3
doi:10.2777/76892