Bridges are an integral part of the worldwide traffic infrastructure and long-span bridges especially contribute to mobility and economy of time in travelling. Improvements of the steel-plated cross-sections of steel and composite bridge structures help to enhance the competitiveness of such bridges. Herein the aims are the valorisation and dissemination of the knowledge and results which have been acquired within the preceding RFCS research project 'Competitive steel and composite bridges by improved steel plated structures — Combri' for practitioners with regard to plate buckling verifications. The outcome is the Combri design manual consisting of two parts which provide clearly arranged and concise documents for daily use. Part I 'Application of Eurocode rules' covers two composite bridge structures — a twin-girder and a box-girder bridge — on the basis of worked examples for which the knowledge is written down in a descriptive manner and references are given to current Eurocode rules. Part II 'State-of-the-art and conceptual design of steel and composite bridges' presents the current practice in several European countries and common bridge types as well as unusual bridges for special purposes or development projects. Improvements which can be provided to the design of steel and composite bridges are discussed and the possibilities and restrictions given by the current Eurocode rules are highlighted. In this report, proposals are also formulated to implement the newly gained state-of-the-art knowledge into standardisation via nationally determined parameters (NDP), non-contradictory complementary information (NCCI) and suggestions for the next revision of the Eurocodes.

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European Commission

Research Fund for Coal and Steel

Combri
Valorisation of knowledge for competitive steel and composite bridges

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FINAL SUMMARY

Objectives of the project

The objectives of this project are the valorisation and dissemination of the knowledge and the results which have been acquired within the related preceding RFCS research project "Competitive Steel and Composite Bridges by Improved Steel Plated Structures – COMBRI" [COMBRI2007] with regard to plate buckling verifications in order to promote and to encourage the wider use of steel plated structures in bridges. To achieve this, a design manual is created particularly for practitioners. It consists of two parts: in Part I "Application of Eurocode rules" [6] two composite bridge structures are covered on the basis of worked examples incorporating the advantageous use of ***EBPlate*** software [12] developed in the frame of the COMBRI research project. In Part II "State-of-the-Art and Conceptual Design of Steel and Composite Bridges" [7] the current practice and possibilities as well as restrictions of current Eurocode rules are presented. The aim of the design manual is not only to bring together the knowledge and the practice from different European countries but also to give practitioners ideas for an out-of-the-box thinking.

Another objective, and final conclusive step, is the implementation of the newly gained state-of-the-art knowledge into standardisation via Nationally Determined Parameters (NDP), Non-Contradictory Complementary Information (NCCI) and suggestions for the next revision of the Eurocodes to improve the range of application and bring forward economic advantages. Then, practitioners will benefit in the long run from this common European basis for the design of steel plated structures on an up-to-date level.

Thus, the three major objectives of this project can be summarised as:

- Preparation of the COMBRI Design Manual
- Promotion and dissemination in seminars and workshops
- Formulation of proposals for implementation in standardisation

Comparison of initially planned activities and work accomplished

The work of this project closely followed the original work plan and the distribution of work according to the different work packages was kept. During the coordination meetings minor adjustments to the COMBRI Design Manual were thoroughly discussed and agreed on to better achieve the objectives of the project.

Initially within WP 1 the design example of only one twin-girder composite bridge with a fully-detailed calculation was planned. During the first coordination meeting the partners decided to focus especially on the topics related to COMBRI research project, i.e. plate buckling verifications. For instance, the study about buckling of stiffened plates in pure compression can be valorized only if a box-girder bridge example is carried out. The level of detail of the different verifications to be performed has been adjusted according to the main topics of the COMBRI research project. For instance, reasonable values were chosen for other topics such as fatigue issues, justification of the reinforced concrete slab, weld sizes, without detailed calculation. Finally, the partners decided to design two composite bridge examples - a twin-girder and a box-girder bridge - instead of one as a basis for illustrating the practical use of Eurocode rules and for allowing the quantification of the design options and new application rules developed within COMBRI project.

During the third coordination meeting the partners agreed to change the originally planned title of WP 2 "Conceptual Design of Steel Bridges and Composite Bridges" into "State-of-the-art and Conceptual Design of Steel and Composite Bridges" because a comparison with current practice supports better the highlighting of possibilities and restrictions of current Eurocode rules. Thus, the contents of Part II of the manual have been enhanced with examples on current practice in several European countries and common bridge types as well as unusual bridges for special purposes or development projects in an introductory section.

Description of activities and discussion

During the project runtime the necessary close cooperation between all partners was ensured by four coordination meetings. The intermediate results were reported regularly in the technical reports. More than 70 internal working documents were prepared as contributions to the COMBRI design manual and to the proposals for an implementation in standardisation.

The preparation of the COMBRI Design Manual was accomplished hand in hand between all partners and it allowed taking into account the national experience and knowledge in bridge design. The 18-month runtime which has been applied for in the beginning was absolutely necessary and paid off in the proper preparation of an English reference document and translation of the manual into French, German and Spanish.

Mainly the national partners were responsible for the organisation and marketing of their national dissemination activities at the end of 2008. In several seminars and workshops the design manual and ***EB****Plate* software were disseminated and received a positive feedback throughout. Compliments were received especially for the highlighting of the possibilities and restrictions of current Eurocode rules and the overview on best national practices in the different European countries. The presentation of ***EB****Plate* software was highly appreciated and suggestions for future development were given.

Based on the results from the COMBRI research project, the preparation of the manual and discussions at seminars and workshops, proposals were formulated finally for an implementation of the project developments into standardisation.

Conclusions

This section summarises in short the conclusions which have been drawn from the COMBRI Design Manual and the proposals for implementation of the results into standardisation.

An overview of the bridge types in the participating partner's countries - Belgium, France, Germany, Spain and Sweden – show the current practice in those countries. It can be stated that there are notable differences between the practices of the countries and these differences are to some extent caused by differences between the national design standards but more often they are caused by different traditions and practice. Thus, the solutions presented are intended to serve as inspiration for the conceptual design of new bridges.

EN 1993-1-1 covers steel grades up to and including S460 but EN 1993-1-12 extends the range of permitted steel grades up to S700. It is shown that in most cases such high grades are not feasible. The problem is usually that the fatigue requirements limit the full utilisation of the strength. The grade S460 seems to be the most suitable for normal road bridges and S355 for normal rail bridges. It is also shown that hybrid girders with higher strength in the flange than in the webs are economic in many applications. The large span box-girder from Part I of the manual was redesigned from S355 to a hybrid girder with S460 and S690 and it turned out that the costs of the material was reduced by 10% in the spans and 25% at the piers. In addition, there is a reduction of the fabrication costs.

Flanges as bottom flanges in box-girders are in most cases stiffened and different types of stiffeners are discussed. It is shown that large trapezoidal stiffeners are favourable as they give two stiffened lines for the same welding effort as one open stiffener. Further, their torsional stiffness increases the critical stress and this can be calculated with the ***EB****Plate* software which has been developed in the COMBRI research project. Another topic is the double composite action with both top and bottom flanges being composite which has been used for some large bridges in Germany and France. The design of bridges with double composite action is more complicated than the design of a normal composite bridge so that past experience is summarised and recommendations for design are given.

With regard to webs the focus was on to what extent stiffeners should be used. It is common that transverse stiffeners are used at the locations of the cross bracings of which the transverse stiffeners form a part. The effect of the transverse stiffeners on the resistance of the web is an increase in the shear buckling resistance. However, unless the distance between the transverse stiffeners is very short this effect is small and it does not justify the cost of the stiffeners. The possibility of omitting the transverse stiffeners is discussed. Besides that, longitudinal stiffeners on webs increase the resistance for bending as well as for shear. The economy of using longitudinal stiffeners was studied and if the method with effective cross section in EN 1993-1-5 is applied it is shown that longitudinal stiffeners are not economical for web depths below ca. 4 m. The detailing of longitudinal stiffeners has been discussed as well and the main point is the intersection with the transverse stiffeners. One solution is to use discontinuous stiffeners and another is to put the transverse and the longitudinal stiffeners on opposite sides of the web.

Cross bracings and diaphragms for I-girder bridges and box-girders have the function to prevent lateral torsional buckling and to transfer lateral loads on the girders to the deck. Traditional cross bracings can be of truss type or frame type including transverse stiffeners on the webs. The distance between the cross bracings is typically up to 7 to 10 m in I-girder bridges. Although it is not much material used for cross bracings, from an economical point of view it is important to minimize the man hours for fabrication. This was discussed in terms of eliminating parts and possibly also the transverse stiffeners leading to straightforward solutions. For box-girders, the cross bracings or diaphragms also have the function of preventing cross sectional distortion and in many cases they also support the bridge deck. Therefore the distance between the cross bracings is rather small, typically 4 to 5 m.

The technique of launching bridges is very popular and verification methods have been improved in the COMBRI research project. They allow now the utilisation of quite long loaded lengths and accordingly quite high resistance can be achieved. This may make it possible to launch bridges with parts of the concrete slab or the reinforcement in place. For the twin-girder bridge of Part I of the manual, these two possibilities have been studied and the results are compared. If it is useful to have the concrete slab or the reinforcement already in place, the outcomes of the COMBRI research project are very helpful and may lead to more economic solutions.

The proposals which have been formulated for an implementation into standardisation were classified according to Nationally Determined Parameters (NDP), Non-Contradictory Complementary Information (NCCI) and amendments to be used in the next revision of Eurocode in order to provide a concise background and proposal scheme. The transfer of these proposals to CEN is ensured through the collaborative work of the project members in that committee.

Exploitation and impact of the research results

The proper exploitation and impact of the COMBRI research project is the main objective and actual origin of this valorisation and dissemination project in order to reach a wider audience and to ensure the application of the outcomes of the COMBRI research project.

Within this project, the COMBRI Design Manual has been created, Part I and Part II being a 277-page and 121-page document which is available in English, French, German and Spanish. In Part I "Application of Eurocode rules" two composite bridge structures are covered on the basis of worked examples and the advantageous use of ***EB**Plate* software developed in the frame of the COMBRI research project is shown. In Part II "State-of-the-Art and Conceptual Design of Steel and Composite Bridges" the current practice and possibilities as well as restrictions of current Eurocode rules are presented. Thus, the design manual including worked examples, state-of-the-art and conceptual design issues contributes to a better understanding and assessment of the knowledge gained along with the possibility to highlight relevant topics.

The effective and wide dissemination of the design manual and its contents including ***EB**Plate* software was carried out by the organisation of seminars and workshops in each participating partner's country. The dissemination activities took place according to Table 3-1. The feedback from the seminars and workshops was positively throughout. Compliments were received especially for the highlighting of the possibilities and restrictions of current Eurocode rules and the overview on best national practices in the different European countries. The presentation of ***EB**Plate* software was highly appreciated and suggestions for future development were given.

Within this project, proposals for an implementation of the results into standardisation via Nationally Determined Parameters (NDP), Non-Contradictory Complementary Information (NCCI) and suggestions for the next revision of Eurocode were also formulated to ensure the transfer of knowledge and results in the long run so that a common basis is created from which practitioners can benefit from in the future.

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1 INTRODUCTION

The objectives of this project are the valorisation and dissemination of the knowledge and the results which have been acquired within the related preceding RFCS research project “Competitive Steel and Composite Bridges by Improved Steel Plated Structures – COMBRI” [5] with regard to plate buckling verifications in order to promote and to encourage the wider use of steel plated structures in bridges. Work has been performed on the following topics and is presented as follows:

- Preparation of the COMBRI Design Manual, see Chapter 2
- Promotion and dissemination in seminars and workshops, see Chapter 3
- Formulation of proposals for implementation in standardisation, see Chapter 4

2 COMBRI DESIGN MANUAL

2.1 General

The COMBRI Design Manual is an outcome of the research project RFS-CR-03018 “Competitive Steel and Composite Bridges by Improved Steel Plated Structures - COMBRI” [5] and it has been prepared in the frame of this successive dissemination project. The manual is available in English, French, German and Spanish. The essential knowledge which has been acquired within the RFCS research project to enhance the competitiveness of steel and composite bridges has been incorporated in the COMBRI Design Manual. It was promoted and disseminated in the frame of several seminars and workshops, see Chapter 3. The manual is subdivided into two parts to provide the reader with clearly arranged and concise documents:

► Part I: Application of Eurocode rules, see Section 2.2

In the research project the different national background of each partner how to apply and interprete Eurocode rules was brought together and a European melting pot of background information and general knowledge has been created. In order to maintain this valuable information two composite bridge structures - a twin-girder and a box-girder bridge - are covered in Part I of the COMBRI Design Manual on the basis of worked examples for which the knowledge is written down in a descriptive manner. The examples include references to current Eurocode rules.

► Part II: State-of-the-Art and Conceptual Design of Steel and Composite Bridges, see Section 2.3

The national state-of-the-art in bridge design can be different so that firstly bridge types of the project partners’ countries - Belgium, France, Germany, Spain and Sweden - are introduced. They reflect the current practice in those countries and common bridge types as well as unusual bridges intended to solve special difficulties and some solutions being part of development projects are presented in Part II of the COMBRI Design Manual. Also, improvements which can be provided to the design of steel and composite bridges are discussed and the possibilities and restrictions given by the current Eurocode rules are highlighted.

Moreover, the features of software ***EB**Plate* [12] developed in the research project to determine the elastic critical plate buckling stresses are presented in its contributive application for bridge design.

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2.2 Part I

Part I of the COMBRI Design Manual [6] deals with the calculation of two composite bridges according to the Eurocodes.

In the COMBRI research project [5] the different national background of each partner how to apply and interprete Eurocode rules was brought together and a lot of background information and general knowledge has been created. In order to facilitate the implementation of Eurocodes EN 1993-1-5 “Plated Structural Elements” [13], EN 1993-2 “Steel bridges” [14] and EN 1994-2 “Composite bridges” [15] with regard to plate buckling verifications, it was decided to cover two steel-concrete composite bridges - a twin-girder and a box-girder bridge - in order to present the knowledge with the help of worked examples and in a very descriptive manner. As the examples focus in detail on the application and interpretation of Eurocode rules which are related to plate buckling verifications, the overall view on bridge design cannot be covered. In this context, Figure 2-1 shows how many standards can be involved in the design of a composite bridge.

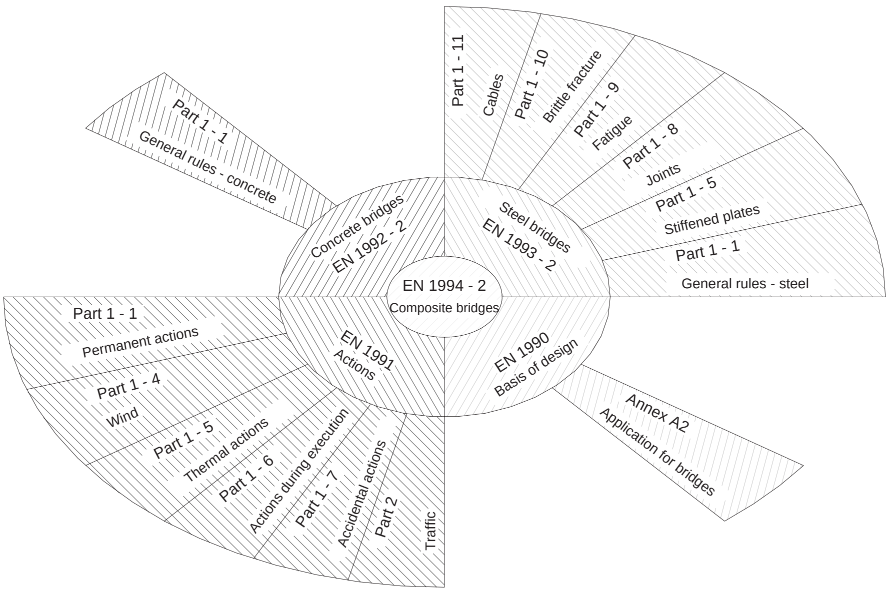


Figure 2-1: Eurocodes to be used in a composite bridge design

In some parts, this design manual introduces general assumptions e.g. on actions without aiming to present the theoretical background or the modelling in detail. In addition to that, it is assumed that the reader is familiar with general design and modelling issues of bridges because this design manual gives a detailed view on plate buckling topics but it does not cover all other topics related to the verification of the design. For further information to the aforementioned topics, the reader is referred to relevant literature.

In the document, the worked examples are presented in a double-sided layout with comments, background information and interpretation issues on the left-hand side and the example calculation on the right. All relevant references to current Eurocode rules are provided. As mentioned above, the examples are a twin-girder and a box-girder bridge which allows looking at a design without and with longitudinal stiffeners.

In chapter 2 the deck of the twin-girder and the box-girder bridge is described and the global analysis of both bridges is introduced. For this purpose, an overview on the bridge geometry, material distribution and construction sequences is given firstly. Secondly, a general section follows in which common data such as material properties and actions as well as combinations thereof are given. Last but not least, the global analysis is presented for both bridges and the relevant results - internal forces and moments - are summarised for the verifications. Based on that, chapters 3 and 4 look at the verifications during the final stage and the execution stage. Here, each chapter is subdivided into a part dealing with the verifications of the twin-girder bridge or the box-girder bridge.

