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Volume 5
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Steel Construction

Design and Research

- Hybrid steel plate girders subjected to patch loading
- New resistance model for plate girders under patch loading
- Design of trapezoidal corrugated web girders under combined actions
- Intermediate transverse stiffeners in plate girders
- Reduced stress design of plates under biaxial compression
- Innovation brokers for managing interdisciplinary research
- China's longest single-pylon cable-stayed bridge span
- Reconstruction of "Olympic" stadium in Kiev

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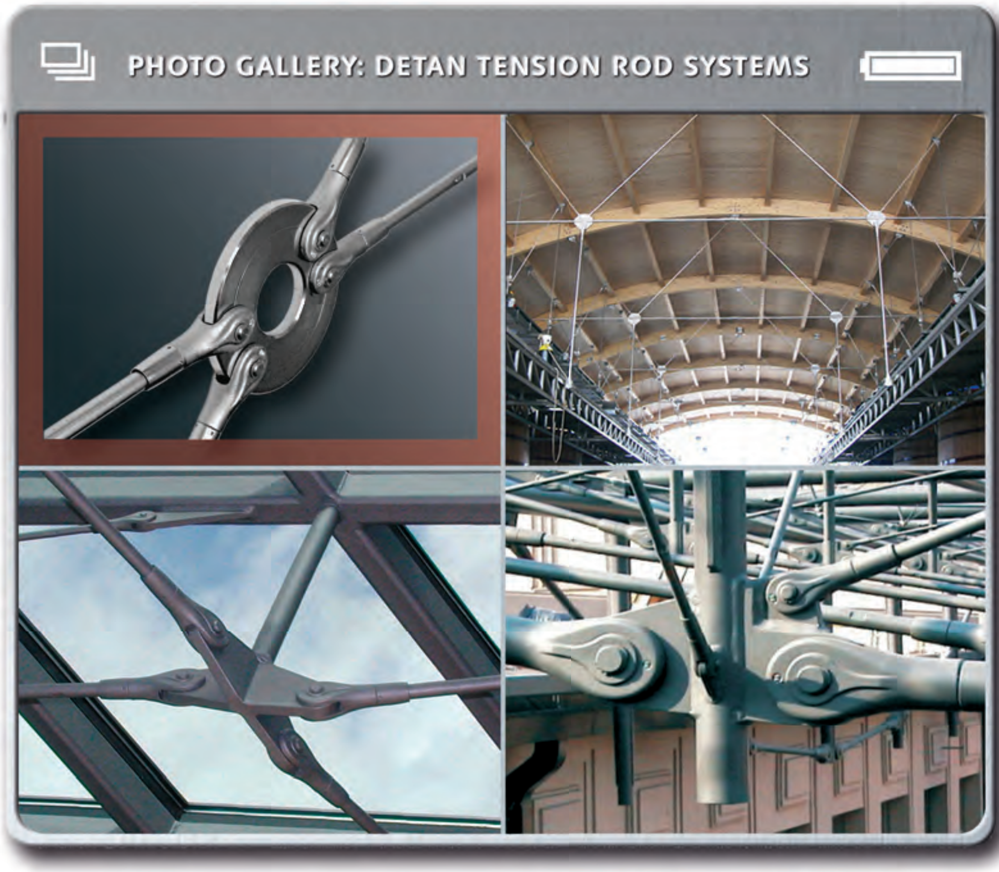
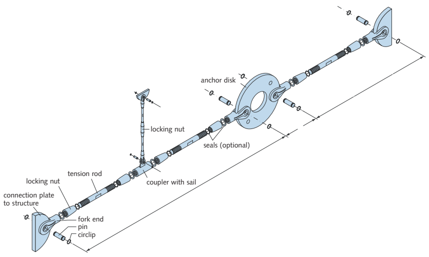


Form and function are perfectly combined in the DETAN tension rod system. Individual system solutions make it possible to realise even the most complicated designs and aesthetic details both indoors and outdoors.



DETAN tension rod structures can consist of one or all of the standard components shown in this example.

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The Huangpu Pearl River Bridge is the key project on the second ring highway of Guangzhou, China. The main bridge over the northern channel is a single-pylon cable-stayed bridge with a main span of 383 m and a steel box girder, which was constructed using the balanced cantilever method (see p. 53).

Steel Construction 1

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ECCS CECM EKS Journal for ECCS members

Tamed, but not tame

The name AlpspiX depicts a spectacular viewing platform about 1,000 m over the Hell Valley, being opposite of Germany's highest mountain, the Zugspitze. Two light platforms protrude in the shape of an X into the abyss. However, in case of purposeful movements of the visitors, these platforms vibrated that much that the operator, the Bayerische Zugspitzbahn Bergbahn AG, decided to make the platform also accessible to tourists which are more sensitive to vibrations. The "Damping with residual adventure factor" was implemented by Maurer Söhne, the world wide renowned specialist for structural protection systems.

The inauguration of the AlpspiX platforms was in July 2010. They are located at the "Osterfelderkopf", which is only 50 m above the mountain terminus of the Alpspitz Track, and thus they also can be reached by inexperienced hikers. This in contrast to the Alpspitz peak which is 550 m further up and which will usually be reached by means of a fixed rope route.
The AlpspiX viewing platform facilitates an undisturbed view at its finest. Only a glass plate prevents the free fall at the tip of the platform. And a grid iron allows a look into the abyss 1,000 m below. But that the steel platform were prone to undamped vibrations was not exactly palatable for many visitors. Already a few individual visitors created continuing vibrations in the platforms – not to mention the jaunty groups who purposely created these vibrations.

Bild 1. AlpspiX: spektakuläre Aussichtsplattform 1.000 m über dem Höllental und gegenüber der Zugspitze

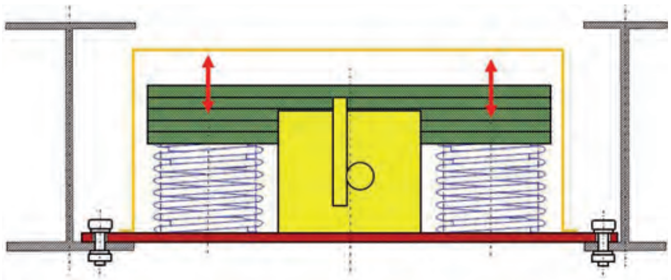

Bild 2. Querschnitt durch Massendämpfer: Die viereckigen, flachen Boxen (orange) beherbergen die schwingende Tilgermasse (grün) von je 150 kg, die in mehrere Platten aufgeteilt ist. In der Mitte ist ein zusätzliches hydraulisches Dämpfelement (gelb) integriert. Der Plattenstapel liegt auf den vier Federn (blau) auf, welche auf der Bodenplatte (rot) befestigt sind. Die Befestigung am Bauwerk erfolgt mittels vier Schrauben zu den Doppel-T-Trägern. (Foto/Grafik: Maurer Söhne)

Dampen, but not fixate

The job instruction for the damper experts of Maurer Söhne was therefore to reduce the vibrations but not completely eliminate, by way of tuned mass dampers. Next to the higher attractiveness of the visitors the owner also considered the service life of the platforms: the less vibrations, the less material fatigue, and thus longer maintenance intervals and longer service life.
The two AlpspiX arms which are each about 17.50 m long are situated on top of each other and protrude in the shape of an X into the abyss. Their load carrying structure consists of two each double-T-girders with cross members. The two platforms are separately constructed and clamped into two foundations each.
The tuned mass dampers had to be installed at the tip of the cantilever, because there the vibrations are the strongest. Furthermore, the 2 tuned mass dampers each should not disturb the free view. Therefore, the 340 mm high dampers were placed under the grid iron, between the longitudinal girders at the tip of the platform.
The four tuned mass dampers of each 340 kg have the shape of flat square boxes with a footprint of 650 × 650 mm. The swinging damper mass of 150 kg each is separated into several steel plates such that it can be adjusted retroactively. The plate stack rests on four steel coils which are mounted on a base plate. The base plate is bolted to the longitudinal girders. For each damper, the frequency of the coils is tuned to the natural frequency of the platform and lie between 2.03 and 2.68 Hz.

Coils plus hydraulics

However, the countereffect of the swinging mass plates alone would not suffice to damp the structure against vibrations which are created by groups acting willfully. For this reason, Maurer integrated in the centre of the mass damper an additional hydraulic damping element which is connected with the mass and acts as internal brake. It converts motional energy into heat, and this way structural vibrations decay very fast. In comparison to the "undamped" state (i.e. without hydraulic damping element) it not only takes more people to trigger a perceptible vibration, but the vibration also decays in the course of a few seconds.
The four mass dampers were installed from above in summer 2011, by way of detaching the grid irons, lifting the damper boxes to their location and fixing them. After the inauguration, final measurements were made as well as a fine adjustment of the damper mass.

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Application of organo-ceramic coating product gives phenomenal adhesion on stainless steel

It was in summer 2011 that Ceram Kote International GmbH launched the innovative, solvent-free coating product PROGUARD CN-OC with its excellent protective properties. Extensive test series in cooperation with an independent research institute as well as relevant practical experience verify the outstanding performance of PROGUARD CN-OC.

The new solvent-free coating product has an excellent chemical resistance. That means stability against:
- various acids and alkaline solutions (pH value 4–11), in some cases up to operating temperatures of 110 °C,
- all kinds of hydrocarbons, and
- “killer-solution” – already > 8000 h (> 11 months!) of uninterrupted storage in 98 % sulphuric acid, pure methanol and 3 % sodium chloride solution with a mixing ratio of 1:3 [1]in each case and a constant temperature of 50 °C.

Furthermore, it has phenomenal adhesion:
- on concrete (direct application without the need for a primer first is possible),
- on carbon steel, and
- on stainless steel.

Remarkable results are achieved when curing at room temperature and there is a great increase in adhesion at elevated temperatures. The test procedures and results are described below.

Adhesion tests with PROGUARD CN-OC

To identify the adhesion value, shear strengths were tested with specimens of stainless steel (see Fig. 1). Firstly, cylinders and base plates were sandblasted, then a coating of PROGUARD CN-OC was applied in the centre of the base plates and the cylinders placed on these without applying pressure. The coating was allowed to cure at room temperature for 24 h. Afterwards, different tempering steps were carried out to examine the behaviour of the coating under thermal influences.
The prepared specimens were sheared off with an “Instron 4469” tensile testing machine. The determination of bonding strengths included different analyses of shear strengths (average of five specimens of each kind) and the fracture behaviour of the coating.

Interpretation of test results

The progression of adhesive strength on stainless steel for PROGUARD CN-OC is highly interesting from the scientific viewpoint. After curing at room temperature, the adhesion on stainless steel reached an average value of 17.7 MPa. Tempering at 100 °C increases the adhesion dramatically by 74 %.
A cohesion fracture occurred in the coating at the average value of 30.7 MPa. That means the internal strength of PROGUARD CN-OC in shear tests is defined by this value. The true adhesion of the coating on stainless steel can be considerably higher. To assess the actual bonding strength, further tensile tests with other procedures would be necessary. What causes this extreme increase in adhesion on stainless steel? More surface analyses were carried out to obtain the scientific explanation for this phenomenon.

Surface analyses of stainless steel samples using x-ray photoelectron spectroscopy (XPS)

Preparation: All the base plates of the stainless steel specimens from the previous shear tests were cleaned in N-methyl-2-pyrrolidone (NMP) at 60 °C and for 72 h using an ultrasonic process to remove the coating thoroughly. The plates were coated beforehand only in the area of the affixed cylinder. Thus, a direct comparison between the coated and uncoated metal was possible to define the effect of PROGUARD CN-OC.
Test apparatus: “Theta Probe Thermo VG Scientific”, 400 mm region of examination, 10 nm penetration depth
Assay method: (source: Wikipedia[2])
X-ray photoelectron spectroscopy (XPS) is a method by which the chemical composition of solid objects and their surfaces can be determined. It uses qualitative elemental analysis, i.e. it reveals the chemical elements contained in a solid object. Merely helium and hydrogen cannot usually be detected because of their low cross-sections.

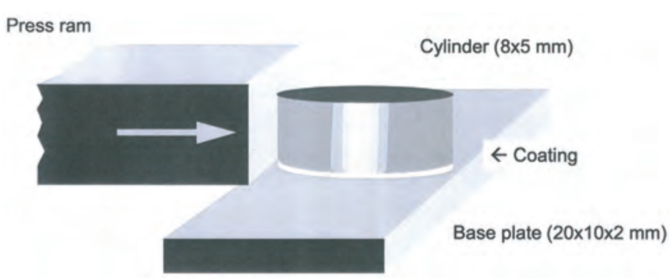

Fig. 1. Shear test in accordance with DIN EN 1465

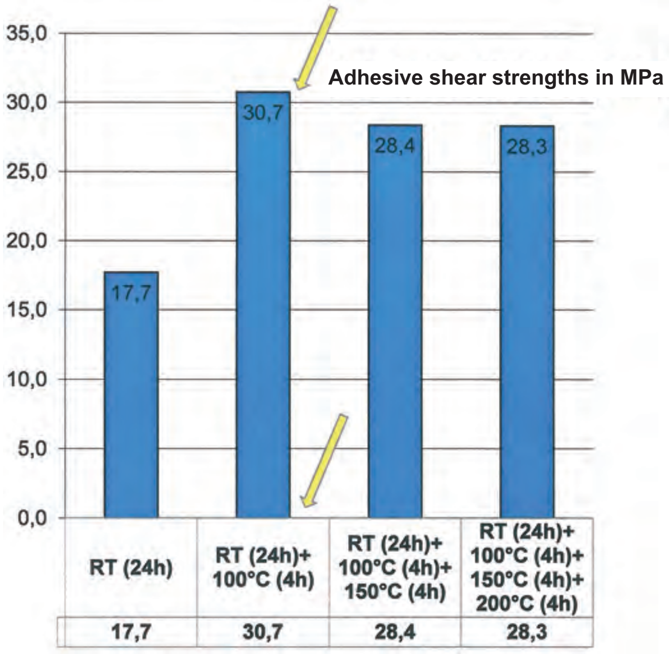

Fig. 2. *Interpretation of test results* *(© Ceram Kote International)*

XPS analysis findings – result of examination, exemplary after tempering at 100 °C for 4 h

The different C1s spectra of the two surfaces demonstrate unequal chemical carbon constitutions in terms of their oxidation state or molecular character.

Stainless steel, uncoated (red graph):

– The carboniferous film on the surfaces caused by environmental influences features a constant chemical carbon constitution irrespective of the tempering.

Stainless steel, coated beforehand (green graph):

– The organo-ceramic components of the coating adhere to the surface and cannot be removed by ultrasonic cleaning.
– The carboniferous film on the surface also exhibits other chemical carbon constitutions in comparison with environmental carbon sediments.
– By increasing the tempering, one kind of chemical carbon constitution increases and another kind decreases proportionally.

Conclusion

The scientific surface analysis shows that the uniquely formulated high-performance coating PROGUARD CN-OC contains particular, innovative organo-ceramic fillers that form specific organic compounds even with slight tempering. These compounds cause chemical interactions with compounds in the surface of the metal and therefore give rise to extreme adhesion on stainless steel substrates.

Currently, this chemical behaviour of PROGUARD CN-OC is being explored in depth. We are also carrying out research to determine at which tempering thresholds the chemical reactions of both materials occur in order to identify the highest possible adhesion of PROGUARD CN-OC on stainless steel.

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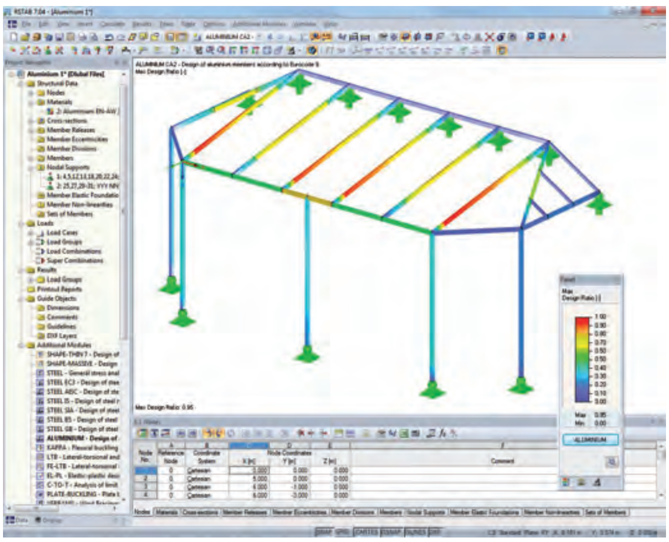
Aluminium design to Eurocode 9 with RSTAB and RFEM

Dlubal's two analysis programs RSTAB and RFEM can now be upgraded with an add-on module for the design of aluminium members according to European standard 1999-1-1:2007 (EC 9). ALUMINIUM is the answer to designing for tension, compression, bending, shear and combined internal forces.

The program is able to take into account various cross-sections, e.g. I-sections (symmetric and unsymmetric), C-sections, T-sections, rectangular hollow sections, square sections, round sections, angles with equal and unequal legs, steel flats and round bars.

Features of ALUMINIUM
In addition to analysing general internal forces, ALUMINIUM designs the structure for stability and serviceability. The program considers several stability cases, such as flexural buckling, torsional buckling and lateral-torsional buckling. The critical buckling load and the elastic critical moment for general loading and support conditions are determined automatically by means of an integrated FEA program (eigenvalue analysis). Furthermore, it is possible to define lateral intermediate supports for members selected for the design.
Serviceability limit state design is carried out for the characteristic, frequent or quasi permanent design situation as required. During the design process in ALUMINIUM the user has the chance to optimize particular cross-sections within the same cross-section table. Subsequently, it is possible to transfer the optimized cross-sections to RSTAB or RFEM. A cross-section classification is performed automatically by the program.

Working with ALUMINIUM
The mode of operation is similar to that of other Dlubal applications. Structure and loads are entered in RSTAB or RFEM, and the design is performed in the add-on module. For aluminium design it is important to note that the data specified in RSTAB/RFEM for materials, loads and load combinations must be entered in accordance with the design concept described in the Eurocode. The RSTAB/RFEM library already contains appropriate materials. ALUMINIUM provides the parameters presented in the national annexes (NAs) of the following countries: Germany, Italy, Czech Republic, Cyprus, Denmark, Ireland and Slovakia. The preset values for intermediate lateral supports and effective lengths can be adjusted. Where continuous members are being designed, the user can define individual support conditions and eccentricities for each intermediate node.



ALUMINIUM provides the parameters given in the national annexes of these countries: Germany, Italy, Czech Republic, Cyprus, Denmark, Ireland and Slovakia. (© Dlubal)

Output of results
The results tables are clearly laid out. For example, the first table shows the maximum design ratio with the corresponding analysis results for each load case, group or combination that has been analysed. Moreover, the output includes governing internal forces as well as parts lists. The complete module data is part of the RSTAB/RFEM hardcopy report. The report contents and the extent of the output can be selected specifically for the individual designs.

Further information and demo versions:
Ingenieur-Software Dlubal GmbH,
Am Zellweg 2, 93464 Tiefenbach, Germany,
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Grontmij to design largest shopping mall of Scandinavia

Grontmij has been awarded the structural design of the largest shopping mall in Scandinavia; the 'Mall of Scandinavia'. The assignment was awarded by Peab, one of the leading construction companies in the Nordic region. The total contract value is € 392 million. Grontmij's fee is over € 5 million. Grontmij's works will run until the end of 2014.
The assignment of Grontmij includes the structural design and the coordination of sub suppliers' engineering work related to the structure of the shopping mall.
The 'Mall of Scandinavia' is one of the largest building projects in Sweden. After completion it will be the biggest shopping centre in Scandinavia with approximately 100,000 m^2 of retail space. The mall will include a multiplex cinema and 250 shops and restaurants. The Mall of Scandinavia is part of a larger complex which includes a hotel, office spaces and dwellings. It will be located centrally in the Stockholm region. The opening is scheduled for autumn 2015.
Claes Ullén, Project Manager Peab for the Mall of Scandinavia. 'We received a very positive response from Grontmij. They demonstrated clear commitment, technical competence and creativity in the contractual framework.'
Leif Bertilsson, CEO of Grontmij Sweden: 'It's an honour to be involved in this landmark project for Sweden and Stockholm in particular. It will be one of the most exciting building projects in Sweden for the next years. Together with our Danish Grontmij colleagues we'll be glad to contribute. This is a good example of our international capability delivered locally.'

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Versatile floor system with improved performance

Arval, the flooring brand from ArcelorMittal for sheet steel construction elements, can boast a new lightweight, cost-effective solution for building long-span flooring structures: the Cofraplus 220 floor system. The concept, developed for car parks, permits the design of continuous slabs and can also be used in combination with various floor beam configurations in multi-storey building applications. Besides its advantages during construction and in the finished state, the system is a winner due to its simple, quick assembly.

The Cofraplus 220 floor system is based on a 220 mm sheet metal section with two webs, a ribbed crown and a cover width of 750 mm. The thickness of the sheet metal can be varied and so the system can be finely adapted to different load-span specifications.
Car parks and multi-storey buildings benefit from the long spans without propping and the possibility of introducing continuous slabs for a cost-effective design. The sheets are made of highly anti-corrosive pre-painted sheet steel, available in a choice of pure metal coating or a number of organic coatings with various features. The range of colours is even more extensive. The light flooring substrate produced by the sheet metal section provides friendly, secure surroundings; colourful accents are an aid to horizontal and vertical orientation in the building.

Fig. 1. *Concreting the Corfaplus 220 floor system*

Fig. 2. *Connection detail of the Cofraplus 220 floor system – welded support "wings" and flooring*

Fig. 3. *Cofraplus 220 floor system with continuity effect for better economy (© Arcelor Mittal)*

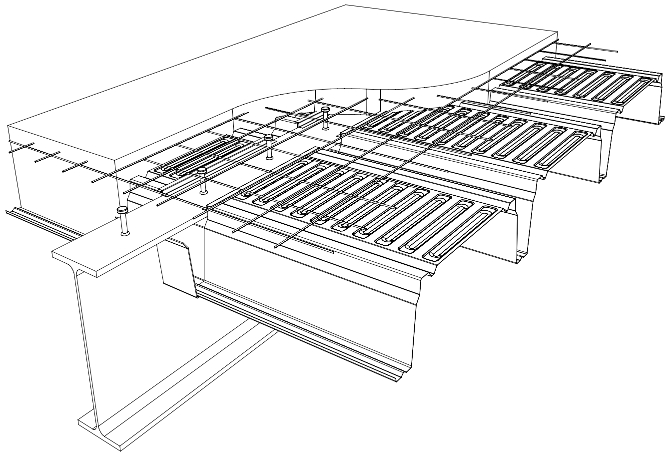

Fig. 4. *The system can be finely adapted to different load-span specifications*

Continuous slabs for cost-effective loadbearing characteristics

The floor system is dimensioned taking into account its additive effect. Here, the concrete cross-section of the ribbed floor and the sheet metal are each involved in the load transfer separately, with no connection to each other. The total loadbearing capacity of the floor is thus achieved through simple addition of the two loadbearing portions. In addition to the considerable moment of inertia of the 220 mm floor slab, particularly favourable in critical designs, the Cofraplus 220 System permits a static continuous slab design above the beams acting as intermediate supports. The sheet metal sections are supported via so-called wings: 3 mm thick sheet metal parts fitted to the profile, which are factory-welded directly to the web of the composite steel beams. The attachment of the wings to the web can vary depending on the specific project. If required, the crown of the sheet metal section can be positioned over the upper edge of the support flange. In this way, the composite steel beam, as the primary loadbearing element, can be optimized in a more independent manner without increasing the depth of the concrete floor between beams or increasing the loads. Since they are supported via the wings, the sheet metal sections are brought right up to the web, along with the ribs of the floor system which will be concreted at a later date. This construction achieves a high transverse loadbearing capacity between support and sheet metal, providing the prerequisite for redistribution of moments of inertia and enabling a more cost-effective design on the basis of a continuous slab.

Generally no need for reinforcement in the corrugations

Depending on the load–span specifications and the fire resistance required, there is generally no need for reinforcement in the corrugations. Likewise, complicated transverse reinforcement over the beams can be dispensed with. Since public car parks with spans of 15 to 17 m and beam spacings of 5.0 to 6.0 m do not require stricter fire resistance requirements, only anti-shrinkage reinforcement is necessary. Depending on the requirements for crack control reinforcement, which are generally calculated based on the corrosion protection and the planned surface finishes, it may be necessary to allow for more reinforcement over the beams. Perimeter zones, ramps and other details must be considered on an individual basis.

Spans of up to 6.3 m without propping – even while construction is in progress

The loads generated in the construction phase are generally a crucial factor when designing long-span floor slabs because the fresh concrete has not yet solidified and the “permanent formwork” is the only element ensuring load transfer. Thus, it is the condition of the concrete that represents a critical load for the sheet metal section, even if a more favourable safety factor in the design concept can be selected here. Depending on the weight of the concrete and the depth of the floor above the sheet metal, the Cofraplus 220 System permits spans of up to 6.3 m without propping in the construction phase. If temporary joists are used during the construction phase, a clear span of up to 9.0 m is even possible in the final construction. In such cases, the ribs must be held by appropriate supports in order to prevent a local stability failure.

Prefabrication and erection with minimum tool requirements

The composite steel beams are already prefabricated to a large extent, including wings. This enables extremely fast and easy erection because the system generally requires no time-consuming assembly of complex parts and provides a thick shell for the concreting process. For sheet metal mounted at a higher level, openings between the steel beams and the floor sheets are simply closed off with a small, continuous Z-section. Owing to their light weight, the sheet metal panels can easily be positioned on each floor, saving time and crane capacity. A structure without propping in the construction phase therefore makes it possible to concrete several floors at a time without having to wait for the concrete to harden.

In conjunction with an integrated Slim Floor Beam from ArcelorMittal (CoSFB – Composite SLIM FLOOR BEAM), the system can achieve clear grid dimensions of approx. 9 × 10 m. This underscores the performance of the Cofraplus 220 floor system as a lightweight, cost-effective solution for long-span floor structures in multi-storey buildings, and particularly in car parks.

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ED.: ECCS – EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK

Design of Steel Structures
EC3: Design of Steel Structures.
Part 1-1: General Rules and Rules for Buildings.

April 2010.
446 pages, 295 fig., 105 tab., Softcover.
€ 70,–*

ISBN 978-3-433-02973-2

■ This book introduces the basis design concept of Eurocode 3 for current steel structures, and their practical application. Numerous worked examples will facilitate the acceptance of the code and provide for a smooth transition from earlier national codes to the Eurocode.

ED.: ECCS

Fire Design of Steel Structures
EC1: Actions on Structures.
Part 1-2: Actions on Structures exposed to Fire.
EC3: Design of Steel Structures.
Part 1-2: Structural Fire Design.

May 2010.
428 pages, 134 fig., 21 tab., Softcover.
€ 70,–*

ISBN 978-3-433-02974-9

■ This publication sets out the design process in a logical manner giving practical and helpful advice and easy to follow worked examples that will allow designers to exploit the benefits of the new approach given in the Eurocodes to fire design.

ED.: ECCS

Design of Plated Structures
EC3: Design of Steel Structures.
Part 1-5: Design of Plated Structures.

January 2011.
272 pages, 139 fig., 19 tab., Softcover.
€ 55,–*

ISBN 978-3-433-02980-0

■ This design manual provides practical advice to designers of plated structures for correct and efficient application of EN 1993-1-5 design rules and includes numerous design examples.

ED.: ECCS

Fatigue Design of Steel and Composite Structures
EC3: Design of Steel Structures.
Part 1-9: Fatigue.
EC4: Design of Composite Steel and Concrete Structures.

October 2011.
311 pages.
250 fig. Softcover.
€ 55,–*

ISBN 978-3-433-02981-7

■ This volume addresses the specific subject of fatigue, a subject not familiar to many engineers, but still relevant for proper and good design of numerous steel structures. It explains all issues related to the subject: Basis of fatigue design, reliability and various verification formats, determination of stresses and stress ranges, fatigue strength, application range and limitations. It contains detailed examples of application of the concepts, computation methods and verifications.

ED.: ECCS

preliminary

Design of Cold-formed Steel Structures
EC3: Design of Steel Structures.
Part 1-3: Design of Cold-formed Steel Structures.

I. Quarter 2012.
approx. 512 pages.
approx. 300 fig. Softcover.
approx. € 70,–*

ISBN 978-3-433-02979-4

■ The book is concerned with design of cold-formed steel structures in building based on the Eurocode 3 package, particularly on EN 1993-1-3. On this purpose, the book contains the essentials of theoretical background and design rules for cold-formed steel sections and sheeting, members and connections.

ED.: ECCS

preliminary

Design of Connections in Steel and Composite Structures

II. Quarter 2012.
approx. 500 pages, approx. 300 fig.
Softcover.
approx. € 70,–*

ISBN: 978-3-433-02985-5

■ This volume elucidates the design rules for connections in steel and composite structures which are set out in Eurocode 3 and 4. Numerous examples illustrate the application of the respective design rule.

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Eurocode 3 Part 1.5: Plated Structures – new chances and developments

Ulrike Kuhlmann

Large steel structures such as bridge girders or girders for heavy overhead cranes are usually built up from steel plates welded together – and are so slender that they are highly prone to buckling. With the introduction of the Eurocodes, in most European countries the design of slender steel plates is now covered by Eurocode 3 Part 1.5. Within the process of European harmonization, traditional methods as well as new research developments have found their way into the code put together by a very knowledgeable project team headed by *Bernt Johansson*. Earlier publications, also in this journal (e.g. "Steel Construction", No. 4, 2009), have provided some background.

Although the code covers many common stability problems for plated structures, there is still a need for, and possibility of, further development. For a long time, the technical development of rules for plate buckling has been driven forward by ECCS Technical Working Group (TWG) 8.3, a subgroup of ECCS Technical Committee (TC) 8 "Stability". Again and again over the years, research issues have often been motivated by questions coming from practice, such as bridge launches and the introduction of high concentrated forces (patch loading) associated with this, or the design of plated structures stiffened by modern trapezoidal stiffeners with high torsional rigidity. These are some of the topics discussed by experts from universities and industry. Joint research has been initiated and has resulted in several PhD theses with supervisors from different European universities. So true European collaboration has evolved, with some of the results being presented in "Steel Construction". An overview of recent developments, with a special focus on bridge design, was presented by *Ulrike Kuhlmann, Benjamin Braun, Hervé Degée* and *Antonio Zizza* in "Steel Construction" No. 4, 2011. A more extensive presentation with five papers of recent results of the work of ECCS TWG 8.3 can be found in this issue.

The good collaboration between members of TWG 8.3 has also resulted in the ECCS Eurocode design manual "Design of Plated Structures". This was produced in order to provide practical advice to designers of plated structures, so that they can apply the EN 1993-1-5 design rules correctly and efficiently.

On the other hand, CEN/TC250/SC3, responsible for Eurocode 3 in general, has created several expert groups (called Evolution Groups) for the maintenance and ongoing development of the different parts of Eurocode 3; one of those groups is the Evolution Group for EN 1993-1-5. As the two groups ECCS TWG 8.3 and CEN/TC250/SC3/EG 1-5 had a chairperson and many members in common, it was decided to bring both together in order to avoid duplication of effort. The first joint meeting took place in May 2011 in Paris, a second one in Stuttgart in November 2011. The first amendments for the future version of EN 1993-1-5 have been discussed and are partially reflected in the contributions on the following pages.

In this issue there is a focus on the patch loading situation, which is typical for crane runway beams and launching of bridge girders. An economically interesting alternative to homogeneous girders for such applications is the hybrid steel plate girder, with flanges of high-strength steel and a web of mild steel. *Rolando Chacón, Marina Bock, Enrique Mirambell* and *Esther Real* report on research into patch loading resistance – and reveal contradictions in the existing code rules. The method they devised is integrated into a new resistance model for steel plate girders subjected to patch loading, which has widely been discussed in TWG 8.3 because it summarizes the findings of several doctoral theses and offers a new, improved formulation for EN 1993-1-5. The statistical evaluation of this new resistance model is presented by *Rolando Chacón, Benjamin Braun, Ulrike Kuhlmann* and *Enrique Mirambell*.

Another interesting design alternative to flat web plates is the corrugated web, which allows for higher shear and also patch loading forces and avoids numerous expensive transverse stiffeners and frames. The joint work on the design of girders with trapezoidal corrugated webs under patch loading, shear and bending interaction is summarized, also in the light of new rules for EN 1993-1-5, by *Balász Kövesdi, Benjamin Braun, László Dunai* and *Ulrike Kuhlmann*.

One of the main discussion points during the drafting of the existing Eurocode 3 Part 1.5 was the design axial force in intermediate transverse stiffeners. So this subject has been on the agenda of TWG 8.3 for some time; it is also an important issue from the viewpoint of industrial practice. Meanwhile, recent research has opened up an interesting possibility that might solve this question through a minimum requirement for the stiffness of the intermediate transverse stiffener. This solution and its background are explained by *Franc Sinur* and *Darko Beg*.

At a very late stage of the development of EN 1993-1-5, a second method of verification was introduced into the code based on reduced stresses instead of reduced "effective" sections. This method is especially powerful for multi-axial stress situations and irregular geometries for which no other design rule exists. However, its traditions are mainly rooted in German design practice and it is not very well known in other parts of Europe. In "Reduced stress design of plates under biaxial compression", *Benjamin Braun* and *Ulrike Kuhlmann* outline the background to this method applied to a special stress state, identify deficiencies and also offer an improved solution.

At this point I want to thank all the contributors for their intensive collaboration and also the colleagues of ECCS TWG 8.3 and CEN/TC250/SC3/EG 1-5 for the many valuable discussions and contributions. And I would also like to thank the journal for the opportunity of this special issue.

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Rolando Chacón*
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Esther Real

Hybrid steel plate girders subjected to patch loading

Hybrid girders represent an economical alternative to homogeneous girders because they achieve greater flexural capacity with less material. One of the potential applications of hybrid steel plate girders is their use in bridges. One potential method of construction for these bridges is the push launch method in which patch loading may affect the design. The aim of this paper is to present the advanced conclusions of research work dealing with these two fields simultaneously: hybrid steel plate girders subjected to the particular case of patch loading. It is shown that, contrary to the EN 1993-1-5 formulation, the influence of the f_{yf}/f_{yw} ratio (namely, the hybrid grade) is negligible for both unstiffened and longitudinally stiffened girders according to the EN 1993-1-5 assumptions. Suggestions for considering these findings in design codes are provided at the end of the paper.

1 Introduction

The collapse behaviour of patch-loaded homogeneous plate girders has been widely reported [1, 2]. Researchers have proposed several expressions for the elastic critical loads and physical models which, referring to these expressions, accurately reproduce the limit state of the plates at ultimate load. Most of these approaches agree with a vast number of experimental results obtained from various sources. However, it has been pointed out that the vast majority of these studies only deal with the resistance of homogeneous plate girders [3–6].

The resistance of both unstiffened and longitudinally stiffened hybrid steel plate girders subjected to patch loading is studied in this work. The paper is based on the conclusions previously given in [3–6]. Such works demonstrate that, contrary to the EN 1993-1-5 provisions, the varying yield strength of the flange f_{yf} (and hence the hybrid grade) does not play any role in the resistance of girders subjected to concentrated loading. Highlights concerning this proposal are presented at the end of the paper.

2 Review of earlier work

A steel girder is deemed as being hybrid when it is fabricated with different steel strengths for the flange and web panels. This type of girder is popular because compared with a homogeneous girder, greater flexural capacity is obtained at lower cost and weight [7–8]. Extensive experimental, theoretical and numerical research on hybrid design can be found in the literature [9–13].

Patch loading phenomena has been widely analysed since the early 1960s. Experimental and theoretical analyses have pinpointed the failure mechanisms of girders subjected to patch loading and, consequently, ultimate load predictions have been provided [14–20].

Despite the vast amount of research devoted to hybrid girders and patch loading separately, research work covering both subjects is rather scant. *Schilling* [21] presented the first publication related to hybrid steel girders dealing explicitly with concentrated loading. Moreover, when studying the behaviour of slender girders subjected to patch loading, *Granath* [22] reached conclusions about the influence of the moment capacity of the flanges on the bearing capacity of plate girders subjected to concentrated loads. Following this thread, the authors of this paper subsequently presented several research works [3–5].