2.3 Part II

Part II of the COMBRI Design Manual [7] deals with the state-of-the-art in several countries and addresses improvements that can be provided to the design of steel and composite bridges. It is focused on the conceptual design of steel bridges and the steel parts of composite bridges. Design of steel bridges is a very wide field which can not be covered completely in the manual so that a selection of topics has been made. A short summary of the content and conclusions is given hereafter:

In chapter 2 of the manual an overview of bridge types in the countries participating in the project is given: Belgium, France, Germany, Spain and Sweden. It reflects the current practice in those countries and presents common bridge types as well as unusual bridges intended to solve special difficulties and some solutions being parts development projects. There are notable differences between the practices of the countries and these differences are to some extent caused by differences between the national design standards but more often they are caused by different traditions and praxis. Thus, the solutions presented are intended to serve as inspiration for the conceptual design of new bridges.

In chapter 3 of the manual the choice of steel grades is discussed. EN 1993-1-1 covers steel grades up to and including S460 but EN 1993-1-12 extends the range of permitted steel grades up to S700. However, in most cases such high grades are not feasible. The problem is usually that the fatigue requirements limit the full utilization of the strength. The grade S460 seems to be the most suitable for normal road bridges and S355 for normal rail bridges. It is also shown that hybrid girders with higher strength in the flange than in the webs are economic in many applications. The large span box-girder from Part I of the COMBRI Design Manual is redesigned from S355 to a hybrid girder with S460 and S690 and it turns out that the cost of the material is reduced by 10% in the spans and 25% at the piers. In addition, there will be a reduction of the fabrication cost as well.

Flanges are dealt with in chapter 4 of the manual and the main topic is bottom flanges in box-girders. Such flanges are in most cases stiffened and different types of stiffeners are discussed. Large trapezoidal stiffeners are favourable as they give two stiffened lines for the same welding effort as one open stiffener. Further, their torsional stiffness increases the critical stress and this can be calculated with the software ***EB**Plate* [12] which has been developed in the COMBRI research project. Another topic is the double composite action with both top and bottom flanges being composite which has been used for some large bridges in Germany and France. The top flange is as usual the bridge deck and the bottom flange has a concrete slab at the piers where the bottom flange is in compression. The design of bridges with double composite action is more complicated than the design of a normal composite bridge so that past experience is summarised and recommendations for design are given.

Webs have been discussed in chapter 5 of the manual with the focus on to what extent stiffeners should be used. It is common that transverse stiffeners are used at the locations of the cross bracings of which the transverse stiffeners form a part. The effect of the transverse stiffeners on the resistance of the web is an increase in the shear buckling resistance. However, unless the distance between the transverse stiffeners is very short this effect is small and it does not justify the cost of the stiffeners. The possibility of omitting the transverse stiffeners is discussed. It should be noted that EN 1993-1-5 does not require any transverse stiffeners except at the supports. Besides that, longitudinal stiffeners on webs increase the resistance for bending as well as for shear. The economy of using longitudinal stiffeners has been studied and if the method with effective cross section in EN 1993-1-5 is applied it is shown that longitudinal stiffeners are not economical for web depths below ca. 4 m. The detailing of longitudinal stiffeners has been discussed as well and the main point is the intersection with the transverse stiffeners. One solution is to use discontinuous stiffeners and another is to put the transverse and the longitudinal stiffeners on opposite sides of the web.

Chapter 6 of the manual covers cross bracings and diaphragms for I-girder bridges and box-girders. Functional requirements are described and ways to meet them are discussed. The main functions are to prevent lateral torsional buckling and to transfer lateral loads on the girders to the deck. Traditional cross bracings can be of truss type or frame type including transverse stiffeners on the webs. The distance between the cross bracings is typically up to 7 to 10 m in I-girder bridges. It is not much material used for cross bracings but from an economical point of view it is important to minimize the man hours for fabrication. This is discussed in terms of eliminating parts and possibly also the transverse stiffeners leading to straightforward solutions. For box-girders, the cross bracings or diaphragms also have the function of preventing cross sectional distortion and in many cases they also support the bridge deck. Therefore the distance between the cross bracings is rather small, typically 4 to 5 m.

Launching has been studied in detail in the COMBRI research project and it is dealt with in chapter 7 of the manual. The technique of launching bridges has been improved and the method is very popular. It is described in some detail including the equipment that is used. At launching the resistance to patch loading is of importance as very high support reactions have to be resisted in combination with high bending moments. This has been studied in the project and it resulted in improved design rules which will be finally proposed for inclusion in EN 1993-1-5. The rules allow the utilisation of quite long loaded lengths and accordingly quite high resistance can be achieved. This may make it possible to launch bridges with parts of the concrete slab or the reinforcement in place. For the twin-girder bridge of Part I of the COMBRI Design Manual, these two possibilities have been studied and the results are compared. If it is useful to have the concrete slab or the reinforcement already in place, the outcomes of the COMBRI research project are very helpful and may lead to more economic solutions.

3 DISSEMINATION ACTIVITIES

3.1 General

The effective and wide dissemination of the design manual and its contents including ***EB**Plate* software was carried out by the organisation of seminars and workshops in each participating partner's country. The dissemination activities took place according to Table 3-1. A short report about the national seminars and workshops can be found in Sections 3.2 to 3.6. The feedback from the seminars and workshops was positively throughout. The presentation of ***EB**Plate* software was highly appreciated and suggestions for future development were given.

3.2 Belgium

For the dissemination activity in Belgium, The University of Liège (ULg) obtained the support of the "Centre Information Acier" (CIA-Infosteel) for the promotion and the organization of the seminar. The one-day workshop was held centrally (Walloon region) at New Hotel de Lives near Namur. The workshop was promoted by e-mail to the subscribers of the CIA and an announcement homepage on CIA website: http://www.infosteel.be. The background of the participants stretched out over administration offices, researchers, engineering offices and construction companies.

Table 3-1: Schedule of the dissemination activities

Country	Date	Place	Number of Participants
Belgium	30/10/2008	New Hôtel de Lives, Namur	24
France	11/12/2008	Centre de conférence BSA, Paris	55
Germany	07/11/2008	Stahl-Zentrum, Deutscher Stahlbau-Verband (DSTV), Düsseldorf	48
Spain	30/06/2008	Sede de la Fundación LABEIN, Derio - Bizkaia	24
	22/09/2008	Universidad de Burgos, Escuela Politécnica Superior Ingeniería de Caminos Canales y Puertos, Burgos	7
	21/10/2008	Plataforma Tecnológica Española del Acero, PLATEA, sede de AITEMIN en Porriño - Vigo	32
	28/10/2008	Colegio de Ingenieros de Caminos, Canales y Puertos, Madrid	144
	21/11/2008	Universidad de Burgos. Escuela Politécnica Superior Ingeniería de Caminaos Canales y Puertos, Burgos	174
Sweden	16/10/2008	Kungliga Tekniska Högskolan, Stockholm	35

To begin the seminar, Jo Naessens (CIA) introduced the program of the day and the different activities which CIA is in charge (as organisation of others RFCS project). First, Professor René Maquoi at the *University of Liège* (*ULg)* spoke about "The objectives of Eurocodes". Then, some general principles on the conception on concrete-steel composite bridge were introduced by Hervé Degée (from *ULg*). After that, two design examples were introduced during the seminar by Hervé Degée and Nicolas Hausoul (from *ULg*): one concerned a composite steel-concrete twin-girder bridge and another one concerned a composite steel-concrete box-girder bridge. A design manual including the 2 design examples were distributed to the participants. To end the seminar, Professor René Maquoi presented the software ***EB****Plate* with some application examples. During the all seminar, some aspects on Eurocode was explained and discussed with the audience.

a) Presentation of one design example

b) Participants

Figure 3-1: Pictures of the workshop in Namur, Belgium

c) Hervé Degée

d) Participants

Figure 3-1 (continued): Pictures of the workshop in Namur, Belgium

3.3 France

For the dissemination in France, the French seminar took place in Paris on the 11th of December 2008. It had been previously promoted by a printed flyer, announcements in several issues of the CTICM bulletin “Construction Métallique Informations”, which is distributed at about 7500 copies, a dedicated webpage on the CTICM website (www.cticm.com) and a general emailing to French engineers involved in steel construction (see Figure 3-2). Finally 55 participants attended the meeting, coming from engineering schools, universities, engineering offices and steel construction companies. Each participant received a printed copy of the two COMBRI+ Guides and a USB key containing the pdf files of the Guides and the ***EB**Plate* software.

The seminar was divided into two parts. The morning session was dedicated to the state-of-the-art and to the standard environment for the construction of steel and composite bridges. A panorama of the most common types of bridges and constructional details used in France has been drawn, also presenting some original solutions as adopted in several European countries. The normative context has been described, showing the progressive introduction of Eurocodes in French standards. The problem of plate buckling and its specific treatment in EN 1993-1-5 has been looked at, and finally the COMBRI project and its outcomes were presented.

The afternoon session dealt with some specific issues in the design of steel and composite bridges and with the solutions recommended by the COMBRI project. It began by a presentation of the ***EB**Plate* software. It was followed by a talk about the use of high strength steel in the bridge construction. An example of application illustrated the economical interest of hybrid girders. The three last lectures were dedicated to important steps in the design of a bridge: the design of the bottom flange of box girders, the launching during the erection and the resistance to lateral torsional buckling of the structure.

The speakers of the seminar were French engineers involved in the COMBRI+ project (L. Davaine, A. Petel, J. Raoul and P-O. Martin) and two invited personalities (Dr. Eng. D. Bitar, head of the Bridges Unit – CTICM and J.M. Vigo from ConstruirAcier, a French organization for the promotion of steel).

The seminar ended by fruitful exchanges between participants and speakers. This was the opportunity to promote several tools available to help designers in the use of the Eurocodes, including the COMBRI+ Guides. An investigation showed that participants were very satisfied of this seminar and of the outcomes of the COMBRI project (Guides, software…).

Figure 3-2: Promotional tools of the French workshop

Figure 3-3: Participants of the workshop in Paris, France

3.4 Germany

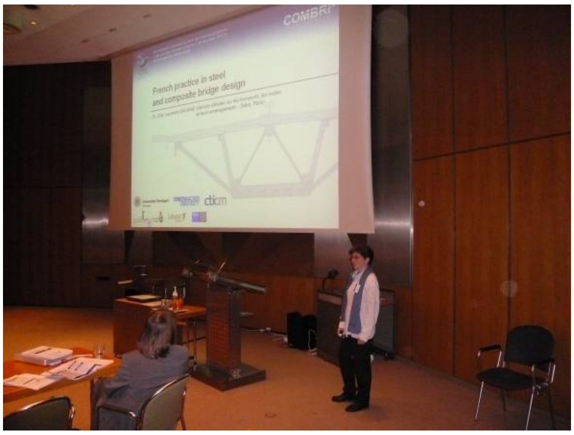
For the dissemination activity in Germany, Universität Stuttgart obtained the support of Deutscher Stahlbau-Verband (DSTV), namely *Fachgemeinschaft Brückenbau*, and the one-day workshop was held centrally at the German Steel Centre in Düsseldorf. The workshop was promoted by mailing list, printed flyer, an announcement homepage, several important websites in Germany and announcements in the German journals “Stahlbau” and “Bauingenieur”. Eventually it could attract not only participants from Germany but also from Austria. Amongst the participants, students, researchers, engineers from engineering offices and construction companies could be found.

For the workshop presentations the support of *Ministerialrat Dipl.-Ing Joachim Naumann* of the *German Federal Ministry of Transport, Building and Urban Affairs* was gained. He spoke about “Eurocodes and their implementation in Germany”. Also *Dr. Eng. Laurence Davaine* of the *Service d'études sur les transports, les routes et leurs aménagements*, Paris, was invited and reported about the “French practice in steel and composite bridge design”.

In the end, compliments were received especially for the highlighting of the possibilities and restrictions of current Eurocode rules and the overview on best national practices in the different European countries.

a) Greetings of host *Dr.-Ing. Volkmar Bergmann*

b) Invited speaker *Dipl.-Ing. Joachim Naumann*



c) Invited speaker *Dr. Eng. Laurence Davaine*

d) Participants

Figure 3-4: Pictures of the workshop in Düsseldorf, Germany

3.5 Spain

LABEIN - Tecnalia has been supported by the Spanish Independent Steel Promotion Organisation, *Asociación para la Promoción Técnica del Acero (APTA)*, in addition, one of the workshops was organised in cooperation with the Spanish Steel Technology Platform, *Plataforma Tecnológica Española del Acero (PLATEA)* and one of the seminars was organized in cooperation with the Technical University of Burgos.

All the dissemination activities were promoted between professionals related to civil works and related to the steel construction. The contacts were done via e-mail and by Internet through the homepage of APTA, PLATEA and LABEIN-Tecnalia.

The dissemination events organized by LABEIN-Tecnalia can be classified in two ways:

a. Workshops to discuss with practicing professionals about the contents developed in COMBRI and disseminated through COMBRI+:
    i. Competitiveness of the composite bridges attending to project, fabrication and construction stages. Derio -Bizkaia.
    ii. Implementation of the Eurocodes for bridge design. Burgos.
    iii. R&D to increase the competitiveness of the steel construction sector for bridges. Vigo.
b. Seminars to disseminate the outcomes of COMBRI and the *Design Manual COMBRI*:

i. Dissemination to practicing civil engineers including lectures performed by acknowledged Spanish civil engineers to demonstrate the competitiveness of the composite bridges. Madrid.

ii. Dissemination to the academy and students of civil engineering including lectures performed by acknowledged Spanish civil engineers to demonstrate the competitiveness of the composite bridges. Burgos.

As main conclusion, the participants acknowledged the *Design Manual COMBRI* as a very useful tool for the efficient implementation of the Eurocodes related to the bridge design.

a) Seminar organised at the *Colegio de Ingenieros de Caminos Canales y Puertos* in Madrid

b) Lecture by Julio Martínez Calzón at the seminar in Madrid. Other keylectures were presented by Javier Manterola, Francisco Millanes, José Romo and Guillermo Capellán

c) Seminar organised at the University of Burgos

d) Workshop organised in cooperation with PLATEA in Vigo

Figure 3-5: Pictures of the dissemination activities organised in Spain

3.6 Sweden

The dissemination of the COMBRI results in Sweden took place at the Royal Institute of Technology in Stockholm. It was originally planned as a half day seminar but it was decided to make it a full day by adding more general information on steel bridges. The seminar was arranged by professor Peter Collin, Ramböll and LTU in cooperation with the Swedish Steel Construction Institute which used its large address register to promote the seminar. The Design Manuals were given to the participants free of charge.

A summary of the two bridge examples from the Part I of the Design Manual was presented by Mr. Jörgen Eriksen from LTU. The content of Part II of the design manual was covered in three different presentations. Professor Peter Collin talked about recent bridges in Sweden and professor Milan

Veljkovic presented what we can learn from other countries. Professor emeritus Bernt Johansson talked about conceptual design of composite bridges with possibilities and restrictions given by the Eurocodes.

Mr. Anders Spåls, Ruukki gave a presentation on fabrication and erection of steel girders for composite bridges with focus on limiting expensive details. After that followed a presentation by Dr. Lars Pettersson, Skanska on the use of composite bridges from a general contractor's view with focus on the competiveness of the composite concept. Finally, Peter Collin informed about some ongoing research projects on bridges including integral abutments using steel piles and full depth concrete elements for bridge decks.

Figure 3-6: Participants of the workshop in Stockholm, Sweden

An additional promotion of the Combri results was undertaken at a course on Eurocodes for bridge engineers held 21 October, 2008, in Borlänge, Sweden. The 30 participants were bridge engineers from the National Road Administration and the National Rail Administration. The Design Manual was handed out to the participants.

Figure 3-7: Participants at course for bridge engineers in Borlänge, Sweden

4 IMPLEMENTATION FOR STANDARDISATION

4.1 General

In the COMBRI research project [5] several aspects concerning an improvement of EN 1993-1-5 [13] have been addressed. In this project, these topics have been prepared for an implementation for standardisation. All topics are structured in such a manner that the affected section is addressed and a point of discussion is given. After that, a proposal is formulated which is supported by a section with background information. In most cases, this background information is available through the COMBRI final report [5]and its background documents. However, in some cases new results became available and this information has been included in this document. In the following the topics are summarized based on their classification as Nationally Determined Parameter (NDP), Non-Contradictory Complementary Information (NCCI) and amendments for the next revision of Eurocode.

The following topic is intended to be addressed as Nationally Determined Parameter (NDP):

- Patch loading resistance of longitudinally stiffened girders, see Section 4.2.1

An overview of the topics addressed in terms of Non-Contradictory Complementary Information (NCCI) is given below:

- Ultimate resistance of longitudinally stiffened plates under uniform compression, see Section 4.3.1
- Effective area of stiffened plates, see Section 4.3.2
- Shear resistance of unstiffened and longitudinally stiffened girder, see Section 4.3.3
- Interaction between shear- and patch loading, see Section 4.3.4

Furthermore the following topics are also foreseen as amendment within a future revision:

- Ultimate resistance of longitudinally stiffened plates under uniform compression, see Section 4.4.1
- Effective area of stiffened plates, see Section 4.4.2
- Shear resistance of unstiffened and longitudinally stiffened girder, see Section 4.4.3
- Patch loading resistance of unstiffened girders, see Section 4.4.4

NOTE: References written in normal type in the boxed sections - e.g. equation numbers and section numbers - refer to current EN 1993-1-5 [13]. However, references written in italic type in the boxed sections correspond to a new numbering and they are used in the explanatory text.

4.2 Proposals for Nationally Determined Parameters

4.2.1 Patch loading resistance of longitudinally stiffened girders

Affected section

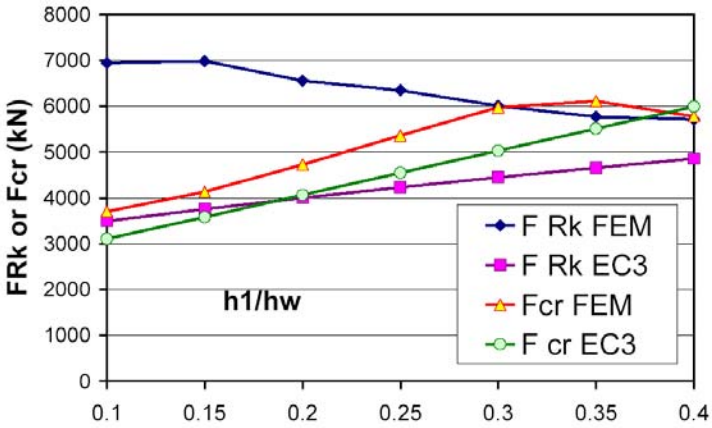
Section 6.4(2), EN 1993-1-5 [13]

"χ–λ approach"

Point of discussion

Section 6.4(2) of EN 1993-1-5 [13] deals with the calculation of the critical load F_{cr} for a longitudinally stiffened web through a note (leaving choice to the National Annex) and with the reduction factor χ_F for the patch loading resistance.

To be efficient for the patch loading resistance, the longitudinal stiffener should be as close as possible to the loaded flange (the EN 1993-1-5 scope imposes $b_1/h_w \leqslant 0.3$). In the design of a launched bridge, the typical location is $b_1/h_w = 0.25$. The failure mode is then a local buckling in the subpanel closest to the loaded flange. However the k_F formula in EN 1993-1-5 corresponds to the first buckling mode of the web which is a buckle in the deepest web subpanel. Consequently if the longitudinal stiffener moves away from the loaded flange, the depth of the non directly loaded web subpanel decreases, the critical load according to EN 1993-1-5 increases and finally the design resistance increases. This is contradictory to the fact that the web is less stiffened for patch loading (the subpanel closest to the loaded flange is deeper). In [8] this contradictory behaviour has also been highlighted through Finite Elements calculations by *Davaine* [8], see Figure 4-1.



Note:

Results for a 8-m-long and 3-m-deep web panel with a single flat longitudinal stiffener located at a distance h_1 from the patch load

Figure 4-1: Contradictory variation of the design resistance F_{Rk} according to EN 1993-1-5 and to Finite Element calculations [8]

To solve this contradiction, a new way for calculating the critical load of a longitudinally stiffened web submitted to a patch load has been proposed in [8]. The same study has also shown that the use of the coefficient m_2 for the plastic resistance F_y was not justified in comparison to FE simulations. This point about F_y has been further studied and confirmed by Gozzi in [19].

If a change is introduced in one of the steps of the χ–λ approach (here the calculation of F_{cr} and of F_y), then a new reduction factor χ_F should be derived to ensure the safety level of the model as a whole. This has been done in [8] by using the format proposed in Annex B of EN 1993-1-5.

Proposal

To 6.4 (2) Note

For webs with longitudinal stiffeners, the critical load F_{cr} may be determined by using the following procedure :

$$k_{F,2} = \left(0{,}8\frac{s_s + 2t_f}{a} + 0{,}6\right)\left(\frac{a}{b_1}\right)^{0{,}6\frac{s_s+2t_f}{a}+0{,}5}$$

$$F_{cr,2} = k_{F,2}\frac{\pi^2 E}{12\left(1-\nu^2\right)}\frac{t_w{}^3}{b_1}$$

$$F_{cr} = \frac{F_{cr,1}F_{cr,2}}{F_{cr,1}+F_{cr,2}}$$

where $F_{cr,1}$ is given by Equation (6.5) using the buckling coefficient k_F according to Equations (6.6) and (6.7).

The reduction function χ_F should be determined from the following equation :

$$\chi_F = \frac{1}{\varphi_F + \sqrt{\varphi_F^{\,2} - \bar{\lambda}_F}} \leq 1{,}0 \quad \text{with} \quad \varphi_F = 0{,}5\left(1+0{,}21(\bar{\lambda}_F - 0{,}8) + \bar{\lambda}_F\right)$$

To 6.5

For determination of the yield load F_y, the effective loaded length ℓ_y should be determined according to Equation (6.10) with $m_2 = 0$.

Background information

The interpolation between $F_{cr,1}$ and $F_{cr,2}$ proposed in [8] relies on the idea to superimpose the first buckling mode as proposed in EN 1993-1-5 (i.e. a buckle in the deepest not directly loaded web subpanel) and the buckling mode in the directly loaded web subpanel as shown in Figure 4-2. This interpolation solves the contradictory behaviour observed in Figure 4-1.

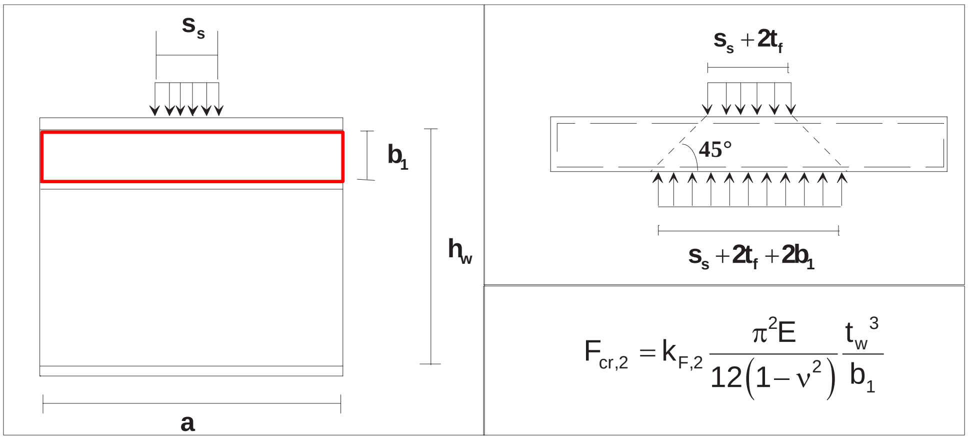


Figure 4-2: Buckling mode in the directly loaded subpanel

An extensive database of numerical simulations can be found in [8] for scanning bridge dimensions and completing the experimental database. These data have been used to calibrate the coefficients $\alpha_F = 0{,}21$ and $\bar{\lambda}_0 = 0{,}8$ in the reduction factor. The format of this reduction factor results from a transition between the plastic design of the web without buckling ($\chi_F = 1$) and a web plate buckling ($\chi_F = 1/\bar{\lambda}$ according to Von Karman hypothesis). By analogy with the European buckling curves, the reduction function is then the solution of the following equation: $(1-\chi_F)(1-\chi_F\bar{\lambda}) = \alpha_F(\bar{\lambda} - \bar{\lambda}_0)\chi_F$ where the coefficients α_F and $\bar{\lambda}_0$ should be calibrated with the experimental and numerical databases.

In the subsequent studies at LTU by Gozzi and Clarin, the interpolation between $F_{cr,1}$ and $F_{cr,2}$ has been improved by using $min(F_{cr,1};F_{cr,2})$ and the suppression of the coefficient m_2 in the yield resistance has been confirmed. This new change in calculating F_{cr} needs then a new calibration of the coefficients α_F and $\bar{\lambda}_0$. The new proposal is detailed in Section 4.4.4.

4.2.2 Patch loading resistance according to Chapter 10

Affected section

Section 10(5), EN 1993-1-5 [13]:

Point of discussion

In section (10.5) the reduction factor ρ_z for transverse patch stresses taking into account plate-like behaviour may be determined according to section 4.5.4 which corresponds to the reduction factor ρ_x for longitudinal stresses. However, in [26] it was shown that in case of column-like behavior the interpolation function used for the determination of the reduction factor ρ_z overestimates the resistance, see Figure 4-3. Thus, a transferability of the interpolation function from longitudinal stresses is not given.

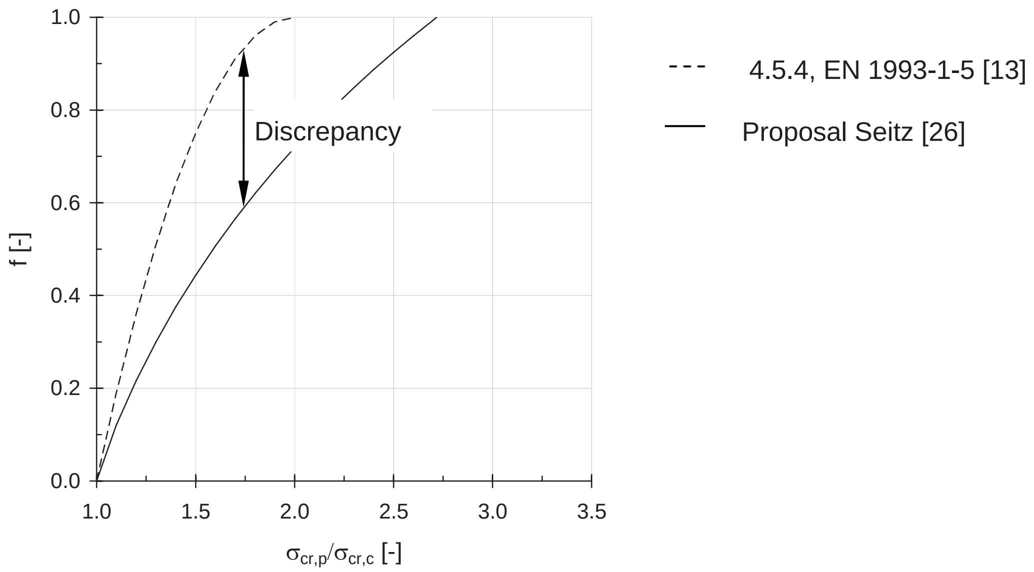


Figure 4-3: Comparison of the current interpolation function according to EN 1993-1-5 and the required interpolation function according to Seitz [26]

Proposal

To 10(5)

The reduction factor ρ may be determined using either of the following methods:

a) the minimum value of the following reduction factors:

[…]

ρ_z for transverse stresses from Annex B.1(3) taking into account column-like behavior where relevant;

[…]

Background information

The discrepancy of the interpolation function can be solved either by defining a new interpolation function or by choosing an appropriate reduction curve. During the revision of the German DIN-Fachbericht 103 [11] it was decided to adopt the reduction curve for transverse stresses according to Annex B, EN 1993-1-5.

4.3 Proposals for Non-Contradictory Complementary Information

4.3.1 Ultimate resistance of longitudinally stiffened plates under uniform compression

Affected section

Section 4.5.4(1) of EN 1993-1-5 [13]:

“Interaction between plate and column buckling”

Point of discussion

The determination of the weighting factor ξ is an extensive calculation procedure as it requires the use of several different sections of EN 1993-1-5 [13], namely Section 4.5.3, Annex A.1, Annex A.2.1 and Annex A.2.2. Formulae for the direct calculation of the weighting factor ξ should be provided.

Particularly in combination with the proposal for revision given in section 4.4.1 of this report, the direct calculation of ξ would lead to a strong simplification and consolidation of the calculation procedure.

Proposal

Two paragraphs are added to Section 4.5.4 of EN 1993-1-5 [13] to allow the direct determination of the weighting factor ξ. The proposal for modification of Section 4.5.4 is as follows:

To 4.5.4

(1) The final reduction factor ρ_c should be obtained by interpolation between χ_c and ρ as follows:

$$\rho_c = (\rho - \chi_c)\,\xi\,(2 - \xi) + \chi_c \tag{4.13}$$

where $\xi = \dfrac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1$ but $0 \le \xi \le 1$

$\sigma_{cr,p}$ is the elastic critical plate buckling stress, see Annex A.1(2) and A.2.2(1), respectively;

$\sigma_{cr,c}$ is the elastic critical column buckling stress according to 4.5.3(2) and (3), respectively;

χ_c is the reduction factor due to column buckling.

ρ is the reduction factor due to plate buckling, see 4.4(1).

(2) For longitudinal stiffened plates the weighting factor ξ may be obtained directly from one of the equations given in 4.5.4(3) and 4.5.4(4).

(3) For orthotropic plates with at least three stiffeners the weighting factor ξ may be obtained from

$$\xi = k_{\sigma,p} \cdot \alpha^2 \cdot \frac{b_{s\ell,1}}{b_c} \cdot \frac{1+\delta}{1+\gamma} - 1 \qquad \text{for } \psi \neq 1 \tag{4.14}$$

and

$$\xi = \frac{\left(1+\alpha^2\right)^2 + \gamma}{1+\gamma} - 1 \qquad \text{for } \psi = 1 \tag{4.15}$$

but $0 \le \xi \le 1$

Note: All parameters $k_{\sigma,p}$, α, $b_{s\ell,1}$, b_c, δ and γ are specified in Annex A.1 and Figure A.1, respectively.

(4) For plates with one or two stiffeners the weighting factor ξ may be obtained from

$$\xi = k_{\sigma,p} \cdot \alpha^2 \cdot \frac{b_{s\ell,1}}{b_c} \cdot \frac{A_{s\ell,1} \cdot t^2}{I_{s\ell,1} \cdot 12 \cdot \left(1-\nu^2\right)} - 1,$$ where $k_{\sigma,p}$ is known from relevant computer simulations *(4.16)*

or

$$\xi = \frac{a^4 \cdot b \cdot t^3}{4 \cdot \pi^4 \left(1-\nu^2\right) \cdot b_1^2 \cdot b_2^2 \cdot I_{s\ell,1}},$$ acc. to Annex A.2 *(4.17)*

but $0 \le \xi \le 1$

Note: The parameters $k_{\sigma,p}$, α, a, b, b_1, b_2, t, ν, $A_{s\ell,1}$ and $I_{s\ell,1}$ are specified in Annex A.2. The geometric values $b_{s\ell,1}$ and b_c, from the stress distribution are specified in Figure A.1.

Background information

The added formulae for the direct determination of the weighting factor ξ are derived by application of Section 4.5.3, Annex A.1, Annex A.2.1 and Annex A.2.2, thus they are non-contradictory to EN 1993-1-5 [13].

The explanation of the used symbols can be taken from EN1993-1-5 [13], if not specified hereafter. In detail the derivations of the formulae are as follows:

Equation *(4.14)* and *(4.15)*:

The elastic critical buckling stress of a plate is given by $\sigma_{cr.p} = k_{\sigma,p} \cdot \sigma_e$. The values for the buckling coefficient $k_{\sigma,p}$ can be taken from buckling charts, relevant computer simulations (e.g. with *EBPlate*) and Annex A.1, respectively. The Euler-stress σ_e is given by

$$\sigma_e = \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot b^2 \cdot \left(1-\nu^2\right)}$$

The elastic critical column buckling stress of the orthotropic plate can be derived by an equivalent Euler-column with the geometric properties of the stiffener and its adjacent parts:

$$\sigma_{c,s\ell} = \frac{\pi^2 \cdot E \cdot I_{sl}}{a^2 \cdot A}.$$

With

$$I_{sl} = \frac{b \cdot t^3}{12 \cdot \left(1-\nu^2\right)} \cdot \left(1+\gamma\right),$$

$$A = \left(A_p + \sum A_{sl}\right) = b \cdot t \cdot \left(1+\delta\right),$$

$$a = \alpha \cdot b$$

the elastic critical column buckling stress reads

$$\sigma_{cr,s\ell} = \left(\frac{\sigma_e}{\alpha^2}\right) \cdot \frac{1+\gamma}{1+\delta} \, .$$

For the final comparison with the elastic critical plate buckling stress $\sigma_{cr.p}$, the critical column buckling stress $\sigma_{cr.s\ell}$ has to be extrapolated to the plate edge according to the stress distribution, cp. NOTE of section 4.5.3(3), EN 1993-1-5 [13]. Hence the elastic critical column buckling stress at the plate edge with the maximum compression is given by

$$\sigma_{cr,c} = \sigma_{cr,s\ell} \cdot \frac{b_c}{b_{s\ell,1}}$$

which can be rewritten to

$$\sigma_{cr,c} = \frac{b_c}{b_{s\ell,1}} \left(\frac{\sigma_e}{\alpha^2}\right) \cdot \frac{1+\gamma}{1+\delta} \, .$$

Thus the resulting weighting factor ξ to obtain the final reduction factor ρ_c reads:

$$\xi = k_{\sigma,p} \cdot \alpha^2 \cdot \frac{b_{s\ell,1}}{b_c} \cdot \frac{1+\delta}{1+\gamma} - 1 \text{ , but } 0 \le \xi \le 1$$

For uniform compression ($\psi = 1$) the buckling coefficient $k_{\sigma,p}$ is given by

$$k_{\sigma,p} = \frac{\left(1+\alpha^2\right)^2 + \gamma}{\alpha^2 \cdot (1+\delta)} \text{ , see also NOTE 2}$$

and the equation for the resulting weighting factor ξ can be simplified to

$$\xi = \frac{\left(1+\alpha^2\right)^2 + \gamma}{1+\gamma} - 1 \text{, for } (\psi = 1)$$

NOTE 1: To get save sided results, the orthotropic plate parameters δ and γ for the determination of the weighting factor ξ should be calculated with the same number of stiffeners, which has been used for the calculation of $k_{\sigma,p}$. Computer programs and buckling charts usually increase the number of stiffeners by one $\left(n_x = n_{stiffeners} + 1\right)$ for the computation of the orthotropic plate, as it leads to an higher consistency between calculations with discrete and smeared stiffeners. This has be taken into account for the determination of ξ.