3 EN 1993-1-5

Verification of patch loading according to EN 1993-1-5 F_{Rd} is based on simplifications of the procedures provided in [1] and [2]. The general approach currently included in EN 1993-1-5 is based on a plastic resistance F_y which is partially reduced by means of the resistance function (Eq. (1)). The plastic resistance includes the key length parameter l_y, which is the yield-prone effectively loaded length. This length can be calculated from the geometrical and mechanical properties of the girders using Eqs. (2) and (3). More details concerning the EN 1993-1-5 formulation are given in a companion paper [23].

$$F_{Rd} = \frac{\chi_F \cdot F_y}{\gamma_{M1}} = \frac{\chi_F \cdot f_{yw} \cdot l_y \cdot t_w}{\gamma_{M1}} \le \frac{\chi_F \cdot f_{yw} \cdot a \cdot t_w}{\gamma_{M1}} \quad (1)$$

$$l_y = s_s + 2 \cdot t_f \cdot \left(1 + \sqrt{m_1 + m_2}\right) \le a \quad (2)$$

$$m_1 = \frac{f_{yf} \cdot b_f}{f_{yw} \cdot t_w} \qquad m_2 = 0.02 \cdot \left(\frac{h_w}{t_f}\right)^2 \quad (3)$$

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4 Numerical study

A numerical study was carried out using the FE-based software Abaqus-Simulia [24]. The software was used as a simulation tool and therefore numerical databases of unstiffened and longitudinally stiffened steel plate girders subjected to patch loading could be modelled and constructed. Details of those models can be found in [6].

4.1 Unstiffened girders

The simulations were performed on a single panel centrically loaded with a patch as sketched in Fig. 1a. The numerical database was constructed by varying the parameters depicted in Table 1, which resulted in 192 specimens.

4.2 Longitudinally stiffened girders

4.2.1 Open stiffeners

The simulations were performed on a single panel centrically loaded with a patch as sketched in Fig. 1b. The numerical database was constructed by varying the parameters given in Table 2. Other parameters such as the distance between transverse stiffeners (a = 9000 mm), the web yield strength (f_{yw} = 235 N/mm^2), the web depth (h_w = 3000 mm), the flange dimensions (80 × 900 mm), the transverse stiffener dimensions $t_{st} \cdot b_f$ (30 × 900 mm), the longitudinal stiffener width (b_{sl} = 300 mm) and the stiff bearing length (s_s = 2250 mm) were kept constant. Table 2 summarizes the set of variations.

4.2.2 Trapezoidal closed stiffeners

Several girders were provided with a closed, trapezoidal longitudinal stiffener (Fig. 1c). In this case, both the position and the stiffener rigidity were kept constant. Other parameters such as the distance between transverse stiffeners (a = 9000 mm), the web yield strength f_{yw} = 235 N/mm^2, the web depth h_w = 3000 mm, the flange dimensions (80 × 900 mm), the transverse stiffener dimensions (30 × 900 mm) and the stiff bearing length s_s = 2250 mm were kept constant as well. Table 3 gives the geometrical data of the specimens tested numerically. It is worth pointing out that for all cases, the longitudinal stiffener has the same thickness t_{sl} as the web plate t_w. Fig. 2 shows the geometrical proportions of the closed longitudinal stiffener.

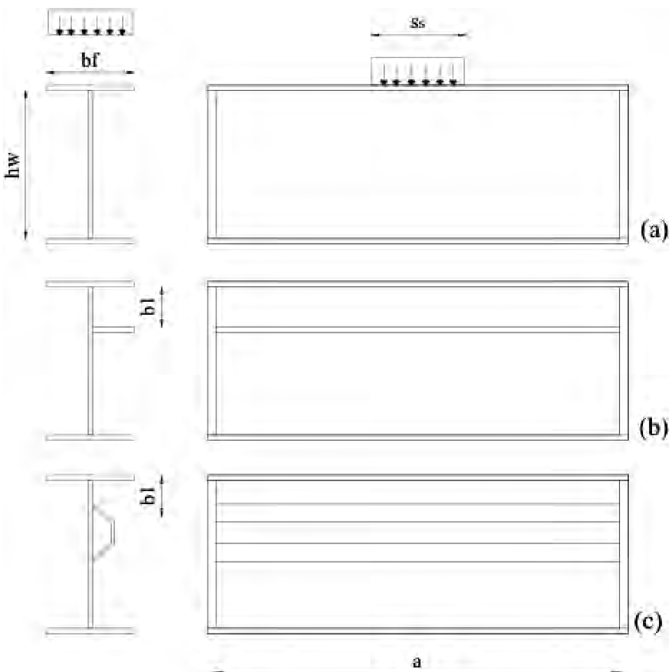


Fig. 1. Geometry of the modelled prototypes

Table 1. Numerical database of unstiffened girders

Numerical database variations	Group			
	U0	UI	UII	UIII
Web yield strength f_{yw} (N/mm^2)	235	235	235	235
Flange yield strength f_{yf} (N/mm^2)	235	235	235	235
	275	275	275	275
	355	355	355	355
	460	460	460	460
h_w (mm)	1000	2000	3000	4000
a (mm)	1000	2000	3000	4000
	2000	4000	6000	8000
	3000	6000	9000	12000
t_w (mm)	8	12	15	15
	12	20	25	30
s_s (mm)	250	500	750	1000
	500	1000	1500	2000
Flange dimensions (mm^2)	800 × 60	900 × 80	1000 × 80	1200 × 100
Stiffener thickness (mm)	40	60	60	80
Girders per group	48	48	48	48
Total number of numerical simulations	192			

Table 2. Numerical database of longitudinally stiffened girders, open stiffeners

Dimensions	LS0	LSI	LSII	LSIII	LSIV
h_w (mm)	3000	3000	3000	3000	3000
t_w (mm)	10	15	20	25	30
f_{yf} (N/mm^2)	235	235	235	235	235
	355	355	355	355	355
	460	460	460	460	460
b_1 (mm)	300	300	300	300	300
	600	600	600	600	600
	900	900	900	900	900
	1200	1200	1200	1200	1200
t_{sl} (mm)	10	10	10	10	10
	20	20	20	20	20
	30	30	30	30	30
	40	40	40	40	40

Table 3. Numerical database of longitudinally stiffened girders, closed stiffeners

Dimensions	LSa	LSb	LSc	LSd	LSe
h_w (mm)	3000	3000	3000	3000	3000
$t_{sl} = t_w$ (mm)	8	10	12	15	20
f_{yf} (N/mm^2)	235	235	235	235	235
	355	355	355	355	355
	460	460	460	460	460
b_1 (mm)	900	900	900	900	900

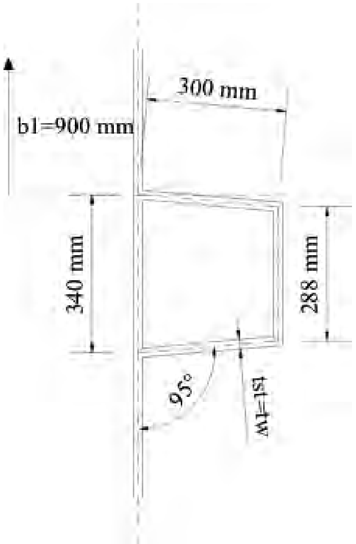


Fig. 2. Geometric proportions of the closed longitudinal stiffener

5 Numerical results

Results in the form of ultimate load capacity F_u, load-displacement plots, load-stress plots and thorough comparisons with EN 1993-1-5 are widely available in previously published papers from the authors [3–5]. Tables with precise data concerning each numerically simulated prototype as well as several plots and contours of the simulations are also provided. In this paper, and for the sake of conciseness, only the key results are displayed. These results aim at show the most remarkable findings of the present research work.

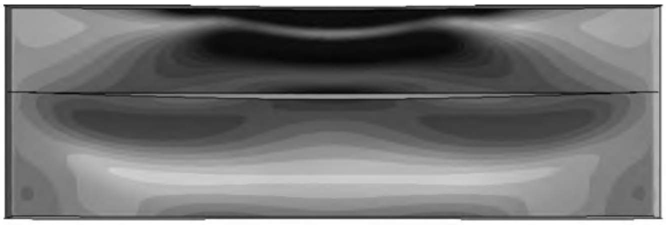
5.1 Ultimate load capacity

The ultimate load capacity of the steel plate girders subjected to patch loading has been obtained as the maximum load the prototype is able to carry in an incremental process. The observed failure modes are in accordance with the typical failure modes associated with patch loading. Slender girders tend to fail by local instability, whereas stocky girders tend to fail by an intertwined local yielding-buckling mode. Fig. 3 shows a qualitative plot in which local folding of the loaded panel is noticeable.

Moreover, numerical results for F_u are illustrated graphically in Fig. 4a for unstiffened girders and Fig. 4b for longitudinally stiffened girders. These results are plotted on the χ-λ space for comparison purposes.

It is possible to draw the following conclusions from these plots:

- All results are located above the EN 1993-1-5 (on the safe side).
- Generally, the results follow a typical *Euler*ian hyperbolic shape.
- Some girders highlighted in Fig. 4a show anomalous results. Their geometrical proportions are such that the EN 1993-1-5 formulation must be modified slightly and therefore these girders do not follow the same trends.



Web folding of the loaded panel

Web folding of both loaded panels

Fig. 3. Typical failure modes

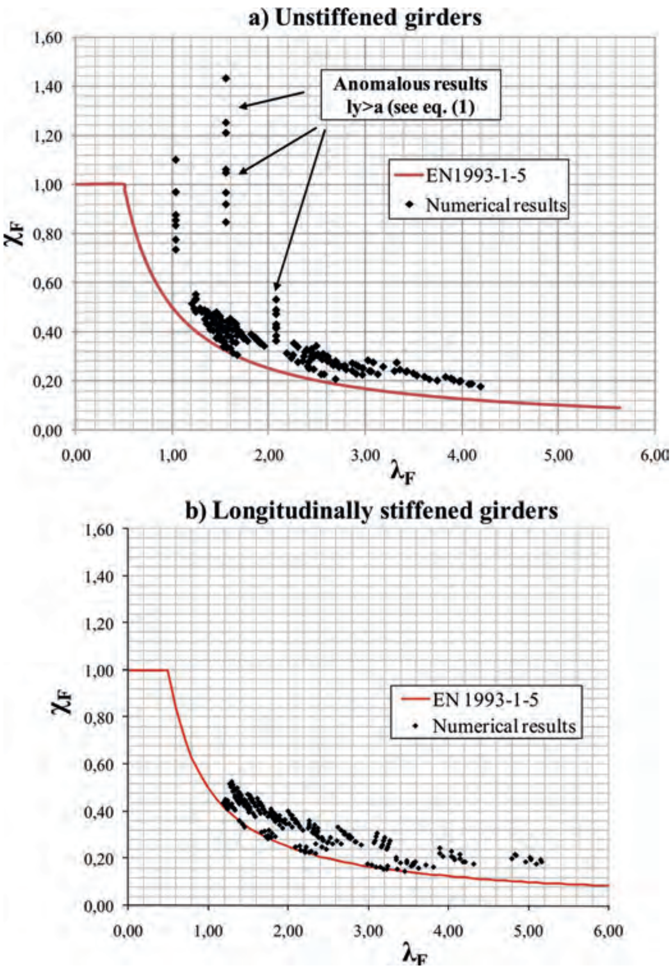


Fig. 4. Ultimate load capacity of the numerical prototypes

These girders have closely spaced transverse stiffeners (in these cases, the effectively loaded length l_y is greater than the distance between transverse stiffeners a, i.e. l_y > a) and their failure modes do not fully match with patch loading. Additional research concerning this topic has been addressed by the authors [25].

5.2 Load-deflection plots

Load-deflection plots characterize the response of the steel plate girders subjected to patch loading. In a hypothetical incremental process, the applied load is plotted against the vertical displacement of a point located on the loaded flange. The response is initially linear but becomes gradually non-linear until the load reaches a maximum. From this maximum load onwards, a softening branch is recorded and the deformation increases sharply.

Fig. 5 shows four load-deflection plots of selected prototypes from the numerical database of unstiffened girders. The main varying parameter between the four plots is the web depth of the specimens and therefore the web slenderness h_w/t_w. Fig. 5a represents the stockiest girders, whereas Fig. 5d represents the most slender prototypes.

Within each plot, load-displacement curves as well as values of F_u are shown for girders with a varying hybrid grade f_{yf}/f_{yw}. The following features can be gleaned from these plots:

- Irrespective of the web slenderness, all load-displacement curves are identical because the f_{yf}/f_{yw} ratio is varied. The response of the girders does not depend on the variation of the flange yield strength.
- Irrespective of the web slenderness, the ultimate load capacities F_u are identical because the f_{yf}/f_{yw} ratio is varied. F_u is not dependent on the flange yield strength either.
- Stocky girders present a quite marked linear branch, whereas in slender prototypes the curves are non-linear from low levels of load onwards.

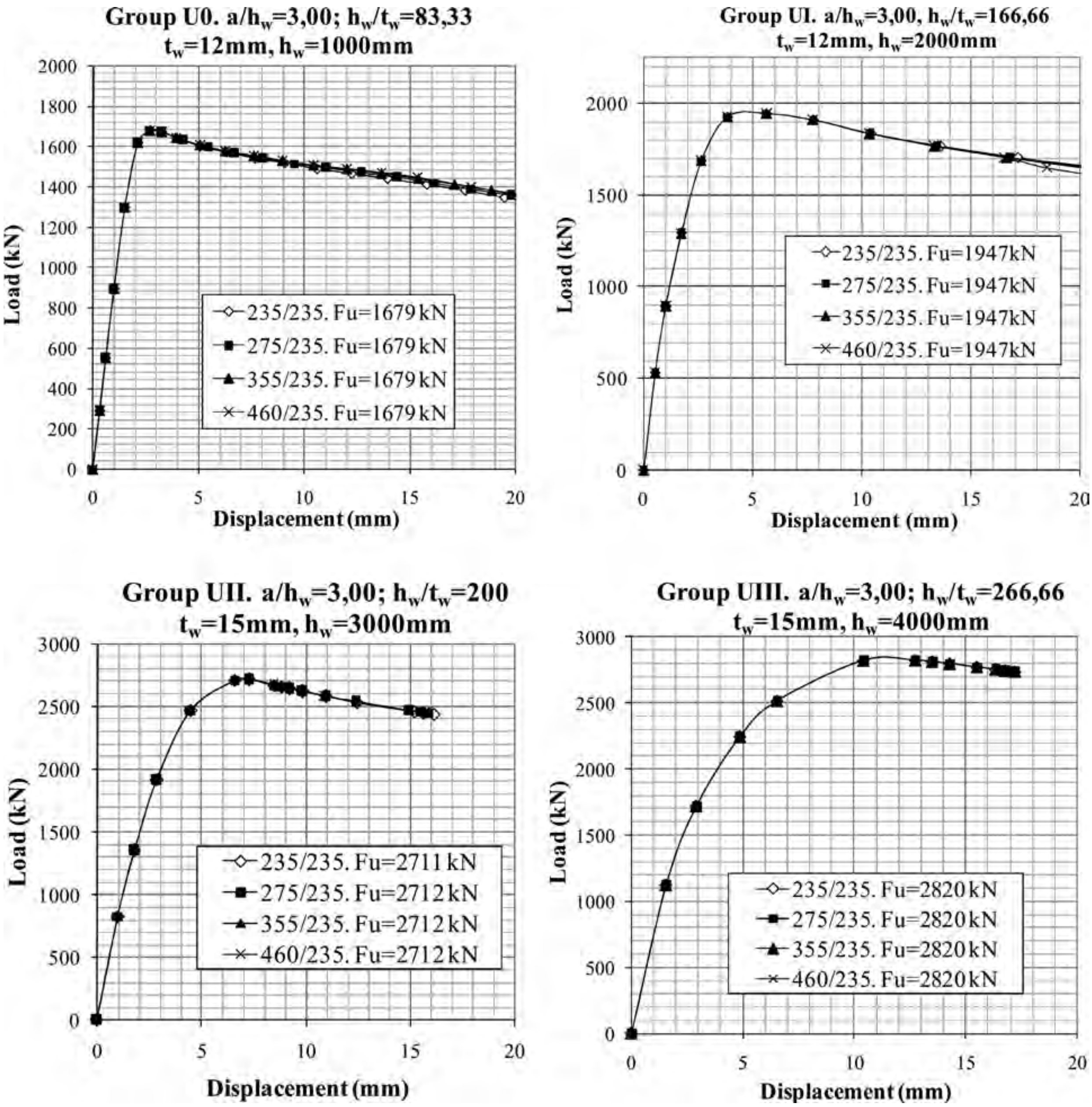


Fig. 5. Load displacement plots for unstiffened girders

On the other hand, Fig. 6 shows four load-deflection plots of selected prototypes from the numerical database of longitudinally stiffened girders. The main varying parameter between the four plots is the position of the longitudinal stiffened b_1 and therefore the b_1/h_w ratio. Fig. 6a represents the longitudinal stiffener closest to the loaded flange ($b_1/h_w = 0.1$), whereas Fig. 6d represents the longitudinal stiffener with a relative position $b_1/h_w = 0.4$.

Within each plot, load-displacement curves as well as values of F_u are shown for girders with varying hybrid grade f_{yf}/f_{yw}. The following features should be pointed out:

- Irrespective of the position of the longitudinal stiffener, all load-displacement curves are identical because the f_{yf}/f_{yw} ratio is varied. The response of the girders does not depend on the variation of the flange yield strength.
- Irrespective of the position of the longitudinal stiffener, the ultimate load capacities F_u are identical because the f_{yf}/f_{yw} ratio is varied. F_u is not dependent on the flange yield strength either.

5.3 Hybrid vs. homogeneous prototypes – numerical and EN 1993-1-5 results

The ultimate load capacity F_u of a given hybrid girder $F_{u,hyb}$ can be compared with the ultimate load capacity F_u of its equivalent homogeneous girder $F_{u,hom}$. If F_u depends on f_{yf}/f_{yw}, then $F_{u,hyb}$ should differ from $F_{u,hom}$. Fig. 7 shows the ratio $F_{u,hyb}/F_{u,hom}$ as f_{yf}/f_{yw} is varied for unstiffened girders (Fig. 7a) and longitudinally stiffened girders (Fig. 7b). Numerical as well as EN 1993-1-5 results are included in each plot.

From these figures, the following features should be pointed out:

- For both unstiffened and longitudinally stiffened girders, the numerical results show very little dependency on f_{yf}/f_{yw}. The $F_{u,hyb}/F_{u,hom}$ ratios vary, ranging from $F_{u,hyb}/F_{u,hom}$ = 1.00 to 1.02 (variation that might arguably be attributed to numerical reasons).
- For both unstiffened and longitudinally stiffened girders, the EN 1993-1-5 results show a significantly strong dependency on f_{yf}/f_{yw}. The $F_{u,hyb}/F_{u,hom}$ ratios vary,

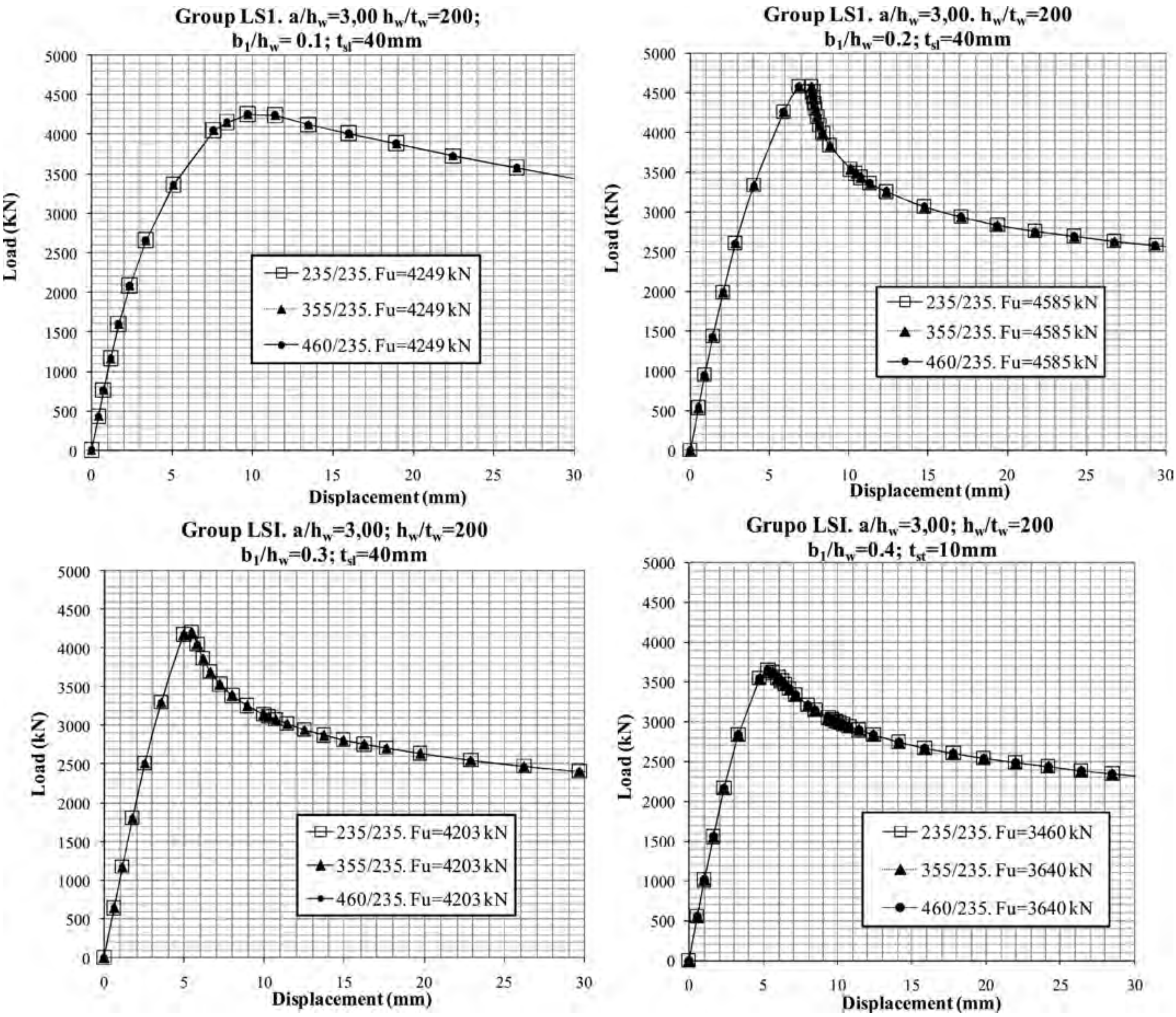


Fig. 6. Load displacement plots for longitudinally stiffened girders

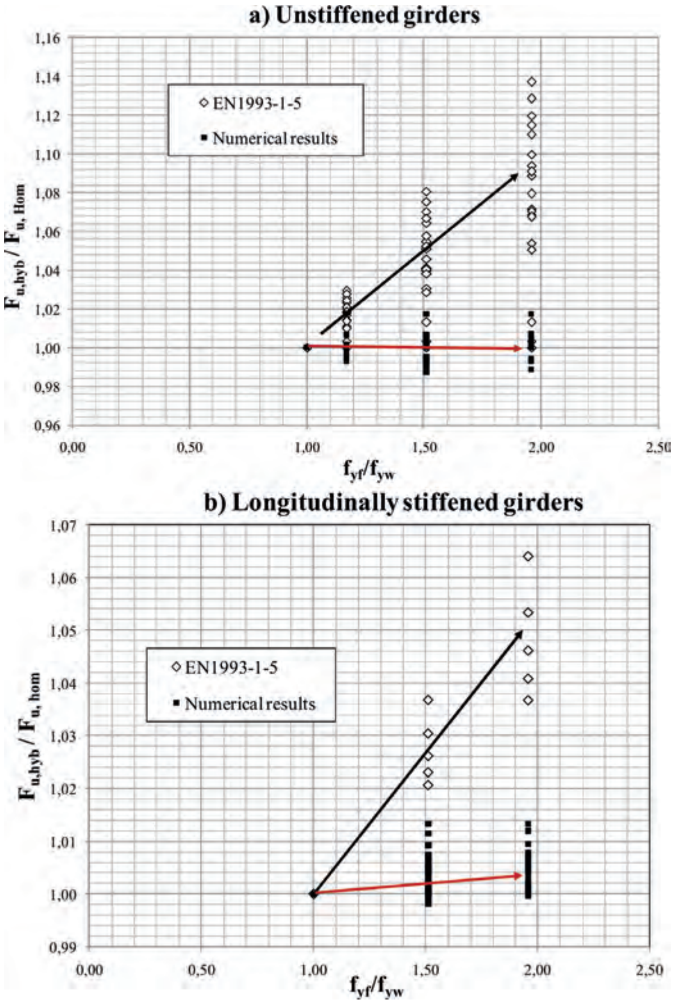


Fig. 7. $F_{u,hyb}/F_{u,hom}$

ranging from $F_{u,hyb}/F_{u,hom}$ = 1.02 to 1.14. This anomaly is structurally unsafe and should be corrected in the formulation.

6 Design proposal

The results obtained suggest that the patch loading resistance should not depend on the hybrid parameter f_{yf}/f_{yw} to any extent because the flange yield resistance does not seem to play any role in the development of the collapse mechanism.

For the sake of correcting the aforementioned anomaly, a modification to the current EN 1993-1-5 formulation that enhances the results quite satisfactorily is provided. Readably, it is proposed that the m_1 coefficient in Eq. (3) be replaced by m_1^*, see Eq. (4):

$$l_y = s_s + 2 \cdot t_f \left(1 + \sqrt{\frac{b_f}{t_w} + 0{,}02 \cdot \left(\frac{h_w}{t_f} \right)^2} \right) \qquad m_1^* = \frac{b_f}{t_w} \qquad (4)$$

This proposal has been tested both structurally and statistically. The results lead to a satisfactory improvement of the formulation. The results obtained with the upgraded coefficient m_1^* are structurally sound and on the safe side.

7 Conclusions

A vast numerical study of more than 800 hybrid and homogeneous steel plate girders has been presented in this paper. The numerical study is based on girders with geometrically realistic proportions. The parametric variation of the most relevant variables influencing the resistance of girders to patch loading is quite profuse, although the particular emphasis was on the hybrid grade f_{yf}/f_{yw}.

The numerical results presented for both transversally and longitudinally stiffened girders do not agree with the results provided by EN 1993-1-5 when the focus is on the effect of the flange yield strength (and therefore the hybrid parameter f_{yf}/f_{yw}). Numerically, it is predicted that for girders with widely spaced transverse stiffeners ($l_y < a$), f_{yf}/f_{yw} has no influence on the ultimate load capacity of patch-loaded girders. The current formulation of EN 1993-1-5 takes this ratio into account in such a way that the greater the ratio f_{yf}/f_{yw}, the higher is the ultimate load capacity of the girders. It is worth pointing out that this parameter appears explicitly in the term m_1.

The current formulation of EN 1993-1-5 leads to structurally unsafe results. An attempt to correct this anomaly involves making the f_{yf}/f_{yw} ratio equal to 1.0 in the current expressions for F_{Rd}. This attempt leads to reshaping the effectively loaded length l_y in Eq. (4), in which m_1 is replaced by m_1^*. This proposal has been tested both structurally and statistically in [3–5]. For reliability purposes, further research concerning the recalibration of the resistance function and the partial safety factor is addressed in a companion paper [23].

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Keywords: hybrid girders; patch loading

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Peoples

Dr. Norbert Bannenberg reappointed to the Board of Management

The Supervisory Board of the Aktien-Gesellschaft der Dillinger Hüttenwerke has again appointed Dr. *Norbert Bannenberg* (59) as a member of the Board of Management for the period 1 June 1 to 31 December 2015.

Dr. *Bannenberg* has been a member of the Dillinger Hütte Board of Management since 1 June 2002, and is responsible for the „Technology“ division.

The other members of the Board of Management are Dr. *Karlheinz Blessing*, Chief Executive Officer with responsibility for the Commercial Division, *Fred Metzken*, Executive Officer, Financial Division and *Peter Schweda*, Executive Officer, Human Resources and Labour director.

Norbert Bannenberg reappointed to the Board of Management until 31 December 2015

Rolando Chacón*
Benjamin Braun
Ulrike Kuhlmann
Enrique Mirambell

DOI: 10.1002/stco.201200002

Statistical evaluation of the new resistance model for steel plate girders subjected to patch loading

This paper presents a new resistance model for steel plate girders subjected to patch loading. The model aims to correct the EN 1993-1-5 formulation concerning the resistance to transverse forces with the help of findings presented in several doctoral studies on this topic. In addition, a statistical calibration of the new model is addressed. This calibration includes a new proposal for the resistance function χ-λ. The proposal is conceived for fulfilling safety levels of the partial safety factor that match the current values included in EN 1993 for instability-related problems. These new proposals are in accordance with the design philosophy found in EN 1993-1-5 for the design of plated structures.

1 Introduction

The adequate and safe formulation of the patch loading resistance is crucial in the design of launched steel bridges. The resistance of plate girders subjected to patch loading has been studied thoroughly. A large number of contributions concerning the web resistance to patch loading are available in the literature. Two milestone research works [1, 2] provide the framework for the formulation for the patch loading resistance presently implemented within EN 1993-1-5 [3]. In these works, the patch loading resistance is formulated by following a similar procedure found in other instability-based verifications, namely the χ-λ approach. In this approach, plastic and elastic critical buckling resistances (F_y and F_{cr} respectively) are defined separately and subsequently related to the ultimate load by a resistance function $\chi = f(\lambda)$.

Recent contributions [4–9] underpin the need for further modification of the plastic resistance F_y which appears in the current version of EN 1993-1-5. According to these studies, the definition of the plastic resistance overestimates the patch loading capacity in certain cases (hybrid girders), whereas this capacity is slightly underestimated for others (very slender girders).

This paper provides a brief description of the modification needed. Furthermore, a design proposal including a new patch loading resistance model and a new resistance function – calibrated based on hundreds of experimental and numerical results – are outlined.

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2 Current formulation in EN 1993-1-5

The verification of patch loading F_{Rd} is based on simplifications of the procedures provided in [1] and [2]. The general approach currently included in EN 1993-1-5 is based on a plastic resistance F_y which is reduced by means of the resistance function (Eq. (1)). The plastic resistance includes the length l_y, which can be calculated from the geometrical and mechanical properties of the girders using Eqs. (2) and (3). Plate instability is accounted for by means of Eq. (4). The buckling coefficient k_F is given in Eqs. (5), (6) and (7). This formulation can be applied even if the patch load is applied on only one side of the steel girder with the assumption that the flange is prevented from rotating about its longitudinal axis. The plastic resistance as well as the elastic critical buckling loads are both related to the ultimate load by means of the resistance function (Eq. (8)) and the relative slenderness (Eq. (9)). The formulation includes a partial safety factor γ_{M1}.

$$F_{Rd} = \frac{\chi_F \cdot F_y}{\gamma_{M1}} = \frac{\chi_F \cdot f_{yw} \cdot l_y \cdot t_w}{\gamma_{M1}} \le \frac{\chi_F \cdot f_{yw} \cdot a \cdot t_w}{\gamma_{M1}} \tag{1}$$

$$l_y = s_s + 2 \cdot t_f \cdot \left(1 + \sqrt{m_1 + m_2}\right) \le a \tag{2}$$

$$m_1 = \frac{f_{yf} \cdot b_f}{f_{yw} \cdot t_w} \qquad m_2 = 0.02 \cdot \left(\frac{h_w}{t_f}\right)^2$$

$$\text{if } \overline{\lambda}_F \ge 0.5, \text{ otherwise } m_2 = 0 \tag{3}$$

$$F_{cr} = 0.9 \cdot k_F \cdot E \cdot \frac{t_w^3}{h_w} \tag{4}$$

$$k_F = 6 + 2 \cdot \left(\frac{h_w}{a}\right)^2 \tag{5}$$

$$k_F = 6 + 2 \cdot \left(\frac{h_w}{a}\right)^2 + \left(5.44 \frac{b_1}{a} - 0.21\right)\sqrt{\gamma_s} \tag{6}$$

$$\gamma_s = 10.9 \frac{I_{sl}}{h_w \cdot t_w^3} \le 13 \cdot \left(\frac{a}{h_w}\right)^3 + 210 \cdot \left(0.3 - \frac{b_1}{a}\right) \tag{7}$$

$$\chi_F = \frac{0.5}{\overline{\lambda}_F} \le 1.0 \tag{8}$$

$$\overline{\lambda_F} = \sqrt{\frac{f_{yw} \cdot l_y \cdot t_w}{F_{cr}}} \tag{9}$$

3 Recent discussions on the EN 1993-1-5 formulation

Ever since the implementation of the latest version of EN 1993-1-5, major studies concerning the patch loading resistance of both homogeneous and hybrid transversally and longitudinally stiffened girders have been undertaken in France [4], Sweden [5], [6] and Spain [7–9]. These studies have pointed out that, due to safety concerns, the effectively loaded length l_y given in Eq. (2) requires a major modification.

3.1 Research work performed in France

In 2005, *Davaine* [4] studied the resistance of longitudinally stiffened steel plate girders subjected to patch loading. The main purpose of her thesis was to validate the aforementioned χ-λ approach. The question was whether the χ-λ approach was valid or not for the domain of steel plate girders with realistic proportions and not only those tested in laboratories worldwide (the main basis used by [1] and [2] to calibrate their models). After a rigorous analysis based upon results obtained with the FE models, it was suggested to maintain the χ-λ approach for the resistance to patch loading. Despite the fact that the results were deemed to be rather too far on the safe side, this approach led to the best statistical fit when compared with other methods. A new resistance function for recalibrating EN 1993-1-5 was provided based on the work presented by *Müller* [10]. Nevertheless, a very important issue concerning the patch loading phenomenon arose: throughout the development of the thesis, *Davaine* questioned the physical meaning of the term m_2 (see Eq. (3)). The thesis did not focus on understanding the validity of this term, but it paved the way for further studies.

3.2 Research work performed in Sweden

Subsequently, major doctoral studies [5, 6] were conducted in Sweden. These studies followed the trends depicted by [1], [2] and [4] concerning the validity of the χ-λ approach as the fundamental method for calculating the resistance of steel plate girders subjected to patch loading. Numerical studies on steel plate girders subjected to patch loading were carried out for the sake of evaluating the validity of the prediction of the effective loaded length l_y (Eq. (2)). In particular, the main focus was on the term m_2. The effective loaded length was determined from the numerical observations according to Eq. (10). Such works drew similar conclusions to those in [4].

3.3 Research work performed in Spain

Chacón et al. [7–9] have presented a research work on patch loading based on a larger research project concerning the design of hybrid girders (with $f_{yf} > f_{yw}$). The numerical results presented for both transversally and longitudinally stiffened girders did not agree with the results provided by EN 1993-1-5 when the focus was on the effect of the flange yield strength (and hence the hybrid parameter f_{yf}/f_{yw}). Numerically, it was predicted that f_{yf}/f_{yw} has absolutely no influence on the ultimate load capacity of patch-loaded girders. The current formulation in EN 1993-1-5 takes this ratio into account in such a way that the greater the ratio f_{yf}/f_{yw}, the higher is the ultimate load capacity of the girders. It is worth pointing out that this parameter appears explicitly in the term m_1.

Summarizing, Fig. 1 shows the timeline of studies concerning the patch loading resistance in recent years. In addition, although not referenced in the paper, the figure lists the researchers who set up the framework for the resistance of steel plate girders subjected to patch loading from 1960 onwards.

4 New proposals for patch loading resistance

Two independent modifications of the patch loading resistance have been separately proposed to ECCS Technical Working Group TWG.8.3 (Plate Buckling).

4.1 Deletion of m_2

The results obtained by *Davaine* [4], *Gozzi* [5] and *Clarin* [6] consistently show a better prediction of the patch loading resistance when the term m_2 is omitted from the formula. According to these studies, the effective loaded length should be modified (Eq. (10)):

$$l_y = s_s + 2 \cdot t_f \left(1 + \sqrt{\frac{f_{yf} \cdot b_f}{f_{yw} \cdot t_w}}\right) \tag{10}$$

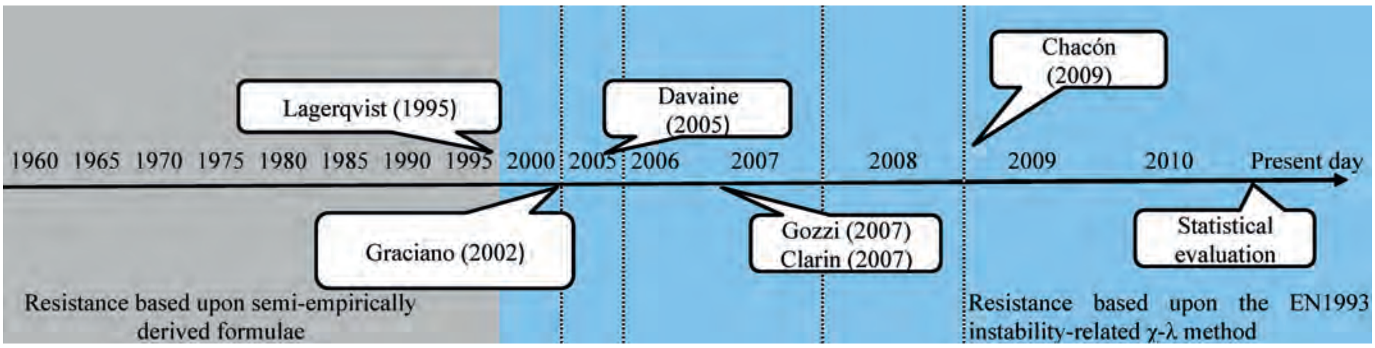


Fig. 1. Timeline of recent studies concerning patch loading

4.2 Replacing m_1 with m_1^*

The results obtained by *Chacón* et al. [7–9] suggest that the patch loading resistance should not depend on the hybrid parameter f_{yf}/f_{yw} to any extent because the flange yield resistance does not seem to play any role in the development of the collapse mechanism.

For the sake of correcting the aforementioned anomaly, a modification of the current EN 1993-1-5 formulation provides quite satisfactory results. Thus, it is proposed, as given below, that the m_1 coefficient be replaced by m_1^* (Eq. (11)):

$$l_y = s_s + 2 \cdot t_f \left(1 + \sqrt{\frac{b_f}{t_w} + 0.02 \cdot \left(\frac{h_w}{t_f}\right)^2}\right) \tag{11}$$

4.3 Final modification

After scrutiny and debate within TWG 8.3, it has been decided to merge both proposals into a single formula (Eq. (12)) that predicts the effective loaded length when a steel plate girder carries a concentrated load. This final proposal is based on identical assumptions of the prior formulation.

$$l_y = s_s + 2 \cdot t_f \left(1 + \sqrt{\frac{b_f}{t_w}}\right) \tag{12}$$

For the sake of completeness, a recalibration of the resistance function χ-λ was performed in a collaborative task between the research group from the University of Stuttgart [11] and the group from the UPC [12].

5 Resistance function for patch loading χ_F-λ_F

The resistance function relates the reduction factor χ_F to the relative slenderness λ_F of the plate. The current formulation χ_F-λ_F for the patch loading resistance according to EN 1993-1-5 is based on slight modifications of the proposals presented in [1] and [2] (Eq. (13)):

$$\chi_F = \frac{0.5}{\bar{\lambda}_F} \tag{13}$$

This resistance function has been statistically calibrated to fit both the plastic resistance F_y and the elastic critical buckling load F_{cr} depicted in Eqs. (1) to (9) and to merge both magnitudes in a single formulation. The calibration has been undertaken primarily with the experimental databases existing until 1995 for unstiffened girders and 2002 for longitudinally stiffened girders.

It is understood that any major change to such a formula implies recalibrating the resistance function. Furthermore, where additional experimental or numerical studies are available, these prototypes may be included in the pool of tests to obtain a higher level of statistical significance. For the sake of recalibrating the resistance functions with the newly proposed plastic resistance F_y, two diploma theses have been recently performed at the University of Stuttgart and at the Technical University of Catalonia [11, 12]. These works gather together all the experimental and numerical tests available on the welded steel plate girders subjected to patch loading. These databases include all the specimens used by [1] and [2] for their prior calibration as well as the newer tests available since then.

After consensus within TWG 8.3, the group suggested that any modification of the resistance curve should be based on the proposal presented by *Müller* [10], which takes the form of the equation included in EN 1993-1-5, Annex B. This proposal is advantageous since it harmonizes the resistance function shape for all verifications of compressed members. Eqs. (14) and (15) show the general form of this proposal:

$$\chi_F = \frac{1{,}0}{\phi_F + \sqrt{\phi_F^2 - \bar{\lambda}_F}} \le 1{,}0 \tag{14}$$

$$\phi_F = \frac{1}{2}\left(1 + \alpha_{F0} \cdot \left(\bar{\lambda}_F - \bar{\lambda}_{F0}\right) + \bar{\lambda}_F\right) \tag{15}$$

In these equations, α_{F0} and $\bar{\lambda}_{F0}$ are tuneable magnitudes that may be calibrated for the sake of achieving the desired level of safety. The former is an imperfection factor and the latter the plateau length of the resistance function. For the sake of obtaining sound values for these magnitudes, a systematic study has been carried out to establish the sensitivity of the safety level of the formulation based on these values.

The main random variable accounted for in this statistical study is the ratio between the ultimate load capacity F_u and the EN 1993-1-5 prediction F_{Rk} ($b = F_u/F_{Rk}$).

The main statistics used in the sensitivity evaluation are the mean, the minimum and maximum values b_{min} and b_{max}, the standard deviation, the coefficient of variation and the percentage of unsafe values of the random variable (if $b_i < 1.0$, it is not on the safe side).

Table 1 shows the sensitivity analysis of the aforementioned magnitudes when both α_{F0} and $\bar{\lambda}_{F0}$ are systematically varied in unstiffened girders. Table 2 shows a similar table for longitudinally stiffened girders. Different rows represent different values of the plateau length $\bar{\lambda}_{F0}$[0.2; 0.3; 0.4; 0.5; 0.6; 0.7], whereas different columns represent different values of the imperfection factor α_{F0}[0.25; 0.50; 0.75; 1.0].

Both tables employ colours for a better understanding. The sensitivity of the formulation of $\bar{\lambda}_{F0}$ can be observed by following a given column from top to bottom. Closer inspection leads to the following conclusions:

- The greater the plateau length, the less conservative is the formulation. The mean, minimum and maximum values decrease, whereas the percentage of values below the safety threshold increases.
- The standard deviation and the variation remain nearly constant, which is statistically sound.

The sensitivity of the formulation with regard to the imperfection factor α_{F0} can be assessed by following columns with the same colours (e.g. all red columns). Closer inspection of the table leads to the following conclusions:

- The greater the imperfection factor, the safer is the formulation.
- The coefficient of variation remains constant, whereas the standard deviation increases slightly.

Table 1. Statistics obtained for the patch loading resistance of unstiffened girders, $n = 478$

α_{F0} / $\bar{\lambda}_{F0}$	0.25						0.5						0.75						1.0					
	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation
0.2	1.21	0.77	2.08	16.53	0.23	0.19	1.46	0.94	2.42	2.09	0.28	0.19	1.70	1.09	2.83	0.00	0.32	0.19	1.92	1.23	3.22	0.00	0.36	0.19
0.3	1.19	0.76	2.01	19.25	0.23	0.19	1.43	0.92	2.41	2.09	0.27	0.19	1.65	1.06	2.80	0.00	0.31	0.19	1.86	1.19	3.19	0.00	0.35	0.19
0.4	1.17	0.75	1.99	23.22	0.22	0.19	1.40	0.89	2.39	4.60	0.26	0.19	1.61	1.03	2.78	0.00	0.30	0.19	1.81	1.15	3.16	0.00	0.34	0.19
0.5	1.16	0.73	1.98	26.75	0.22	0.19	1.37	0.87	2.37	4.60	0.26	0.19	1.61	1.03	2.78	0.00	0.30	0.19	1.75	1.11	3.12	0.00	0.33	0.19
0.6	1.14	0.72	1.97	30.96	0.22	0.19	1.34	0.85	2.35	5.23	0.25	0.19	1.52	0.96	2.72	0.84	0.30	0.19	1.69	1.07	3.09	0.00	0.33	0.20
0.7	1.12	0.71	1.96	32.43	0.21	0.19	1.30	0.83	2.33	10.25	0.25	0.19	1.48	0.90	2.68	4.18	0.29	0.20	1.64	0.93	3.05	0.84	0.33	0.20

Mean | Minimum | Maximum | $\%F_u/F_{Rk} < 1$ | Standard D | Variation

Table 2. Statistics obtained for the patch loading resistance of longitudinally stiffened girders, $n = 770$

α_{F0} / $\bar{\lambda}_{F0}$	0.25						0.5						0.75						1.0					
	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation	Mean	Minimum	Maximum	$\%F_u/F_{Rk} < 1$	Standard D	Variation
0.2	1.19	0.75	1.68	6.78	0.17	0.14	1.43	0.91	2.04	0.42	0.20	0.14	1.65	1.05	2.38	0.00	0.23	0.14	1.85	1.19	2.48	0.00	0.24	0.14
0.3	1.17	0.74	1.66	10.59	0.17	0.14	1.40	0.89	2.01	0.85	0.20	0.14	1.60	1.02	2.33	0.00	0.23	0.14	1.79	1.15	2.50	0.00	0.25	0.15
0.4	1.15	0.72	1.64	13.14	0.16	0.14	1.36	0.87	1.92	1.69	0.19	0.14	1.55	0.99	2.27	0.42	0.23	0.15	1.72	1.11	2.47	0.00	0.26	0.16
0.5	1.12	0.71	1.62	19.07	0.16	0.14	1.32	0.84	1.94	2.54	0.19	0.15	1.49	0.96	2.22	0.85	0.23	0.16	1.66	0.99	2.50	0.42	0.27	0.17
0.6	1.10	0.70	1.60	25.42	0.16	0.14	1.28	0.82	1.90	3.81	0.19	0.15	1.44	0.93	2.17	1.69	0.23	0.16	1.59	0.95	2.43	0.85	0.27	0.17
0.7	1.08	0.69	1.58	36.44	0.16	0.14	1.24	0.80	1.86	5.80	0.19	0.14	1.40	0.90	2.12	2.68	0.23	0.14	1.54	0.96	2.36	1.79	0.26	0.13

Mean | Minimum | Maximum | $\%F_u/F_{Rk} < 1$ | Standard D | Variation

It can be seen that any decision concerning the imperfection factor and the plateau length alters the statistics related to the safety of the formulation. As the formulation also includes a partial safety factor γ_{M1}, it is inferred that this decision would also alter the value of γ_{M1} in the following way:

- If a combination of α_{F0} and $\bar{\lambda}_{F0}$ is selected from the top left area of Tables 4 and 5 (a less safe combination), the partial safety factor would be rather high ($> \gamma_{M1} = 1.1$) because it would "correct" the prior, unsafe level.
- If a combination of α_{F0} and $\bar{\lambda}_{F0}$ is selected from the bottom right area of Tables 4 and 5 (a very safe combination), the partial safety factor would be rather low (tending towards $\gamma_{M1} = 1.0$).

6 Partial safety factor γ_{M1}

The partial safety factor is determined by means of the standard evaluation procedure design method assisted by testing described in EN 1990 [13]. The partial safety factor accounts for the unavoidable uncertainties inherent in the design model, which is given by Eq. (16):

$$F_{Rk} = \chi_F \cdot f_{yw} \cdot l_y \cdot t_w \tag{16}$$

The design model comprises physical magnitudes related to the geometry of the prototypes (width, thickness) as well as the material of the plate (yield strength). These magnitudes present a certain variation that has been quantified by means of extensive statistical studies performed mainly by the steel industry. The procedure is based on a set of sequential steps that involve complex statistical concepts. These steps are summarized in Table 3, together with the main objectives and quantities involved for each one. Further information concerning this matter can be found in [11–13].

Once the procedure is applied, the values of the corrected partial safety factor γ^*_{M1} can be obtained. Studies concerning the sensitivity of γ^*_{M1} with regard to different combinations of α_{F0} and $\bar{\lambda}_{F0}$ are available in [11] and [12].

It has been decided within TWG 8.3 to provide two different combinations of the imperfection factor α_{F0} and the plateau length $\bar{\lambda}_{F0}$. The aim of these combinations is to obtain alternative values of $\gamma^*_{M1} = 1.0$ or $\gamma^*_{M1} = 1.1$. As a result, once the decision is taken as to which final partial safety factor will be used, adequate values of α_{F0} and $\bar{\lambda}_{F0}$ can be selected. Table 4 shows the required and adequate values of α_{F0} and $\bar{\lambda}_{F0}$ for target values of the final partial safety factor $\gamma^*_{M1} = 1.0$ or $\gamma^*_{M1} = 1.1$.

7 Conclusions

Based on the doctoral studies presented in section 2, two independent modifications of the patch loading resistance have been separately proposed to ECCS Technical Working Group TWG.8.3 (Plate Buckling):

- Deletion of m_2 (original proposal: France and Sweden)
- Replacing m_1 with m^*_1 (original proposal: Spain)

After scrutiny and debate within TWG 8.3, it has been decided to merge both proposals into a single formula that predicts the effective length loaded when a steel plate girder carries a concentrated load. This final proposal is based on identical assumptions of the prior formulation. For the sake of completeness, a recalibration of the resistance function χ-λ is depicted throughout the document.

Table 3. Procedure for determining the partial safety factor

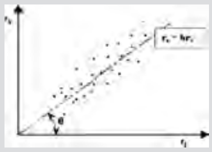
<table>
<tr><th>Step</th><th colspan="3">Feature</th><th>Objective</th></tr>
<tr><td>Definition of the design mode</td><td colspan="3">$F_{Rk} = c_F \cdot f_{yw} \cdot l_y \cdot t_w$</td><td>To identify the basic variables</td></tr>
<tr><td>Experimental (or numerical) vs. predicted values</td><td colspan="3">r_e (exp)  r_t (theoretical)</td><td>To compare the predicted values with the resistances obtained experimentally and numerically</td></tr>
<tr><td>Estimation of the correction factor b</td><td colspan="3">$r = b \cdot r_{i,the} \cdot \delta$</td><td>To create a probabilistic model of resistance. “b” is the least square best fit of the slope, “δ” is the error term</td></tr>
<tr><td>Estimation of the variation of the error V_δ</td><td colspan="3">$\delta_i = \frac{r_{exp,i}}{b \cdot r_{the,i}}$</td><td>To determine the variation of the errors in the formulation which result from the given formula</td></tr>
<tr><td>Normality test</td><td colspan="3">Kolmogorov-Smirnov</td><td>To test the normality of the distribution of the errors</td></tr>
<tr><td rowspan="4">Definition of the variation of the fundamental variables V_{xi}</td><td></td><td>Mean X_i</td><td>V_{xi}</td><td rowspan="4">To include a proper value (statistically determined) of the variations of the fundamental variables</td></tr>
<tr><td>fy</td><td>$1.14 \cdot f_{y,nom}$</td><td>0.07</td></tr>
<tr><td>Thickness</td><td>t_{nom}</td><td>0.05</td></tr>
<tr><td>Width</td><td>b_{nom}</td><td>0.005</td></tr>
<tr><td>Definition of the variation of the numerical results V_{FEM}</td><td colspan="3">Numerical and experimental results are compared with each other to determine the accuracy of the modelling of the steel girders.</td><td>This term is included for the sake of defining a certain level of random nature in the calculation of resistance with deterministic numerical models</td></tr>
<tr><td>Definition of the final coefficient of variation V_r</td><td colspan="3">$V_r^2 = \left(V_\delta^2 + 1\right) \cdot \left[\prod_k \left(V_{Xk}^2 + 1\right)\right] - 1 + V_{FEM}^2$</td><td>To include all variations in a single formula</td></tr>
<tr><td>Definition of the characteristic resistance</td><td colspan="3">$r_k = b \cdot g_{rt}(X_m) \cdot e^{(-k_\infty \cdot Q - 0.5 \cdot Q^2)}$</td><td>$k_{\infty} = 1.64$ $Q = \sqrt{\ln\left(V_r^2\right) + 1}$</td></tr>
<tr><td>Definition of the new design value</td><td colspan="3">$r_d = b \cdot g_{rt}(X_m) \cdot e^{(-k_{d,\infty} \cdot Q - 0.5 \cdot Q^2)}$</td><td>$k_{d,\infty} = 3.04$ $Q = \sqrt{\ln\left(V_r^2\right) + 1}$</td></tr>
<tr><td>Partial safety factor γ_{M1}</td><td colspan="3">$\gamma_{M1} = \frac{r_k}{r_d}$</td><td>To relate the characteristic to the design resistances by means of a single coefficient</td></tr>
<tr><td>Corrected partial safety factor γ_{M1}^*</td><td colspan="3">$\gamma_{M1}^* = \frac{r_k}{r_d} \cdot \frac{r_n}{r_k}$</td><td>To adapt the partial safety factor to better statistical variations</td></tr>
</table>

Table 4. Values of the imperfection factor and the plateau length to be used for achieving the desired level of safety

$\gamma_{M1}^* = 1.0$		$\gamma_{M1}^* = 1.1$	
$\bar{\lambda}_{F0}$	α_{F0}	$\bar{\lambda}_{F0}$	α_{F0}
0.5	1.0	0.5	0.75

This recalibration is carried out by assuming target values of the partial safety factor which are consistent with the values presently included within the core of the Eurocodes concerning the strength and stability of steel structures.

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News

Indispensable: fibre composites in architecture

4th International Symposium
on Composites in Architecture
Weimar, 5–6 December 2011
“Structures & Skins”

It all began with the idea of bringing together the world of fibre composites and the world of construction in one place, a place steeped in history. This latest symposium was the fourth time that top international architects and engineers had gathered in the charming ballroom of the “mon ami” youth and arts centre in Weimar to exchange experiences with fibre composite specialists. “Structures & Skins” was the topic this time.

As ever, the itinerary applied the concept of comparing innovations in building with composites with innovations in other materials. Such an approach enables a good overview of the future course of the construction industry and the contribution that fibre composites can make to that course.

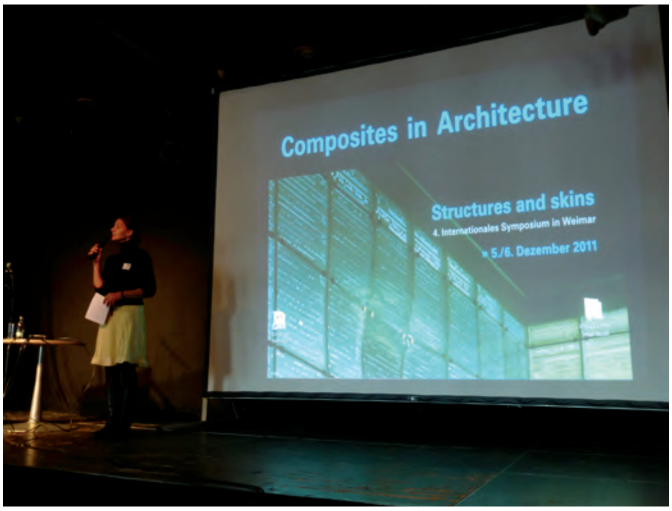
Day 1: Façades & structures
A wide range of projects was on offer, including active solar façades (Prof. *Tina Wolf*, Munich TU), challenging glass façades (Prof. *Jens Schneider*, Darmstadt TU), breathtaking, flexible membrane structures (Prof. *Jan Cremers*, Stuttgart HFT), composites in mobile structures (Prof. *Göran Pohl*, University of Jena) and composites in competitive wall assemblies (Prof. *Josef Kurath*, ZHAW, Winterthur). Without doubt, composites have become indispensable, especially in smart façade systems.

Day 2: Natural forms & natural materials
Clearly evident on the second day was the way the speakers employed a different approach to their treatment of the most diverse materials: the focus was on the apparently negative properties, which were then used to advantage. Prof. *Jan Knippers* (Knippers Helbig, Stuttgart/New York) presented shading louvres that, owing to the low elastic modulus of the composite material, can adapt and deform to suit the position of the sun. This project is currently under construction (pavilion for Expo 2012, Yeosu, Korea; architects: SOMA Architecture, Vienna). Prof. *Achim Menges* (ICD, Stuttgart TU) closed the day with fantastic curves in timber – structures that derive from the same idea: exploiting the low stiffness of the material.

As with the previous events, satisfied exhibitors and enthusiastic participants were the hallmarks of the 4th International Symposium on Composites in Architecture. In the meantime, the 100 or so delegates can be taken as an indication that building with composites faces a bright future.

The next Composites in Architecture event will take place on 3–4 December 2012.

Dr.-Ing. *Elke Genzel*
SKZ Das Kunststoff-Zentrum
www.skz.de/composites



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Design of girders with trapezoidal corrugated webs under the interaction of patch loading, shear and bending

The corrugated steel plate has been used in many applications for a long time because of its favourable properties. Engineers have realized a potential application in bridge structures, too, especially hybrid bridges. Among modern bridge erection methods, incremental launching is one of the most competitive. When incremental launching is used to erect a bridge, the girder is subjected to a combined action of transverse force (F), shear force (V) and bending moment (M), resulting in a complex stress field and a combined loading situation. The current version of EN 1993-1-5, Annex D, includes design methods for the bending and shear resistance of corrugated web girders, but no recommendations for the patch loading resistance calculation or consideration of different interactions (F+V; F+M). Therefore, this paper focuses on the experimental and numerical investigations into the structural behaviour of corrugated web girders under all the aforementioned actions. Design proposals are developed to determine the patch loading resistance of corrugated web girders and two interaction equations are proposed to take into consideration the reduction in resistance due to the combined loading situation.

1 Introduction

The corrugated steel plate is a structural element widely used in many fields because of its many favourable properties. This structural layout has spread in the field of bridges, too, especially for hybrid and composite bridges. In practice, during launching of a bridge structure, before the launching device reaches the next pier, large shear, bending and transverse forces can be introduced simultaneously at the previous pier as shown in Fig. 1. In this case the corrugated web is loaded by a combination of large bending moment and shear force together with transverse forces in such a way that their interaction should certainly be considered in the design. This paper focuses on the development of a design method for corrugated web girders under this combined loading situation. The current version of EN 1993-1-5 [1] does not give recommendations for determining the patch loading resistance of corrugated web girders and there are no interaction equations for such girders. This paper summarizes the design proposals developed for the patch loading resistance and interaction equations which can be applied in the design of corrugated web girders.

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The previous experimental, analytical and numerical investigations have been analysed from the point of view of bridge construction. These investigations were extended to typical bridge structures and design methods developed which can be applied in the parameter range relevant to bridges. Patch loading tests were conducted at the Budapest University of Technology & Economics, Hungary. The aim of the tests was to determine the patch loading resistance of corrugated web girders with a typical corrugation profile of a real bridge girder under combined patch load and bending moment. Based on the experimental background, a finite element model was developed and verified. Covering the wide range of parameters typical for bridges, a comprehensive parametric study was conducted to investigate the patch loading resistance of corrugated web girders. Based on the numerical model, the different interaction behaviours (F+V; F+M) were also studied. The different failure modes, load–deflection curves and ultimate loads were determined under each of the different loading conditions. The structural behaviour under the combined effects was also analysed and the interaction of the different failure modes characterized. The intensity of the influence of complex loading conditions was determined, evaluated and design interaction equations developed to take into account the reduction in resistance due to the combined loading situation. The results reported in the current paper support the design of corrugated web girder bridges. The parameters applied are shown in Fig. 2.

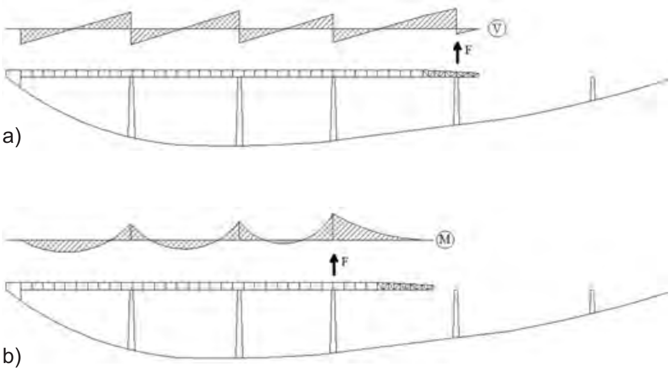

Fig. 1. Internal forces during launching
a) F-V interaction: "pier reached" construction stage
b) F-M interaction: "cantilever" construction stage

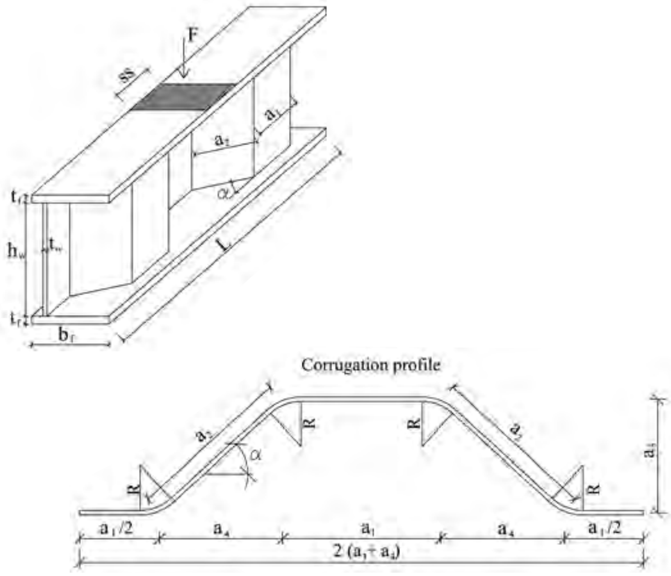

Fig. 2. Geometry of the trapezoidal corrugated girder

2 Review of previous investigations

2.1 Patch loading resistance of corrugated web girders

Experimental research on the patch loading resistance of girders with trapezoidal corrugated webs was started in 1987 by *Leiva-Aravena* and *Edlund* [2]. Research in this field was continued by *Kähönen* in 1988 within the scope of laboratory tests [3]. Based on the experimental investigations, a design model was developed to determine the patch loading resistance of girders with corrugated webs. This design formula is based on the four plastic hinge failure mechanism [1] developed by *Rockey & Roberts* for flat web girders [4]. *Kähönen* modified this formula for corrugated web girders [3] and the design method was calibrated to the experimental results by several modification factors. *Elgaaly* and *Seshadri* carried out experiments in 1997 to determine the patch loading resistance of corrugated web girders [5]. A finite element model was developed to analyse the patch loading failure mode and a numerical parametric study was conducted to examine the influence of the different parameters on the patch loading resistance. A design method was proposed on the basis of the experimental and numerical investigations. Numerical investigations were also conducted by *Luo* and *Edlund* in 1996 [6]. They proposed an empirical design formula based on a simple design model for flat web girders.

2.2 Interaction of patch loading and shear force

Elgaaly presented the first experimental investigation available on this topic for flat web girders in 1975 [7]. *Zoetemeier* conducted experiments on hot-rolled I-girders in 1980 [8]. Experimental research activity was continued by *Oxfort* and *Weber* on three welded test specimens in 1981 [9]. *Roberts* and *Shahabian* carried out experiments on 24 welded I-girders in 2000 and developed an interaction equation which is published in [10–12]. *Kuhlmann* and *Braun* researched this topic extensively in 2006 [13]. Based on a large number of numerical calculations, a new interaction curve was developed, which can be used over an extended parameter range. *Elgaaly* and *Seshadri* [5] first investigated the shear and patch loading interaction of corrugated web girders in 1997. Five specimens were tested under local transverse force. The main aim of their research was to determine the patch loading resistance of corrugated web girders. Their experiments led to the building of a numerical model. The pure patch loading resistance and the interaction between bending, shear and patch loading was studied within the scope of a numerical research programme. Based on the numerical calculations, a design interaction curve was proposed in the form of Eq. (1):

$$\left(\frac{V - 0.5 \cdot F}{V_R}\right)^{1.25} + \left(\frac{F}{F_R}\right)^{1.25} \le 1.0 \qquad (1)$$

where:
V_R shear buckling resistance
F_R patch loading resistance
V applied shear force
F applied transverse force

2.3 Interaction of patch loading and bending moment

Research into this topic was started by *Elgaaly* in 1975 [7]. Numerous tests were executed to determine the interaction behaviour and several interaction equations proposed. An overview of the previous investigations in this field for flat web girders can be found in [14]. Based on the previous studies, the current EN 1993-1-5 [1] also includes an interaction equation for bending and transverse force for flat web girders. *Elgaaly* and *Seshadri* [5] first investigated the bending and transverse force interaction of the corrugated web girders in 1997, in the aforementioned research programme. Based on a limited number of test results and numerical calculations, an interaction equation was proposed in the form of Eq. (2):

$$\left(\frac{F}{F_R}\right)^{1.25} + \left(\frac{M}{M_R}\right)^{1.25} \le 1.0 \qquad (2)$$

where:
M_R bending resistance
F_R patch loading resistance
M applied bending moment
F applied patch load

2.4 Summary of previous investigations and research aims

EN 1993-1-5, Annex D [1], deals with members with corrugated webs, but there is no standard design method for determining the patch loading resistance and there are no interaction equations for corrugated web girders. The patch loading resistance and the interaction behaviour of corrugated web girders was barely studied by researchers in the past. Besides that, the results of previous investigations are of only limited use in bridge design because the majority of them deal with shorter loading lengths and with higher global web ratios (h_w/t_w) than those typical for bridges. So most of the tests executed are far from the typical parameter range of bridge structures.

Therefore, an extended analysis was executed within the scope of experimental and numerical investigations and a design method is proposed to determine the patch loading resistance of corrugated web girders. Furthermore, the structural behaviour of the corrugated web girders under the combined effect of bending, shear and patch loading is also investigated because there is only a very limited number of investigations available for corrugated web girders. The applicability of the interaction curves (F+V; F+M) previously developed are studied and evaluated for bridge design, and new design equations proposed which can be used over a larger range of parameters.

3 Patch loading resistance of corrugated web girders

Patch loading tests were executed on 12 simply supported girders at the Department of Structural Engineering , Budapest University of Technology & Economics, Hungary, in 2009. The aim of the tests was to determine the patch loading resistance of corrugated web girders with the typical corrugation profile of a real bridge girder. Ultimate loads were determined, the structural behaviour and typical failure modes analysed and described. The test arrangement and the test specimen can be seen in Fig. 3. Details of the girders tested and the test results are published in [15] and [16].

A numerical model was developed and verified to expand the applicability of the test results. The modelling was based on a full shell model using four-node thin shell elements. Model verification was done by recalculating many experimental results. The ultimate loads were determined by geometrical and material non-linear analysis using imperfections. Based on the verified model, a comprehensive parametric study was executed in order to analyse the patch loading resistance in the parameter range used in bridges. The parameters analysed, which influence the load-carrying capacity, were varied between the following values (for notation, see Fig. 2):

– Web slenderness ratio: $h_w/t_w = 200$ to 500
– Fold slenderness ratio: $a_i/t_w = 13.3$ to 116.7 (in all cases $a_1 = a_2$)
– Loading length: $s_s/h_w = 0.4$ to 0.8 [2]
– Corrugation angle: $\alpha = 15$ to $65°$

The details of the numerical model development, the model verification and the detailed evaluation of the numerical simulations are published in [15] and [16]. Numerical calculations were used for developing the design method which may be applied to the practical design of corrugated web girders. Based on the numerical database developed in [17], an enhanced design method was developed in a co-operation between the Institute of Structural Design, University of Stuttgart, Germany (*Braun* and *Kuhlmann*), and the Department of Structural Engineering, Budapest University of Technology & Economics, Hungary (*Kövesdi* and *Dunai*). The design method developed is based on the four plastic hinge failure mechanism [3] according to the patch loading theory of *Rockey* and *Roberts* [4], which was evolved for flat web girders. The design method was calibrated for corrugated web girders by *Kähönen* [3] and further investigated and modified by *Braun* and *Kuhlmann* for a larger parameter range [18]. Based on the own test results and numerical calculations the design method is further developed by the authors and it can be applied if the failure mode is local buckling of the web. The geometric criteria in Eq. (3) is to exclude the global buckling failure of the whole web panel.

$$F_R = F_{R,w} + F_{R,fl} \quad \text{for} \quad a_i \geq \left(\frac{h_w}{t_w} + 260\right) \cdot \frac{t_w}{11.5} \tag{3}$$

The first part of this equation refers to the resistance of the web and the second part to the resistance of the flange. The web resistance can be calculated using Eq. (4):

$$F_{R,w} = \chi \cdot t_w \cdot f_{yw} \cdot ss \cdot k_\alpha \tag{4}$$

The reduction factor χ should be calculated using Eqs. (5)–(9):

$$\chi = \frac{1.9}{\bar{\lambda}_p} - \frac{0.798}{\bar{\lambda}_p^2} \quad \text{if} \quad \bar{\lambda}_p > 1.273 \tag{5}$$

$$\chi = 1.00 \qquad \text{if} \quad \bar{\lambda}_p \leq 1.273 \tag{6}$$

where:

$$\bar{\lambda}_p = \sqrt{\frac{f_{yw}}{\sigma_{cr}}}, \quad \sigma_{cr} = \frac{k_\sigma \cdot \pi^2}{12 \cdot (1-\upsilon^2)} \cdot E \cdot \left(\frac{t_w}{a_i}\right)^2, \quad k_\sigma = 1.11 \qquad (7), (8), (9)$$

f_{yw} yield strength of web
t_w web thickness
a_i loaded fold length (if more folds are loaded, the maximum fold length)
s_s loading length

The new non-dimensional slenderness curve (λ_p–χ) was derived from *Kähönen*'s formula by *Braun* and *Kuhlmann* [18] and it can be applied to the patch loading resistance of corrugated web girders. The recommendation is illustrated in Fig. 4. Curve ① is the proposal of EN 1993-1-5 [1] for patch loading of flat web girders (points show the results calculated from tests), curve ② shows the proposal derived from the *Kähönen* formula and curve ③ is the recommended enhanced non-dimensional slenderness curve for local buckling of corrugated web girders by *Braun* and *Kuhlmann* [18]. Numerical results show that the corrugation angle has a significant influence on the patch loading resistance. *Luo* and

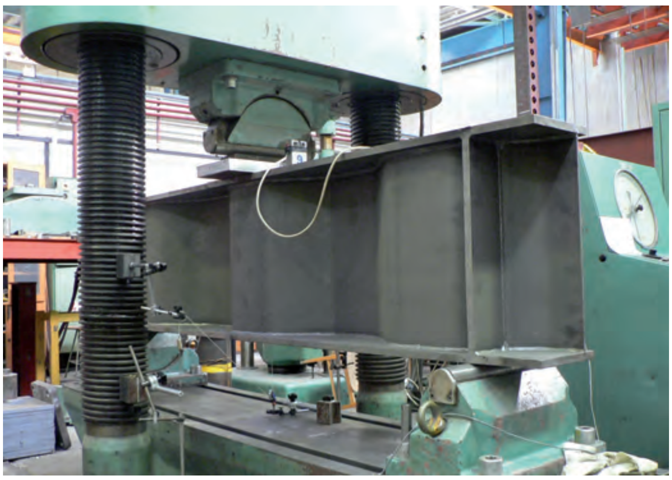

Fig. 3. Test setup

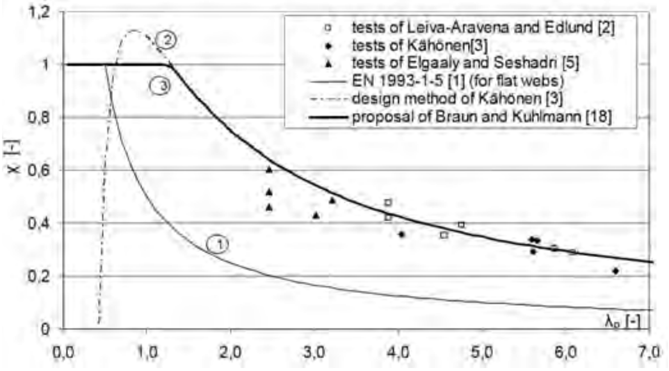

Fig. 4. Non-dimensional slenderness and reduction factor relationship

Edlund [6] developed an empirical design formula that considers the influence of the corrugation angle. Parametric studies show that this curve does not follow the numerical results in the case of relatively large corrugation angles ($\alpha > 40°$). According to the numerical parametric study, the character of the increasing patch loading resistance can be described sufficiently well by referring the real length of the folded web to the projected loading length. Therefore, *Braun* and *Kuhlmann* [18] propose improving the design method by introducing a factor k_α, calculated from Eq. (10):

$$k_\alpha = \frac{a_1 + a_2}{a_1 + a_4} \tag{10}$$

Fig. 5 shows the comparison of the two proposals and the numerical calculations. The results of the numerical calculations are presented by points, the proposal of *Luo* and *Edlund* is illustrated by curve ① and the proposal of *Braun* and *Kuhlmann* by curve ②. The results show that the proposal of *Braun* and *Kuhlmann* tracks the numerical results well and is on the safe side over the whole parameter range analysed. It means that the applicability of the k_α factor developed is proven based on the numerical calculations.

The flange contribution to the patch loading resistance may be calculated using Eq. (11):

$$F_{R,fl} = 2 \cdot \sqrt{n \cdot M_{pl.f} \cdot \chi \cdot t_w \cdot f_{yw}} \tag{11}$$

where:

$$M_{pl.f} = \frac{b_f \cdot t_f^2}{4} \cdot f_{yf} \tag{12}$$

f_{yf} yield strength of flange
b_f flange width
t_f flange thickness
n plastic hinges considered in flange according to Table 1

Factor n depends on the t_f/t_w ratio. If the t_f/t_w ratio is < 4, the complete yield lines in the web and all four plastic hinges in the flange can be evolved in the failure mechanism. If the flange is more dominant ($t_f/t_w \geq 4$), the web failure mechanism is activated earlier and none of the four plastic hinges can develop, and so the patch loading resistance is smaller. This decrease in resistance can be taken into account in the number of plastic hinges considered in the flange and n can be determined according to Table 1.