NOTE 2: Equation (A.2) given in Annex A, EN 1993-1-5 [13], is equal to $k_{\sigma,p} = \frac{\left(1+\alpha^2\right)^2 + \gamma - 1}{\alpha^2 \cdot (1+\delta)}$

Comparative calculations show a discrepancy between Equation (A.2) and numerical results from EBPlate. For $\psi = 1$ this discrepancy can be eliminated completely by erasing the dissenting term "-1".

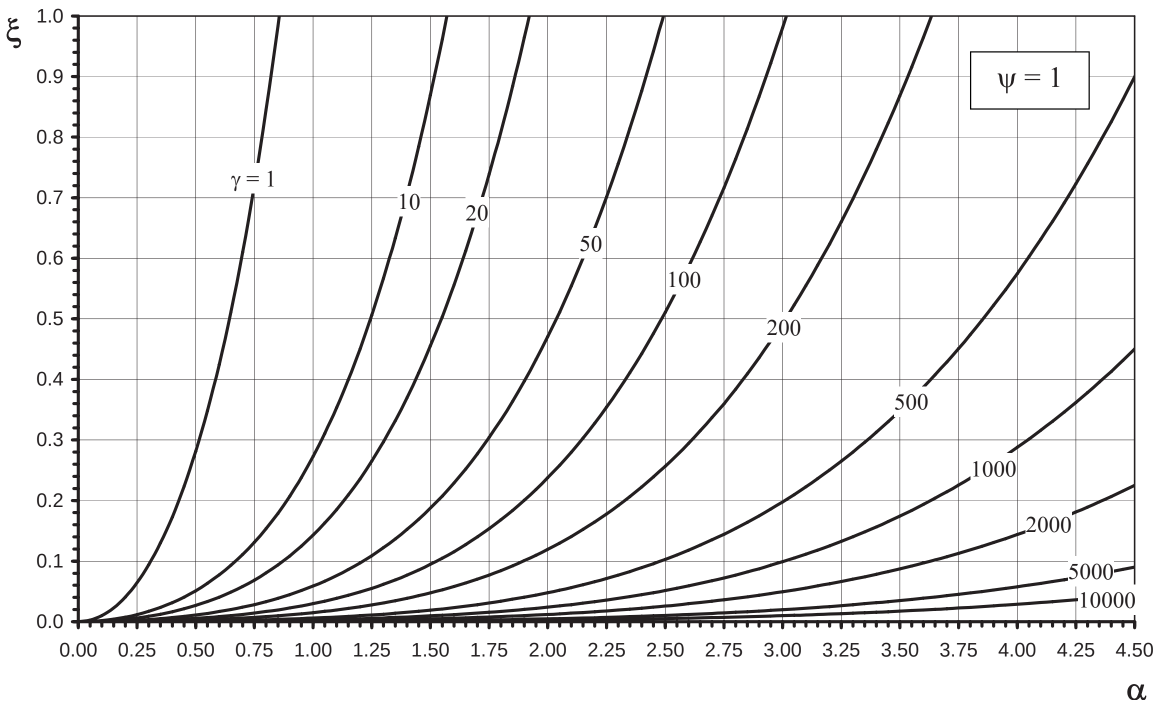


Figure 4-4: Weighting factor ξ in function of the aspect ratio α for pure compression (ψ = 1); Curve parameter: Sum of the relative stiffness γ of the longitudinal stiffeners

Equation *(4.16)*:

The elastic critical buckling stress of a plate is given by $\sigma_{cr.p} = k_{\sigma,p} \cdot \sigma_e$. The values for the buckling coefficient $k_{\sigma,p}$ can taken from buckling charts and relevant computer simulations (e.g. with EBPlate), respectively. The Euler-stress σ_e can whether be determined by

$$\sigma_e = \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot b^2 \cdot \left(1 - \nu^2\right)}$$

or directly be taken from the used computer program.

The elastic critical column buckling stress of the stiffener close to the edge with the highest compression is given by:

$$\sigma_{cr.s\ell,1} = \frac{\pi^2 \cdot E \cdot I_{s\ell,1}}{a^2 \cdot A_{s\ell,1}}$$

where $I_{s\ell,1}$ is the second moment of area of the gross-section of the stiffener and the adjacent parts of the plate, relative to the out-of-plane bending of the plate

$A_{s\ell,1}$ is the cross-sectional area of the stiffener and the adjacent parts of the plate according to Figure A.1, EN 1993-1-5 [13]

NOTE: For plates under bending the width of the adjacent parts has to be determined according to Annex A.2.1.(2) , EN 1993-1-5 [13].

With $a = \alpha \cdot b$ the elastic critical column buckling stress can be written as

$$\sigma_{cr.s\ell,1} = \frac{\pi^2 \cdot E \cdot I_{s\ell,1}}{\alpha^2 \cdot b^2 \cdot A_{s\ell,1}}$$

For the final comparison with the elastic critical plate buckling stress $\sigma_{cr.p}$, the critical column buckling stress $\sigma_{cr.s\ell,1}$ has to be extrapolated to the plate edge according to the stress distribution, cp. NOTE of section 4.5.3(3), EN 1993-1-5 [13].

With the given equations

$$\sigma_{cr,c} = \sigma_{cr,s\ell,1} \cdot \frac{b_c}{b_{s\ell,1}}$$

and

$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1$$

the weighting factor ξ to obtain the final reduction factor ρ_c can be derived to:

$$\xi = k_{\sigma,p} \cdot \alpha^2 \cdot \frac{b_{s\ell,1}}{b_c} \cdot \frac{A_{s\ell,1} \cdot t^2}{I_{s\ell,1} \cdot 12 \cdot (1-\nu^2)} - 1 \text{ , where } 0 \le \xi \le 1$$

Equation *(4.17)*:

In case of a stiffened plate with one longitudinal stiffener located in the compression zone, the elastic critical buckling stress $\sigma_{cr,p}$ can be calculated by a fictitious isolated strut supported on an elastic foundation according to Annex A.2.2, whereas the stiffeners in the tension zone are ignored. For plates with a length of $a \ge a_c$ the weighting factor results to $\xi = 1$, thus the decisive equation for the elastic global plate buckling stress $\sigma_{cr,p}$ for a plate with one stiffener is given by

$$\sigma_{cr,p} = \frac{b_{s\ell,1}}{b_c} \cdot \sigma_{cr,s\ell} = \frac{b_{s\ell,1}}{b_c} \cdot \left(\frac{\pi^2 \cdot EI_{s\ell,1}}{A_{s\ell,1} \cdot a^2} + \frac{E \cdot t^3 \cdot b \cdot a^2}{4 \cdot \pi^4 (1-\nu^2) \cdot A_{s\ell,1} \cdot b_1^2 \cdot b_2^2} \right)$$

with the elastic critical column buckling stress according to Section 4.5.3(3), EN 1993-1-5 [13]

$$\sigma_{cr,c} = \frac{b_{s\ell,1}}{b_c} \cdot \sigma_{cr,s\ell} = \frac{b_{s\ell,1}}{b_c} \cdot \frac{\pi^2 \cdot EI_{s\ell,1}}{A_{s\ell,1} \cdot a^2}$$

and

$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1$$

the weighting factor ξ to obtain the final reduction factor ρ_c can be derived to:

$$\xi = \frac{a^4 \cdot b \cdot t^3}{4 \cdot \pi^4 (1-\nu^2) \cdot b_1^2 \cdot b_2^2 \cdot I_{s\ell,1}} \text{, where } 0 \le \xi \le 1$$

In case of a plate with two longitudinal stiffeners located in the compression zone the elastic critical plate buckling stress should be taken as the lowest of those computed for the three cases $b_1 = b_1^*$, $b_2 = b_2^*$ and $b = B^*$, cp. Figure A.3, EN 1993-1-5 [13].

CAUTION: Expression "$\sigma_{cr,s\ell}$" is used twice in the Eurocode with different meaning: On the one hand for the elastic critical column buckling stress according to Section 4.5.3(3), EN 1993-1-5 [13] and on the other hand for the elastic critical plate buckling stress according to Annex A.2.2, EN 1993-1-5 [13].

4.3.2 Effective area of stiffened plates

Section 4.5, EN 1993-1-5 [13]

"Calculation of the effective[p] area of stiffened plates with longitudinal stiffeners under direct stresses"

Point of discussion

Numerical simulations have shown that the equation (4.5) of EN 1993-1-5 may lead to over-evaluated effective[p] areas and so to unsafe results for plates with weak stiffeners. According to these studies, the results become unsafe when the relative bending stiffness γ of the stiffeners is less than 25.

Proposal

To 4.5

Stiffened plates having weak longitudinal stiffeners should be considered as unstiffened plates regarding their resistance to direct stresses and their effective[p] area should be calculated according to section 4.4 of EN 1993-1-5. Longitudinal stiffeners should be considered as weak stiffeners if their relative bending stiffness γ is less than 25, where γ is defined by:

$$\gamma = \frac{E\, I_s}{b\, D}$$

where E is the Young's modulus

b is the width of the plate

D is the bending stiffness of the plate, defined by $D = \dfrac{E\,t^3}{12\left(1-\upsilon^2\right)}$

t is the web thickness

ν is the Poisson's coefficient

I_s is the second moment of area of the stiffener for out-of-plane bending, its cross-section including a participating width of web of 10 t each side of each stiffener-to-web junction.

Background information

Numerical investigations have been carried out in the frame of WP1 of the COMBRI research project [5], from which an important database has been constituted. Four sets of rectangular plates have been simulated, with two different uniform thicknesses, several lengths and several values of the relative bending stiffness γ of the stiffeners (see Figure 4-5).

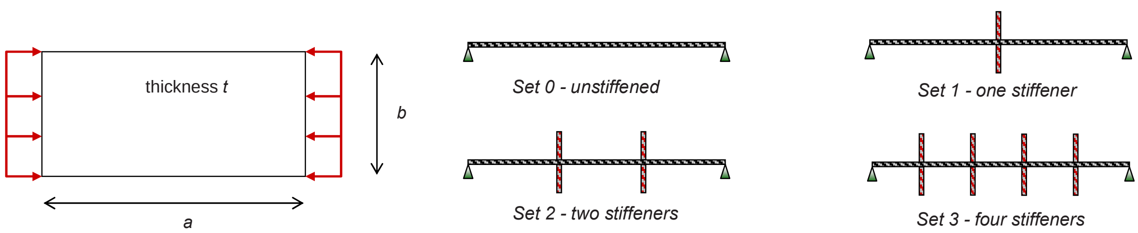


Figure 4-5: Geometries of the plates considered in the numerical simulations

For each case, the final resistance of the plate under pure normal stresses had been determined by performing a non–linear Finite Elements analysis taking into account global and local imperfections according to Annex C of EN 1993-1-5. This resistance has been compared to the resistance calculated according to chapter 4.5 of EN 1993-1-5, more specifically with eq. (4.5).

The comparison shows that the EN 1993-1-5 method may be unsafe (see Figure 4-6) for weak stiffeners. The threshold value of the relative bending stiffness γ of the stiffeners has been evaluated to about 25: for $\gamma > 25$, the EN 1993-1-5 is always safe (except very short plates), and for $\gamma < 25$ it is generally unsafe [23].

This outcome has been verified in the frame of two different ways for assessing $\sigma_{cr,p}$ (cf. 4.5.2 of EN 1993-1-5): either Annex A of EN 1993-1-5 (see Figure 4-6) or the method proposed in the COMBRI project [18] using EBPlate software [12] (see Figure 4-7).

It is to be noted that the calculation of the γ parameter requires taking into account a participating width of the plate, equal to 10 *t* each side of the stiffener-to-web junction (see [24]).

An analysis of the Equation 4-5 of EN 1993-1-5 has been carried out [23] and it has been shown that the model leading to this formulation considers in fact rigid stiffeners. This formulation gives correct results for the resistance where this assumption is verified, but may lead to unsafe results for weak stiffeners. See Section 4.4.2 for a proposal for the improvement of eq. (4.5).

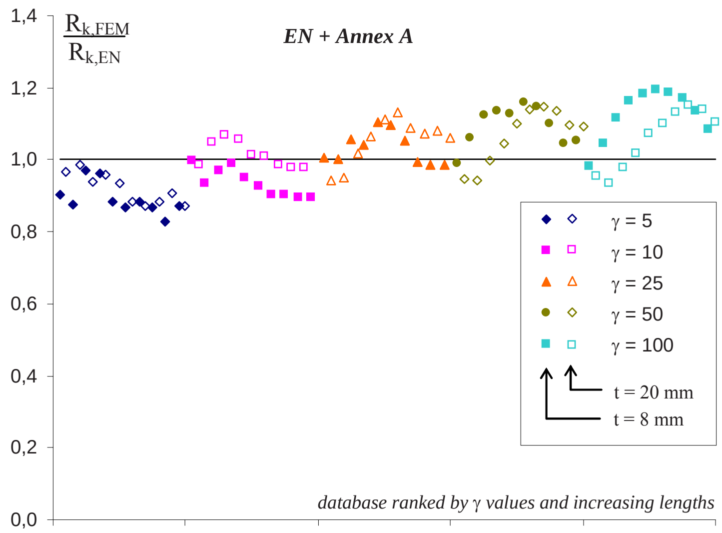


Figure 4-6: Comparisons between numerical simulations and EN 1993-1-5 formulas for the resistance of plates stiffened by one stiffener – $\sigma_{cr,p}$ calculated from Annex A

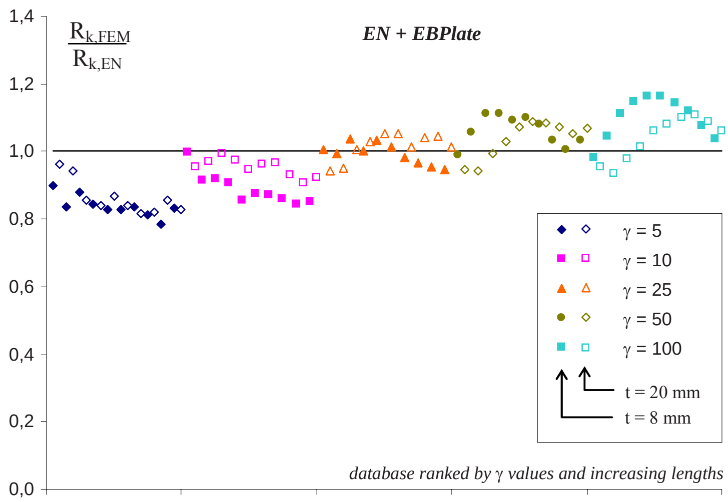


Figure 4-7: Comparisons between numerical simulations and EN 1993-1-5 formulas for the resistance of plates stiffened by one stiffener – $\sigma_{cr,p}$ calculated from EBPlate

4.3.3 Shear resistance of unstiffened and longitudinally stiffened girders

Affected section

Section 5.3(2) to 5.3(4), EN 1993-1-5 [EC3-1-5]

Point of discussion

The proposal deals with the resistance of web panels subjected to shear combined or not with bending and stiffened by one or two longitudinal closed stiffeners. Indeed, though EC3-1-5 is one of the most advanced standards regarding plate buckling, the beneficial effect of closed section longitudinal stiffeners is not explicitly addressed, even if those have a significant positive impact on the stability of sub-panels and on the actual rigidity of transverse stiffeners compared to a design using open section stiffeners.

Experimental tests and numerical simulations performed during the COMBRI project and in complementary studies have shown that the ultimate load provided by tests or simulations is significantly higher than predicted by the codes as soon as shear is concerned (i.e. if the resistance is governed either by shear or by bending-shear interaction).

Proposal

In order to take into account the beneficial effects of closed stiffeners, the following proposals for Non Contradictory Complementary Information are made:

To 5.3(2)

For webs stiffened by closed-section longitudinal stiffeners connected to end posts and vertical stiffeners, end post may always be considered as rigid.

To 5.3(3)

For the assessment of the plate critical buckling stress τ_{cr} of unstiffened webs or of webs stiffened by open section longitudinal stiffeners, simply supported conditions should always be assumed, while for webs stiffened by closed section longitudinal stiffeners, the torsional restraint brought by the flanges and transverse stiffeners may be accounted for.

To 5.3(4)

Due to the high torsional stiffness of closed stiffeners, the reduction of the second moment of area of the stiffeners to 1/3 of their actual value is not required for the calculation of the shear buckling coefficient k_τ.

Background information

This proposal relies on extensive parametric studies based on FE simulations of web-panels with one or two longitudinal closed section stiffeners subjected to bending and shear. Comparisons with application of Eurocode 3 Part 1.5 have been performed, leading to the code improvements proposed above. These proposals are non contradictory since they are dealing exclusively with closed section stiffeners that are not specifically addressed in Eurocode 3.

This study is an extension of investigations performed previously within the ComBri Project [1,2]. FE calculations have been conducted with the software FINELg [17] using shell finite elements and accounting for geometric and material non-linearities as well as initial imperfections. The modelled stiffened panel is composed of two flanges, web, transverse and longitudinal stiffeners. The latter have a trapezoidal cross-sectional shape. Configurations of transverse stiffeners that should be considered either as rigid or non-rigid end-posts are also defined (see Figure 4-8). The constitutive law of the steel material is taken elastic-plastic with strain-hardening. The influence of residual stresses is disregarded since it is known to have only a slight influence on the ultimate strength when shear governs. The panel is supposed to be half of a three-point bending girder situation and the support conditions and loading conditions are adopted accordingly.

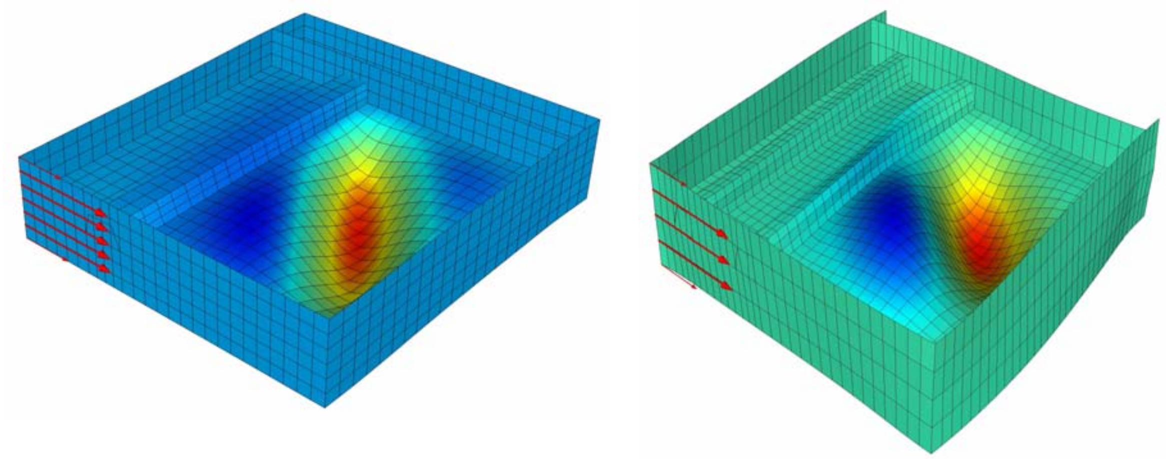


Figure 4-8: Examples of FEM models at ULS (1 stiffener + Rigid end-post; 2 stiffeners + non-rigid end-post)

The different geometric configurations of the parametric study are summarized in Table 4-1. All results provided by the numerical simulations have been compared to those resulting from the application of EN1993-1-5 specifications. In accordance with the latter, the following resistance checks must be performed: bending resistance at mid-span of the member with a gross elastic cross-section (“gross”), bending resistance at a distance of min (0.4 a; 0.5 b) from mid-span (“bending”), shear buckling resistance (“shear”), possible bending-shear interaction at a distance of 0.5 h_w from mid-span (“interaction”) and resistance of the transverse stiffeners (“transverse”). As far as Eurocode results are of concern, the values reported in the following correspond to the minimum of those relative to above-listed criteria.

Results obtained for plates respectively with one or two stiffeners are reported in Figure 4-9. It can be observed that as soon as shear is concerned (i.e. when the governing EN1993 check is either "shear" or "interaction"), the value of the ultimate load provided by the FEM simulation is significantly higher than the value predicted by the code. This had already been evidenced in [1,2] for what regards configurations with one single stiffener and is confirmed in [9] for configurations with 2 stiffeners, with an even more important scattering for the "interaction" criterion.

In order to improve the results obtained with the normative approach, the 3 modifications of the procedure already proposed in [1] for the 1-stiffener configuration have also been applied to the 2-stiffeners configuration in [9]. These modifications can be summarized as follows:

- **Using a dedicated software such as EBPlate [12] for the determination of the elastic critical stress $\sigma_{cr,pl}$ corresponding to the so-called “plate behaviour”.** In contrast with EN1993-1-5, where critical stresses are supposed to be assessed with the assumption of simply supported edges, this software can account for restraining effects provided by both flanges and transverse stiffeners. While the assumption of simple supports is not much disputed for unstiffened web panels or panels stiffened by open cross-section stiffeners, it is found unduly conservative for panels stiffened by closed stiffeners.
- **Adopting the actual stiffener’s inertia for calculating the shear buckling stress of the stiffened panel, instead of 1/3rd of its value as recommended in EN1993-1-5.** This allows accounting for the higher torsional stiffness of the closed trapezoidal stiffeners, since the reduction to 1/3rd proposed by EN1993-1-5 is actually based on investigations on panels with open stiffeners. The proposal is supported by the comparison of FEM results obtained in the presently summarized study [1,9] considering open or close stiffeners.
- **Using the upper shear resistance curve for web contribution (factor χ_w), normally dedicated to rigid end-posts, even in the case of non-rigid end-posts, provided that the longitudinal stiffeners are welded to the transverse stiffeners.** This is justified by the fact that both webs of a usual closed section stiffener stabilize the end-post over a non negligible part of its depth, in such a way that it behaves as rigid.

Figure 4-10 presents a comparison between FEM results and normative assessments once the 3 amendments are applied. The proposed amendments result in a much better quality of the results especially in terms of accuracy. One may also note that the proposed modifications for k_τ are leading to higher failure loads associated with 'interaction' and 'shear' criteria and are thus responsible for the different distributions of failure modes observed when comparing Figure 4-9 and Figure 4-10 in particular for the 2-stiffeners configuration. Indeed, some cases that were assumed to be governed by shear or shear-bending interaction appear finally as governed either by bending or by the resistance of the transverse stiffener.

Table 4-1: Summary of the geometric configurations

	One stiffener – 1st set	One stiffener – 2nd set	Two stiffeners
h_w [mm]	1000 and 2000	2000, 3000 and 4000	4000
t_w [mm]	6 and 12	20	20, 30 and 40
α	1, 3 and 5	1, 1.5 and 2	1, 2 and 4
Stiffener geometry	2 (weak + strong)	2 (weak + strong)	1 (intermediate)
Location of longitudinal stiffeners	$0.3\ h_w$ $0.5\ h_w$	$0.3\ h_w$ $0.5\ h_w$	$0.25\ h_w + 0.5\ h_w$ $0.125\ h_w + 0.375\ h_w$ $1/3\ h_w + 1/3\ h_w$
End post	Rigid + non-rigid	Rigid + non-rigid	Rigid + non-rigid
Resulting number of configurations	384	288	648

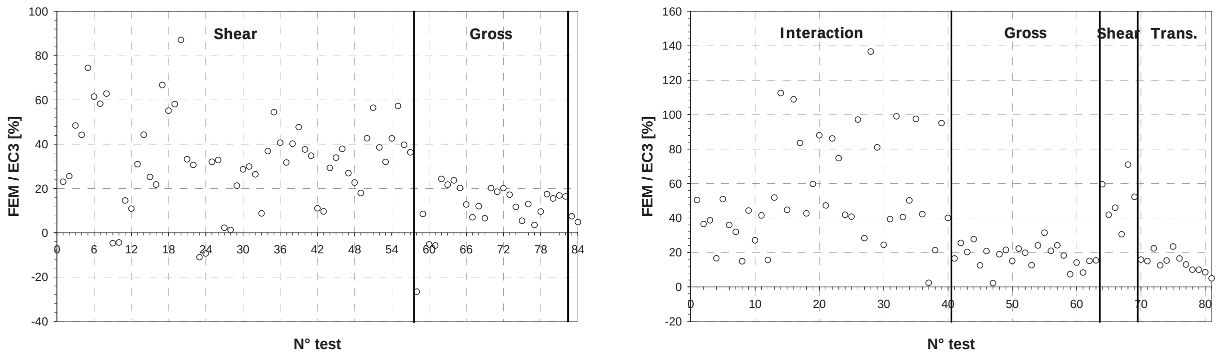


Figure 4-9: Ultimate loads, comparison between application of EN1993-1-5 and FEM results (left: one stiffener, right: two stiffeners)

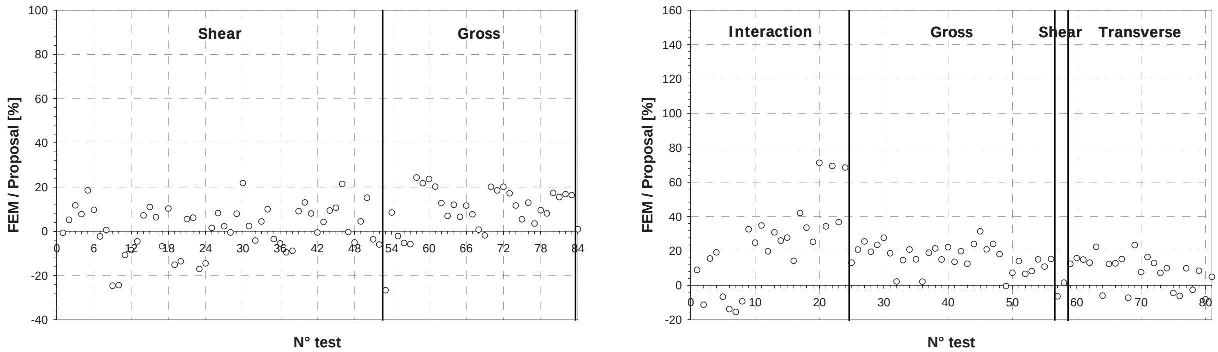


Figure 4-10: Ultimate loads, comparison between application of amended EN1993-1-5 and FEM results (left: one stiffener, right: two stiffeners)

4.3.4 Interaction between shear- and patch loading

Affected section

Section 7, EN 1993-1-5 [13]

“Interaction”

Point of discussion

Currently no formulation for the interaction between transverse force and shear force is given in Section 7, EN 1993-1-5.

Proposal

To 7 Interaction between transverse force and shear force

If the girder is subjected to a concentrated transverse force acting on the compression flange in conjunction with shear force, the resistance should be verified using 4.6, 5.5 and the following interaction expression:

$$\left[\eta_3 \cdot \left(1-\frac{F_{Ed}}{2 \cdot V_{Ed}}\right)\right]^{1.6} + \eta_2 \quad \leq \quad 1.0$$

Background information

Currently no formulation for the interaction between transverse force and shear force is given. All studies on the interaction between shear and patch loading have in common that they subdivide the combined loading into two basic load cases. Thus, the influence of shear stresses caused by the transverse forces can be accounted for because it is already included in the patch loading model. Figure 4-11 shows the basic load cases “pure patch loading” and “pure shear force” which in combination leads to the investigated type of interaction. This subdivision is also the basis on which the interaction equation (7.3) is defined.

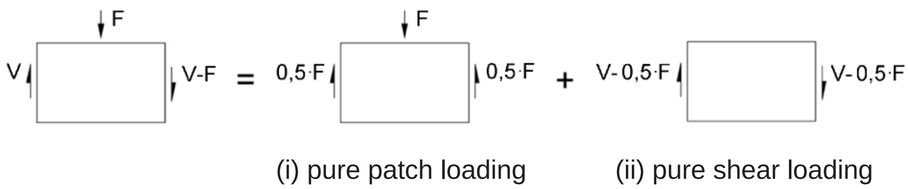


Figure 4-11: Subdivision into basic load cases

In the frame of the COMBRI research project, experimental and numerical studies on steel plated girders have been conducted in order to review and to propose an interaction equation for combined shear and patch loading. Based on the own test girders a Finite-Element model has been created and the own experiments as well as tests from literature were recalculated successfully with the numerical model. The numerical model was then used for parameter studies in which the most important parameters with regard to bridge launching conditions were varied. The behaviour of longitudinally stiffened girders, which may show global or local buckling, is found to be covered within the interaction range of the unstiffened girder cases and a common interaction equation according to Equation (7.3) has been developed,. The statistical evaluation of the proposed interaction equation has been done with the shear resistance model according to Section 5, EN1993-1-5. For the patch load resistance, several models exist which have been developed in order to improve the current formula of Section 6, EN1993-1-5. Here, these models have been evaluated and it is shown that the interaction equation is safe sided for these models not only for unstiffened but also for longitudinally stiffened girders. Figure 4-12 shows this representatively for the improved patch loading resistance models for unstiffened girders according to Gozzi [19] and for longitudinally stiffened girders according to

Davaine [8]. Another approach for longitudinally stiffened girders has been proposed by Clarin [4]. In all cases the applicability of the current resistance models according to EN1993-1-5 and for improved resistance models is proven.

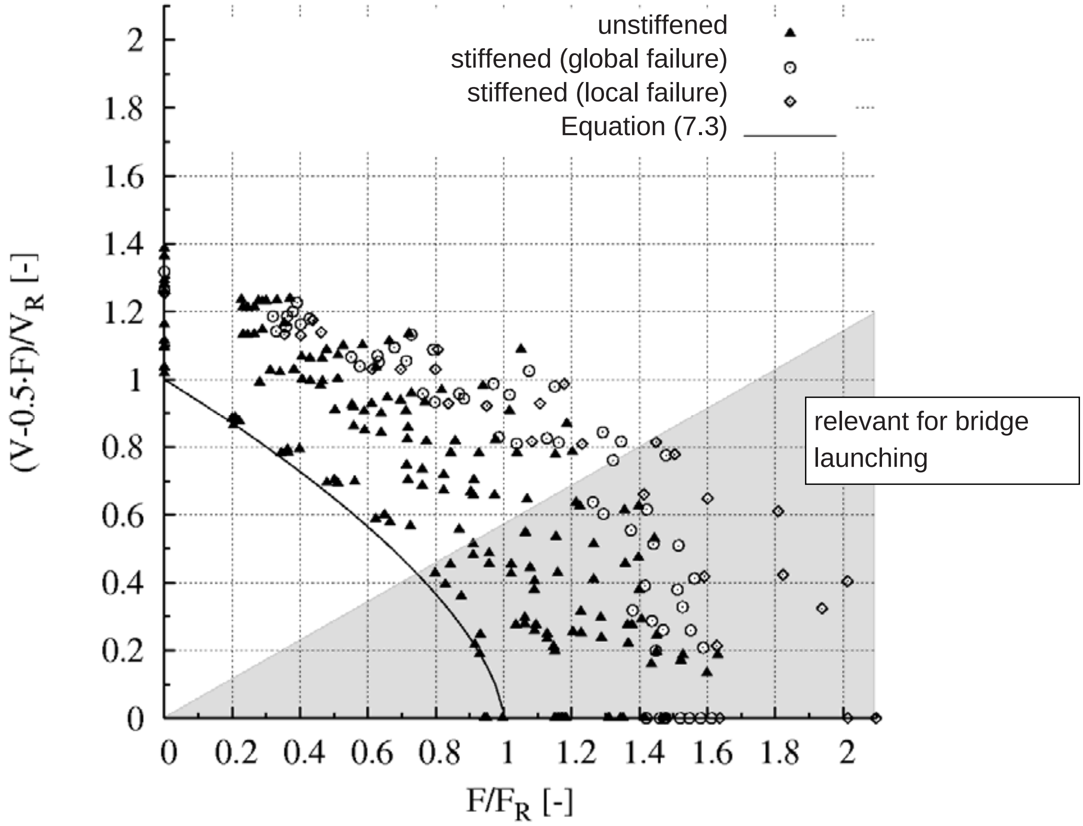
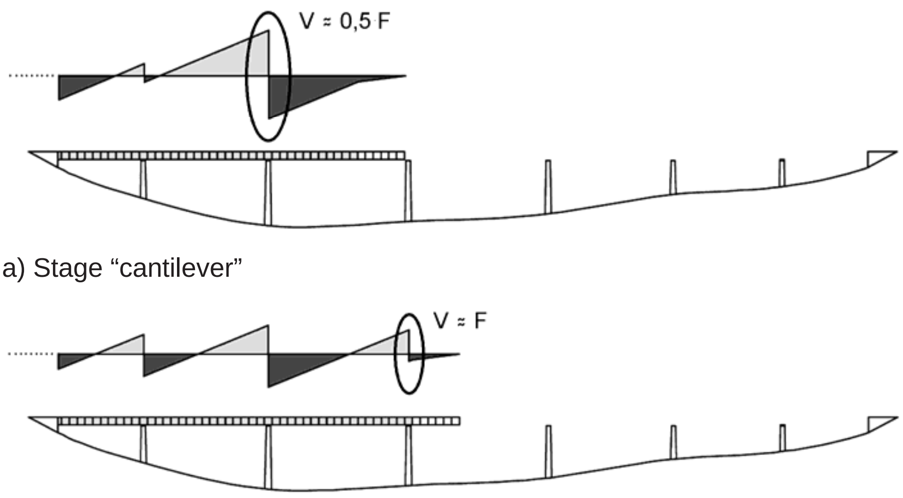


Figure 4-12: Experimental and numerical results compared to proposed interaction equation based on different resistance models ($F_{R,unstiffened}$: Gozzi [19], $F_{R,stiffened}$: Davaine [8] and V_R: EN1993-1-5 [13]).



a) Stage "cantilever"

b) Stage "pier reached"

Figure 4-13: Shear force diagrams for construction stages during bridge launching.