Table 1. Number of the considered plastic hinges in the flange

t_f/t_w	n
$t_f/t_w < 4$	4
$4 \leq t_f/t_w \leq 7$	3
$7 < t_f/t_w$	2

The new design proposal is compared with the results of the numerical and experimental database in Fig. 6. The vertical axis of the diagram shows the numerical and experimental results, the horizontal axis the results calculated based on the proposed design method. The comparison shows a good correlation between the design method and the numerical as well as experimental results. The mean value of the ratio of the results to the design method is 1.008 and the standard deviation is 0.06. The design method introduced in this section determines the mean value of the patch loading resistance of the corrugated web girders. In order to be able to use this method in the design of structures, a partial factor is derived in [19] to determine the design resistance level. Based on the statistical evaluation of the resistance model, a reduction factor of 1.35 can be applied to reach the design resistance level. The partial safety factor includes all the material and design method uncertainties and already consists the $\gamma_{M1} = 1.1$ what is recommended in the EN1993-2 [20] for bridges.

The design method developed is valid for corrugation angles $15° \leq \alpha \leq 65°$, for loading lengths in the range s_s/h_w = 0.4 and 0.8 [4] if inclined and parallel folds have the same

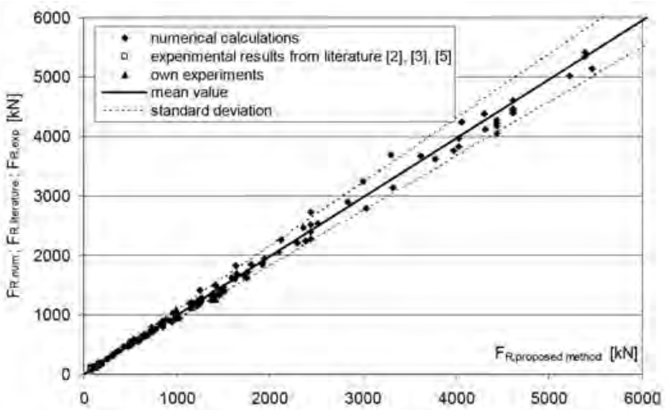

Fig. 5. Influence of the corrugation angle

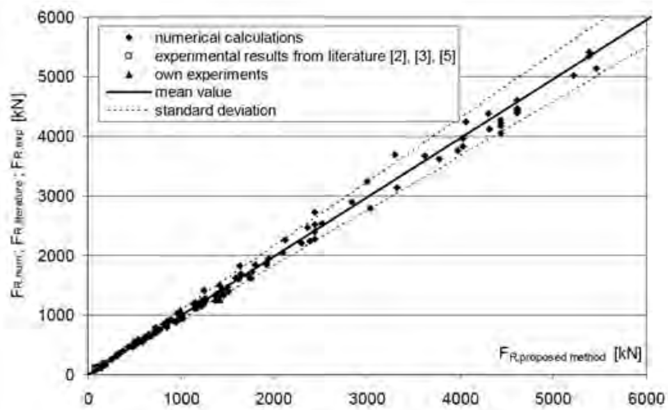

Fig. 6. Comparison between the numerical and experimental results and the design method

length ($\alpha_1 = \alpha_2$). This design formula can be used between web ratios $h_w/t_w = 200$ to 500 and fold ratios $\alpha_1/t_w = 15$ to 100, if the failure mode is local buckling of one or more web folds. This parameter range is relevant for bridges.

4 Interaction between patch loading and shear force

During launching of a bridge structure, large shear and transverse force can be introduced at the same cross-section. In this case, the corrugated web is loaded by large shear and transverse forces, an interaction that should be considered in the design. Numerical calculations have been carried out to study the interaction behaviour of corrugated web girders. The numerical model developed, the supports and the loading conditions are introduced in Fig. 7. Details of the numerical model development and the model verification can be found in [20]. Different failure modes under pure patch loading, pure shear force and the combined loading situations have been analysed in order to obtain the structural behaviour of the shear and patch loading interaction. A detailed evaluation and comparison of the calculation results are published in [20].

The interaction behaviour of the corrugated web girders was analysed in a large parameter range within the scope of the numerical parametric study. The parameters investigated were as follows (for notation, see Fig. 2):
- $h_w/t_w = 100$, 125, 150, 200 and 250
- $\alpha_1/t_w = 15$, 20, 25, 30 and 35
- $\alpha = 20°$, 30°, 40° and 60°
- $s_s/h_w = 0.2$, 0.4, 0.5, 0.6 and 0.8

The ratio of F to V was changed for all the girder geometries analysed and calculations were executed with different F/V ratios in order to analyse the interaction behaviour. The results obtained, together with *Elgaaly*'s numerical calculations, are presented in Fig. 8. The vertical axis of the diagram in Fig. 8 show the results as a function of (V-0.5F)/$V_{R,num}$. The reason of it is that the shear stresses due to "pure patch load" are already included in the patch loading resistance model and a reduction of the load carrying capacity is caused only by the additional shear stresses coming from "pure shear force". The results show that the

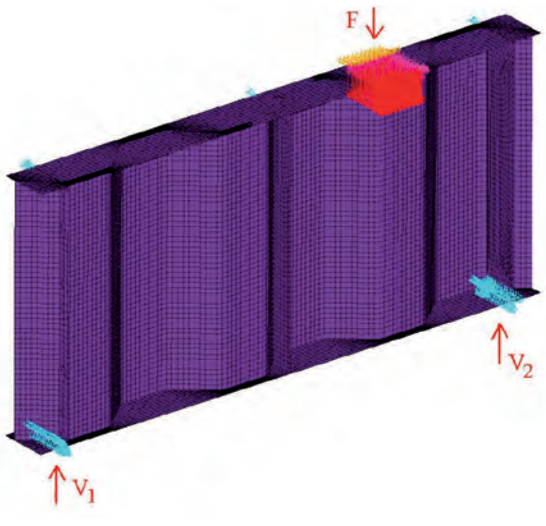

Fig. 7. Numerical model for investigating shear and patch loading interaction

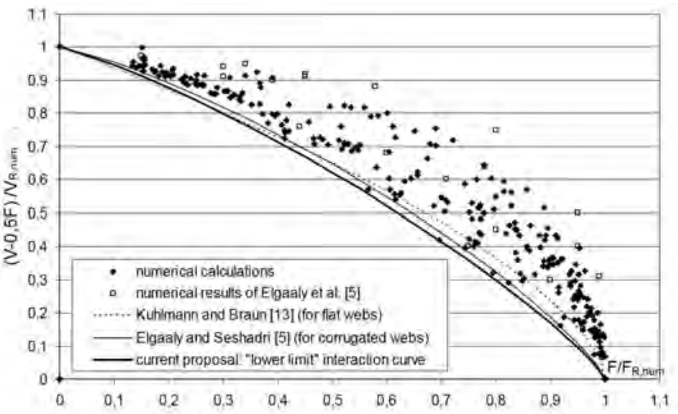

Fig. 8. Interaction curve for shear and patch loading

interaction curve of *Kuhlmann* and *Braun* [13], which was developed for flat web girders, overestimates the interaction behaviour for corrugated web girders. The recommended interaction curve of *Elgaaly* and *Seshadri* [5] gives a good approximation in the extended parameter range as well. Of all the numerical calculations, only three results fall below this interaction curve and all of them are at the extremes of the parameter range analysed (long loading length s_s, large α and large a_1 at the same time). In order to take these special cases into account, a lower limit interaction curve is proposed in the form of Eq. (13). It is recommended to use this interaction curve in the design of corrugated web girders:

$$\left(\frac{V - 0.5 \cdot F}{V_R}\right)^{1.2} + \left(\frac{F}{F_R}\right)^{1.2} \leq 1.0 \tag{13}$$

Due to the relatively large scatter of the numerical results, parameters were studied separately to ascertain their effects on the interaction criterion. The effect of the different geometric parameters are determined and evaluated in [19]. The analysis shows that the interaction behaviour of corrugated web girders depends on the $F_{R,w}/F_{R,fl}$ ratio (web-to-flange patch loading resistance). This ratio can be calculated using Eqs. (4) and (11). Based on this assumption, a parameter-dependent interaction curve was developed as an alternative, which takes into account the effect of the most influential parameters and therefore can lead to a more economic design. But this parameter-dependent interaction curve does not consider the effect of the simultaneous bending moment, which can reduce the safety of the patch loading and shear interaction curve. Therefore, the application of this interaction curve is proposed only in cases of small bending moments. The general use of this interaction curve needs more background investigations with combined bending moments.

5 Interaction between patch loading and bending moment

During the launching of a bridge structure, large bending moments and transverse forces act in the same cross-section above the pier. The bending moment is mainly resisted by the flanges, and the shear and transverse forces are carried mainly by the web. But the flange loaded by the transverse force can have an important role in the patch loading resistance as well, especially in the case of bridges where

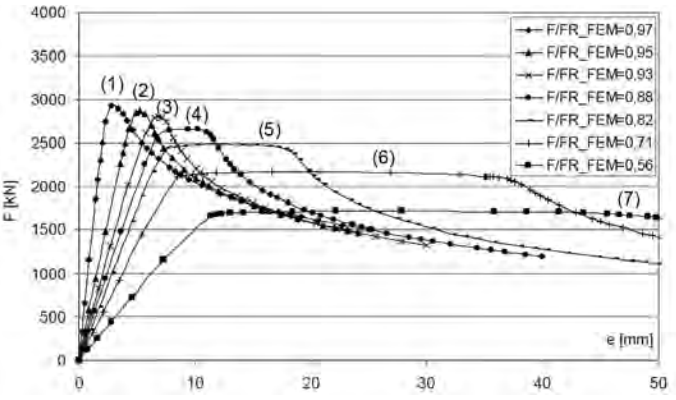

Fig. 9. Comparison of load–deflection curves

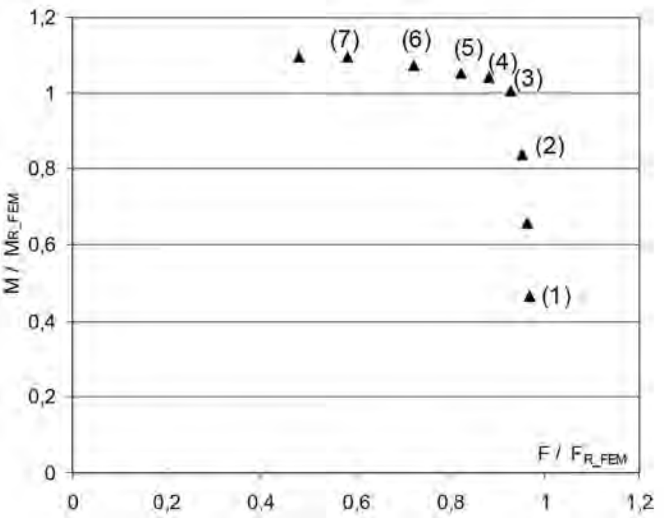

Fig. 10. Interaction behaviour

the flanges are relatively thick. Therefore, the interaction of bending moments and transverse forces should also be considered in the design. Numerical calculations were conducted to analyse the structural behaviour of the corrugated web girders under the combined loading. The typical failure modes, load–deflection curves and load-carrying capacities were studied under transverse force, bending moment and the combined loading condition. The comparison of the load–deflection curves calculated and the load-carrying capacities in the interaction field are presented in Figs. 9 and 10. The typical failure modes are web crippling [5]and bending. Under the combined loading situation, the interaction of these two failure modes is typical. A uniform transition between them can be observed in the interaction zone, and no strict limit can be drawn. The uniform transition can be also observed in the load–deflection diagram in Fig. 10. Points (1) to (3) in Fig. 10 are failures where patch loading dominates, and for points (4) to (7) it is bending that is dominant.

The interaction behaviour was analysed over a large parameter range within the scope of a numerical parametric study. The parameters investigated were the same as described in Section 4. The results of the numerical and experimental investigations are presented in Fig. 11. The diagram shows that the combined bending moment and transverse force hardly decreases the load-carrying capacity. The reason is the well-known "accordion effect" of corrugated web girders. Based on this effect, the bending moment is mainly resisted by the flanges and the web carries the transverse force, and so the reduction in resistance is smaller than that for flat web girders. Besides that, the results show that the interaction curve according to EN 1993-1-5 [1] for flat web girders is on the unsafe side if a large transverse force is acting on the girder with a small bending moment. If both effects are relatively large, the Eurocode curve is well on the safe side. These statements are only valid if the reference values for the bending moment resistance (M_{R_FEM}) and the patch loading resistance (F_{R_FEM}) are calculated by finite element analysis, and not by design formulas. The results show that the interaction proposal of *Elgaaly* and *Seshadri* [5] is conservative in the parameter range analysed; therefore, a new interaction equation (Eq. (14)) is proposed for corrugated web girders, which applies to the aforementioned parameter range:

$$\left(\frac{F}{F_R}\right)^{1.20} + \left(\frac{M}{M_R}\right)^{8} \le 1.0 \tag{14}$$

Fig. 12 evaluates the results based on the bending moment resistance according to EN 1993-1-5 [1] and the patch loading resistance model proposed in this paper supplemented by the partial factor (1.35) determined in [19] for corrugated web girders. The comparison in Fig. 12 shows how the model scatter of the aforementioned design formulas influences the interaction. However, the statistical quality of the design formulas is not on the same level. The patch loading resistance model was developed on the basis of a small experimental and numerical database, which results in a large partial factor. Therefore, the distance between the data and the interaction curves is large.

6 Conclusions

Despite the increasing use of girders with trapezoidal corrugated webs in bridges, there are no design rules for determining the patch loading resistance nor its interaction with bending moment and shear force. This paper outlines design rule proposals that fit into the verification format of EN 1993-1-5, Annex D [1].

The review of previous research work in this field has revealed not only a lack of data but also its focus on building structures such that the parameters studied must be expanded to apply to bridge structures. The key achievement is the development of a patch loading resistance model in-

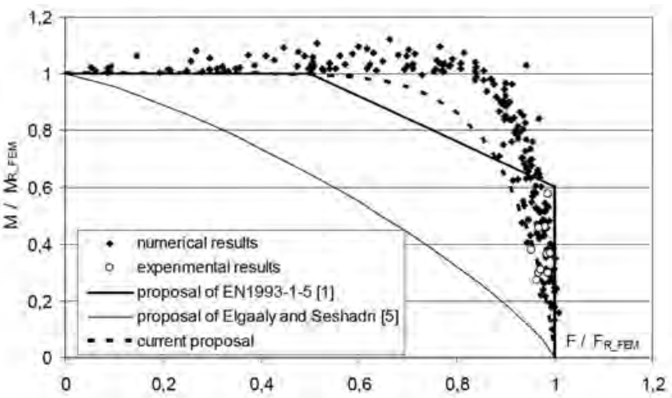

Fig. 11. Interaction curves, numerical and experimental results – based on resistances by FEM

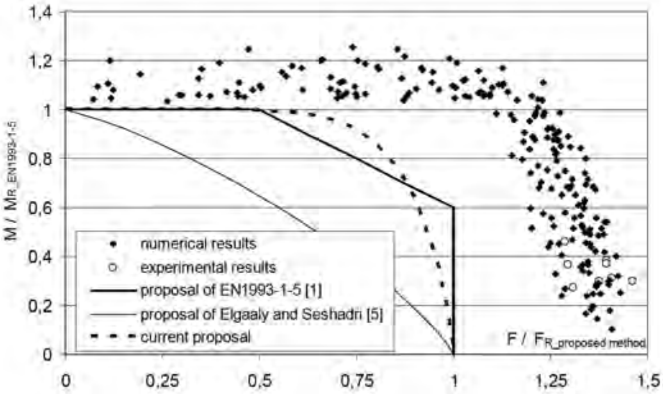

Fig. 12. Interaction curves, numerical and experimental results – based on resistances by design formulas

cluding the statistical assessment of the partial factor so that a ready-to-use design rule can be proposed. Based on the numerical and experimental database, interaction equations for patch loading and bending moment (F-M) as well as patch loading and shear force (F-V) are derived and the newly developed patch loading resistance model successfully evaluated and supplemented. Although the interaction between F-M and F-V is addressed in separate equations at the moment, it is perceived that this is useful to the designer because the internal force distribution during bridge launching usually leads to patch loading either with dominant bending moment or dominant shear force. Nevertheless, future work will have to focus on merging both interaction equations in order to achieve a consistent definition of the F-M-V interaction surface.

Acknowledgements

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Keywords: corrugated web; patch loading resistance; shear and patch loading interaction; bending and patch loading interaction

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Intermediate transverse stiffeners in plate girders

The aim of this paper is to study the design requirements for rigid intermediate transverse stiffeners in longitudinally stiffened plate girders. Firstly, two representative design codes, AASHTO and EN 1933-1-5, are discussed and compared. Then the results of two tests on 1.5 m deep plate girders and the results of the extensive numerical parametric study are presented. The analysis of the results shows that the axial force in the stiffener due to tension field action is overestimated in the design codes and that all the design requirements for rigid transverse stiffeners can be covered by defining a minimum required second moment of area.

1 Introduction

Transverse stiffeners in plate girders are usually designed as rigid stiffeners that do not deflect beyond certain limits and even at the ultimate limit state provide a solid support along the line of connection between the web plate of the web itself and the longitudinal stiffener.

The main consequence of this is that the plate panel between adjacent rigid transverse stiffeners may be analysed as an isolated panel with well-defined boundaries. To be on the safe side, the panel is assumed to be restrained for transverse movements and free to rotate along the edges. Further, the buckling length of longitudinal stiffeners is well defined and can be assumed to be equal to the spacing of the transverse stiffeners.

Transverse stiffeners at end and intermediate supports are subjected to large axial forces from support reactions. For this reason, they are mostly designed as double-sided stiffeners and are not dealt with in this paper. Intermediate transverse stiffeners are installed to increase the strength and stiffness of the web panel and to prevent distortional effects on the plate girder cross-section. Normally, they are designed as single-sided open stiffeners with a range of different cross-sections. The most typical are flat, L- or T-stiffeners.

In most cases, intermediate transverse stiffeners do not carry large external forces. If they are subjected to large external forces, they have to be designed in a similar way to support stiffeners, taking account of any eccentricity if they are single-sided. Usually, intermediate transverse stiffeners are mainly subjected to forces arising from the following two sources:

– Deviation forces from longitudinal stresses in the web panels adjacent to the stiffener, which develop due to the out-of-plane imperfect geometry of the stiffener. These deviation forces induce out-of-plane bending moments in the stiffener and are subjected to second-order effects.
– Tension field action that develops in the post-buckling state in shear. It develops in the form of a diagonal tension band in the web plate and induces an axial compression force in the stiffener (truss analogy).

Both actions can appear individually or simultaneously. To resist these actions and to limit deformability, rigid transverse stiffeners should, in principle, be designed for strength and stiffness criteria at the ultimate limit state.

The first author to address the design issues of transverse stiffeners was *Timoshenko* [1]. Although his approach was limited to geometrically perfect plates and stiffeners, the traces of his work can still be detected in current design rules for deviation forces as well as for shear buckling problems.

Since 1950, many authors have dealt with transverse stiffeners, but mainly for the effects of the shear loading in the web panel: *Stein & Fralich* [2], *Basler* et al. [3], *Rockey* et al. [4, 5], *Evans* et al. [6], *Höglund* [7], etc. Two design rules were typically developed: an expression for calculating the minimum required second moment of area of the transverse stiffener and an expression for the axial force in the stiffener resulting from the tension field action in the web panel, or, as an alternative, an expression for the minimum required cross-sectional area of the stiffener taking account of local buckling but usually ignoring overall flexural buckling of the stiffener. A representative design code of this kind is the American bridge design specification AASHTO [8, 9], which will be discussed in more detail later.

Recently, Euronorm EN 1993-1-5 [10] introduced a more general approach to stiffener design. According to this, standard rigid intermediate transverse stiffeners should be explicitly designed for strength and stiffness criteria, both at the ultimate limit state. Formulas for calculating deviation forces and the axial force in the stiffener are also provided.

The common drawback of all the design rules mentioned is that the axial force in the stiffener induced by tension field action in the post-buckling state is overesti-

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mated to a large extent, as has been demonstrated by several authors [11–15] and will be demonstrated again later in this paper. This will be done based on two tests performed on 1.5 m deep plate girders and on the results of an extensive numerical parametric study. The main aim is to find out if it is possible to cover all relevant design issues regarding the rigid intermediate transverse stiffeners of plate girders by the stiffness approach alone – defining the minimum required second moment of area of the stiffener.

2 Brief overview of AASHTO and EN 1993-1-5 design standards

The two representative design standards, AASHTO and EN 1993-1-5, are discussed below.

2.1 AASHTO

In AASHTO-1996 [8], transverse stiffeners have to fulfil the following second moment of area and cross-sectional area requirements:

$$I_{st} = \max\left(2.5\left(\frac{h_w}{a}\right)^2 - 2;\ 0.5\right) a t_w^3 \tag{1}$$

$$A_{st} \geq \left[0.15 B h_w t_w (1-c)\frac{V}{V_u} - 18 t_w^2\right]\frac{F_{yw}}{F_{crs}} \tag{2}$$

where:
- h_w depth of web plate
- a spacing of transverse stiffeners
- c ratio of shear buckling resistance to shear yield strength
- F_{yw} specified minimum yield strength of stiffener
- F_{crs} elastic local buckling stress of stiffener

The minimum stiffness requirement is needed for the web to develop the shear buckling resistance with near zero lateral deflection along the transverse stiffeners. The transverse stiffener has to resist the vertical component of the tension field action, and therefore Eq. (2) has to be fulfilled. This expression is derived from the condition that the maximum stress in the stiffener due to tension field action does not exceed the elastic local buckling stress in the stiffener. If the local buckling stress in the stiffener is higher than the yield stress of the material, critical stress is replaced with yield stress. In Eq. (2), the influence of eccentricity for single-sided stiffeners is taken into account with factor B, which results in 1.0 for double-sided stiffeners, 1.8 for single-sided angle stiffeners and 2.4 for single-sided flat stiffeners. The effective part of the web plate, $18t_w^2$, is also accounted for in Eq. (2). In the minimum area condition, only the post-buckling part of the force acting on the stiffener due to the tension field action is taken into account.

According to AASHTO-1996 [8], the transverse stiffener of the longitudinally unstiffened web plate is only designed for the influence of tension field action. For longitudinally stiffened girders, Eq. (2) can be used to determine the transverse stiffener area with h_w equal to the whole girder depth if the tension field develops over the whole web and h_w equal to the maximum subpanel depth when tension fields in each subpanel develop independently. Additionally, the transverse stiffener used in the longitudinally stiffened web panels must also satisfy:

$$I_{st} \geq \left(\frac{b_{st}}{b_{sl}}\right)\left(\frac{h_w}{3a}\right) I_{sl} \tag{3}$$

where I_{sl} denotes the second moment of area of the longitudinal stiffener including an effective width of the web equal to $18t_w$ and calculated for the neutral axis of the combined section b_{st} and b_{sl} as shown in Fig. 1.

In the latest edition of AASHTO-2007 [9], the intermediate transverse stiffener that supports web subjected to tension field action and deviation forces is given by the maximum of Eq. (1) and:

$$I_{st} \geq \frac{h_w^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \tag{4}$$

where:
- I_{st} second moment of area of transverse stiffener taken around the edge in contact with the web
- ρ_t maximum of F_{yw}/F_{crs} and 1
- F_{yw} specified minimum yield strength of web plate
- F_{crs} local buckling stress for stiffener, given by:

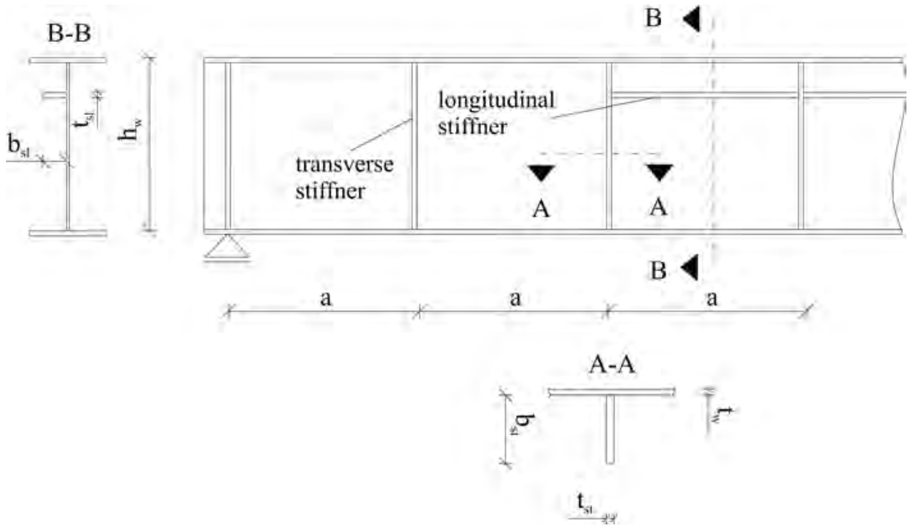

Fig. 1. Notation of the stiffened panel

$$F_{crs} = \frac{0.31E}{\left(\frac{b_{ts}}{t_{st}}\right)^2} \le F_{ys} \tag{5}$$

In Eq. (5), b_{st} and t_{st} denote the width and thickness of the stiffener respectively, and F_{ys} is the specified minimum yield strength of the stiffener.

2.2 EN 1993-1-5

According to Eurocode EN 1993-1-5 [10], the stiffener has to meet strength and stiffness criteria given as:

– The maximum stress σ_{max}. in the stiffener should not exceed the yield strength:

$$\sigma_{max} \le \frac{f_y}{\gamma_{M1}}$$

– The additional lateral deflection w_{max} should not exceed:

$$w_{max} \le \frac{h_w}{300}$$

When a transversely stiffened plate is loaded with pure shear only, the transverse stiffener has to fulfil minimum stiffness criteria:

$$I_{st} \ge \frac{1.5h_w^3 t_w^3}{a^2} \text{ for } \alpha = \frac{a}{h_w} \le \sqrt{2} \tag{6a}$$

$$I_{st} \ge 0.75 h_w t_w^3 \text{ for } \alpha = \frac{a}{h_w} > \sqrt{2} \tag{6b}$$

This is similar to the stiffness requirement given by Eq. (1) of AASHTO [8, 9]. Due to tension field action in the web plate and truss analogy, the stiffener is subjected to an axial force defined as:

$$N_{st,ten} = V_{Ed} - \frac{1}{\lambda_w^2} t h_w \frac{f_y}{\sqrt{3}\gamma_{M1}} \tag{7}$$

where V_{Ed} is a design shear force and $\overline{\lambda_w}$. is the slenderness of the web panel adjacent to the stiffener. Double-sided stiffeners may be simply designed as columns subjected to concentric compression. In the case of a single-sided stiffener, the stiffener should be verified for the axial force and for bending moments due to the eccentricity of the axial force. Local buckling of open stiffeners is closely linked to the torsional buckling mode and in EN 1993-1-5 local buckling is prevented with the following expression:

$$\sigma_{cr} \ge \theta f_y \tag{8}$$

where σ_{cr} is the elastic critical stress for torsional buckling of the stiffener and θ is a parameter linked to the stiffener's cross-sectional shape (2 for flat stiffeners, 6 for stiffeners that possess warping torsional stiffness).

Longitudinal compression stresses in the web plate and in longitudinal stiffeners due to bending moments and axial forces induce transverse deviation forces. The magnitude of these deviation forces q_{dev} is related to the stiffener imperfection amplitude w_0 and is subjected to second-order effects. Fig. 2 shows the corresponding calculation model adopted in EC 1993-1-5.

In most practical design cases, the web plate is subjected to a combination of shear and normal longitudinal stresses. The stiffener is subjected to an axial load $N_{st,ten}$ due to tension field action and to deviation forces q_{dev} due to normal stresses acting on an initially imperfect stiffener. For these load combinations, the stiffener should be designed taking into account second-order effects, and both strength and stiffness criteria should be met. EN 1993-1-5 does not explicitly prescribe the design checks for this general case, which is critical, especially for single-sided stiffeners, because the overestimated axial force due to tension field action acts eccentrically on the stiffener.

A relatively simple design check that covers strength and stiffness criteria, including second-order effects, is given in *Beg* et al. [16] or *Johansson* et al. [17]).

Table 1 summarizes the minimum dimensions b_{st}/t_{st} of a flat stiffener calculated according to AASHTO-1996 [8], AASHTO-2007 [9] and EN 1993-1-5 [10] for a selected plated girder with dimensions h_w/t_w = 2000/8 mm, α = a/h_w = 1 and steel grades S235 and S355. The actual load acting on the stiffener is important only when the stiffener is designed according to EN 1993-1-5. In this particular example, the stiffener was designed for the most unfavourable

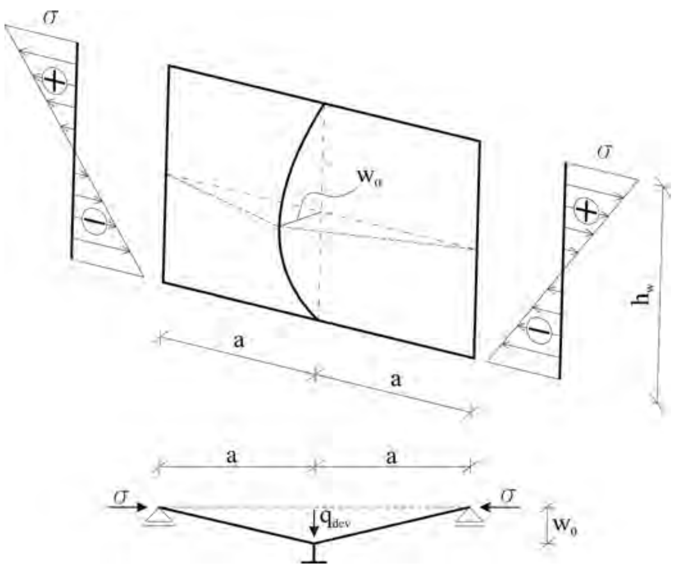

Fig. 2. Calculation model for deviation forces in EN 1993-1-5

Table 1. Transverse stiffener required to fulfil conditions given in EN 1993-1-5 and AASHTO

b_{st}/t_{st} (cm⁴)	AASHTO (1996) Eq. (2)	AASHTO (2007) Eq. (4)	EN 1993-1-5 $\sigma_{max,w}$	EN 1993-1-5 Eq. (6)
h_w = 200 cm, t_w = 0.8 cm, a = 200 cm, S235	175/19	195/24	195/25	110/14
h_w = 200 cm, t_w = 0.8 cm, a = 200 cm, S355	190/20	230/28	205/25	110/14

case, taking into account a shear load equal to the shear resistance of the web plate and a bending moment equal to the bending resistance of the web plate.

By increasing the yield strength, larger stiffener dimensions are required in all design provisions except in the case of minimum stiffness criteria (Eq. (6) given in EN 1993-1-5). The latest AASHTO-2007 results in much larger stiffeners than those of AASHTO-1996.

Recent studies regarding the design of transverse stiffeners have shown that the formulation for the axial force due to tension field action (truss analogy) produces very conservative values. The influence of the stiffness of the transverse stiffener on a transversely stiffened girder under shear was studied by *Lee* et al. [12, 13]. In their study they proved that the axial force in the stiffener is much smaller than that predicted by design specifications. The result of their work is a new model that describes the post-buckling behaviour, called "shear cell analogy", and a new proposal for the stiffness requirement for a transverse stiffener.

The test results of the transverse stiffeners of *Basler* et al. [3] and *Evans* et al. [6] were evaluated by *Höglund* [11] and compared with the rules in EN 1993-1-5. The formulation in EN 1993-1-5 for determining the axial force from tension field action was found to be conservative.

An extended numerical investigation into transversely stiffened girders has also been carried out by *Handy & Presta* [14, 15]. A new proposal for the axial force acting on a transverse stiffener was proposed:

$$F_{Ed} = V_{Ed} - V_{bw,Rd} \geq 0 \tag{9}$$

where V_{Ed} is the design shear load and $V_{bw,Rd}$ is the design shear resistance of the web only. This simplification was only proposed for transversely stiffened girders without longitudinal stiffeners.

3 Experimental work

Two tests on intermediate transverse stiffener were performed. Fig. 3 shows the layout of the girder tested and the load positions for tests S1 and S2. The dimensions of the girder cross-sections are listed in Table 2. The intermediate transverse stiffeners were designed for the effects of deviation forces and tension field action. The deviation forces were calculated from the stress distribution due to the pure bending resistance of the plate. Only half of the axial force due to tension field action, calculated according to Eq. (7), was considered in the design. In both cases

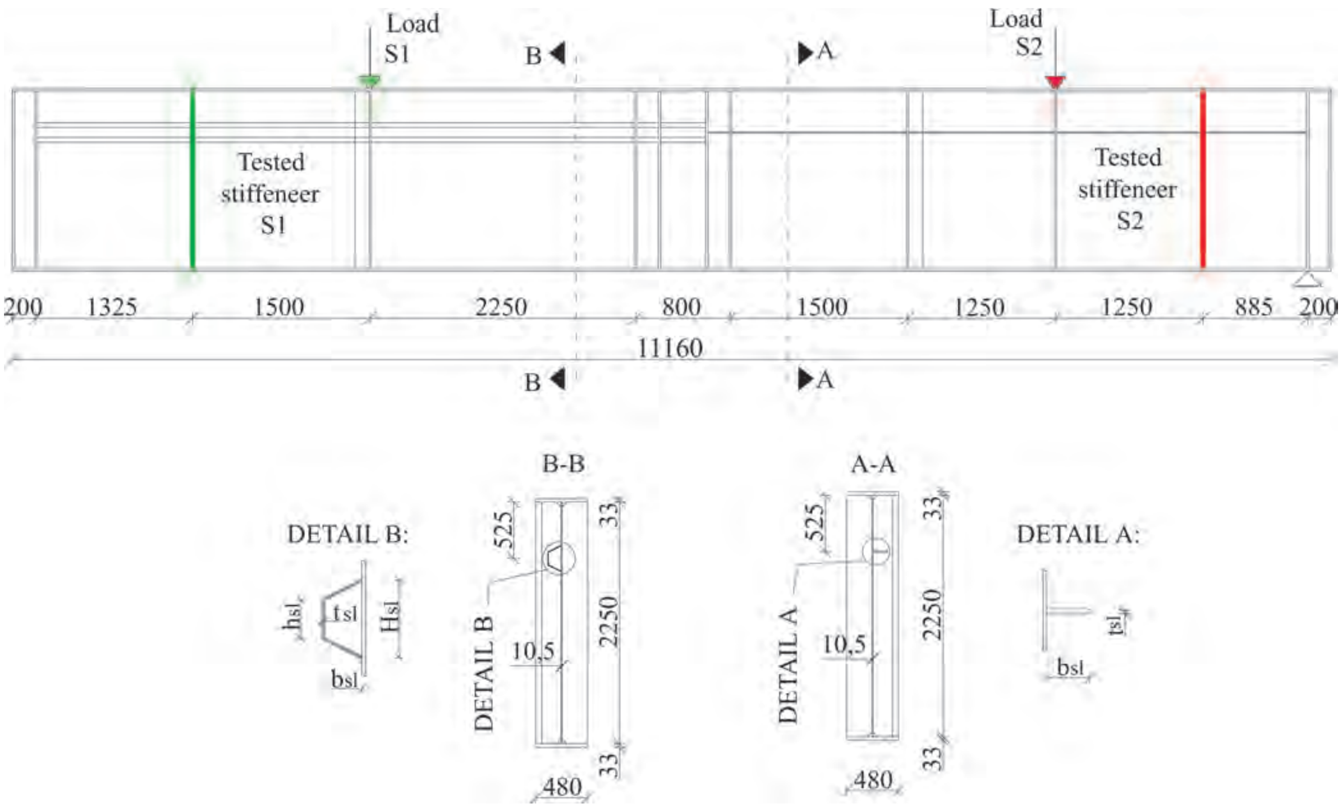

Fig. 3. Layout of the girder tested and loading positions for tests S1 and S2

Table 2. Geometry of the steel plate girders tested

	Web			Top flange		Bottom flange		Longitudinal stiffener			
Specimen	h_w [mm]	t_w [mm]	a [mm]	b_{f1} [mm]	t_{f1} [mm]	b_{f2} [mm]	t_{f2} [mm]	H_{sl} [mm]	h_{sl} [mm]	b_{sl} [mm]	t_{sl} [mm]
SO	1500	7	1500	320	22	320	22	–	–	90	10
SC	1500	7	2250	320	22	320	22	160	80	80	5
UO	1800	6	1800	250	20	450	20	–	–	100	10
UC	1800	6	2700	250	20	450	20	300	180	80	5

studied, the dimensions of the transverse stiffener were finally set to $b_{st}/t_{st} = 120/15$ mm.