From the investigations, it is obvious that the interaction between shear and patch loading is not negligible. Although the interaction equation and diagram appears severely at first sight, for bridge launching the specific conditions have to be taken into account. For this reason, Figure 4-12 shows two relevant construction stages: a) when the bridge girder is about to arrive at the support and a cantilever exists; b) when the bridge girder has reached the pier. In stage a) the introduced patch load is almost equally equilibrated resulting in a pure patch loading situation where the shear is already considered in the patch load model. In stage b) the maximum internal shear force approximates the value of the applied patch load which leads to an asymmetric patch loading condition. For this situation the interaction becomes relevant, the average will result in reductions of around 10%, see Figure 4-12. However, for the verification of the cross-section the interaction load case "patch loading and bending moment" is decisive.Further background information is given in COMBRI documents [3], [5] and [21].

4.4 Proposals for revision

4.4.1 Ultimate resistance of longitudinally stiffened plates under uniform compression

Affected section

Section 4.5.3(4) and 4.5.3(5), EN 1993-1-5 [13]

"Column type buckling behaviour"

Point of discussion

The use of two different slendernesses $\bar{\lambda}_p$ and $\bar{\lambda}_c$ to determine the reduction factor for plate type behaviour ρ and the reduction factor for column type behaviour χ_c leads to a labour-intensive and non consistent calculation procedure. It is furthermore contradictory to the procedure given in section 10 of EN 1993-1-5 [13], where only one slenderness $\bar{\lambda}_p$ is used for the determination of all reduction factors, cp. Section 10(5) EN 1993-1-5 [13]. For more details see Background information below.

Proposal

Section 4.5.3(4) EN 1993-1-5 [13] is cancelled.

Section 4.5.3(5) EN 1993-1-5 [13] is modified in the following way:

(5) The reduction factor χ_c should be obtained from 6.3.1.2 of EN 1993-1-1. The provided slenderness $\bar{\lambda}$ is calculated from equation (4.7). For unstiffened plates $\alpha = 0{,}21$ corresponding to buckling curve *a* should be used. For stiffened plates its value should be increased to:

$$\alpha_e = \alpha + \frac{0{,}09}{i/e} \tag{4.12}$$

with $i = \sqrt{\frac{I_{s\ell,1}}{A_{s\ell,1}}}$

e = max (e1, e2) is the largest distance from the respective centroids of the plating and the one-sided stiffener (or of the centroids of either set of stiffeners when present on both sides) to the neutral axis of the effective column, see Figure A.1;

α = 0,34 (curve b) for closed section stiffeners;

= 0,49 (curve c) for open section stiffeners.

Background information

The ultimate resistance of a plate under compression is not necessarily limited to its elastic critical buckling stress σ_{cr}, as it is the case for bar-like structures. Due to stress redistributions at the unloaded edges, plates are able to exceed the elastic critical buckling stress limit up to an over-critical value σ_x, cp. Figure 4.14.

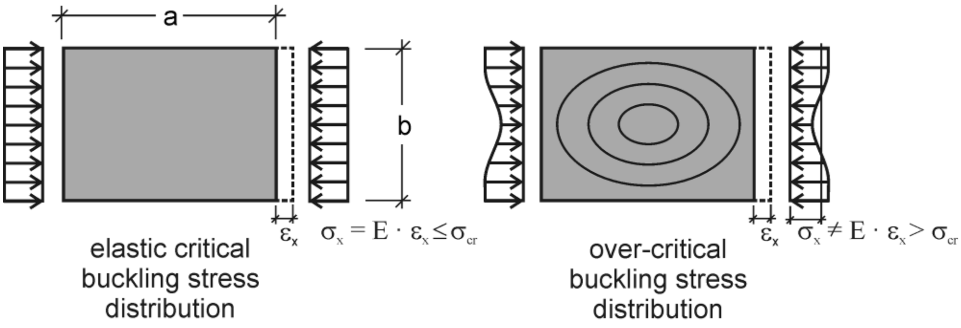


Figure 4.14: Stress distribution for elastic critical buckling and over-critical buckling

The magnitude of this over-critical reserve depends on the capability of the plate to redistribute the load to the supported edges and its initial imperfection. Very wide or strongly stiffened plates are only able to redistribute a minor part of the load. These plates react more column-like and the reduction factor has to be determined via an interpolation between Winter and column-buckling curve.

To determine whether a plate reacts more column or plate-like Section 4 of EN1993-1-5 [13] uses an established procedure, which compares the existing elastic critical buckling stress $\sigma_{cr,p}$ of the plate to those of an equivalent column ($\sigma_{cr,c}$). The closer both values are together, the more column-like is the behaviour of the plate and the column buckling curve has to be applied. Different to other calculation rules (e.g. in Section 10 of EN1993-1-5 [13] and DIN 18800 Teil 3 [10]) Section 4 does not only use the critical buckling stress $\sigma_{cr,c}$ of the equivalent column to determine the weighting factor ξ for the interpolation between the two limits ρ and χ_c but also for the determination of the lower limit χ_c itself. Thus the interpolation does not take place between the two extreme values $\rho(\bar{\lambda}_p)$ and $\chi(\bar{\lambda}_p)$ of the real plate, which is the case for the direct method (cp. green line in Figure 4.15), but between one value $\rho(\bar{\lambda}_c)$ determined with the real plate properties and one value $\chi(\bar{\lambda}_c)$ determined with a not existing fictitious column (red broken line).

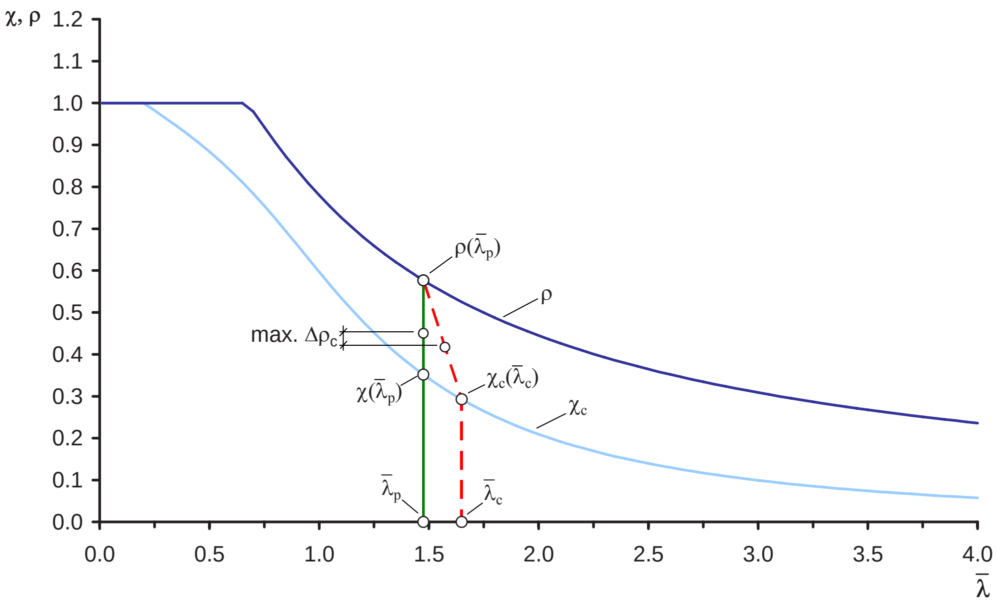


Figure 4.15: Maximum possible difference $\Delta\rho_c$ for an imperfection factor $\alpha_0 \geq 0,34$

This procedure is not only less consistent and contradictory to Section 10, but makes the calculation procedure even more laborious. Furthermore it impairs the efficiency of other improvements as e.g. the ones proposed in section 4.3.1 of this report.

Beside these facts it has to be mentioned that the total effect on the finally determined reduction factor for column-type behaviour ρ_c is smaller than one might expect. The maximum possible difference between the reduction factors ρ_c determined in the two different ways, described above, takes place for a plate buckling slenderness of $\overline{\lambda}_p = 1{,}475$ in combination with an elastic critical column buckling stress of $\sigma_{cr,c} = 0{,}8 \cdot \sigma_{cr,p}$. In this case the difference $\Delta\rho_c$ between the both reduction factors is about 7,36 %. This situation is given in Figure 4.15. In any other case the difference in result between both calculation procedures is smaller. For those cases where the weighting factor ξ is outside its limits $0 \le \xi \le 1$, the difference is 0,0 %.

Figure 4.16 displays the magnitude of the possible differences between both approaches, whereas for each plate buckling slenderness $\overline{\lambda}_p$ every possible combination of $1 \le \sigma_{cr,p}/\sigma_{cr,c} \le 2$ has been checked. Considering the same occurrence probability for all possible combinations ($\overline{\lambda}_p$; $\sigma_{cr,p}/\sigma_{cr,c}$) a mean deviation between both approaches of ≤ 2,62 % can be determined. With increasing imperfection factor α_0 the difference between both approaches is even getting smaller. A typical imperfection factor for single sided stiffeners is about $\alpha_0 \approx 0{,}6$.

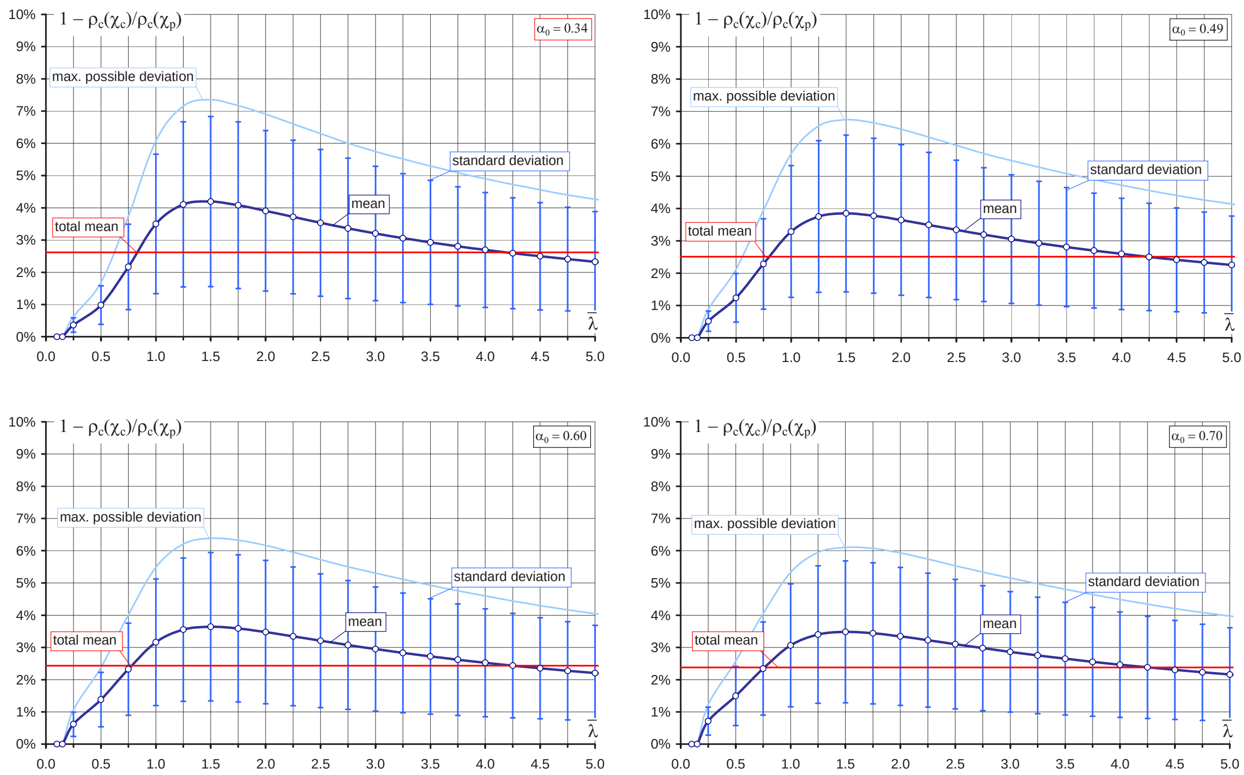


Figure 4.16: Maximum possible deviation between both approaches for $0 \le \xi \le 1$

Considering all these aspects the clear conclusion has to be drawn that only one slenderness $\overline{\lambda}$ should be used to make EN 1993-1-5 more consistent and user-friendly.

Particularly with regard to the NCCIs proposed in section 4.3.1 of this report, the use of one single slenderness $\overline{\lambda}_p$ instead of two different ones will lead to a strong simplification and consolidation of the given design procedure.

4.4.2 Effective area of stiffened plates

Affected section

Section 4.5(4), EN 1993-1-5 [13]

"Calculation of the effective[p] area of stiffened plates with longitudinal stiffeners under direct stresses"

Point of discussion

Numerical simulations have shown that the equation (4.5) of EN 1993-1-5 may lead to over-evaluated effective[p] areas and so to unsafe results for plates with weak stiffeners. According to these studies, the results become unsafe when the relative bending stiffness γ of the stiffeners is less than 25.

Proposal

To Equation (4.5)

$$A_{c,eff} = \rho_c A_{c,eff,loc} + \Sigma\, \rho_{edge}\, b_{edge}\, t \qquad (4.5)$$

where: ρ_{edge} is the reduction factor for the edge subpanel, calculated from equation (4.2) with the relative slenderness $\bar{\lambda}_{p,edge}$ defined as:

$$\bar{\lambda}_{p,edge} = \sqrt{\frac{f_y}{\min(\sigma_{cr,p};\sigma_{cr,sp})}}$$

$\sigma_{cr,p}$ is the elastic critical buckling stress for the global buckling of the plate (see 4.5.2)

$\sigma_{cr,sp}$ is the elastic critical buckling stress of the edge subpanel considered as isolated.

When $\sigma_{cr,p}$ is calculated with Annex A, ρ_{edge} can be calculated by the following modified relation:

$$\rho_{edge} = \min(\rho_{edge,loc}\ ;\ \rho)$$

where: $\rho_{edge,loc}$ is the reduction factor for the edge subpanel calculated according to 4.4 (2)

ρ is the reduction factor for the global buckling of the plate, calculated according to 4.5 (2).

Background information

In the frame of the COMBRI project, equation (4.5) of EN 1993-1-5 has been proved to be unsafe (see Section 4.3.1) for plates stiffened by weak stiffeners ($\gamma < 25$).

By studying the effect of the relative bending stiffness within the EN 1993-1-5 model of resistance, it has been shown that the model for stiffened plates is not consistent with the model for unstiffened plates when the relative bending stiffness γ tends to zero. In this latter condition, the resistance of the plate does not tend to the resistance of the unstiffened plate, as it would have to, but to a higher value (see Figure 4-17).

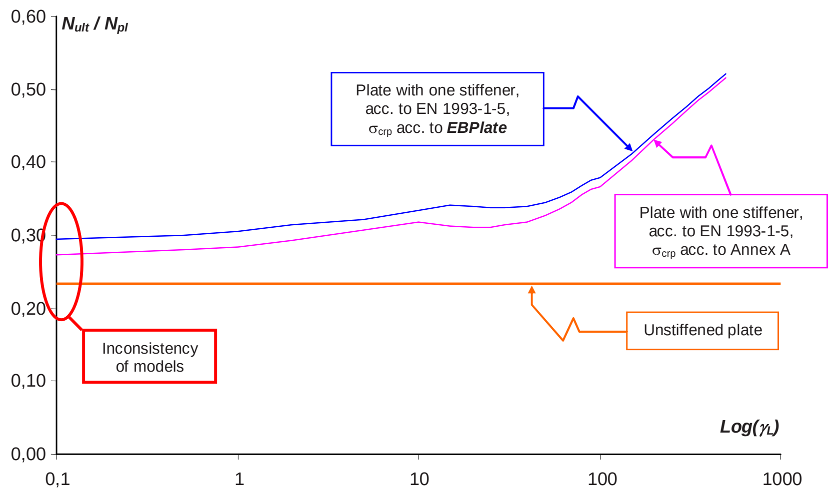


Figure 4-17: Influence of the relative bending stiffness of the stiffener γ on the resistance of a plate under compression

This over-evaluation may be related to the fact that the effective width attached to the edge of the plate ($b_{edge,eff}$ in eq (4.5) of EN 1993-1-5) is calculated assuming that the stiffeners are sufficiently rigid so that no influence of the global buckling behaviour of the plate is expected to affect this effective width. This can be accepted as far as the first critical mode of the plate is the buckling of the sub panels (i.e. fully rigid stiffeners) but it can be clearly criticized if the first critical mode is global (i.e. flexible stiffeners). This latter case implies an influence of this global mode of the edge effective width, the global critical stress being lower than the critical stress for the elastic buckling of the sub-panel, see Figure 4-18.

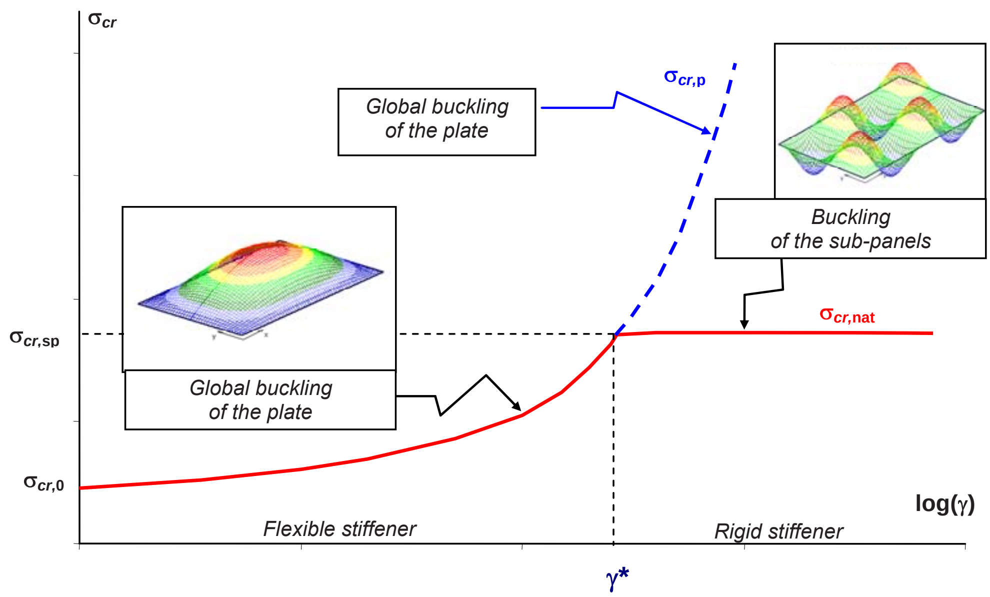


Figure 4-18: Influence of the stiffness of a stiffener on the buckling of the plate

The idea to correct the inaccuracy of the model is then to calculate the effective width $b_{edge,eff}$ of an edge subpanel by considering the actual critical stress for the buckling which reduces the resistance of this subpanel (see Figure 4-18):

- either the stiffener is sufficiently rigid so that the first buckling mode of the plate is a local buckling of the subpanel; $b_{edge,eff}$ is then calculated using $\sigma_{cr,sp}$ the elastic critical stress of the subpanel (as it is done in the present formulation of EN 1993-1-5);

- or the stiffener is not sufficiently rigid so that a global buckling of the plate occurs before the local buckling of the subpanel. In this case, the global buckling will affect the resistance of the edge panel and it is proposed to use $\sigma_{cr,nat} = \sigma_{cr,p}$, where $\sigma_{cr,p} < \sigma_{cr,sp}$.

Finally, the proposal can be summarized as follows:

$\sigma_{cr,nat} = \min(\sigma_{cr,p}, \sigma_{cr,sp})$ and then $\bar{\lambda}_{p,edge} = \sqrt{\dfrac{f_y}{\sigma_{cr,nat}}}$

ρ_{edge} is calculated from $\bar{\lambda}_{p,edge}$ with equation (4.2) of EN 1993-1-5

$b_{edge,eff} = \rho_{edge}\, b_{edge}$

This proposal has been evaluated and calibrated using the database constituted in the frame of WP1 of the COMBRI and described in Section 4.3.1 (see [23]). The results are clearly improved compared to the ones given by the present formulation of equation (4.5) and nearly all of them are on the safe side (see Figure 4-19 when using Annex A or Figure 4-20 when using EBPlate [18] for the calculation of $\sigma_{cr,p}$). It is to be noted that the proposal does not change the results for plates with rigid stiffeners.

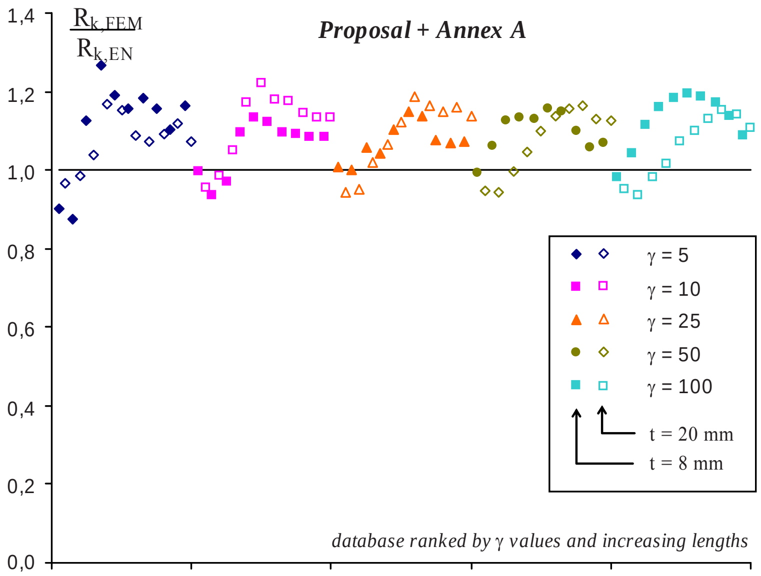


Figure 4-19: Calibration of the proposal for a plate stiffened by one stiffener $\sigma_{cr,p}$ calculated with Annex A

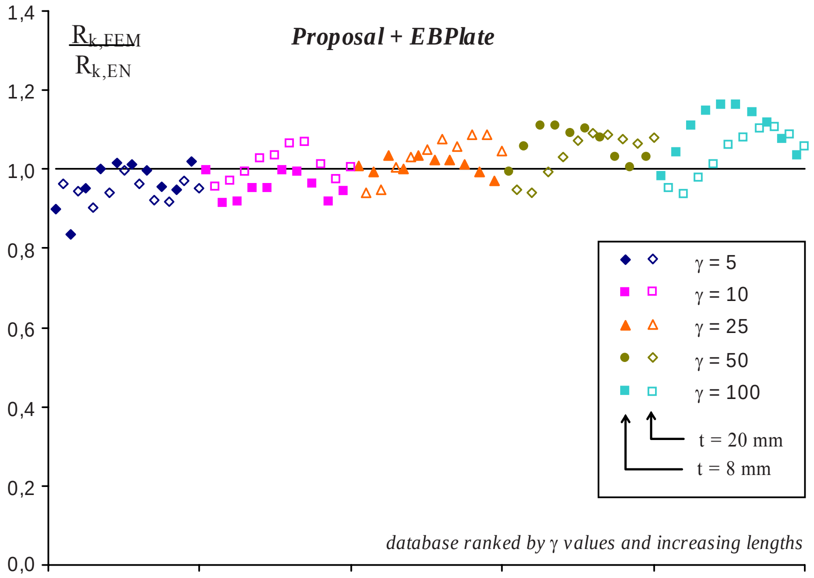


Figure 4-20: Calibration of the proposal for a plate stiffened by one stiffener $\sigma_{cr,p}$ calculated with EBPlate

When Annex A is used to assess $\sigma_{cr,p}$, the following modified relation could be used instead of the previous one: $\rho_{edge} = \min(\rho_{edge,loc}; \rho)$, where $\rho_{edge,loc}$ is the reduction factor for the edge subpanel calculated according to 4.4 (2) and ρ the reduction factor for the global buckling of the plate calculated according to 4.5.2. The difference between the two methods is in the coefficient $\beta_{A,c}$ used for the reduced slenderness of the equivalent plate in the equation (4.7). See Figure 4-21 for the calibration of this simplified proposal. This simplification cannot be used when $\sigma_{cr,p}$ is not assessed with Annex A, excluding so the method with EBPlate [18].

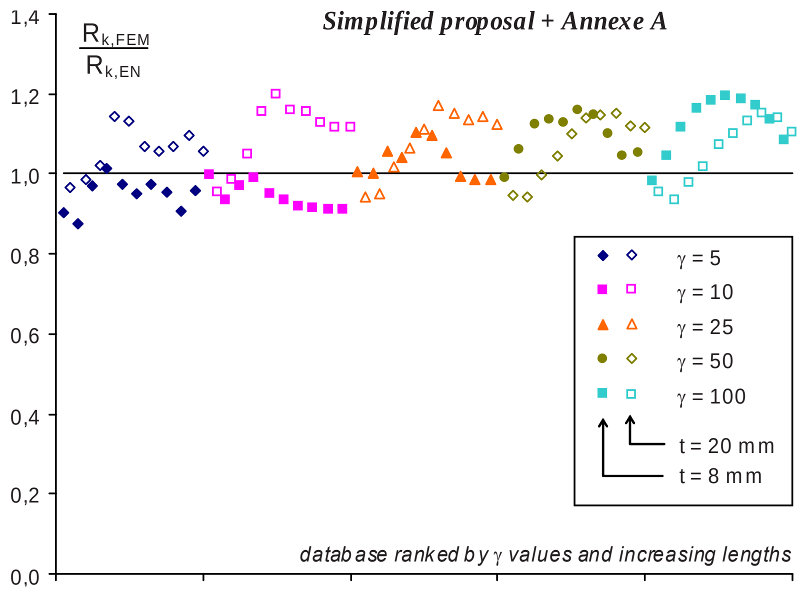


Figure 4-21: Calibration of the simplified proposal for a plate stiffened by one stiffener $\sigma_{cr,p}$ calculated with Annex A

4.4.3 Shear resistance of unstiffened and longitudinally stiffened girders

Affected section

Section 5.3(2) to 5.3(4), EN 1993-1-5 [13]

Point of discussion

See Section 4.3.3 of the present document. The Non-Contradictory Complentary Information (NCCI) proposed regarding web panels stiffened by closed stiffeners should preferably be included in the main text of EN 1993-1-5 in a next revision.

Proposal

Section 4.3.3.

Background information

Section 4.3.3.

4.4.4 Patch loading resistance according to Chapter 6

Affected section

Section 6, EN 1993-1-5 [13]

Point of discussion

The present rules for patch loading resistance are safe but sometimes too safe-sided and uneconomical. The rules use the same format as for other plate buckling phenomena but there are quite large differences between the reduction functions. A step towards harmonization is desirable.

Proposal

It is suggested that the following text replaces the present Section 6 of EN 1993-1-5:2005.

6 Resistance to transverse forces

6.1 Basis

(1) The design resistance of the webs of rolled beams and welded girders should be determined in accordance with 6.2, provided that the compression flange is adequately restrained in the lateral direction and for case (c) in 6.1(2) both flanges.

(2) The load is applied as follows:

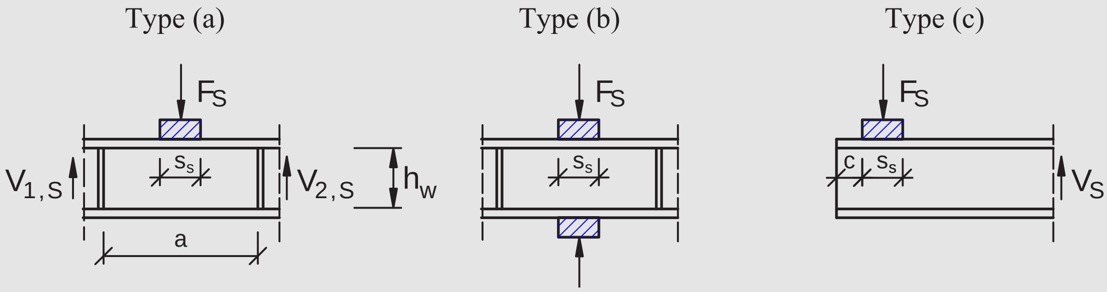
a) through the flange and resisted by shear forces in the web, see Figure 6.1(a);

b) through one flange and transferred through the web directly to the other flange, see Figure 6.1(b);

c) through one flange adjacent to an unstiffened end, see Figure 6.1(c) where the flanges are restrained from relative lateral movements.

(3) For box girders with inclined webs the resistance of both the web and flange should be checked. The internal forces to be taken into account are the components of the external load in the plane of the web and flange respectively.

(4) The interaction of the transverse force, bending moment and axial force should be verified using 7.2.



$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 \qquad k_F = 3{,}5 + 2\left(\frac{h_w}{a}\right)^2 \qquad k_F = 2 + 6\left(\frac{s_s + c}{h_w}\right) \leq 6$$

Figure 6.1: Buckling coefficients for different types of load application

6.2 Design resistance

(1) For unstiffened or stiffened webs the design resistance to local buckling under transverse forces should be taken as

$$F_{Rd} = \frac{\chi_F\, f_{yw}\, \ell_y\, t_w}{\gamma_{M1}} \qquad (6.1)$$

where t_w is the thickness of the web;

f_{yw} is the yield strength of the web;

ℓ_y is the effective loaded length, see 6.5, appropriate to the length of stiff bearing s_s, see 6.3;

χ_F is the reduction factor due to local buckling, see 6.4(1).

6.3 Length of stiff bearing

(1) The length of the stiff bearing s_s on the flange should be taken as the distance over which the applied load is effectively distributed at a slope of 1:1, see Figure 6.2. However, s_s should not be taken as larger than h_w.

(2) If several concentrated forces are closely spaced, the resistance should be checked for each individual force as well as for the total load with s_s as the distance between the outer loads.

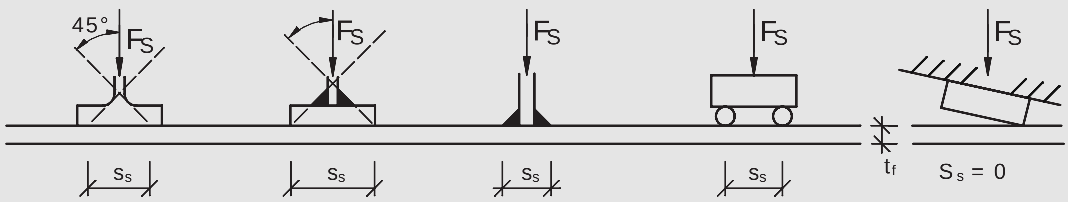


Figure 6.2: Length of stiff bearing

(3) If the bearing surface of the applied load rests at an angle to the flange surface, see Figure 6.2, s_s should be taken as zero.

6.4 Reduction factor χ_F for buckling resistance

(1) The reduction factor χ_F should be obtained from:

$$\chi_F = \frac{1}{\varphi_F + \sqrt{\varphi_F^2 - \bar{\lambda}_F}} \leq 1.2 \qquad (6.2)$$

where $$\bar{\lambda}_F = \sqrt{\frac{\ell_y t_w f_{yw}}{F_{cr}}} \qquad (6.3)$$

$$\varphi_F = \frac{1}{2}\left(1 + 0,5\left(\bar{\lambda}_F - 0,6\right) + \bar{\lambda}_F\right) \qquad (6.4)$$

(2) For webs without longitudinal stiffeners F_{cr} should be obtained from Equation *(6.5)* with k_F according to Figure 6.1.

$$F_{cr} = 0,9\, k_F E \frac{t_w^3}{h_w} \qquad (6.5)$$

NOTE 1 The values of k_F in Figure 6.1 are based on the assumption that the load is introduced by a device that prevents rotation of the flange.