The out-of-plane displacements in the panel and in the transverse stiffener investigated were measured at discrete points by displacement transducers. Strains in the transverse stiffener and the web plate in the vicinity of the stiffener were measured in addition to displacements, as shown in Fig. 4. The strain gauges were positioned on either side of the stiffener and the web plate.

The initial geometrical imperfection of the transverse stiffener was measured and in both cases bow imperfection in the direction of the intermediate transverse stiffener was detected, with a maximum amplitude of 4.7 mm for stiffener S1 and 3.7 mm for stiffener S2.

3.1 Test results

The load-deflection curves for both tests are plotted in Fig. 5. In the first test (S1), the load was increased up to 2572 kN. The test was stopped just before the maximum capacity of the girder was reached. In the second test (S2), the test specimen was loaded up to the maximum resistance of 2659 kN – well into the softening range in terms of displacements.

The average strains over the thickness of the transverse stiffener are plotted in Fig. 6. The resulting maximum membrane stress for stiffener S1 is obtained at the position of strain gauge L12 in the web plate with a value of –210 MPa, which is less than the measured yield stress of the material. In test S2, plastic strains were observed at point L11 only, with a maximum average strain of 0.46 %. This strain was obtained after the peak load had already been attained. The strain at maximum load was 0.30 %. In both tests, the maximum average strains were measured in the cross-section near the longitudinal stiffener. This is due to the fact that this cross-section is directly in the area where the diagonal tension field is anchored into the transverse and longitudinal stiffener. In this cross-section (1-1, see Fig. 7) the strains are relatively high, whereas in the other two cross-sections (2-2 and 3-3) the strains are smaller and also

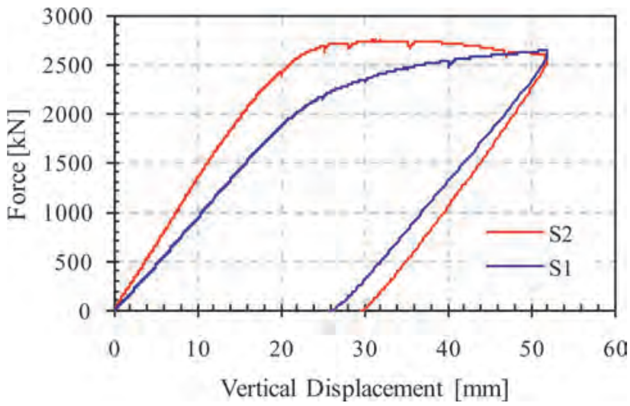

Fig. 5. Load-deflection curves for tests S1 and S2

more representative for determining the axial force in the intermediate transverse stiffener.

The axial forces were evaluated from the measured strains for each cross-section. The results are listed in Table 3. For comparison, 100 % (100 % TFA) and 50 % (50 % TFA) of the axial force due to tension field action according to EN 1993-1-5 are given. As can be seen, the maximum compression is found in section 1-1 (see Fig. 7). One of the measured points (L11) was directly in this diagonal tension field where the strains are extremely high. In the middle section (2-2), the axial force is much smaller, whereas section 3-3 contains the smallest value. It is reasonable to assume that section 2-2 is relevant for determining the representative (average) value of the axial force in the stiffener. This axial force represents only 56 % of the calculated axial force arising from tension field action using truss analogy.

4 Numerical simulations

4.1 Model verification

The numerical model was developed in the multi-purpose program ABAQUS [18] and was verified against the test results. The measured initial geometrical imperfections and

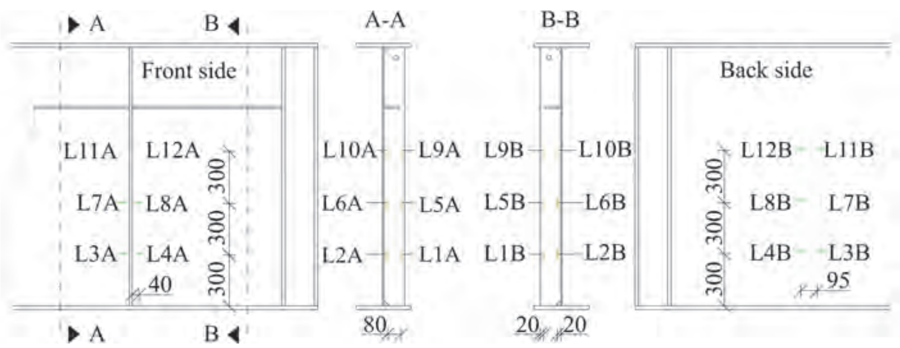

Fig. 4. Positions of strain gauges on stiffener and web

Table 3. Axial force in transverse stiffener at maximum girder resistance, taking into account effective part of web $15\varepsilon t_w$

N_{ten} [kN]	Stiffener S1			Stiffener S2		
SECTION	1-1	2-2	3-3	1-1	2-2	3-3
TEST	– 329.1	**– 290.0**	– 223.4	– 653.9	**– 280.7**	– 160.4
100 % TFA	– 514			– 504		
50 % TFA	– 257			– 252		

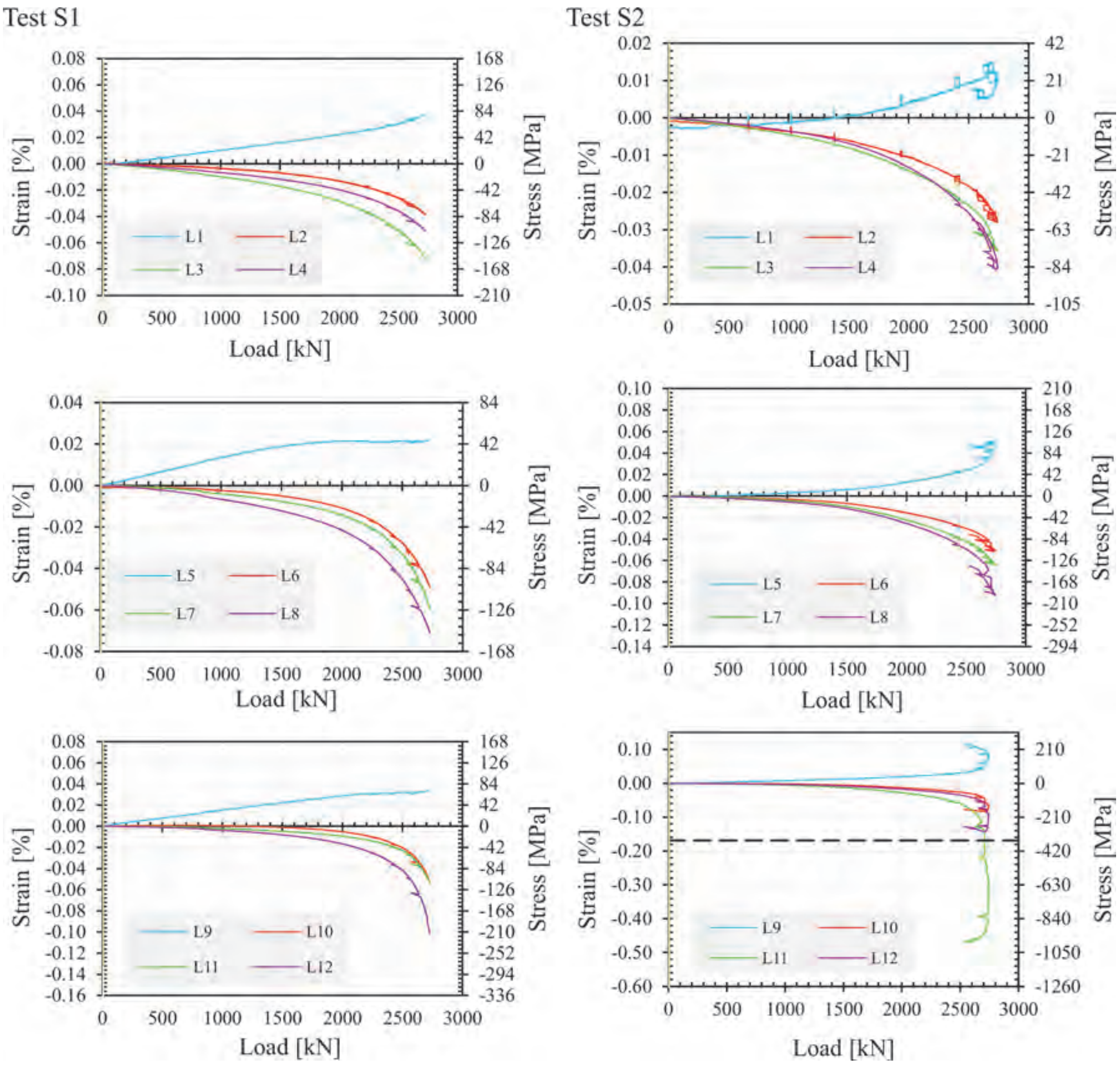

Fig. 6. Strain measurements in transverse stiffeners

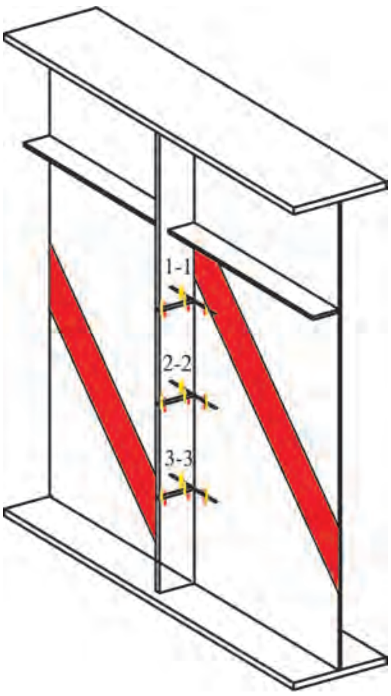

Fig. 7. Cross-sections in stiffener where axial forces were evaluated

non-linear material behaviour based on tensile tests were considered. Verification of the numerical model was performed by comparing the initial stiffness, maximum capacity and load-deflection curve.

Fig. 8 compares the experimental and numerical results by way of the load-deflection curves. The initial stiffness of numerical model S1 is slightly higher than the experimental one. The transition from the elastic to the plas-

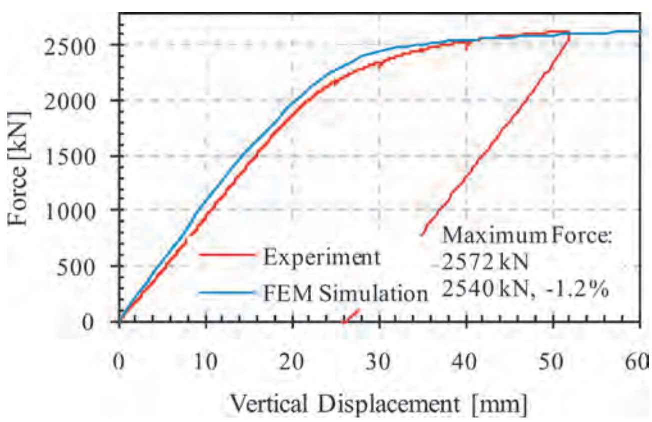

Fig. 8. Comparison of load-deflection curves for test S1

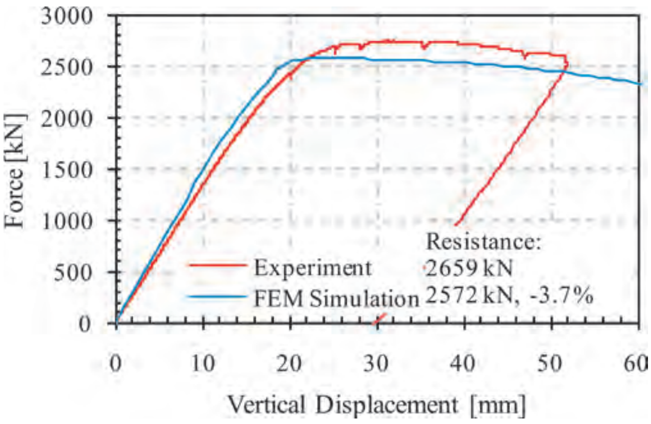

Fig. 9. Comparison of load-deflection curves for test S2

tic zone is very similar, but the maximum capacities cannot be compared because the test was stopped before the resistance was reached. However, comparing the load obtained at the same vertical displacement, the difference between numerical and experimental values is small. As noted in the previous case, the initial numerically obtained stiffness of test S2 (see Fig. 9) is also slightly higher than the experimental one. The calculated resistance is 3.7 % lower compared with the experimental results.

4.2 Parametric study

Two sets of numerical analyses were performed. First of all, the influence of the stiffness of transverse stiffeners was studied on girders that were equivalent to the test girders S1 and S2. The second series was performed on girders loaded with a combination of high bending and high shear loads (see Fig. 10). All important parameters were varied in the analysis.

A bilinear material model with yield strength f_y = 355 MPa was used in the parametric studies. According to EN 1993-1-5, the equivalent geometric imperfection of the transverse stiffener under consideration was taken as a half sine wave imperfection with amplitude $w_0 = h_w/300$. Each adjacent stiffener was straight (see Fig. 2). Both imperfection directions were studied. It was found that the imperfection in the direction of the transverse stiffener is decisive due to larger eccentricity of the tension field action.

4.2.1 Stiffeners S1 and S2

The size of the transverse stiffener stiffness was varied for numerical simulations on test girders S1 and S2. Table 4 summarizes the dimensions and stiffnesses of the stiffeners used. The stiffness of the stiffener was normalized with the minimum required stiffness (see Table 5) given by Eq. (6) and with the stiffness required to fulfil strength and stiffness requirements taking account of tension field action at the ultimate shear resistance of the girder and deviation forces due to the corresponding bending moment in the adjacent panel. The simplified stiffener resistance model from *Beg* et al. [16] was used for this purpose.

The results of the numerical simulations are presented through:

a) The evolution of out-of-plane displacements along the transverse stiffener.

Table 5. Stiffness of stiffener according to different requirements

I_{req} (cm^4)	EN 1993-1-5 Eq. (6)	100 % TFA	50 % TFA
S1	115.2	545.4	179.4
S2	165.9	522.4	161.2

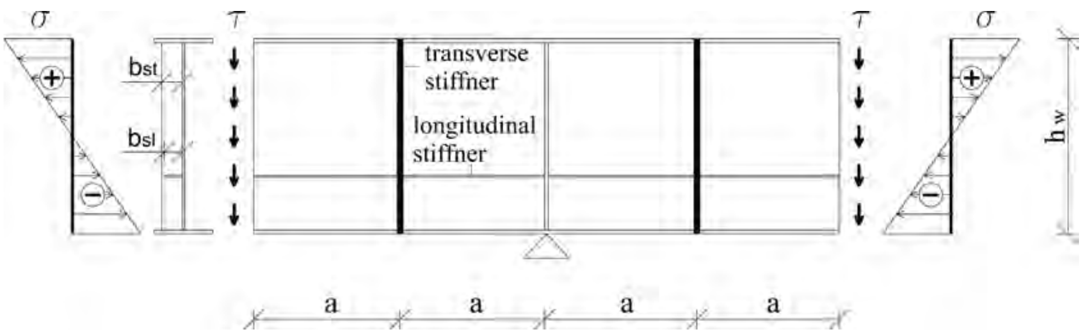

Fig. 10. Numerical model of stiffened plate girder under combination of high bending moment and shear force

Table 4. Stiffness variation of transverse stiffeners in tests S1 and S2

TEST\STIFFENER	I1	I2	I3	I4	I5	I6	I7	I8
S1- b_{st}/t_{st} [mm]	60/6	70/7	80/8	110/11	120/12	130/13	140/14	150/15
I/I_{req} Eq. (6)	0.33	0.58	0.94	2.90	3.92	5.15	6.63	8.37
I/I_{req} 100 % TFA	0.07	0.12	0.20	0.61	0.83	1.09	1.40	1.77
I/I_{req} 50 % TFA	0.21	0.37	0.60	1.86	2.52	3.31	4.26	5.38
S2- b_{st}/t_{st} [mm]	60/6	70/7	80/8	100/10	120/12	140/14	150/15	158/16
I/I_{req} Eq. (6)	0.23	0.40	0.65	1.44	2.72	4.60	5.81	6.99
I/I_{req} 100 % TFA	0.07	0.13	0.21	0.46	0.86	1.46	1.85	2.22
I/I_{req} 50 % TFA	0.24	0.42	0.67	1.49	2.80	4.74	5.98	7.19

b) The amplitude of the maximum displacement and maximum capacity for transverse stiffeners with different stiffnesses.

Fig. 11 shows the out-of-plane displacements obtained at the maximum force for cases S1 and S2. The shape of the out-of-plane displacement is directly linked with the stiffness of the stiffener. The "S" shape of the out-of-plane displacement of the transverse stiffeners was observed for stiffnesses I1 and I2. These stiffeners are not rigid so they do not prevent shear buckling over the depth of the web. The lateral deflection over the height of the stiffener is governed by the buckling in the panel. The amplitudes for weak stiffeners were –10.5 and 6.2 mm for cases S1-I1 and S2-I1 respectively. By increasing the stiffness of the transverse stiffener, the shape was transformed from the "S" to the "C" (I3) shape with much smaller amplitudes.

Fig. 12 shows the influence of the stiffness of transverse stiffener on the maximum resistance and maximum amplitude of the out-of-plane displacement. On the horizontal axis, the actual stiffness of the transverse stiffener is normalized with stiffness I_{req} calculated as:

a) Minimum stiffener requirement according to Eq. (6).
b) Required stiffness calculated to fulfil strength and stiffness requirements under the influence of deviation forces and tension field action (EN 1993-1-5).
c) Required stiffness calculated to fulfil strength and stiffness requirements under the influence of deviation forces and 50 % of tension field action (EN 1993-1-5).

An instantaneous decrease in the maximum out-of-plane displacements and increase in the girder capacity of girders S1 and S2 had already been found at small I/I_{req} ratios. The increase in the girder capacity plotted on the vertical axis in Fig. 12 was very small. The difference between maximum and minimum values was only 3.5 % for girder S1 and 2 % for girder S2. As can be seen from the diagram, the maximum displacement of the stiffener is already below the $h_w/300$ limit for the small stiffener I2 (see Table 4). To

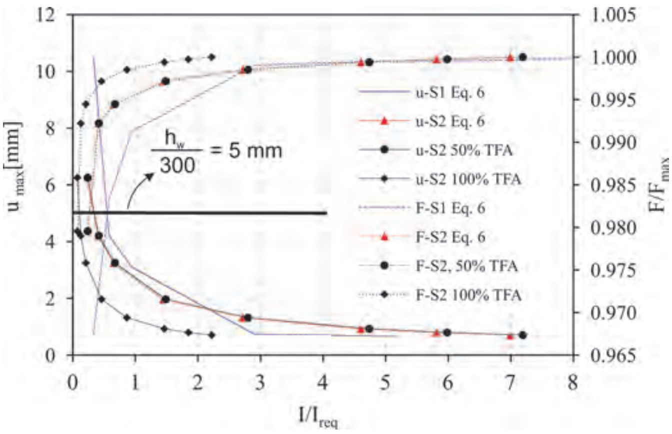

Fig. 12. Influence of stiffness of transverse stiffener on girder resistance and out-of-plane displacement of stiffener

fulfil the displacement criterion ($w \le h_w/300$) in case a), 40 % of the required stiffness is needed for girder S1 and 60 % for girder S2, in case b), 13 % for S2, and in case c), 42 % for S2. In all cases the stiffness requirement was found to govern. The stresses in the stiffener were always below yield strength, except in the part of the web where the tension field formed.

4.2.2 Stiffeners subjected to high bending moment and shear force

The parameters in the parametric analysis were: stiffness of transverse stiffener, web slenderness, ratio of flange area to web area, stiffness of longitudinal stiffener and panel aspect ratio. Seven different girder cross-section geometries were analysed. The basic geometry of the girder (G1 in Table 6) is defined with the following parameters: $h_w/t_w = 250$, $A_f/A_w = 0.7$, $\gamma/\gamma^* = 3.0$ and $\alpha = a/h_w = 1.0$, where γ denotes the relative stiffness of the longitudinal stiffener and γ^* the limit stiffness of the longitudinal stiffener which prevents overall buckling of the web panel in pure shear, including

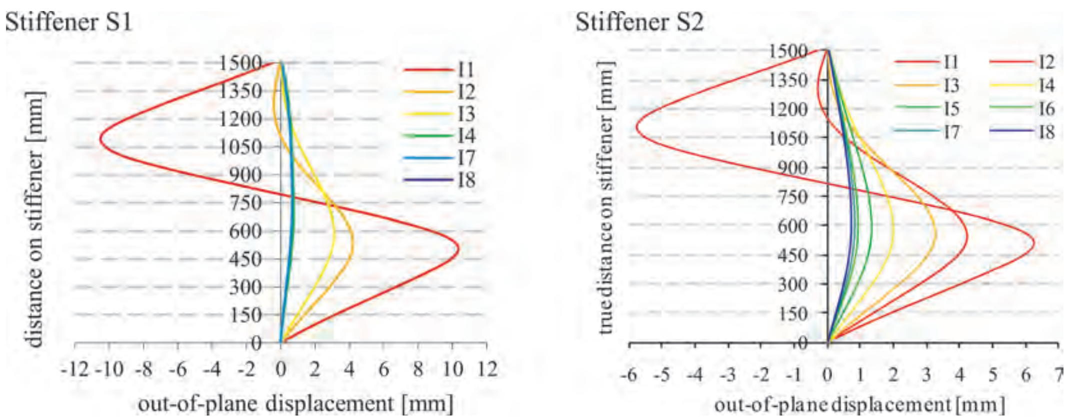

Fig. 11. Out-of-plane displacement of transverse stiffeners S1 and S2

Table 6. Parameters taken into account for girders loaded with high bending and shear load

STIFFENER	I1	I2	I3	I4	I5	I6	I7	I8
b_{st}/t_{st} [mm]	20/2	40/4	60/6	80/8	100/10	120/12	150/15	200/20
GIRDER	G1	G2	G3	G4	G5	G6	G7	G8
	$h_w/t_w = 250$	$A_f/A_w = 0.3$	$A_f/A_w = 1.1$	$\gamma/\gamma^* = 0.30$	$\gamma/\gamma^* = 1.00$	$\alpha = 0.5$	$h_w/t_w = 150$	$h_w/t_w = 350$

Table 7. Required stiffener stiffnesses considering different requirements

I_{req} (cm^4)	AASHTO Eq. (1)	EN 1993-1-5 Eq. (6)	100 % TFA	50 % TFA
G1–G5	51.2	153.6	3617.4	887.1
G6	409.6	614.4	1696.0	697
G7	237.0	711.1	710.2	349.6
G8	18.7	56.0	2529.7	753.8

deflection of this stiffener. In all other cases (girders G2–G8), only one parameter was changed compared with the basic girder (G1). Within each girder, the additional parameter was the stiffness of the transverse stiffeners. The geometry of the transverse stiffeners in the parametric study is given in Table 6.

Table 7 lists the second moments of area for all transverse stiffeners (I1–I8) calculated for different code requirements – AASHTO (Eq. (1)), EN 1993-1-5 (Eq. (6)) – and in the last two columns according to strength and stiffness checks assuming deviation forces and forces due to tension field action. The column "100 % TFA" denotes that full tension field action was considered, and "50 % TFA" denotes that only 50 % of tension field action was taken into account in the design of the stiffener.

The out-of-plane deflections of transverse stiffeners loaded with a combination of shear force and bending moment are plotted in Fig. 13. The curves are plotted for girder G1 and for different transverse stiffeners (see Table 6). The deflected shape depends on the stiffness of the stiffener. By increasing the stiffness, the deflection of the stiffener is transformed from the "S" shape to the "C" (I5) shape. The resistances of the girders obtained at a deflection of $h_w/300 = 6.67$ are plotted in Fig. 14. The resistance

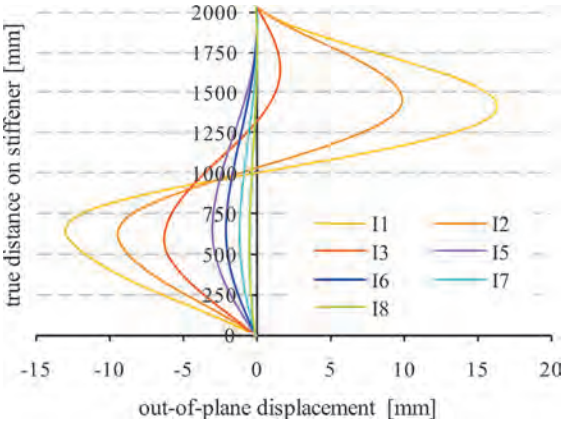

Fig. 13. Out-of-plane displacement along transverse stiffener for girder $h_w/t_w = 250$, $\alpha = 1$, $\gamma/\gamma^* = 3.0$, $A_f/A_w = 0.7$

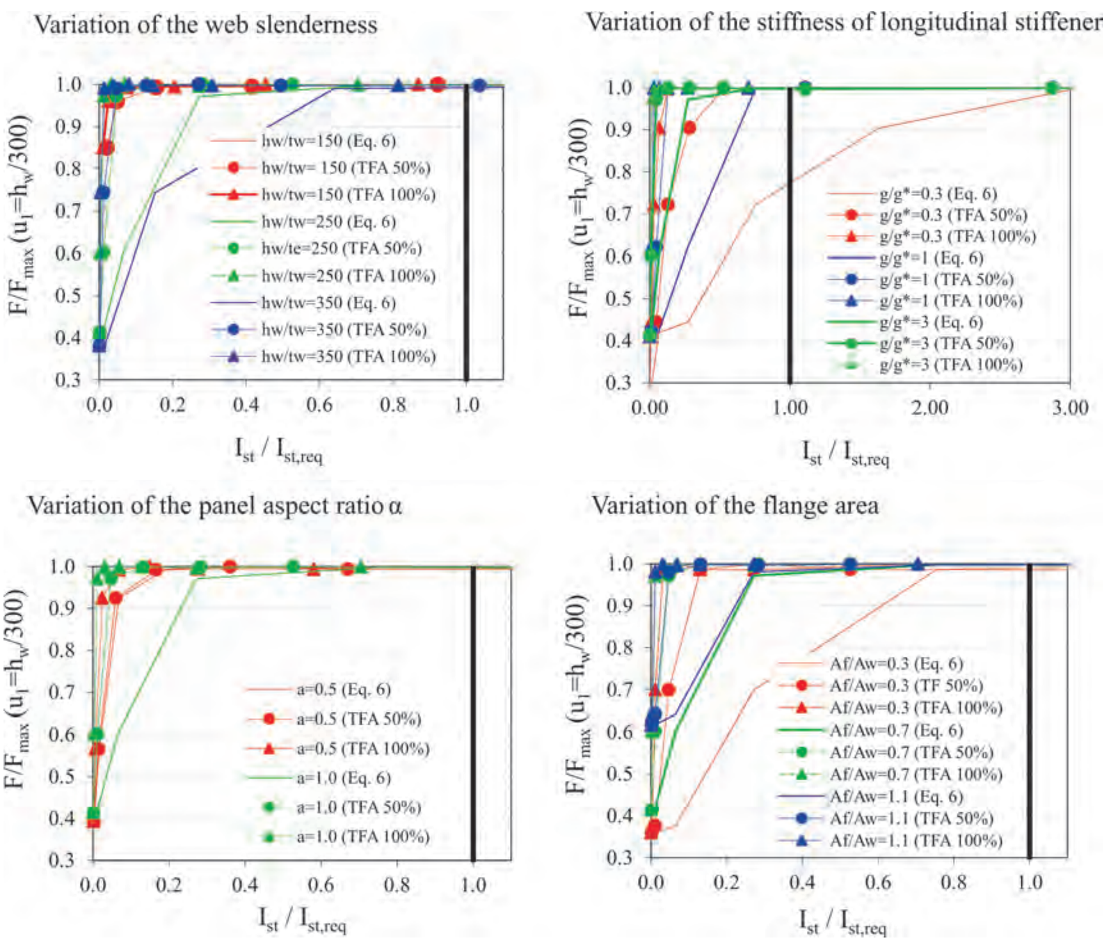

Fig. 14. The normalized force obtained at an out-of-plane displacement of $h_w/300$ for different stiffener stiffnesses

was normalized with the maximum force obtained in all girders with the same cross-section properties which were analysed, whereas the actual stiffness is normalized with the required stiffness given by Eq. (6) and with the stiffness required to fulfil strength and stiffness conditions taking into account 50 or 100 % of the tension field action. When the value 1 is reached on the y axis, the resistance of the girder proves its maximum resistance and at the same time the out-of plane deflection of the intermediate transverse stiffener is below the limit value of $h_w/300$. At this point, minimum stiffness can be read to fulfil stiffness criteria. As it can be seen, the EN 1993-1-5 stiffness requirement generally covers most of design cases because the value 1 on the y axis is reached before value 1 on the x axis.

5 Conclusions

If the transverse stiffener of a plate girder is designed to accommodate deviation forces from bending moments and axial forces from tension field action according to EN 1993-1-5, this results in much bigger stiffener than that obtained from numerical simulations. This can be attributed to the overestimation of the axial forces in the stiffener due to tension field action. The actual force, measured in our own tests, represents 56 % of the force calculated according to Eq. (7). A similar effect was also observed in [12–15].

This overestimation of the axial force in the stiffener is very important for the single-sided stiffeners mostly used for intermediate transverse stiffeners. It has been demonstrated in this paper that the required performance of rigid intermediate transverse stiffeners of longitudinally stiffened plate girders may be obtained by fulfilling simple stiffness criteria given by Eq. (6) increased by a factor that is no larger than 3 but can in most cases be equal to 1. This simplifies the design procedure for intermediate transverse stiffeners while considering all relevant effects.

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Reduced stress design of plates under biaxial compression

European standard EN 1993-1-5 provides the effective width method and the reduced stress method as calculation methods for plate buckling assessment. This article focuses on the reduced stress method, for which the global slenderness concept was introduced into EN 1993-1-5. The concept offers advantages associated with today's numerical procedures which allow the elastic critical load factor of the full stress field to be determined in a single step, thus taking interaction into account in a very elaborate way. However, this article shows that the interaction verification in its pure format based on the von Mises *yield criterion is not able to represent the actual mechanical behaviour of biaxially compressed plates. A simple modification is proposed which leads to appropriate and plausible results.*

1 Introduction

Slender plates and plated structures subjected to compressive in-plane loadings are intrinsically sensitive to buckling. As buckling is a complex process that involves nonlinear stress distributions, load-shedding effects between cross-sectional parts and the influence of imperfections as well as boundary conditions, even today, designers usually favour easy-to-handle calculation methods over non-linear numerical analyses. European standard EN 1993-1-5 [3] offers two different calculation methods for such a plate buckling assessment: the effective width method and the reduced stress method. The former is highly efficient for standard geometries because it accounts for load-shedding between cross-sectional parts. As a requirement, it subdivides the stress field into stress components for which individual plate buckling checks are required (EN 1993-1-5, chapters 4 to 6) and which are finally combined in interaction rules (chapter 7). However, the effective width method is not applicable for non-uniform geometries and certain types of loading. In contrast, the reduced stress method applies to almost any geometries and loadings due to the generic concept that takes into account the full stress field and its interaction (EN 1993-1-5, chapter 10). This article focuses on the reduced stress method and solves shortcomings for biaxially compressed plates in particular.

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2 Design methodology

2.1 General

Although the elastic critical buckling load has been the starting point for an understanding of plate buckling behaviour, linear buckling theory alone is not an appropriate predictor of the ultimate buckling strength. The asset of linear buckling theory is, however, that the elastic critical buckling load can be explicitly mathematically defined, assuming perfect geometry and linear material behaviour. This makes it, even today, well suited as an input parameter for the common approach to determine the ultimate buckling strength via a reduction in plastic strength:

$$\bar{\lambda} = \sqrt{\frac{R_{pl}}{R_{cr}}} \;\rightarrow\; R_{ult} = \rho\left(\bar{\lambda}\right) \cdot R_{pl} \tag{1}$$

where:
R_{pl} plastic strength
R_{cr} elastic critical buckling strength
R_{ult} ultimate buckling strength
$\bar{\lambda}$ slenderness
$\rho(\bar{\lambda})$ reduction factor, function of the slenderness

The reduction factor ρ (also denoted χ or κ) encompasses the influence of structural and material imperfections. Moreover, it reflects the considerable post-critical strength reserve and the detrimental column-like behaviour that plates can possess. In EN 1993-1-5 [3], the effective width and reduced stress methods make use of the design methodology in different ways. Since from the European viewpoint the reduced stress method is often perceived as a German speciality, the following section tries to give an overview on the procedure and background of the concept in EN 1993-1-5, chapter 10 as well as of DIN 18800-3.

2.2 Reduced stress method

2.2.1 EN 1993-1-5 concept

EN 1993-1-5, chapter 10 [3], provides the framework for the reduced stress method in sections 10(1), 10(3), 10(4) and 10(5b). Section 10(7) may also be relevant for stiffener design and detailing. The basic idea behind the global slenderness concept in EN 1993-1-5 follows former German guideline DASt-Ri 012 [1] dating from 1980. It differs from

German standard DIN 18800-3 [2] which is currently in use, see section 2.2.2.

Section 10(1) explains the application range in brief. At present, load-shedding between cross-sectional parts in plated structures is not accounted for. Thus, the weakest part of the cross-section usually governs the resistance of the whole cross-section, which may lead to significant differences when results are compared with the effective width method. However, it was shown in [11] that the reduced stress method could be expanded to overcome this.

Section 10(3) introduces the global slenderness $\bar{\lambda}_p$:

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} = \sqrt{\frac{f_y}{\sigma_{eq,cr}}} \tag{2}$$

where:

$\alpha_{ult,k} = \dfrac{f_y}{\sigma_{eq,Ed}}$; characteristic resistance load factor

$\alpha_{cr} = \dfrac{\sigma_{eq,cr}}{\sigma_{eq,Ed}}$; elastic critical load factor

f_y yield strength

$\sigma_{eq,cr}$ equivalent elastic critical buckling stress

$\sigma_{eq,Ed}$ design equivalent stress

Expressed in plain terms, the global slenderness relates the yield strength f_y to the equivalent elastic critical buckling stress $\sigma_{eq,cr}$. The reduced stress method is used most efficiently when $\sigma_{eq,cr}$ can be determined with numerical procedures in one step. Since the output of a linear bifurcation analysis is commonly a load factor instead of a stress value, the elastic critical load factor α_{cr} has been introduced for convenience. Section 10(6) describes a manual calculation procedure for α_{cr} which is, however, quite tedious. By relating the elastic critical load to the design equivalent stress $\sigma_{eq,Ed}$, the same has to be done for the characteristic resistance. However, both quotients can be reduced in the end, see Eq. (2).

Determining the global slenderness for unstiffened plates usually means performing a single calculation for the full stress field. For stiffened plates, at least two separate calculations are required which take into account either local buckling of subpanels or global buckling of the stiffened panel.

In the next step, the global concept fades into a semi-global one because the determination of the reduction factors and the evaluation of column-like buckling are attributed to the corresponding stress component, see Eq. (3). The designer has to choose the appropriate reduction function depending on the boundary conditions that represent the system. In EN 1993-1-5, this means choosing from section 4.4(2) and/or Annex B for plate buckling, 4.5.3(5) for column buckling and 5.3(1) and/or Annex B for shear buckling:

$$\rho_x\left(\bar{\lambda}_p\right); \rho_z\left(\bar{\lambda}_p\right); \chi_w\left(\bar{\lambda}_p\right) \tag{3}$$

where:

ρ_x reduction factor for longitudinal stresses, interpolated value $\rho_{x,c}$ where column-like buckling is relevant

ρ_z reduction factor for transverse stresses, interpolated value $\rho_{z,c}$ where column-like buckling is relevant

χ_w reduction factor for shear stresses

The interpolation between plate-like and column-like buckling follows section 4.5.4(1) of EN 1993-1-5. It is based on the ratio between the elastic critical plate buckling stress $\sigma_{cr,p}$ and the elastic critical column buckling stress $\sigma_{cr.c}$. The determination of $\sigma_{cr,p}$ and $\sigma_{cr.c}$ is carried out for each relevant stress component separately assuming identical boundary conditions.

The general verification format according to Eq. (4) is given in section 10(5b). The interaction between stress components is based on a variation of the *von Mises* yield criterion:

$$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{\sigma_{z,Rd}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)\cdot\left(\frac{\sigma_{z,Ed}}{\sigma_{z,Rd}}\right) + 3\cdot\left(\frac{\tau_{Ed}}{\tau_{Rd}}\right)^2 \le 1 \tag{4}$$

where

σ_{Ed}; τ_{Ed} design load values

$\sigma_{x,Rd}$ $= \rho_x \cdot f_{yd}$; design resistance value

$\sigma_{z,Rd}$ $= \rho_z \cdot f_{yd}$; design resistance value

τ_{Rd} $= \chi_w \cdot f_{yd}$; design shear resistance value

f_{yd} design yield strength

Sections 10(2) and 10(5a) offer an alternative verification format according to Eq. (5). Both sections correspond with each other and it is presumed that the minimum value of all reduction factors is taken. Without doubt, it may be convenient in the preliminary dimensioning process but it is usually too conservative in the final design of plates when different loadings are present:

$$\sigma_{eq,Ed} \le \rho_{min} \cdot f_{yd} \tag{5}$$

where:

$\sigma_{eq,Ed}$ design equivalent stress

ρ_{min} $= \min(\rho_x;\rho_z;\chi_w)$; minimum value of all reduction factors

f_{yd} design yield strength

2.2.2 DIN 18800-3 concept

The reduced stress method is incorporated in a different way in German standard DIN 18800-3 [2]. In the following, DIN notation has been slightly adjusted to compare with EN notation and only the most prominent differences are outlined. Instead of the semi-global concept of EN 1993-1-5, DIN 18800-3 focuses on stress components from the beginning. Thus, a separate slenderness for each relevant stress component is determined:

$$\bar{\lambda}_{x,p} = \sqrt{\frac{f_y}{\sigma_{x,cr}}};\ \bar{\lambda}_{z,p} = \sqrt{\frac{f_y}{\sigma_{z,cr}}};\ \bar{\lambda}_{\tau,p} = \sqrt{\frac{f_y}{\tau_{cr}\cdot\sqrt{3}}} \tag{6}$$

where:

f_y characteristic yield strength

$\sigma_{x,cr}$; $\sigma_{z,cr}$; τ_{cr} elastic critical buckling stress

In the next step, separate reduction factors are determined for each slenderness:

$$\kappa_x\left(\bar{\lambda}_{x,p}\right);\ \kappa_z\left(\bar{\lambda}_{z,p}\right);\ \kappa_\tau\left(\bar{\lambda}_{\tau,p}\right) \tag{7}$$

where:
κ_x reduction factor for longitudinal stresses, interpolated value $\kappa_{x,PK}$ where column-like buckling is relevant
κ_z reduction factor for transverse stresses, interpolated value $\kappa_{z,PK}$ where column-like buckling is relevant
κ_τ reduction factor for shear stresses

Finally, the general verification format to account for interaction between stress components according to Eq. (8) is based on an interaction equation with the parameters $e_{i=1,2,3}$ and V:

$$\left(\frac{|\sigma_{x,Ed}|}{\sigma_{x,Rd}}\right)^{e_1}+\left(\frac{|\sigma_{z,Ed}|}{\sigma_{z,Rd}}\right)^{e_2}-V\cdot\left(\frac{|\sigma_{x,Ed}\cdot\sigma_{z,Ed}|}{\sigma_{x,Rd}\cdot\sigma_{z,Rd}}\right)+3\cdot\left(\frac{\tau_{ed}}{\tau_{Rd}}\right)^{e_3}\le 1 \tag{8}$$

where:
σ_{Ed}; τ_{Ed} design load values
$\sigma_{x,Rd}$ $= \kappa_x \cdot f_{yd}$; design resistance value
$\sigma_{z,Rd}$ $= \kappa_z \cdot f_{yd}$; design resistance value
τ_{Rd} $= \kappa_\tau \cdot f_{yd}$; design shear resistance value
f_{yd} design yield strength
e_1 $= 1 + \kappa^4_x$
e_2 $= 1 + \kappa^4_z$
e_3 $= 1 + \kappa_x \cdot \kappa_z \cdot \kappa^2_\tau$
V $= (\kappa_x \cdot \kappa_z)^6$ when $\sigma_{x,Ed}$ and $\sigma_{z,Ed}$ are both compression, otherwise $V = \dfrac{\sigma_{x,Ed}\cdot\sigma_{z,Ed}}{|\sigma_{x,Ed}\cdot\sigma_{z,Ed}|}$

As the global slenderness was introduced in EN 1993-1-5, so the concept of the reduced stress method in DIN 18800-3 was varied. In DIN 18800-3 the interaction verification is only allocated to Eq. (8), for which parameters $e_{i=1,2,3}$ and V have been calibrated to fit the database in [8]. In contrast, the interaction in EN 1993-1-5 is taken into account both at elastic critical load level and finally by the interaction equation according to Eq. (4). Thus, it is not possible to transfer simply the values of the parameters determined for the concept in DIN 18800-3 to the concept in EN 1993-1-5.

3 Plates under biaxial compression
3.1 General

Figs. 1 and 2 illustrate the procedures described in section 2.2 for square plates under uniform biaxial compression. Explanatory notation is given in Fig. 3. It can be seen that the global slenderness concept in EN 1993-1-5 leads to a varying slenderness for a given b/t ratio, thus taking interaction into account in a very elaborate way. In contrast, the slenderness according to DIN 18800-3 is based on separate stress components and thus is constant for a given b/t ratio. The projection of the interaction curves onto the ρ_x–ρ_z plane is compared in Figs. 4 and 5 for representative b/t ratios. It can be shown that the curves are significantly different. But this does not depend on the reference strengths at the axes; rather, it depends on the capability of the interaction surface, which is defined by the format of the underlying interaction equation, see also [7, 13]. In fact, when the slenderness approaches the border to class 3 cross-sections, i.e. b/t ≈ 30 here, a stability-induced reduction is expected under biaxial compression, which is covered by EN 1993-1-5 but not by DIN 18800-3. On the other hand, a linear interaction should be approached with increasing

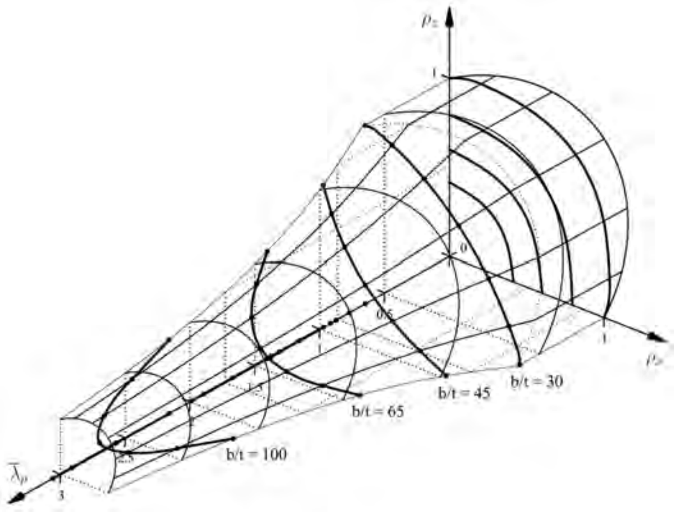


Fig. 1. Interaction scheme according to EN 1993-1-5, chapter 10 ($\alpha = 1$, $f_y = 355$ N/mm²)

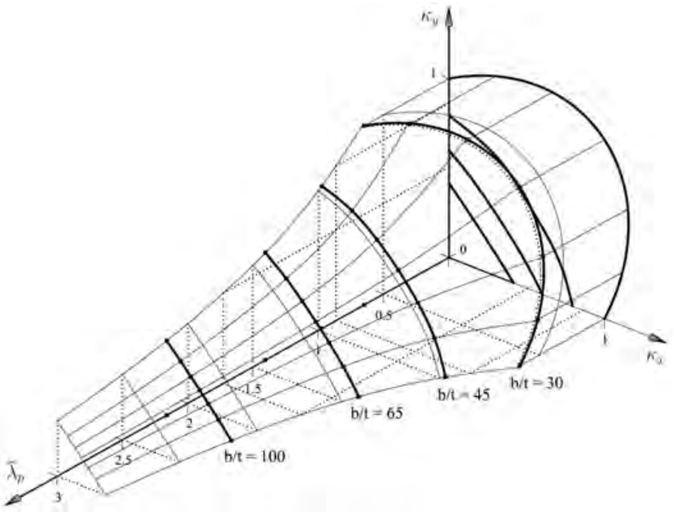


Fig. 2. Interaction scheme according to DIN 18800-3 ($\alpha = 1$, $f_y = 355$ N/mm²)

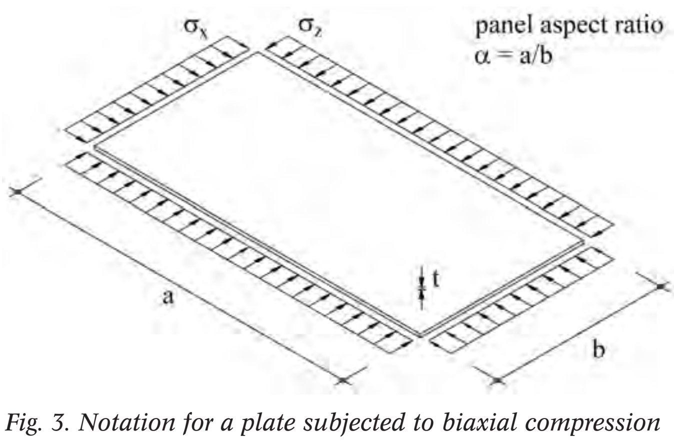
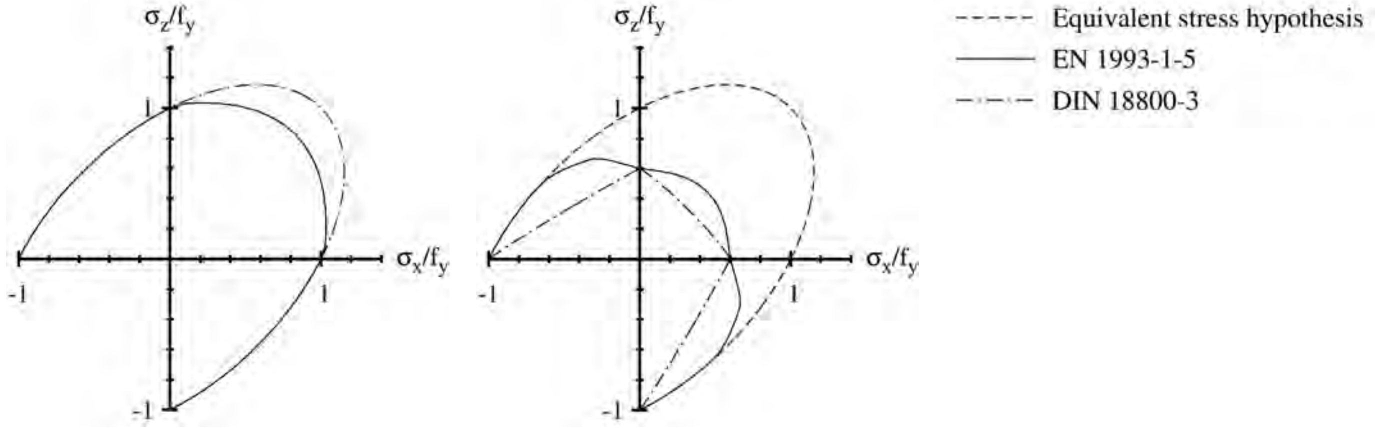
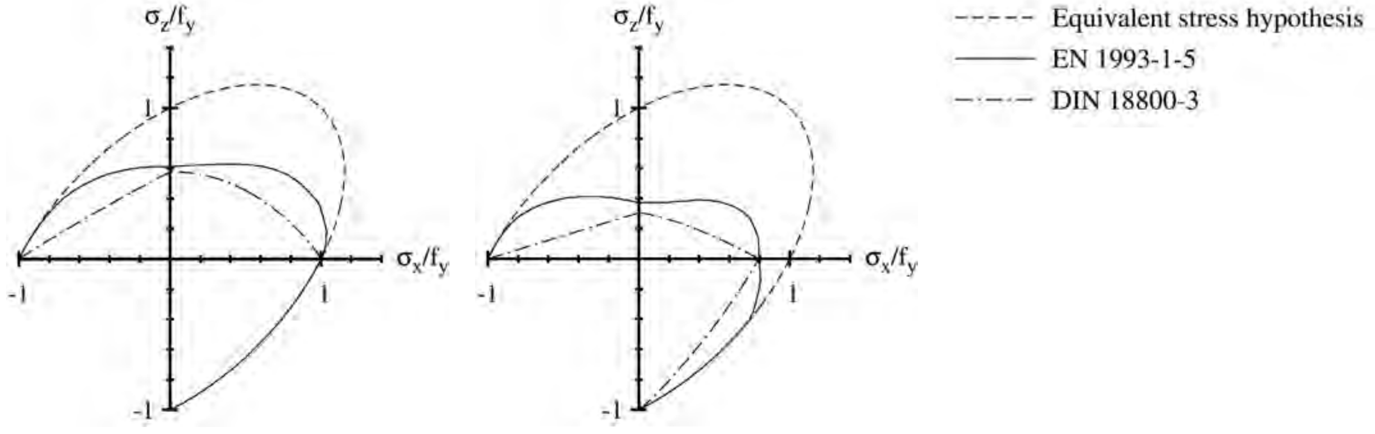


Fig. 3. Notation for a plate subjected to biaxial compression



a) b/t = 30 *b) b/t = 65*

Fig. 4. Comparison of interaction curves for biaxial loading ($\alpha = 1$, $f_y = 355$ N/mm^2)



a) b/t = 30 *b) b/t = 45*

Fig. 5. Comparison of interaction curves for biaxial loading ($\alpha = 3$, $f_y = 355$ N/mm^2)

slenderness – covered by DIN 18800-3 but not by EN 1993-1-5. In principle, the interaction equation should basically reflect the mechanical behaviour irrespective of the verification format. Since the EN 1993-1-5 concept considers the full stress field, it is better suited to describing the mutual influence of stresses. However, the non-negligible discrepancies for high slenderness require clarification.

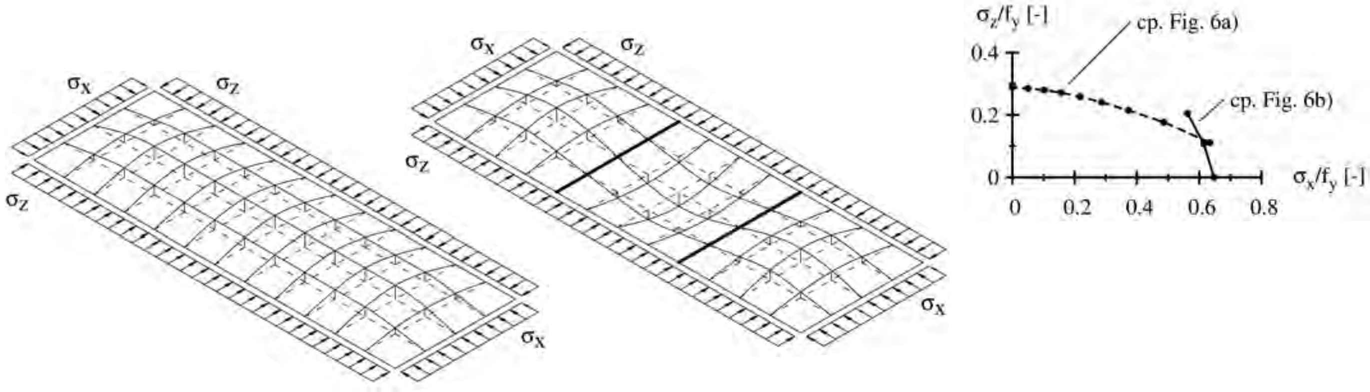
3.2 Consequences of earlier work

Earlier research work comprises a large number of studies, though few experimental ones, which date back to the 1970s and 1980s. Later work mainly focused on summarizing and re-evaluating these studies. The evaluation revealed a large knowledge base where, however, the large number of influential parameters always led to a discussion about the quality and usability of the results. For this reason, additional simulations were carried out in [4] which have been used for reassessing earlier work and which form the principal basis for the proposal in section 4. Some aspects related to the parameter study of biaxially compressed plates are presented in the following.

3.3 Effect of imperfections and panel aspect ratio

For a rectangular plate, the imperfection shape that leads to the smallest resistance depends on the number of half-waves and the corresponding loading direction. Figs. 6a and 6b show two modal imperfection shapes of a biaxially loaded plate with a panel aspect ratio of $\alpha = 3$. For each of the uniaxial loadings, the corresponding mode leads to the smallest resistance. However, the one-half-wave mode increases the resistance when the short edges are mainly loaded because the out-of-plane deformation usually prevents the plate from switching to its natural three-half-wave mode. On the other hand, in the case of biaxial compression, the inflexion points of the three-half-wave mode act like a stiffening of the plate (thick lines in Fig. 6b).

In the parameter study, plates with a panel aspect ratio of $\alpha = 1$ and 3 were considered. A large number of initial mode shape combinations was studied in [5] for panel aspect ratios of $\alpha = 3$ but a smaller failure resistance than that achieved with the three- and one-half-wave mode shapes could not be produced. Measurements of the initial imperfection shapes occurring in normally fabricated plates [6] support that, in principle, the one-half-wave mode shape is the one most likely to occur in practice. Thus, the three-half-wave mode shape is usually on the safe side when the loading of the short edges begins to dominate. The interaction curve shape in Fig. 6c can therefore be seen as an example of a lower bound. In terms of imperfections, the square plate is the simplest one, albeit the worst case, because the imperfection shapes for each stress component usually coincide.



a) one-half-wave mode *b) three-half-wave mode* *c) effect on interaction curve*

Fig. 6. Comparison of imperfection shapes ($\alpha = 3$, $f_y = 355$ N/mm^2)

The imperfection amplitude has a pronounced influence in the small to medium slenderness range. In the parameter study, equivalent geometric imperfections were used with amplitude values of min(a/200; b/200) according to EN 1993-1-5, Annex C, and min(a/420; b/420) based on [12]. It was shown that recalculating buckling curves succeeds only with the reduced amplitude, see also [9]. Residual stresses were studied but it was shown that they do not increase the quality of the parameter study significantly.

3.4 Effect of edge boundary conditions

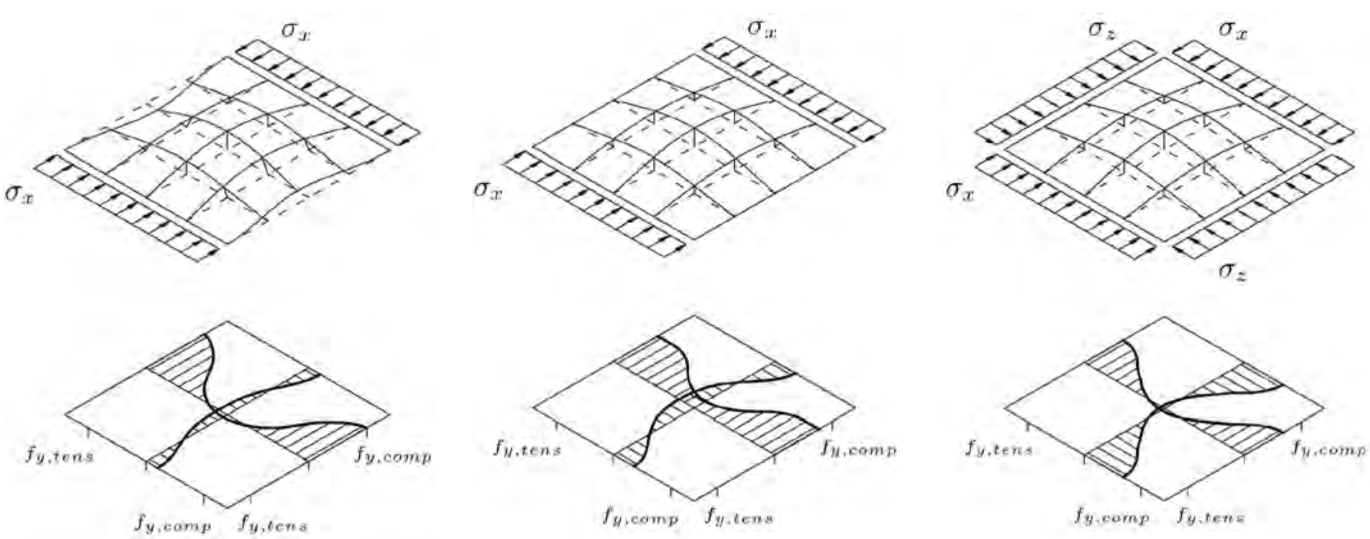
The edge boundary condition can be subdivided into in-plane and rotational restraints. Extreme in-plane edge configurations are the "unconstrained" case (free to move in-plane, stress-controlled) and the "constrained" case (forced to remain straight, strain-controlled). Examples of the configurations and their effect on stress distributions are shown in Figs. 7a and 7b for uniaxially loaded plates. Fig. 7c illustrates the same situation for all edges constrained and biaxial compression. The case with unconstrained edges and biaxial compression was also addressed in the parameter study. It is concluded from the comprehensive recalculations of buckling curves in [4] that an appropriate assessment of the reference strength is required in order to achieve meaningful results. Thus, in the parameter study, the buckling curves chosen represent the corresponding edge boundary condition as precisely as possible. EN 1993-1-5, Annex B, with $\alpha_p = 0.34$ and $\overline{\lambda}_{p0} = 0.70$, is used for the case with all edges unconstrained. In the case of all edges constrained, EN 1993-1-5 does not provide an adequate reduction function, so for this case the authors' own databased buckling curve was used.

Rotational edge boundary conditions are usually covered at elastic critical load level since their effectiveness does not depend on large deflections.

It was found that an appropriate choice of buckling curve facilitates the use of few interaction curves which are then able to cover different edge boundary conditions.

4 Proposal for an improved design rule

In order to achieve an adequate description of the stability behaviour, a modification of the interaction equation in EN 1993-1-5, chapter 10, is required. The proposal focuses on the high slenderness range where, for square plates, a



a) at ultimate load, unloaded edges unconstrained *b) at ultimate load, unloaded edges constrained* *c) at ultimate load, all edges constrained and biaxially loaded*

Fig. 7. Membrane stress distributions, loaded edges constrained ($b/t = 100$, $f_y = 355$ N/mm^2)

linear interaction relationship should be approached. For rectangular plates, the favourable imperfection mode shapes should be accounted for. In all cases, a tunable interaction equation has to be formulated. A generalized form of such an equation based on the *von Mises* yield criterion has already been given in Eq. (8), taken from DIN 18800-3. For biaxially compressed plates, the most relevant parameter with an effect on the interaction curve is the factor V. Thus, a tunable interaction equation can be written in simplified form:

$$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^2+\left(\frac{\sigma_{z,Ed}}{\sigma_{z,Rd}}\right)^2-V\cdot\left(\frac{\sigma_{x,Ed}\cdot\sigma_{z,Ed}}{\sigma_{x,Rd}\cdot\sigma_{z,Rd}}\right)\leq 1 \tag{9}$$

where:
σ_{Ed}; τ_{Ed} design load values
σ_{Rd}; τ_{Rd} design resistance value
V factor

The approach for the factor V is made according to Eq. (10):

$$V=\left(\rho_x\cdot\rho_z\right)^e \tag{10}$$

where:
ρ_x; ρ_z reduction factors
e tunable exponent

In the first step, the results of the parameter study on square plates are used to study the effect of the factor V. Due to the symmetry of the square plate, the curvature can be defined by points located on the bisecting plane at an angle of 45°, see Fig. 8. The comparison of simulations, design rules and proposals in this plane is given in Fig. 9. A distinction is made between unconstrained and constrained in-plane edge restraints. The *Winter* curve from EN 1993-1-5, section 4.4(2), is provided as a reference, but it should not

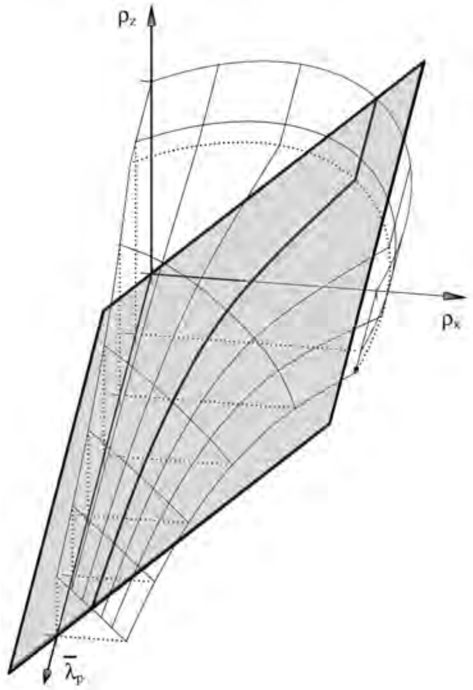
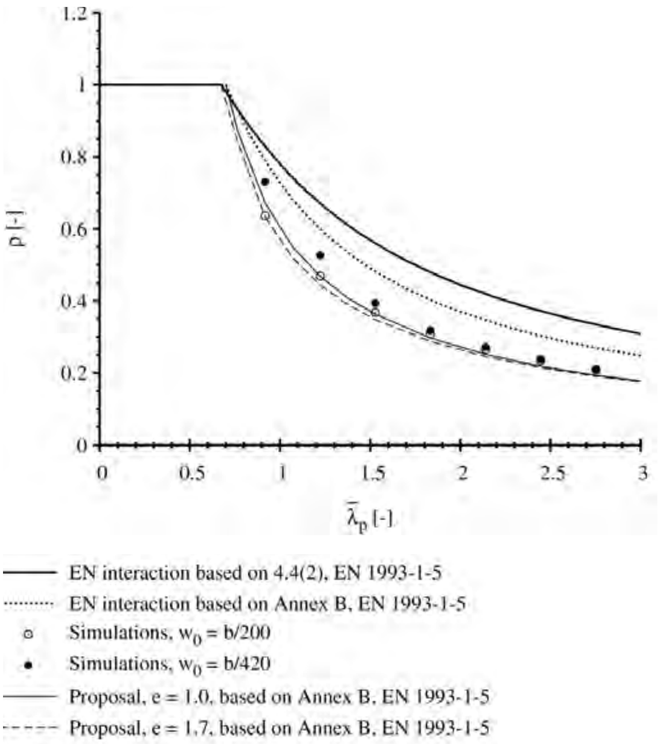
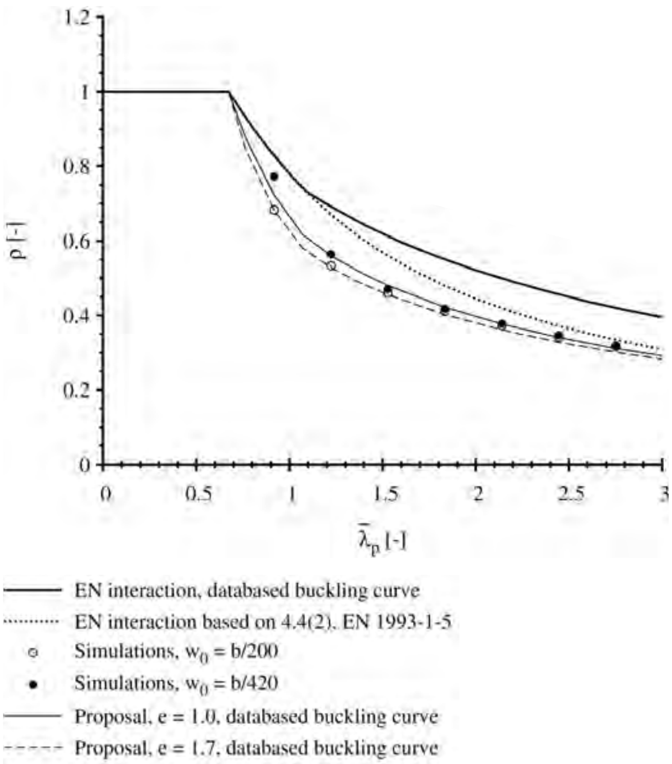


Fig. 8. Plane utilized for calibrating the exponent e in Eq. (10)



a) all edges unconstrained



b) all edges constrained

Fig. 9. Calibration of V and effect of in-plane edge restraints (α = 1, all edges hinged)

be considered further because it represents loaded edges constrained and unloaded edges unconstrained, which is not applicable to the biaxial load scenarios here.

Summing up, the tunable parameter e becomes 1.7 when fitted to the data as a lower bound. It is governed by the medium slenderness range and becomes slightly conservative for high slenderness values. Bearing in mind that the recalculation of the buckling curves in the medium slenderness range is only successful with a reduced imperfection amplitude of b/420, see section 3.3, a similar reasoning applies to the calibration of the interaction equation, and it is shown that e can be simplified to 1.0. Based on this, it is proposed to modify Eq. (10.5) in section 10 (5b), EN 1993-1-5, according to Eq. (11). In order to improve the layout of the verification procedure it is proposed to delete sections (2) and (5a):

$$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{\sigma_{z,Rd}}\right)^2 - V \cdot \left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right) \cdot \left(\frac{\sigma_{z,Ed}}{\sigma_{z,Rd}}\right) + 3 \cdot \left(\frac{\tau_{Ed}}{\tau_{Rd}}\right)^2 \le 1 \tag{11}$$

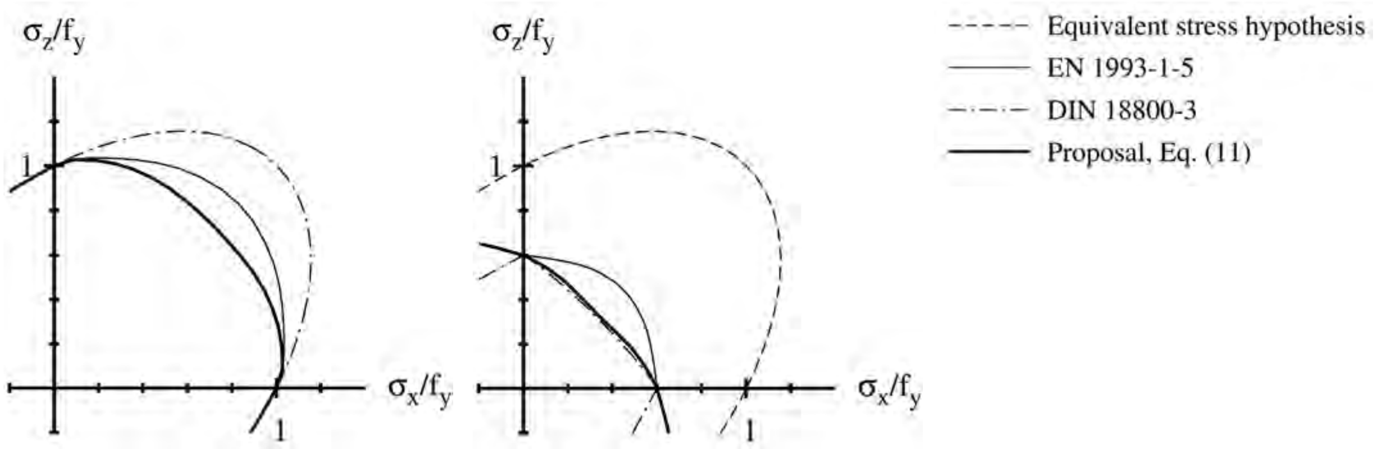
where:

σ_{Ed}; τ_{Ed} design load values
$\sigma_{x,Rd}$ $= \rho_x \cdot f_{yd}$; design resistance value
$\sigma_{z,Rd}$ $= \rho_z \cdot f_{yd}$; design resistance value
τ_{Rd} $= \chi_w \cdot f_{yd}$; design shear resistance value
f_{yd} design yield strength
V $= \rho_x \cdot \rho_z$ when $\sigma_{x,Ed}$ and $\sigma_{z,Ed}$ are both compression, otherwise $V = 1$

Comparing the simulations and the experimental results with the proposal reveals sufficiently good agreement. Eq. (11) is compared with EN 1993-1-5 and DIN 18800-3 in Figs. 10 and 11. For square plates, it can be shown that a linear interaction is approached as the plate becomes more slender. As a result, the proposal more or less coincides with DIN 18800-3. However, for a small slenderness, a stronger interaction has to be taken into account so that the proposed interaction equation becomes more conservative, which is supported by the simulations. For panel aspect ratios other than square, the imperfection shape was proved to have a significant influence on the curvature of the interaction curves. DIN 18800-3 is too conservative, whereas EN 1993-1-5 is the opposite, i.e. too favourable. Here, the proposed interaction equation is able to take the positive influence of the imperfection shape successfully into account.

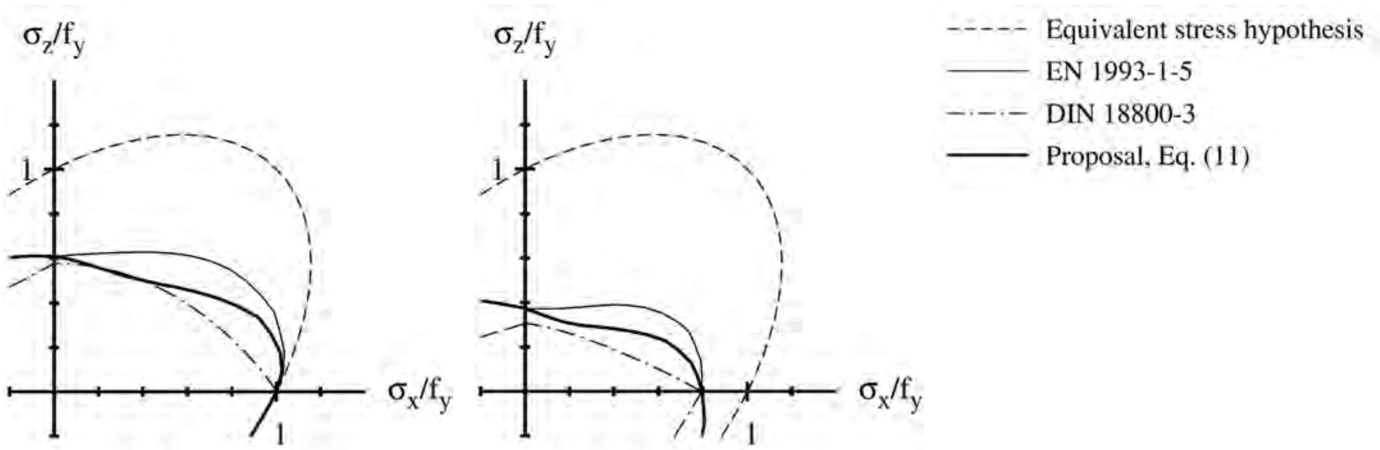
5 Conclusions and implications for further work

As the global slenderness was introduced in EN 1993-1-5, so the concept of the reduced stress method in DIN 18800-3 was varied. The basic idea behind EN 1993-1-5 follows former German guideline DASt-Ri 012. The new concept offers advantages over the today's numerical procedures which allow the elastic critical load factor of the full stress field to be determined in a single step, thus taking interaction into account in a very elaborate way. However, it is shown in this



a) b/t = 30 *b) b/t = 65*

Fig. 10. Comparison of interaction rules and proposal ($\alpha = 1$, $f_y = 355$ N/mm²)



a) b/t = 30 *b) b/t = 45*

Fig. 11. Comparison of interaction rules and proposal ($\alpha = 3$, $f_y = 355$ N/mm²)

article that the interaction verification in its pure format based on the *von Mises* yield criterion is not able to represent the actual mechanical behaviour of biaxially compressed plates. The proposed modification leads to appropriate and plausible results without over-complicating the interaction equation.

A recent publication [10] shows that a further improvement of EN 1993-1-5, chapter 10, has been facilitated based on [4].

Besider that, when the loading changes from compression–compression to compression–tension, the global slenderness concept is able to consider the effect of tensile stresses, see Figs. 4 and 5. However, only few pilot studies have looked at this effect. Since tensile stresses can have not only a stabilizing effect on buckling but also a plastic destabilizing influence, further work in this field will be necessary in the future.

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Keywords: stability; plate buckling; interaction; biaxial compression

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News

Economic and Steel Market Outlook 2012–2013

Eurofer's Q1-2012 economic and steel market outlook shows that while the EU economy probably slipped into a shallow recession in the final quarter of last year, activity of downstream steel-users has remained rather firm.

The latest economic data confirm that the Eurozone's debt crisis is eating its way into the real economy. Since mid-2011, economic sentiment has come down sharply on the back of the further intensification of the sovereign debt crisis. Risk aversion in the financial sector has led to a contraction in credit supply. While it is widely recognised that austerity is required to avoid the sovereign debt crisis from getting worse, it will also stifle economic growth and affect employment. Investment and private consumption will be subdued until confidence returns to the markets.