(3) For webs with longitudinal stiffeners and loading type (a) F_{cr} may be taken as the smallest of *(6.9)* and *(6.10)* with $k_{F,1}$ and $k_{F,2}$ according to Equations (6.6) and *(6.7)*

$$k_{F,1} = 6 + 2\left[\frac{h_w}{a}\right]^2 + \left[5,44\frac{b_1}{a} - 0,21\right]\sqrt{\gamma_s} \qquad (6.6)$$

$$k_{F,2} = \left[0,8\left(\frac{s_s + 2t_f}{a}\right) + 0,6\right] \cdot \left(\frac{a}{b_1}\right)^{0,6\left(\frac{s_s+2t_f}{a}\right)+0,5} \qquad (6.7)$$

$$\gamma_s = 10,9\frac{I_{s\ell 1}}{h_w t_w^3} \leq 13\left[\frac{a}{h_w}\right]^3 + 210\left[0,3 - \frac{b_1}{a}\right] \qquad (6.8)$$

where b_1 is the depth of the loaded subpanel taken as the clear distance between the loaded flange and the stiffener;

$I_{s\ell 1}$ is the second moments of area of the stiffener closest to the loaded flange including contributing parts of the web according to Figure 9.1

Equation *(6.6)* is valid for $0,05 \leq \frac{b_1}{h_w} \leq 0,3$ and $\frac{b_1}{a} \leq 0,3$

$$F_{cr,1} = 0,9\, k_{F,1} E \frac{t_w^3}{h_w} \qquad (6.9)$$

$$F_{cr,2} = 0{,}9\, k_{F,2}\, E \frac{t_w^3}{b_1} \qquad (6.10)$$

(4) ℓ_y should be obtained from 6.5.

6.5 Effective loaded length

(1) The effective loaded length ℓ_y should be calculated as follows for type (a) and (b) in Figure 6.1:

$$l_y = s_s + 2t_f \left(1 + \sqrt{\frac{f_{yf} b_f}{f_{yw} t_w}} \right) \leq a \qquad (6.11)$$

For box girders, b_f in expression (6.11) should be limited to $15\varepsilon t_f$ on each side of the web.

(2) For type c) ℓ_y should be taken as the smallest value obtained from equations *(6.11)*, *(6.12)* and *(6.13)*:

$$\ell_y = \ell_e + t_f \sqrt{\frac{f_{yf} b_f}{2 f_{yw} h_w} + \left(\frac{\ell_e}{t_f} \right)^2} \qquad (6.12)$$

$$\ell_y = \ell_e + t_f \sqrt{\frac{f_{yf} b_f}{f_{yw} h_w}} \qquad (6.13)$$

where $\ell_e = \dfrac{k_F E\, t_w^2}{2\, f_{yw} h_w} \leq s_s + c$ *(6.14*

6.6 Verification

(1) The verification should be performed as follows:

$$\eta_2 = \frac{F_{Ed}}{F_{Rd}} \leq 1{,}0 \qquad (6.15)$$

where F_{Ed} is the design transverse force;

F_{Rd} is the design resistance to transverse forces, see 6.2(1).

Background information

The present rules in EN 1993-1-5:2006 are based on [22] and represented a step forward compared to previous rules. They gave a higher resistance particularly for long loaded lengths and they were written in the same format as rules for other buckling phenomena. During the drafting of EN 1993-1-5 the reduction function was modified such that the resistance was lowered especially for very slender webs. The rules for webs with longitudinal stiffeners were based on [20] and they gave an increase of the resistance based on the increase of the first critical load. It was later shown in [8] and [26] that the resistance did not follow that simple assumption.

During the COMBRI research project the patch loading problem was studied further and the results are summarized in [5] and more detailed information is given in [19] and [4]. A brief description of the background to the suggested changes in the design rules will be given here.

The first change to be discussed is the definition of the plastic resistance. For the case of a concentrated load transferred via a flange to the web there is no obvious definition because as the deformation increases the resistance keeps increasing gradually, see Figure 4-22, which shows results from a computer simulation from [19]. This is partially due to strain hardening but also an effect of increased loaded length as the bending of the loaded flange increases.

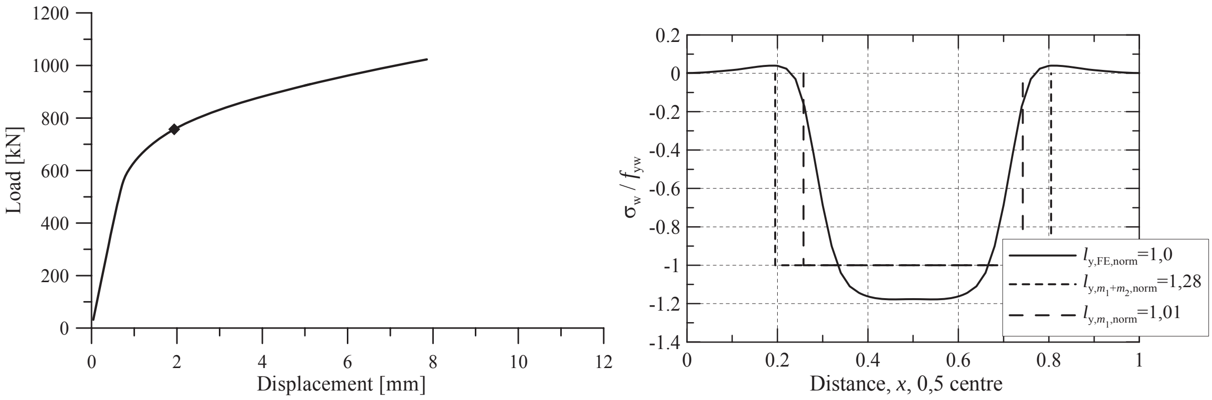


Figure 4-22: Load-displacement curve (left) and vertical stress distribution in the web plate (right) for FE-analysis. Material properties corresponding to a S355 steel.

In an attempt to harmonize the design rules for buckling *Müller* noted that the rules for patch loading differed substantially from other plate buckling rules [25] such that the reduction function was much lower. The reason was stated to be a too high starting value, the plastic resistance. *Davaine* noted the same in her study of patch loading of longitudinally stiffened girders [8] and proposed that the effective loaded length should be taken according to expression (6.9) in the proposed new text for EN 1993-1-5 above. This issue was further studied by *Gozzi* in [19] in which results from computer simulations of several patch loading cases were presented. In the model the web was prevented from buckling. The results in Figure 4-22 are for a girder with web 4x992 mm and flange 16x250 mm and a loaded length of 200 mm. The left diagram shows the relation between load and deformation and the dot on the curve marks the proposed plastic resistance. The right hand figure shows the vertical stresses in the web normalized with the yield strength. The solid line is the FE result and the two rectangular blocks are the old and the new definition of the plastic resistance.

The study in [19] included also a check of reversible behaviour for the case of a travelling load. In order to guarantee a reversible behaviour the load in SLS should fulfil

$$F_{SLS} \leq (0{,}05 + 0{,}44\bar{\lambda}_F)F_R$$

where F_R is the characteristic resistance in ULS.

The new definition of the plastic resistance and the critical load requires a new reduction function. There is today a trend to harmonize the mathematical form of the reduction functions to the type given in Annex B of EN 1993-1-5 and the expressions in (6.2) to (6.5) in the proposed new text are of this type and were developed in [19]. It differs from the one proposed in Annex B in that it gives a lower resistance for $\lambda > 0{,}6$ and a higher for smaller λ. The reduction function was evaluated from 186 tests with small bending moments and the comparison is shown in Figure 4-23. The test data base is the same as the one used by *Lagerqvist* [22] for drafting the present rules except the additional tests from the COMBRI research project which are designated "Tests, author". The statistical evaluation shows a smaller scatter than the present rules in EN 1993-1-5 but actually higher than the original proposal of Lagerqvist. It may still be seen as a step forward in harmonizing design rules for plate buckling. An evaluation according to Annex D of EN 1990 resulted in a recommended value γ_{M1} of 1,0.

For stocky plates it is clear from Figure 4-23 that the resistance exceeds the new plastic resistance. The reasons have been discussed above. In order to partially compensate for this, the reduction function is allowed to continue up to 1,2 in the same way as it does for the shear resistance and in fact for very much the same reasons.

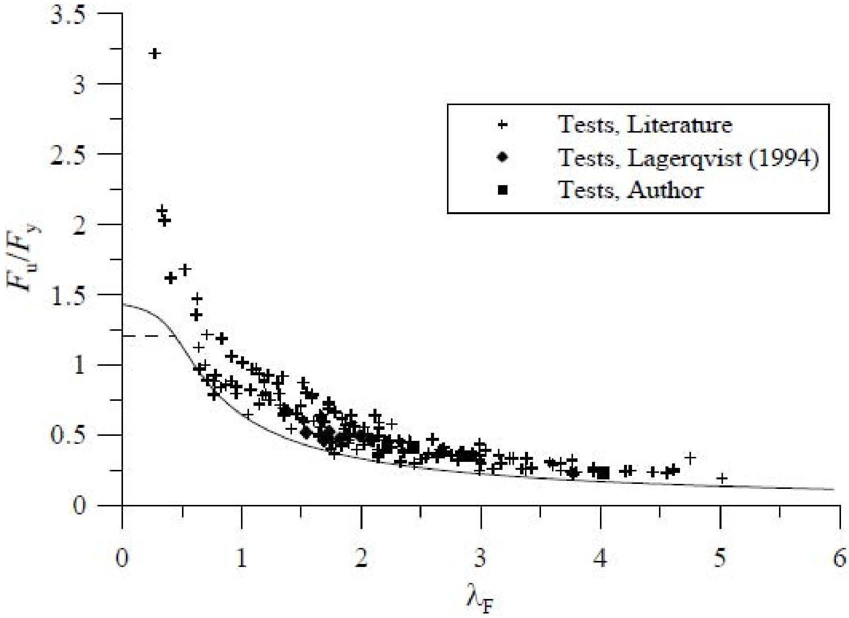


Figure 4-23: Proposed reduction factor for patch loading resistance compared with test results with small bending moment (less than 0,4 times resistance)

Patch loading of girders with longitudinal stiffeners has been studied in two thesis's since the EN 1993-1-5 was written. *Davaine* made extensive computer simulations presented in [8], which showed that the present rules in EN 1993-1-5 were oversimplified. The critical load for global buckling in EN 1993-1-5 was accepted but there was a need also to consider buckling of the loaded panel and an empirical estimate of this critical load was developed, see expression (6.7) in the proposed new text. It was also suggested that an interaction between the two buckling modes should be considered. In a later study by Clarin [4] it was however shown that it was sufficient to use the smaller of the two critical loads in combination with the same reduction function as for unstiffened webs. The validation of this proposal is shown in Figure 4-24 for tests from literature and for computer simulations from [8]. The results are for open stiffeners but also closed stiffeners were studied and it was shown that closed stiffeners gave a larger increase of the resistance. A calibration according to Annex D of EN 1990 showed that $\gamma_{M1} = 1,0$ can be recommended.

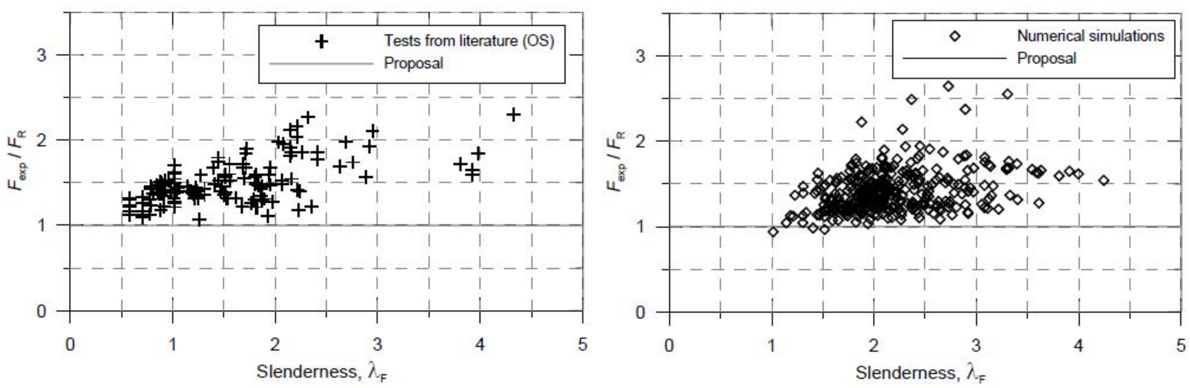


Figure 4-24: Test results (left) and computer simulations (right) for patch loading of girders with longitudinal stiffeners related to the proposed characteristic resistance

4.5 Conclusion

This chapter has summarised a number of proposals which have been developed in the frame of the COMBRI research project [5] and the COMBRI+ project. They are classified according to their possibility of implementation as:

- Nationally Determined Parameter (NDP)
- Non-Contradictory Complementary Information (NCCI)
- Amendment for revision

All amendments clearly aim at an improved economy and easier harmonisation so that they are strongly recommended to be implemented at due time. The same is valid for most of the NCCI. However, some proposals for NCCI tackle questions of safety such as Section 4.2.2 so that a strong recommendation is given to implement them already in the National Annexes drafted by the various countries.

5 FINAL REMARKS

The valorisation of the results of the COMBRI research project was successfully implemented in the COMBRI Design Manual and its promotion and dissemination in several seminars and workshops at national level. Finally the manual was made available in English, French, German and Spanish language. The feedback for participants was positively throughout. Compliments were received especially for highlighting of the possibilities and restrictions of current Eurocode rules and the overview on best national practices in the different European countries. The presentation of ***EBPlate*** software which was developed during the COMBRI research project was highly appreciated. In the following the conclusions which have been drawn in the COMBRI Design Manual are summarised in short:

An overview of the bridge types in the participating partner's countries - Belgium, France, Germany, Spain and Sweden – show the current practice in those countries. It can be stated that there are notable differences between the practices of the countries and these differences are to some extent caused by differences between the national design standards but more often they are caused by different traditions and practice. Thus, the solutions presented are intended to serve as inspiration for the conceptual design of new bridges.

EN 1993-1-1 covers steel grades up to and including S460 but EN 1993-1-12 extends the range of permitted steel grades up to S700. It is shown that in most cases such high grades are not feasible. The problem is usually that the fatigue requirements limit the full utilisation of the strength. The grade S460 seems to be the most suitable for normal road bridges and S355 for normal rail bridges. It is also shown that hybrid girders with higher strength in the flange than in the webs are economic in many applications. The box-girder from Part I of the manual was redesigned from S355 to a hybrid girder with S460 and S690 and it turned out that the costs of the material was reduced by 10% in the spans and 25% at the piers. In addition, there is a reduction of the fabrication costs.

Flanges as bottom flanges in box-girders are in most cases stiffened and different types of stiffeners are discussed. It is shown that large trapezoidal stiffeners are favourable as they give two stiffened lines for the same welding effort as one open stiffener. Further, their torsional stiffness increases the critical stress and this can be calculated with the ***EBPlate*** software which has been developed in the COMBRI research project. Another topic is the double composite action with both top and bottom flanges being composite which has been used for some large bridges in Germany and France. The design of bridges with double composite action is more complicated than the design of a normal composite bridge so that past experience is summarised and recommendations for design are given.

With regard to webs the focus was on to what extent stiffeners should be used. It is common that transverse stiffeners are used at the locations of the cross bracings of which the transverse stiffeners form a part. The effect of the transverse stiffeners on the resistance of the web is an increase in the shear buckling resistance. However, unless the distance between the transverse stiffeners is very short this effect is small and it does not justify the costs of the stiffeners. The possibility of omitting the transverse stiffeners is discussed. Besides that, longitudinal stiffeners on webs increase the resistance for bending as well as for shear. The economy of using longitudinal stiffeners was studied and if the method with effective cross section in EN 1993-1-5 is applied it is shown that longitudinal stiffeners are not economical for web depths below ca. 4 m. The detailing of longitudinal stiffeners has been discussed as well and the main point is the intersection with the transverse stiffeners. One solution is to use discontinuous stiffeners and another is to put the transverse and the longitudinal stiffeners on opposite sides of the web.

Cross bracings and diaphragms for I-girder bridges and box-girders have the function to prevent lateral torsional buckling and to transfer lateral loads on the girders to the deck. Traditional cross bracings can be of truss type or frame type including transverse stiffeners on the webs. The distance between the cross bracings is typically up to 7 to 10 m in I-girder bridges. Although it is not much material used for cross bracings, from an economical point of view it is important to minimize the man hours for fabrication. This was discussed in terms of eliminating parts and possibly also the transverse stiffeners leading to straightforward solutions. For box-girders, the cross bracings or diaphragms also have the function of preventing cross sectional distortion and in many cases they also support the bridge deck. Therefore the distance between the cross bracings is rather small, typically 4 to 5 m.

The technique of launching bridges is very popular and verification methods have been improved in the COMBRI research project. They allow now the utilisation of quite long loaded lengths and accordingly quite high resistance can be achieved. This may make it possible to launch bridges with parts of the concrete slab or the reinforcement in place. For the twin-girder bridge of Part I of the manual, these two possibilities have been studied and the results are compared. If it is useful to have the concrete slab or the reinforcement already in place, the outcomes of the COMBRI research project are very helpful and may lead to more economic solutions.

The proposals which have been formulated for an implementation into standardisation were classified according to Nationally Determined Parameters (NDP), Non-Contradictory Complementary Information (NCCI) and amendments to be used in the next revision of Eurocode in order to provide a concise background and proposal scheme. The transfer of these proposals to CEN is ensured through the collaborative work of the project members in that committee.

European Commission

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