Eurofer director-general *Gordon Moffat*: "When confidence will return to the markets is still difficult to say, but it is encouraging that forward looking indicators such as industrial confidence and manufacturing PMIs appear to have taken a turn for the better in recent weeks. Companies will remain cautious for the time being, but there is no indication that industrial orders will fall off a cliff. Order backlogs are quite strong. The outlook for most steel-users is relatively benign".

EU apparent steel consumption grew 7.2 % in 2011. Steel purchasers waiting at the side-lines after the summer due to rising concerns about the economic situation, downstream business conditions and access to financing and credit together with destocking in the distribution chain caused steel demand to weaken in the 2nd half of 2011.

Early 2012, reduced domestic and import supply appears to be balanced with softer demand. Rather low stock levels – particularly at distributors – will require some replenishment in H1-2012, which will ease later in the year. Buyers anticipating changes in prices and supply could exacerbate this seasonal pattern, despite the fact that inventories will continue to be managed cautiously.

Eurofer director-general *Gordon Moffat* sees the market strengthening again in 2013: "The market will take a pause in 2012, but we foresee only a very mild reduction in demand. Real and apparent steel consumption will strengthen again in 2013".

Please visit www.eurofer.org for more information.

Gregor Nuesse*
Mukesh Limbachiya
Roland Herr
Alan Ellis

DOI: 10.1002/stco.201200006

Managing interdisciplinary applied research on sustainability in construction with the help of an innovation broker

The article reports on how the steel industry in Germany, as a key supplier of materials and innovations to the construction sector, is carrying out a comprehensive optimization of the management of pre-competitive cooperative research (CR) on steel applications in the construction industry. A significant aspect of this is the central use of an innovation broker (IB) role throughout the innovation process. The varying project expectations of science and industry and the related IB tasks are identified as part of an interdisciplinary research cluster on the sustainability of steel in the construction industry. A task profile is drawn up for the IB in cooperative research on the basis of various key person concepts from industrial innovation research. It is expected that the consistent implementation of this profile will increase the innovation successes from CR between the steel industry and the construction sector.

1 Introduction

For the sciences, application-oriented cooperative research (CR) is an important element of their interaction with the economy. It also has a positive effect on the innovation behaviour of the companies participating [1]. The benefits of pre-competitive forms of CR for small and medium-sized enterprises (SMEs) in particular have been clearly demonstrated [2]. The steel industry in Germany supports this kind of application-oriented research by engaging in joint pre-competitive research projects in cooperation with scientific institutions and companies from various steel application sectors. The Research Association for Steel Application (FOSTA) is responsible for managing these projects on behalf of the steel industry in Germany. The projects pursued by this body produce application-oriented results in the interests of all participants. Projects in cooperation with the construction sector account for a significant proportion of this type of steel application research. The starting point in the research management of these projects is described in [3] on the basis of typical innovation attributes in the construction sector and characteristic deficiencies in the existing CR system. These aspects are reproduced in summarized form below as the basis for further examination, and are partly expanded upon. The subsequent discussion on the statistical results of a survey devoted to the project expectations of science and industry in a FOSTA research cluster entitled "Sustainability of steel in the construction sector (NASTA)" enables the development of the task profile for an innovation broker (IB) operating within the framework of integrated management of CR between the steel industry, the construction sector and research institutes [4].

2 Summary of the innovation characteristics of the construction sector and the existing system deficiencies in cooperative research

The construction sector is often characterized as being not very innovative and having a traditional structure [5]. R&D investment and the number of registered patents are both extremely low in relation to other sectors [6, 7]. Innovations occur less within the framework of the sector's own innovation projects and more in response to the special demands of specific construction projects [8, 9]. Innovation opportunities therefore also depend on the many regulations that define specifications for materials, technologies and processes [6, 9]. Consequently, the industry itself often finds itself in a "locked system", from which it is difficult to break away [5]. In the light of this, the construction sector is often seen not primarily as a developer of technologies, but rather in the role of a user of technologies that have originated in innovative companies in the supply chain. According to [10], the industry structure represents an additional challenge to achieving innovation success in the construction sector. This structure is characterized by SMEs that do "not (often) have the resources and the knowledge to develop innovations on their own" [11], that "(often) have an organizational slack in comparison with large construction firms that invest in strategies with long-term return" [12], and that "usually invest in their own R&D only to a small degree" [13]. The steel industry in Germany is a key supplier of materials and products for the construction sector [14] and therefore actively tries to generate new innovations for steel applications in the construction industry within the scope of application-oriented pre-competitive CR with the construction sector. The resulting innovation process can be divided into four phases – early, administrative, project and late – as shown in Fig. 1.

These are in turn subdivided into further phases that essentially depict the individual steps of the innovation process in CR. This involves, in particular, the search for suitable research ideas of interest to the steel

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Fig. 1. Phase model in cooperative research (CR) on the basis of [4]

and construction industries (Initiative) and the subsequent further development of these ideas until a research proposal is ready for submission, including the finding of suitable industrial and academic project partners (Develop). This is followed by the organizing of funding possibilities from industry and the public sector (Finance), early agreement on the use of the potential results (Own) and the actual research process undertaken by research institutes (Research). The subsequent initial application of the research results achieved in cooperation with industrial application partners (Use) and the possible transfer to and use of the research results obtained in other possible industrial steel application sectors (Transfer) conclude the process. Currently, CR in practice sometimes means passing through the individual stages completely, but often only partially. In addition, in order to surmount possible gates between the phases, there are various players, such as universities, research funders or partners from industry, who often operate independently of each other. This makes the whole CR innovation process a complex and sometimes extremely fragmented system. As a logical consequence, "deficiencies" often arise in the course of the process. In the light of this, *Sexton* et al. [11] note, for instance, on relevant CR projects that "unsuccessful innovation often happens because of a lack of market focus (in the project)". *Barnes* et al. [15] are critical of the fact that "the objectives (of a project) were often not specific enough with respect to the expected outcomes and were largely open to interpretation in terms"; they also stress that "there have been inevitable differences in the requirements and expectations of the (project) partners …" For the project phase, *Abbott* et al. [16] comment that "it was felt that at times universities and industry talked different languages". *Couchman* et al. [17] add that "universities are (often) not very effective in commercializing new ideas" and that "the knowledge universities produce is not (often) in a form that can be directly applied or commercially exploited." *Sexton* et al. [11] argue that "current technology transfer mechanisms are not sufficiently informed by, or engaged with, company strategic direction and organizational capabilities". FOSTA would like to counteract these possible deficiencies through the central use of an IB role while seeking to improve the innovation process. This approach is described in greater detail below.

3 Approach to optimizing the management of CR between the steel and construction industries

The steel industry in Germany, as already mentioned at the start, launched an interdisciplinary FOSTA research cluster entitled "Sustainability of steel in the construction sector (NASTA)" in early 2011 as part of its CR with the construction industry. This research cluster, comprising six subprojects in total, is engaged in developing indicators for the quantitative assessment of sustainability in steel construction. There are, in total, 30 research institutes from 11 different disciplines and 200 industry experts involved in the NASTA project [18]. The central role of an innovation broker (IB) was established within the project as part of an integrated approach to innovation management in CR. The aim of the IB is to take account of the overriding requirements in the management of interdisciplinary tasks set but also, in particular, to ensure that the special features of the construction sector in the innovation process are considered while helping to prevent the deficiencies in CR outlined earlier. Publications on innovation management contain various, mostly company-related, key person concepts that can be used to create a possible task profile for the IB. The champion, the promoter and the gatekeeper models can be called the chief markers pointing the way. The respective characteristics of these models are summarized in Table 1.

So far, the concepts cited in Table 1 have been given only scant consideration in the majority of the proposals regarding the establishment of an IB role in construction industry CR. The identification of an IB is often proposed only in the project team, i.e. at the research stage, or, for example, as a key administrative position in the management of a research institution, without any bearing on a specific project or subject. The tasks are then described only in general form without specific reference to the whole innovation process (see, for example, [16, 33]). **The question therefore arises as to how the IB role can best be integrated for CR in the innovation process and how an IB's task profile can be purposefully structured to meet CR requirements. In this context, the questions for the CR under examination here also remain: What interests are the participants from science and industry pursuing through their participation in the project? And: How can these interests be considered appropriately in the project with the IB's help?** With this in mind, so-called industry working groups (IWGs) attached to the respective subprojects have been established in the NASTA project in addition to the IB role, their aim being to provide the innovation project with specialist support. These IWGs do not consist of a limited number of selected companies, as is often the case, but rather are open to innovation and are accessible to all interested industry representatives without restrictions.

4 Survey of project participants: formulation of questions, structure of evaluation, approach to interpretation

In order to answer the questions posed earlier regarding the structur-

Table 1. Summary of varying key person concepts in the innovation process in line with [31] and [32]

The (product) champion model [19], [20], [21], [22]
– One-person concept: an especially dedicated person is driven by their own traits and convictions to push through an innovation in a company against all odds.
– The champion possesses technical expertise, detailed company and market knowledge, drive and political skill.
– That person takes charge of the innovation at the conception stage, frees the innovators from the rules of the organization, convinces others to support the innovation, provides the necessary resources, protects the innovator against disruption and pushes the innovation forward.
– There is no more precise description of the champion's role.
– Its impact on success has been repeatedly demonstrated but the focus on one person can also have negative effects very quickly.
The promoter model [23], [24], [25], [26], [27]
– Departure from the one-person focus.
– Theory-based and empirically tested concept.
– For the bridging of barriers to participation within an organization, it includes a "power promoter" for those not wanting to take part and a "know-how promoter" for those lacking the necessary specialist knowledge.
– The know-how promoter possesses expert knowledge and is often the nucleus of the innovation.
– The power promoter has high hierarchical potential and ensures the resources required and the necessary decision-making for the project's preservation.
– The process promoter, usually from middle management, interlinks both roles, accompanies the innovation process, motivates and coordinates within the process.
– A relationship promoter is added in order to bridge the inter-company barrier of a lack of knowledge among external partners.
– This promoter has a network of personal contacts, both inside and outside the company, and brings together people from various institutions.
– As yet, there has been little research into the motives governing the actions of promoters.
The gatekeeper model [28], [29], [30]
– Project-independent person model whose main task is to pass on and obtain specialist knowledge that transcends organizational boundaries.
– The gatekeeper possesses extensive and up-to-date expertise and is also a networker and communicator.
– This person performs a prominent role in interface management and is the person whose specialist advice is in demand.
– Whereas the relationship promoter tends to be more active in the exploitation of innovations, the gatekeeper imports knowledge for an innovation process.

ing and integration of the IB role in CR and regarding the management of the varying interests in the innovation process within IWGs, a survey of the heads of research institutes (n = 21) and various industry representatives (n = 50) from the IWGs was conducted as part of the NASTA project. This was carried out within the first IWG meetings held in late 2010 and early 2011 once the early and administrative innovation phases of CR had been completed and the project volume amounting to € 6 million had been approved. The survey was conducted with standardized questionnaires based on the following approaches:

1. The **research institutes** were questioned about their expectations of the forthcoming IWGs. They were also asked for a retrospective assessment of the possible IB tasks for the early and administrative innovation phases of CR already completed.
2. The **industry representatives** were asked to give the reasons that resulted in their support for the respective NASTA subproject through their participation in the IWG.
3. **Both groups** were asked about their expectations of the IB's role in the forthcoming project phase and in the late innovation phases of CR between science and industry.

For the replies to approaches 1 and 2, a four-point *Likert* scale was selected with which the possible expectations of the research institutes in relation to the IWGs as listed in the questionnaire, the assessment of the possible IB tasks in the early and administrative innovation phases and the possible reasons for industry's participation in the IWGs could be assessed. The scale provides for an assessment of (1) for "unimportant" to (4) for "very important". For the third approach, which was directed at both groups in the same form, a four-point *Likert* scale was again used, enabling the science and industry representatives polled to assess the possible tasks of the IB role in the project phase and the late innovation phases. The scale provides for a range of responses from (1) for "undesirable" to (4) for "very desirable". If the respondents did not wish to reply to certain questions, they were asked to leave the scale empty. In selecting the respondents, care was taken to ensure that the disciplines included in the NASTA project should also be represented in the survey in accordance with their project participation. Among the research institutes, this included the fields of structural steel engineering/lightweight steel construction/composite construction (43 %), architecture/building services/structural design (24 %), life cycle management (19 %) and construction management/business administration/work organization (14 %). Among the industry representatives, experts were questioned from engineering firms for construction planning and construction project management (30 %), from steel production (20 %), from steel construction (20 %) and from complementary fields (30 %), e.g. manufacturers of building products and construction software, industrial associations. As part of the statistical analysis, the following evaluation steps were performed using the PASW Statistics 18 program for the replies from the research institutes and those from industry separately.

Step 1:
For the purpose of creating a ranking within the individual reply options for a question, the **mean** of the respective assessments was determined at the start. In order to improve the meaningfulness of the mean values, the related **95 % confidence intervals (CI)** were also established. Moreover, a confidence belt with corresponding upper and lower limits was determined for the mean value with a defined probability depending on sample size and respective standard deviations.

Step 2:
As a supplementary aspect for illustrating priorities within the reply options, **the percentages of the persons questioned** who favoured **a specific aspect of the IWG and at the same time a specific IB task** were also ascertained. To this end, positions (1) and (2) of the respective four-point *Likert* scales were summarized within data transformation and interpreted with a somewhat negative slant. In the same way, positions (3) and (4) of the respective *Likert* scales were grouped together and interpreted with a somewhat positive slant. Generating cross-tables from these data between the reply options for IWG assessment and IB tasks means it is possible to identify aspects that have been simultaneously assessed as relatively favourable. A summarized view of the respondents to both questions (IWG and IB) is thus produced at the same time.

Step 3:
In order to identify possible **trend-forming connections** between individual expectations of the IWG and individual IB tasks, *Kendall*'s Tau-b was then determined as the **rank correlation coefficient** for all possible combinations of the aspects prioritized previously. In the process, the ranking of the existing replies is compared in each case with an expectation of the IWG and an IB task and displayed in the form of a correlation coefficient. However, the chief informative value of the correlation determined is only expressed through the significance level ascertained at the same time. Therefore, for the purpose of this analysis, only those correlations with a trend-forming connection defined as significant (95 %) or highly significant (99 %) were interpreted.

5 Statistical analysis

5.1 Expectations of heads of research institutes regarding the IWG accompanying the project and assessment of the corresponding IB tasks

Within the survey of the heads of research institute, the four-point *Likert* scale already described was used at the start to enquire about the various expectations of an IWG. The left-hand column of Table 2 shows results in descending order on the basis of the mean values determined empirically. The upper and lower limits of the related 95 % confidence intervals (CI) are also shown. Based on the same system of classification, the right-hand column of Table 2 shows the results of the assessment of possible IB tasks by the heads of research institutes, again with a four-point *Likert* scale.

The mean values for what the heads of research institutes expect from IWGs range from 2.7 to 3.2, with the related 95 % CIs varying from 2.2 to 3.6. The various aspects are assessed favourably in general, although it is noticeable that from the 8th aspect onwards, the mean for IWG expectations falls short of 3.0 and the lower limit of the 95 % CI drops to 2.4 between the 7th and 8th aspects. The individual assessments of the subsequent aspects 9 and 10 decline correspondingly further and an increase in the corresponding CIs can be seen.

In the assessment of possible IB tasks by the research institute heads questioned, as shown in the right-hand column of Table 2, the mean values range from 2.8 to 3.8, with the corresponding 95 % CIs varying from 2.3 to 4.0. Here, too, there is a generally favourable assessment of possible IB tasks, although an increase in the 95 % CI owing to the drop in the lower limit from 2.9 to 2.7 from IB task 10 onwards is striking. For the first time, the mean value of 3.0 is not reached from IB task 13 onwards.

Since the aim is to develop an IB task profile from the survey on the basis of the expectations of the various participants in the IWG accompanying the project, those IWG aspects and IB tasks that were given a simultaneous assessment tending towards the favourable by the respondents were identified in the second evaluation stage. For the research institute heads, the necessary data transformation and cross-table calculation produced a simultaneously favourable assessment of one of the first six IWG aspects and of the whole of the first 10 IB tasks by 70 to 90 % of the respondents. This means that well over two-thirds of those questioned approved of the first six IWG aspects and the first 10 IB tasks at the same time. For the 7th IWG expectation, a remaining simultaneously favourable assessment could be established at this level (for exactly 76 % of respondents) with the first four IB tasks. A simultaneously favourable assessment in the range of 23 to 66 % of respondents was demonstrated for all other IWG aspects and IB tasks. For further analysis, in the light of the broad simultaneous approval by the research institute heads, the first seven IWG expectations are interpreted as important and the first 10 IB tasks as desirable. This interpretation is also confirmed by the mean values and 95 % CIs given above.

The aim of the third stage of the evaluation is to determine the rank correlations verifiable by calculation between the aspects interpreted as important and desirable and to use this to generate trend-forming assertions regarding a possible connection between key IWG expectations and desirable IB tasks. Table 3, which indicates the respective rank correlation coefficient and the related significance level, summarizes the aspects identified. With the coefficients ascertained, at between 0.30 and 0.49, a moderately strong positive connection can be assumed between the rankings from the *Likert* scales used to assess IWG expectations and IB tasks. The coefficients between 0.50 and 0.69 indicate a strong positive connection between the aspects cited in each case.

In order to complete the whole picture in a further analysis, a brief overview of the IB tasks for the early and administrative innovation phases, as already described in [3], i.e. for the period before the project starts, will now be given: the retrospective assessment of possible IB tasks in the early and administrative innovation

Table 2. How the heads of research institutes view the IWG and the IB role

	Research institutes heads' expectations of the industry working groups (IWG) (Lower limit of the 95 % CI | Mean | Upper limit of the 95 % CI)
1.)	IWGs offer the opportunity to give the research project a **practical orientation** (3.0 | 3.2 | 3.6)
2.)	IWGs offer us and the representatives from industry the opportunity for **joint know-how development** on the research topic (2.9 | 3.2 | 3.5)
3.)	IWGs represent **milestones for the scientific staff over the project period** at which new interim results have to be presented (2.9 | 3.2 | 3.5)
4.)	IWGs offer the opportunity to **gain access to topic-related information** which has not yet been considered (2.9 | 3.2 | 3.5)
5.)	IWGs are important to us in **initiating new projects** with industry involvement (2.9 | 3.1 | 3.5)
6.)	IWGs enable a **practice-oriented evaluation** of the relevant research progress (2.9 | 3.1 | 3.4)
7.)	IWGs offer the opportunity to **present the activities of our research institute** to industry (2.7 | 3.0 | 3.4)
8.)	IWGs are important to **scientific staff for collecting contacts** in industry (2.4 | 2.9 | 3.3)
9.)	IWGs lead during the project period to the **creation of new subtasks** which help to prepare for industrial application (2.5 | 2.8 | 3.1)
10.)	IWGs are important in **initiating consulting services** through us for companies (2.2 | 2.7 | 3.0)

	Research institutes heads' views of the tasks of the innovation broker (IB) in the project (Lower limit of 95 % CI | Mean | Upper limit of 95 % CI)
1.)	Publishing the **final report** (3.6 | 3.8 | 4.0)
2.)	**Organizing events** with the relevant target audience (3.5 | 3.7 | 3.9)
3.)	**Interlinking parallel projects** on similar topics (3.2 | 3.5 | 3.8)
4.)	**Linking up experts** within the framework of the project (3.2 | 3.5 | 3.8)
5.)	Finding **partners for the initial application** of the results (3.2 | 3.5 | 3.8)
6.)	**Discussing interim results** with all participants (3.0 | 3.4 | 3.7)
7.)	Guiding the IWG to an **optimum working model** (3.0 | 3.3 | 3.6)
8.)	Communicating **industrial trends** in the project (2.9 | 3.2 | 3.5)
9.)	Adding **further industry partners** over the course of the project (2.9 | 3.2 | 3.4)
10.)	**Preventing disruptive factors** in the project (2.7 | 3.0 | 3.4)
11.)	Organizing and allocating the confirmed **project funding** among the partners (2.6 | 3.0 | 3.4)
12.)	Procuring **test material** for the project (2.5 | 3.0 | 3.5)
13.)	Monitoring **project tasks** (2.5 | 2.9 | 3.3)
14.)	Maintaining and increasing **motivation in the project** (2.4 | 2.8 | 3.2)
15.)	**Mediating** in the event of conflict (2.4 | 2.8 | 3.2)
16.)	Supporting the **formulation of results** at the end of the project (2.4 | 2.8 | 3.2)
17.)	Including **marketing organizations** linked to the topic (2.3 | 2.8 | 3.2)

Table 3. Rank correlation: what research institutes expect of IWG and IB tasks

Research institutes' expectations of industrial working groups (IWG)	Rank correlation coefficient	Tasks of the innovation broker (IB)
IWGs provide an opportunity to give the research project a **practical orientation**	**0.447*** **0.414***	Guiding the IWG to an **optimum working model** Communicating **industrial trends** in the project
IWGs provide us and the representatives from industry with an opportunity for **joint know-how development** on the research topic	**0.631**** **0.513***	**Linking up experts** on the topic in the project **Interlinking parallel projects** on similar topics
IWGs enable a **practice-oriented evaluation** of the relevant research progress	**0.405***	Guiding the IWG to an **optimum working model**
IWGs provide an opportunity to **present the activities of our research institute** to industry	**0.489***	**Linking up experts** on the topic in the project

*** significant at the 0.05 level | ** significant at the 0.01 level**

phases by the heads of research institutes produces mean values of 3.2 to 3.9 from the survey. The 95 % CI range of mean values varies from 2.9 to 4.0. Overall, the research institute heads therefore assess the use of an IB as desirable, even for the early and administrative innovation phases of CR. In particular, the early brokering of industry contacts (3.0 | 3.3 | 3.6), the central acceptance and collection of outlines for project ideas (2.9 | 3.2 | 3.5) and the organization of pre-evaluation discussions with industry (3.2 | 3.4 | 3.6) are especially highlighted as IB tasks. In the administrative phases, the IB should, in the view of the research institute heads, in particular initiate and, if possible, also conduct communication with prospective funding providers (3.7 | 3.9 | 4.0). Should conflicts of interest arise during the early and administrative innovation phases, the IB should, in the view of the research institute heads, continue to act as a mediator between the parties involved (2.9 | 3.2 | 3.5).

5.2 Industry's expectations of the IWG and the related IB tasks

In the survey of industry representatives, using the four-point *Likert* scale already described, the respondents were asked at the start about the importance of the various reasons for their participation in an IWG. The results are shown in the left-hand column of Table 4 in descending order based on mean values determined empirically. In addition, as with the research institutes, the upper and lower limits of the related 95 % confidence intervals (CI) are indicated. The right-hand side of Table 4 shows, on the basis of the same system of classification, the results of the assessment of possible IB tasks by the industry representatives, also with a four-point *Likert* scale.

The mean values for industry's expectations of the IWGs range from 1.8 to 3.6, with the related 95 % CIs varying from 1.6 to 3.7. Except for the last reason for participation listed, there is a generally favourable assessment of the various aspects, although it is noticeable that from the 9th aspect onwards, the IWG expectations fall short of the mean of 3.0, with the lower limit of the 95 % CI declining from 2.9 to 2.6. The individual assessments for aspects 10 to 12 fall correspondingly further.

The mean values for the assessment of possible IB tasks by the industry representatives questioned range from 2.6 to 3.6, as shown in the right-hand column of Table 4; the corresponding 95 % CIs vary from 2.3 to 3.8. Thus, here, too, there is a generally favourable assessment of the possible IB tasks, although an increase in the 95 % CI from task 10 onwards is apparent owing to the drop in the lower limit from 2.9 to 2.7. The mean of 3.0 is also not reached for the first time from task 10 onwards, albeit remaining constant at 2.9 up to task 14, with the lower limit of the 95 % CI dropping to 2.7 and 2.6 respectively.

In the second evaluation stage, as before, those IWG aspects and IB tasks that had been given a simultaneous assessment tending towards the favourable by the persons polled were identified. Within the results obtained, three areas can be distinguished: (I) 70 % to 95 % of industry representatives gave a simultaneous assessment tending towards favourable for both one of the first eight IWG as-

Table 4. Industry's view of the IWG and the IB role

	Industry's reasons for taking part in an industry working group (IWG) (Lower limit of 95 % CI | Mean | Upper limit of 95 % CI)
1.)	Sharing and increasing **technical knowledge** (3.4 | 3.6 | 3.7)
2.)	Detecting **early trends in technical development** (3.2 | 3.4 | 3.6)
3.)	Making **contacts** with other project participants (3.1 | 3.2 | 3.4)
4.)	Testing **new findings for possible implementation** within their own company (3.0 | 3.2 | 3.5)
5.)	Obtaining **information about the project** as a whole on an ongoing basis (3.0 | 3.2 | 3.5)
6.)	Obtaining **information** on certain **individual aspects** of the tasks set (3.0 | 3.2 | 3.5)
7.)	Intensifying **innovation activities** within their own company (3.0 | 3.2 | 3.4)
8.)	Exerting an **influence on project** progression (2.9 | 3.1 | 3.2)
9.)	Supporting **R&D activities in steel applications** generally (2.6 | 2.9 | 3.1)
10.)	Helping to shape findings that flow into **standardization** (2.6 | 2.8 | 3.1)
11.)	Encouraging **new research projects** on steel application (2.5 | 2.7 | 3.0)
12.)	Obtaining information about **competitors** (1.6 | 1.8 | 2.0)

	Industry‘s view of the role of the innovation broker (IB) in the project (Lower limit of 95 % CI | Mean | Upper limit of 95 % CI)
1.)	Publishing the **final report** (3.4 | 3.6 | 3.8)
2.)	**Discussing interim results** with all participants (3.2 | 3.4 | 3.6)
3.)	**Organizing events** with the relevant target audience (3.1 | 3.3 | 3.5)
4.)	**Linking up experts** within the framework of the project (3.1 | 3.3 | 3.5)
5.)	Supporting the **formulation of results** at the end of the project (3.0 | 3.2 | 3.4)
6.)	Communicating **industrial trends** in the project (3.0 | 3.2 | 3.4)
7.)	**Interlinking parallel projects** on similar topics (3.0 | 3.2 | 3.3)
8.)	Finding **partners for the initial application** of the results (3.0 | 3.2 | 3.4)
9.)	**Preventing disruptive factors** in the project (2.9 | 3.1 | 3.3)
10.)	Maintaining and increasing **motivation in the project** (2.7 | 2.9 | 3.1)
11.)	Monitoring **project tasks** (2.7 | 2.9 | 3.1)
12.)	Guiding the **IWG** to an **optimum working model** (2.7 | 2.9 | 3.1)
13.)	Adding **further industry partners** over the course to the project (2.6 | 2.9 | 3.1)
14.)	Organizing and allocating the confirmed **project funding among the partners** (2.6 | 2.9 | 3.1)
15.)	**Mediating** in the event of conflict (2.5 | 2.8 | 3.0)
16.)	Procuring **test material** for the project (2.5 | 2.7 | 3.0)
17.)	Including **marketing organizations** linked to the topic (2.3 | 2.6 | 2.9)

pects and all of the first six IB tasks; (II) 70 % to 85 % of those polled gave a simultaneous assessment tending towards favourable for both one of the first six IWG aspects and IB tasks 7 to 9; (III) 70 % to 75 % of industry representatives gave a simultaneous assessment tending towards favourable for both one of the first two IWG aspects and IB tasks 10 to 14. This means that, overall, well over two-thirds of those surveyed who favoured the first eight IWG aspects at the same time expressed their approval of the first 14 IB tasks with the grading described. A simultaneously favourable assessment in the range of 12 to 66 % of those questioned was established for all other IWG aspects and IB tasks. For further analysis, in the light of the broad simultaneous approval among industry representatives, the first eight IWG expectations are interpreted as important and the first 14 IB tasks as desirable. Here, too, this interpretation can also be derived from the mean values and 95 % CIs described above.

The third stage of evaluation also aims to ascertain the rank correlations verifiable by calculation between the aspects interpreted as important and the aspects interpreted as desirable, and from this to generate trend-forming assertions regarding a possible connection. Table 5 summarizes the aspects identified, showing the respective rank correlation coefficient and the related significance level.

Here, too, based on the coefficients ascertained, which lie between 0.30 and 0.49, a moderately strong positive connection between the rankings can be assumed. A direct comparison with the coefficients in Table 3 is not possible, however, because the sample size in the survey of industry representatives is higher than that of the research institute heads.

6 Interpretation: project expectations, IB task profile, recommendations for CR

6.1 Varying project expectations among the groups surveyed

The **research institutes' expectations** of an **IWG** which are **interpreted as important** can be represented by means of three different directions of interest for the project phase in CR. Firstly, this includes, in particular, project-related aspects such as the IWG's role in ensuring that the project has a practical orientation, the practice-oriented evaluation of the respective research project's progress in the IWG and the possible access to topic-related additional information disregarded in the project to date. Additionally, the pursuit of personal interests by the respondents can be identified, which includes the joint development of new knowledge related to the research topic with the IWG members. Moreover, interests can be recognized from the IWG expectations which relate to the research establishment as an institution. This includes, above all, the benefit of the IWG meetings as milestones and useful experience for research staff, the scope for generating new projects through cooperation with the IWG and the opportunity to present the research establishment's project activities and range of work to the IWG members (from industry).

The **reasons for industry's participation** interpreted as important in an **IWG** accompanying a CR project can also be summarized based on the three different directions of interest described above. Whereas a relatively uniform weighting within the IWG aspects interpreted as important is discernible among the research institutes, the industry representatives display a clear prioritization of personal interests in their IWG participation in the form of sharing and increasing their own technical knowledge of the research topic. Following close behind, company-related interests are important to the respondents. This includes, in particular, detecting new technical trends, keeping in contact with other project participants also interested in the topic, monitoring the results for possible application in their company and generally intensifying their company's innovation activities. In third place among the IWG aspects deemed important by the industry representatives is their project-related interests. This includes, in particular, receiving general project information on an ongoing basis, but also being able to pursue very specific aspects of the research topic in the IWG. Additionally, the purpose of exerting influence on project progression through IWG participation, identified as important, should be mentioned in this context. Industry representatives regard obtaining information about potential competitors within the IWGs as somewhat unimportant. Fig. 2 provides another summary of the varying interests of science and industry with regard to their IWG participation.

6.2 Developing the IB task profile for steel application research in the construction industry

On the basis of the IB tasks interpreted as desirable, the following IB task profile for pre-competitive CR on steel applications in the construction sector can be drawn up based on the various key person roles outlined at the start:

Table 5. Rank correlation: industry's reasons for taking part in the IWG and IB tasks

Industry's reasons for taking part in an industrial working group (IWG)	Rank correlation coefficient	Tasks of the innovation broker (IB)
Sharing and increasing **technical knowledge**	**0.305***	Organizing **events** with the relevant target audience
Detecting **early trends** in technical development	**0.334***	**Interlinking parallel projects** on similar topics
Making **contacts** with other project participants	**0.398**** **0.327***	**Interlinking parallel projects** on similar topics **Linking up experts** on the topic in the project
Obtaining information about the project as a whole on an ongoing basis	**0.316*** **0.314***	**Interlinking parallel projects** on similar topics **Discussing interim results** with all participants
Exerting an influence on **project progression**	**0.312***	Monitoring the **project tasks**

*** significant at the 0.05 level | ** significant at the 0.01 level**

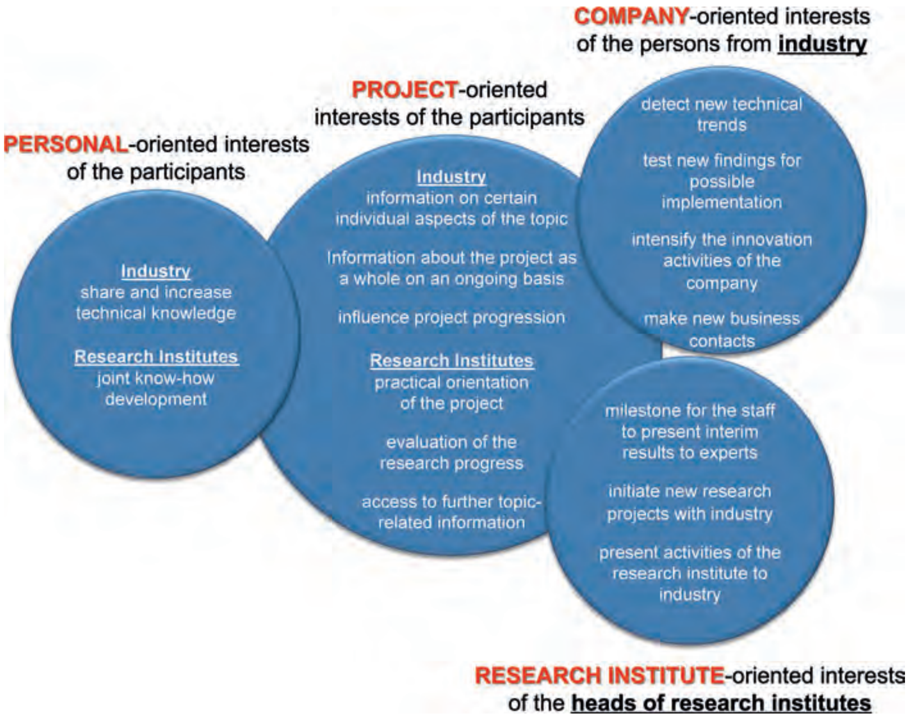

Fig. 2. Varying interests of science and industry in an IWG accompanying a CR project

In the **early innovation phases**, the IB's tasks consist chiefly of active ideas management in which the IB initiates or accepts and collects idea outlines, taking them to the point of industrial pre-evaluation of an initial proposal. The IB also tries very early on, within the scope of network management, to arrange industry contacts to support the idea and promotes the forging of collaboration between the research institutes and the relevant industry representatives. At the very start of the innovation process, the IB is thus performing the groundwork aimed at preventing the possible weakness – outlined at the start – of a lack of practical orientation in the project. This activity, not yet completely project-related, can be likened to that of a **(product) champion role** at the conception stage, which is characteristic of early support for an innovation and through which an idea is promoted based on conviction and with the political skill often required. As part of early ideas management, the IB also tries to initiate and maintain communication between the parties from science and industry, thus helping to define clearly the project goals of both players. Possibly varying expectations of a project criticized at the outset are thus harmonized early on and a clear starting point for the desired project phase comes about. All in all, the activities of the IB during the early innovation phases can be summed up as those of a **coordinating and contact point for innovations**. *Haller* [34] appropriately describes the challenge of such a position in "concentrating on dealings with ideas generators and in supporting their individual needs related to the finding, formulation and implementation of new innovative ideas". In return, the IB confirms a lasting increase in the quality of new research initiatives.

The IB engages in the subsequent arranging of talks with possible funding providers as part of active funding management, which based on the results can be interpreted as the most important aspect of the **administrative innovation phases** from the research institutes' perspective. For this activity, the IB must recognize the process steps required in the administrative phases of a pre-competitive CR project and play a supportive role, taking account of the often differing requirements of the various funding providers in the process of seeking to attract funds. The IB's tasks in this instance can be likened to that of a **process promoter**, who in this situation is acting as a translator between the linguistic worlds of technology and economics. On the basis of possible experience from previous CR projects, the IB can also use the knowledge of specific funding providers across projects, which will give the actual project-related promoter role impetus over time.

In the **project phase**, the IB's activities are related to the project level on the one hand but also require cross-project action on the other. On the basis of the research institutes' view, the IB's tasks can be assigned to five different management areas. For the project phase, as indeed already described for the early innovation phases, this means that the IB should engage in active network management for the project. The research institutes welcome support with finding and linking-up suitable experts from the relevant specialist fields, who are in turn willing to supervise a specific project. In addition, the IB should in this context assist in identifying companies that may be interested in implementing the research results and should promote the recruitment of additional participants for the IWG during the course of the project. Another element in the IB's tasks during the project phase is topic-related information management, which helps the research institutes to obtain information about parallel projects on similar topics and find out about industry trends in the course of the project. Also important to the research institutes is support for discussing interim results, which can be achieved through active knowledge management by the IB. Correspondingly adapted team management, which helps the IB to guide the project group, including the IWG, to an optimum working model and in which disruptive factors in the group are prevented early on or jointly resolved, are also regarded as desirable by the research institutes.

There is a degree of consensus between the industry representatives' views and the research institutes' assessments in relation to the project phase. This concerns the IB tasks of network management and information management. In addition, the industry representatives regard it as desirable for the IB – as part of the IB's knowledge management activities and apart from supporting discussions on interim results – to assist with the formulation of the research results at the end of the project as well. With the IB's team management activities, the industry representatives also see the need for the IB, in addition to steering the IWG and preventing disruptive factors within it, to play an active role in maintaining and increasing motivation among the project partners during the project. As an additional element in the IB's tasks, the industry representatives welcome the assumption of responsibility for project management aspects, such as the monitoring of project tasks already agreed among the participants and the performance-related payment of funds approved at the start of the project over the project term.

Based on the IB tasks determined from the areas of knowledge management and information management, some of the IB's activities for the project phase can be summarized as those of a **know-how promoter**. Team management and project management tasks, which are among the tasks performed by a **process promoter**, were also identified as desirable for the project phase. The additional, desired use of active network management by the IB links the IB role to that of a **relationship promoter**.

Summing up, it can be seen that the IB, even during the project phase, in addition to supporting the project's practical orientation, makes a substantial contribution to the know-how sought by science and industry and to the exchange of information among the project participants. The IB thus makes a significant contribution to eliminating the disadvantages caused by the "different languages" spoken by science and industry, as criticized at the outset, and to clarifying and harmonizing the differing expectations of a CR project. The IB's role in the project phase can thus be interpreted as that of an **intermediate person** between research institutes and industry project partners. This meets the requirements of the role required by *Abbott* et al. [16] for the construction sector of a person who manages interaction at the university-business interface. At the same time, the IB's activities in the IWG support the effectiveness when transferring results. *Blayse* et al. [33] describe how important it is for the achievement of a successful transfer in the construction sector that "the industry partners understand the R&D results and are then actually put in a position to use these results (in day-to-day construction business)".

For the **late innovation phases**, the central publication and dissemination of the final report, as well as the holding of events on the research topic, were stressed as desirable IB tasks by both the research institutes and the industry representatives. As part of active dissemination management, the IB thus supports the research institutes in disseminating their research results and creates – beyond the duration of the project – additional access to the expertise created. For this purpose, the IB should be able to demonstrate specialized knowledge of the results and the IB's role in the late innovation phases could therefore be likened to that of a **know-how promoter**. The additional tasks mentioned regarding the organization of a target group-oriented event and finding suitable partners for the initial application of the research findings can be assigned to the role of a **relationship promoter**.

If the CR phase model shown in Fig. 1 is examined against the background of the IB task profile described, the varying roles of the IB in a comprehensive examination of the innovation process reveal that the IB not only operates at the micro-level, i.e. in the phases themselves, but also frequently ensures the innovation-related flow of information beyond the phase boundaries at the macro-level, thus assuming a prominent role in interface management between the individual innovation phases and the different CR participants. His activities are therefore also similar **across all phases** to that of a **gatekeeper** who is endeavouring to push an innovation process forward from the idea to the application of the findings and to supplement this process with additional topic-related information.

Fig. 3 summarizes the IB tasks identified as desirable by science and industry for CR on steel applications in the construction industry. In addition, the IB's changing roles through the individual innovation phases are shown.

The comprehensive IB task profile depicted shows an initial approach to the implementation of the change in the results transfer in applied research in the construction sector, as demanded by *Abbott* et al. [16]. They describe this as follows: "there is a need to move away from the old fashioned notion of the university developing new ideas through research, and then transferring and disseminating these ideas to (construction) industry in a linear fashion".

6.3 General trends in CR management

The IWG expectations assessed as important (see Fig. 2) indicate the varying interests of the project participants from science and industry. In the same way, the IB tasks assessed as desirable by the respondents (see Fig. 3) form the basis for creating an area of activity for the innovation broker in CR on steel applications in the construction industry. In addition to this opinion, on the basis of the established (highly) **significant correlations** between the IWG expectations assessed as important and the IB tasks assessed as desirable in accordance with Tables 3 and 5, the following trend-forming assertions can be made about CR management:

Based on the results of the survey of heads of research institutes, one can assert the following trend that those research institutes that …

… regard a practice-oriented approach to the project by the IWG as important, to a significant degree also regard it as desirable that the IB should guide the IWG to an optimum working model in the project and communicate trends from the industry involved in the project.

… regard the joint development of know-how with industry in the IWG as important, to a highly significant degree also regard it as desirable that the IB should link up experts on the research topic and (to a significant degree) interlink parallel projects on similar topics.

… regard a practice-oriented evaluation of the interim results within the IWG as important, to a significant degree also regard it as desirable that the IB should assess the guiding of the IWG to an optimum working model.

… regard the possibility of presenting their activities in the IWG as important, to a significant degree also

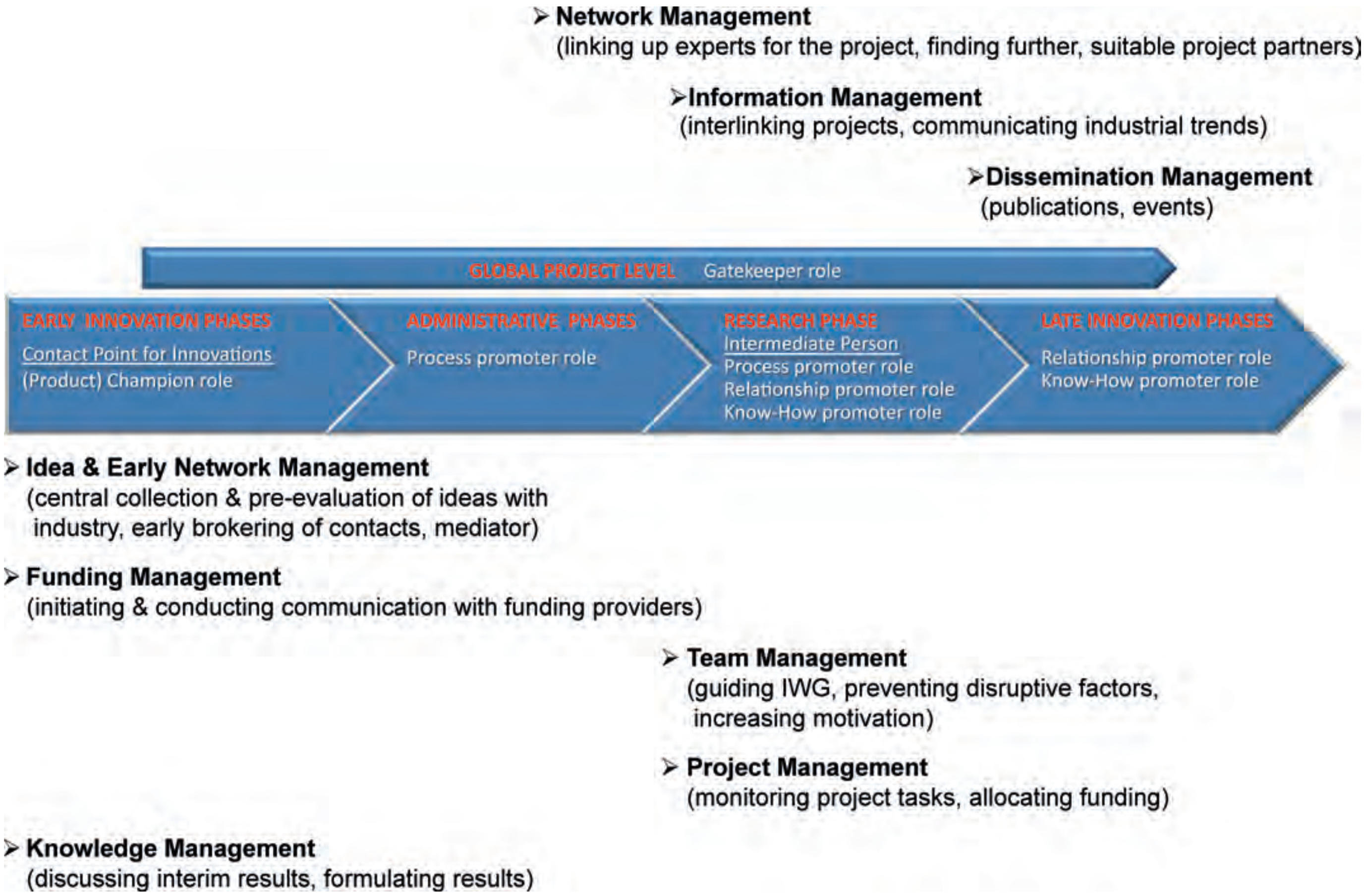

Fig. 3. The tasks and changing roles of the IB in the CR innovation process

regard it as desirable that the IB should assess a link-up of experts on the research topic.

Based on the results of the industry survey, one can assert the following trend that those industry representatives who …

… regard the exchange of technical knowledge in the IWG as important, to a significant degree also regard it as desirable that the IB should assess the holding of an event during or after the project term with the right target audience.

… regard the detection of technical trends through participation in the IWG as important, to a significant degree also regard it as desirable that the IB should assess an interlinking of parallel projects on similar topics.

… regard the contacts established with other project participants within the IWG as important, to a highly significant degree also regard it as desirable that the IB should assess a link-up of parallel projects on similar topics and (to a significant degree) a link-up of experts in the project topic.

… regard the continuous provision of information on the research project in the IWG as important, to a significant degree also regard it as desirable that the IB should assess the interlinking of parallel research projects on similar topics and supporting the discussion of interim results in the IWG.

… regard the possibility of exerting influence on the project through their participation in the IWG as important, to a significant degree also regard it as desirable that the IB should assess the allocation and monitoring of tasks in the IWG.

7 Conclusions

Innovation requires not only the generation of an idea, but also its successful implementation [12]. This must also apply, in particular, to pre-competitive CR between the steel industry and the construction sector, the results of which will benefit a large number of companies. In order to achieve this goal, it is necessary to overcome the deficiencies in the existing CR system mentioned at the start and also to take into account the special features of the construction sector.

In the light of this, the steel industry in Germany, as a key supplier of materials and innovations for the construction sector, is seeking a comprehensive optimization of CR management for steel applications in the construction industry. This article suggests an approach that consists largely of using a central IB role within the whole CR innovation process.

The **various project expectations** of the two groups were identified through a survey of representatives from the scientific community and industry in a FOSTA research cluster which accompanied the project. In summary, these expectations are driven by personal, organization-related and project-related interests.

On the basis of the survey results, a **task profile** has been developed **for the IB** which relates to the whole CR innovation process – from finding ideas to implementing the results obtained. The IB activities interpreted as desirable come from the areas of idea management, funding management, network management, information management, team management, project management, knowledge management and dissemination management, and are distributed across the whole process at both project and cross-project level.

The IB task profile created allows an assignment to familiar **key person roles** according to the content of the individual subtasks. It has emerged, however, that the IB for CR on steel applications in the construction industry cannot be assigned fully to any of the existing key person roles. Rather, the IB's area of activity represents a mixture of existing and new elements. Instead of a rather static assignment of a role to a field of activity, in this case a dynamic crossover is discernible for CR between the varying roles of the different key person concepts during the course of the innovation process. On the basis of this dynamic component and the associated increase in the interface problems between the individual elements of the IB task profile, departing from the one-person concept for the key person in CR on steel applications in the construction industry is not expedient.

Finally, trend-forming assertions about CR management were drawn up in this study, which arose based on **significant correlations** between IWG expectations and IB tasks. These show the IB's firm integration into CR and the acceptance of the IB's activities throughout the innovation process.

As part of its further efforts to optimize the management of the steel industry's pre-competitive CR with its customers, FOSTA intends to continue implementing and optimizing the IB task profile developed thus far within the NASTA research cluster. In particular, the assumption of the tasks identified for the project phase would make a significant contribution to achieving success with innovations. An evaluation of the profile developed to date through individual post-project reviews is planned for the final phase of the six NASTA subprojects. Within the scope of cross-topic management of steel application research by FOSTA, there are plans to enable an institutionalized application of the IB concept for future CR clusters with other steel application sectors.

Acknowledgements

The management aspects put forward here resulted from an ongoing FOSTA research project on innovation management in steel application research with the participation of the Faculty of Science, Engineering & Computing, Kingston University London, and the Faculty of Management, Social Work & Construction of the University of Applied Sciences & Arts, Holzminden. FOSTA is grateful to the research institutes and industrial partners of the NASTA project for their participation in the survey. The NASTA project itself is supported by the Federal Ministry of Economics & Technology through the AiF as part of the programme for promoting industrial cooperative research (IGF) in accordance with a resolution of the lower house of the German parliament.

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Keywords: pre-competitive cooperative research; innovation broker; management of interdisciplinary research; sustainability in steel construction; promoter

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Conferences

10th International Structural Steel Colloquium

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The attending specialists from structural steel engineering companies, engineering consultancies, government departments and agencies, universities and research institutes discussed current topics in the practical use of steel, and the latest research results in this field. Noteworthy projects implemented using heavy plate from Dillingen were naturally again highlighted, among them the Makkah Clock Tower, a skyscraper currently under construction in Saudi Arabia. The centre of interest here is the steel used and the challenges encountered in such remarkable civil engineering projects. The sustainable use of steel in civil engineering was, indeed, a particular focus of this colloquium.

„Steel is a sustainable material; it can be recycled 100 % – again and again and again. As well as this, the use of steel in civil engineering shortens completion times considerably, and therefore reduces environmental problems, such as noise, dirt and disruptions to traffic, to the very minimum during bridge construction projects, for instance", noted Dr. *Karlheinz Blessing*, Chief Executive Officer of Dillinger Hütte, in his speech of welcome, and, as he continued: „The expansion of regenerable energy sources would be inconceivable without innovative steel products. This is why it is so important not to compromise the competitiveness of the European steel industry with emissions rights trading and with the burdens of high energy prices".

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Huangpu Pearl River Northern Channel Bridge, Guangzhou – the longest single-pylon cable-stayed bridge span in China

Yingliang Wang
Chengzao Huang
Yuncheng Feng

The Huangpu Pearl River Bridge is the key project on the second ring highway of Guangzhou, China. The main bridge over the northern channel is a single-pylon cable-stayed bridge with a main span of 383 m and a steel box girder, which was constructed using the balanced cantilever method. This paper describes the tender design, the final design and the construction of the bridge, including foundation, pylon and main girder.

1 Site topography, design standards and criteria

1.1 Site topography

The Huangpu Pearl River Bridge is the key project on the second ring highway at Guangzhou, China. The bridge connects the Central Business District (CBD) of Guangzhou and Panyu district and crosses the Pearl River between the Guangzhou Overseas Ship Maintenance Plant and the Boluomiao Ship Factory. An island about 600 m wide divides Pearl River into a southern and a northern channel at this point.

The total width of the northern channel is about 390 m and the water depth is about 5–8 m. The total width of the southern channel is about 1100 m, the water is 3–30 m deep.

1.2 Design criteria

The key design criteria adopted for the bridge include:

(1) Highway class: 6-lane highway in the short-term, 8-lane highway in the long-term.
(2) Bridge width: 34.5 m (excluding cable anchorage zone width).
(3) Design speed: 100 km/h.
(4) Imposed load standard: Chinese highway class I.
(5) Navigation clearance: the width and height of the northern navigation channel should not be less than 280 and 55 m respectively. The maximum water level for navigation is 7.0 m, the minimum water level for navigation is 3.48 m. According to the requirements of the Guangdong Province Hydraulic Department, the minimum span should be greater than 380 m.
(6) Design wind speed: average maximum 10-minute wind speed is 38.4 m/s, with a 100-year return period and measured 20 m above sea level.
(7) According to the site earthquake safety assessment report, completed by the Guangdong Province Seismic Department, the ground surface horizontal acceleration is 0.228g, with a 100-year return period and a probability of 3 %.

1.3 Design standards

In accordance with the client's requirements, the preliminary and detailed design of the permanent works was carried out according to the Chinese design code for highway bridges, the European Union code (Eurocode) and British Standard BS 5400 [1] for the entire bridge; other compatible codes and technical literature were adopted for specific design issues not covered by the main standards. The entire bridge was designed according to ultimate and serviceability limit states.

2 Tender design for main bridge

2.1 Background to tender design

The bridge was divided into two sections for tendering:

– northern channel main bridge plus the northern approach bridge
– southern channel main bridge, southern approach bridge and middle approach bridge.

A bidder was allowed to tender for one section only, rather than both sections. But each tender had to include the overall general layout of both the southern and northern channel bridges due to their close proximity.

A suspension bridge with a main span exceeding 900 m was preferred for the main bridge over the southern channel.

2.2 Key boundary conditions for main bridge

1) High-voltage corridors on Guangzhou side of northern channel
A high-voltage corridor belonging to Huangpu power station is located on the Guangzhou side of the northern channel, which is fixed on plan. The elevation of the high-voltage cables is about 60 m based on the original design and can be raised no higher than 100 m. The distance between the high-voltage corridor and the Guangzhou bank of the northern channel is about 90 m.
2) Commercial wharf on Guangzhou bank of northern channel
There is a commercial wharf on the Guangzhou bank of the northern channel which is owned by an overseas company. The affect on the commercial wharf must be reduced to a minimum. The distance between pier and bank should be greater than 10 m.

3) Landscaping
   The landscaping should be considered as a unity due to the close relationship between the Northern Channel Bridge and the Southern Channel Bridge

2.3 Tender design proposals

A total of 24 proposals were submitted by seven Chinese tenderers, which are shown in Table 1. Fig. 1 shows the general layout of proposals 1 to 5.

2.4 Tender design results

A proposal for the northern channel using a double-pylon cable-stayed bridge with a span of 450 m was excluded because the side span cable on the Guangzhou side would intersect with the high-voltage cables. A suspension bridge proposal was also rejected: although its cables did not clash with the high-voltage cables, the anchorage on the Guangzhou side would have affected the wharf. In the end, a single-pylon cable-stayed bridge with the pylon located on the island was approved by the expert and the client.

The design of the Southern Channel Bridge, southern and middle approach bridges was awarded to CCCC Highway Consultants Co. Ltd. The design of the Northern Channel Bridge and the northern approach bridges was awarded to CCCC First Highway Consultants Co. Ltd. The entire independent checking of the design and

Table 1. Proposals for Pearl River Bridge

Pro-posal	Proposals for Northern Channel Bridge	Proposals for Southern Channel Bridge	Consultant
1	SPCSB with 398 m main span and steel box girder	SB with 1100 or 900 m main span	Joint venture of Highway Planning & Design Institute, Sichuan province, and Chongqing Communications Research & Design Institute Co. Ltd.
2	SPCSB with 398 m main span and steel-concrete hybrid girder	SB with 1100 or 900 m main span	
3	SB with 468 m main span and steel box girder	SB with 1100 m main span	
4	SPCSB with 398 m main span and steel box girder, connected to the suspension anchorage	SB with 1100 m main span	
5	DPCSB with 450 m main span and steel-concrete hybrid girder	SB with 1100 m main span	
6	DPCSB with 450 m main span and concrete box girder	SB with 1100 m main span	
7	DPCSB with 456 m main span and concrete box girder	SB with 900 m main span	CCCC First Highway Consultants Co. Ltd.
8	DPCSB with 456 m main span and steel box girder	SB with 900 m main span	
9	DPCSB with 456 m main span and steel-concrete hybrid girder	SB with 900 m main span	
10	DPCSB with 430 m main span and concrete box girder	SB with 900 m main span	Highway Planning & Design Institute, Guangdong province
11	DPCSB with 430 m main span and steel concrete hybrid girder	SB with 900 m main span	
12	DPCSB with 430 m main span and steel box girder	SB with 900 m main span	
13	DPCSB with 370 m main span and concrete box girder	SB with 900 m main span	
14	DPCSB with 450 m main span and steel box girder	SB with 900 m main span	Highway Consultants, Anhui province
15	DPCSB with 360 m main span and steel box girder	SB with 900 m main span	
16	DPCSB with 450 m main span and steel box girder	SB with 1100 m main span	CCCC Highway Consultants Co. Ltd.
17	DPCSB with 450 m main span and steel box girder	SB with 760 +760 m main span**	
18	DPCSB with 450 m main span and steel box girder	SB with 960 +380 m main span	
19	DPCSB with 450 m main span and steel box girder	DPCSB with 890 m main span	
20	DPCSB with 450 m main span and steel box girder	SB with 1096 m main span	China Railway Major Bridge Reconnaissance & Design Institute Co. Ltd.
21	DPCSB with 450 m main span and steel box girder	SB with 900 m main span	
22	DPCSB with 450 m main span and steel box girder	SB with 1060 m main span	
23	DPCSB with 450 m main span and steel box girder	DPCSB with 900 m main span	
24	N/A	SB with 900 m main span	SMEDI
Notes	DPCSB: double-pylon cable-stayed bridge; SPCSB: single-pylon cable-stayed bridge; SB: suspension bridge; SMEDI: Shanghai Municipal Engineering Design Institute; ** triple-pylon suspension bridge		

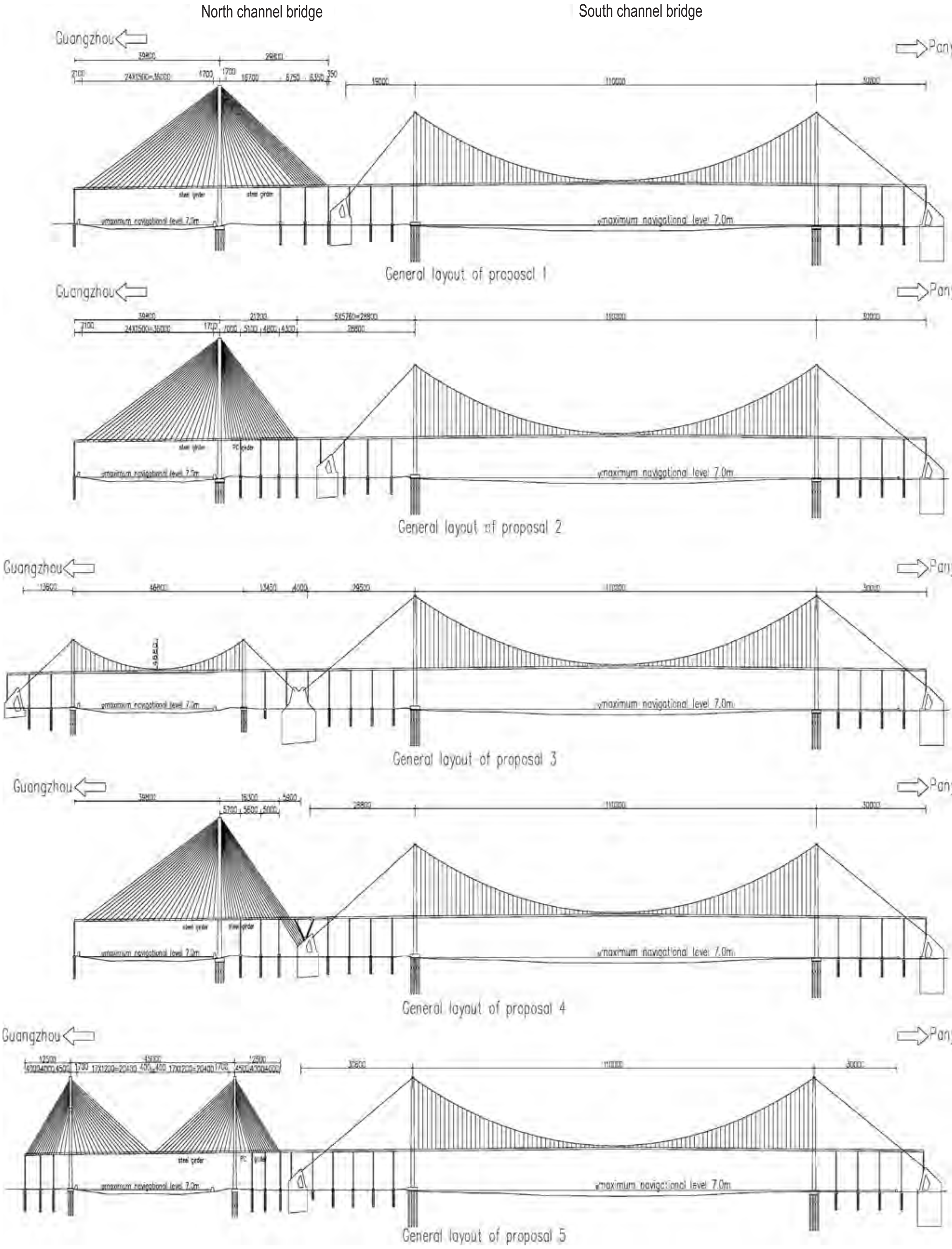

Fig. 1. General layout of the proposals at tender stage (unit: cm)

construction stages was awarded to the joint venture of Highway Planning & Design Institute, the Department of Communication, Sichuan province, and Chongqing Communications Research & Design Institute Co. Ltd. The first author was appointed as chief independent checking consultant by the client for his contribution to proposals 1, 2 and 3.

3 Final proposal and landscaping

3.1 Final design of complete crossing

At the final design stage, the main span of the northern and southern

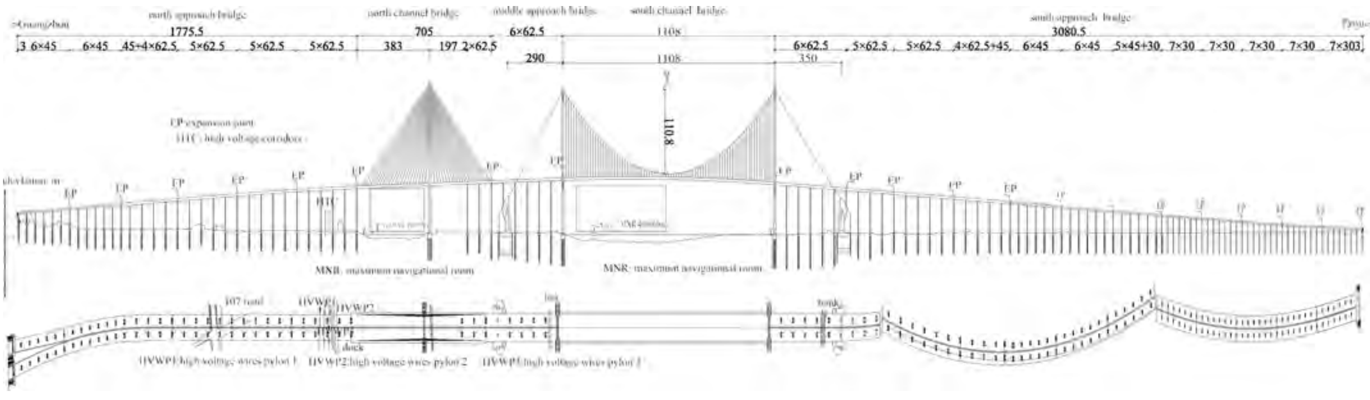

Fig. 2. Elevation and plan of entire bridge, including the locations of piers, abutment and approach bridge (unit: m)

channel bridges were adjusted to 383 and 1108 m respectively. Fig. 2 shows the general layout of the entire crossing.

The total length of the Huangpu Pearl River Bridge is 7049 m. The bridge is divided into:

1) Northern approach bridge – a prestressed concrete continuous girder with a span arrangement of 11 × 30 + (30 + 5 × 45) + 6 × 45 + (45 + 2 × 62.5) + 2 × 62.5 + 5 × 62.5 + 5 × 62.5 m
2) Northern channel main bridge – a steel box girder cable-stayed bridge with a span arrangement of 383 + 322 m
3) Middle approach bridge – a prestressed concrete frame with a span arrangement of 6 × 62.5 m
4) Southern channel main bridge – a steel box girder single-span suspension bridge with a main span of 1108 m
5) Southern channel approach bridge – a prestressed concrete continuous girder with a span arrangement of 6 × 62.5 + 5 × 62.5 + 5 × 62.5 + (4 × 62.5 + 45) + 6 × 45 + 6 × 45 + (5 × 45 + 30) + 7 × 30 + 7 × 30 + 7 × 30 + 6 × 30 + 6 × 30 m.

3.2 Landscaping for main bridge

The landscaping had to consider the bridge as a whole and keep the configuration of the pylons harmonized for both the northern and southern channel main bridges. Although a bidder was allowed to tender for one section only, rather than both sections with their different bridge types, they are connected by the middle approach bridge with a length of just 375 m. Therefore, a similar pylon configuration was adopted. It is very similar to the Chinese character "MEN", which means the "gate of Guangzhou". Fig. 3 shows a photomontage of the main bridge.

Fig. 3. Photomontage of main bridge

4 Design of northern channel main bridge

4.1 General design

The main bridge for the northern channel is a single-pylon cable-stayed bridge with a steel box girder and a span arrangement of 383 + 197 + 62.5 + 62.5 m (Fig. 4). The ratio of side span length to centre span length is 0.8407. Two intermediate piers were adopted to enhance the general rigidity and improve the structural behaviour of main girder and pylon. The approach bridge uses prestressed continuous box girders with main spans of 62.5 and 45 m, which are not included in this paper.

The pylon is located in the shallow water zone, pier 38 is located on the riverbank to avoid affecting the commercial wharf, intermediate piers 40 and 41 are located on the middle island. In order to avoid uplift of the side span, a counterweight is included in this span.

Vertical bearings are included at the pylon and piers 39, 40 and 41; there is a traverse wind bearing on the inner side of the pylon shaft and longitudinal hydraulic dampers at the pylon cross-beam (Fig. 5).

4.2 Main girder

The main girder is a streamlined steel box girder with three cells (Fig. 6). The depth of the box girder is 3.5 m and the total width 41 m, which includes the windshield and the cable anchorage zone. The transverse fall is 2 %. The top and bottom plate thicknesses of standard segments are 16 and 12 mm respectively, and both of them are 20 mm thick near the pylon segment.

Double inner webs divide the box into three cells; the distance between inner web and centre line of bridge is 9.4 m. The longitudinal stiffeners are closed trapezoidal stiffeners. The height, centre-to-centre distance

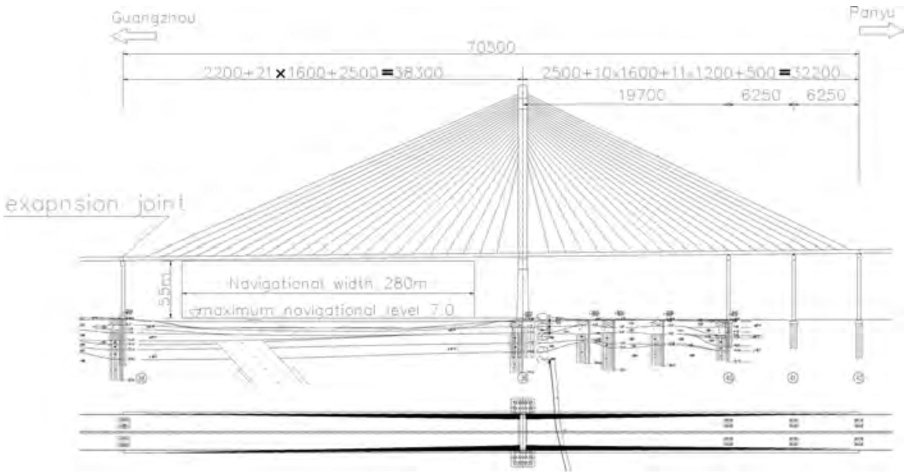

Fig. 4. General layout of northern channel main bridge (unit: cm)

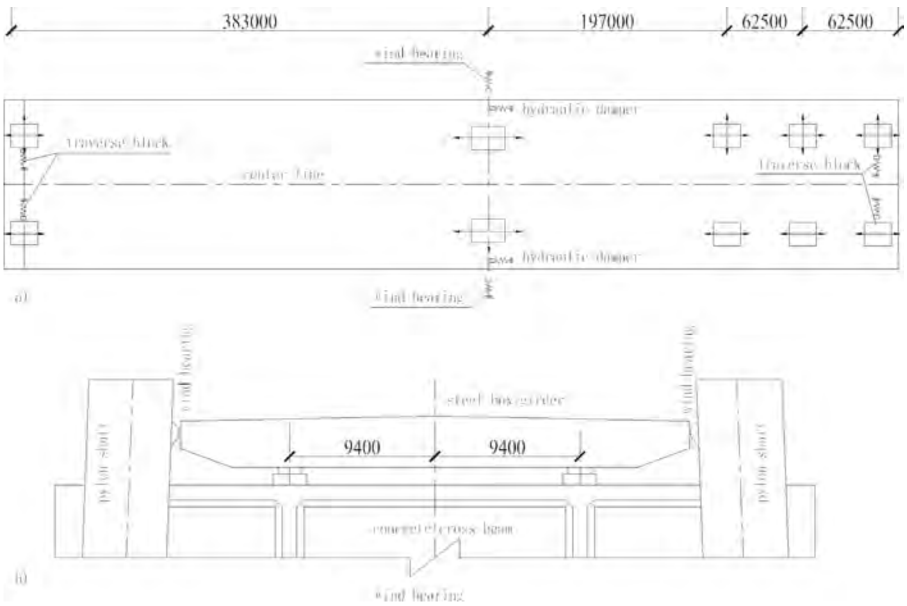

Fig. 5. Bearing system: a) general layout of bridge, b) bearings at pylon cross-beam (unit: mm)

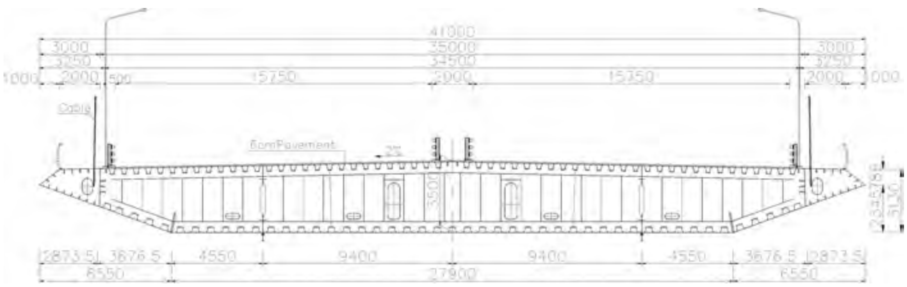

Fig. 6. Section through standard segment steel box girder (unit: mm)

Table 2. Stresses in steel orthotropic deck system

Location	Steel deck plate		Bottom of stiffeners		Mid-span of cross-beams	
Stress	Support of stiffener	Mid-span of stiffener	Support	Mid-span	Top flange	Bottom flange
Stress (MPa)	+16.96	–22.38	–55.33	+76.07	–29.43	+36.30
Note	+ tensile stress					

and thickness of the top flange longitudinal stiffener are 280, 600 and 8 mm respectively. The height, centre-to-centre distance and thickness of the bottom flange longitudinal stiffener are 250, 750 and 6 mm respectively.

The spacing of the cross-beams is 3.2 m, their web thickness is 10 mm, but 14 and 22 mm at the cable anchorages and bearings respectively. The cross-beam is divided into three segments in the transverse direction.

The segment length of the steel box girder was governed by the capacity of the floating crane, the derrick, as well as the construction method and duration of the work. The steel box girder is divided into 52 segments; the standard segment length is 16 m, the maximum weight 321 t.

The site splices of the steel box girder are welded. The total weight of the steel box girder is 15 500 t. Grade Q345qC has been adopted for the steel box girder, which is required to have a yield strength of 345 MPa and an impact test energy of 41 J at 0 °C.

The steel orthotropic deck system was analysed using the Pelikan-Esslinger method; the analysis model is shown in Fig. 7. Table 2 lists the stresses in the deck system.

4.3 Anchorage of stay-cables at main girder

Figs. 8 and 9 show the anchorage device for the stay-cable at the main girder.

The anchorage device was welded to the external side of the web of the steel box girder using full penetration welds. The plate of the external web of the steel box girder requires a Z25 through-thickness property to avoid lamellar tearing. The through-thickness property of the plate was determined with Pre-EN 1993-1-10.

The local stresses in the anchorage region were analysed using the finite element method (Fig. 10).

Fatigue assessment of the full penetration welds under tensile and shear stress interaction was checked using EN 1993-1-9 and the French steel bridge fatigue code (Ponts métalliques et mixtes: Résistance à la fatigue).

4.4 Pylon and foundation

The configuration of the pylon is similar to the pylon of the southern chan-

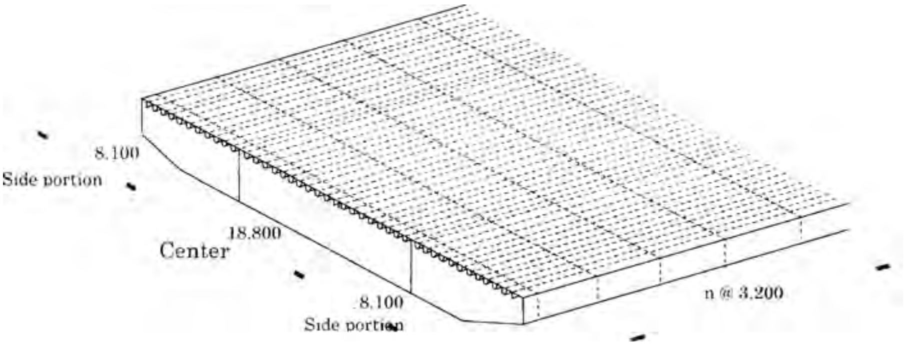

Fig. 7. Analysis model of steel orthotropic deck (unit: m)

Fig. 9. Anchorage device

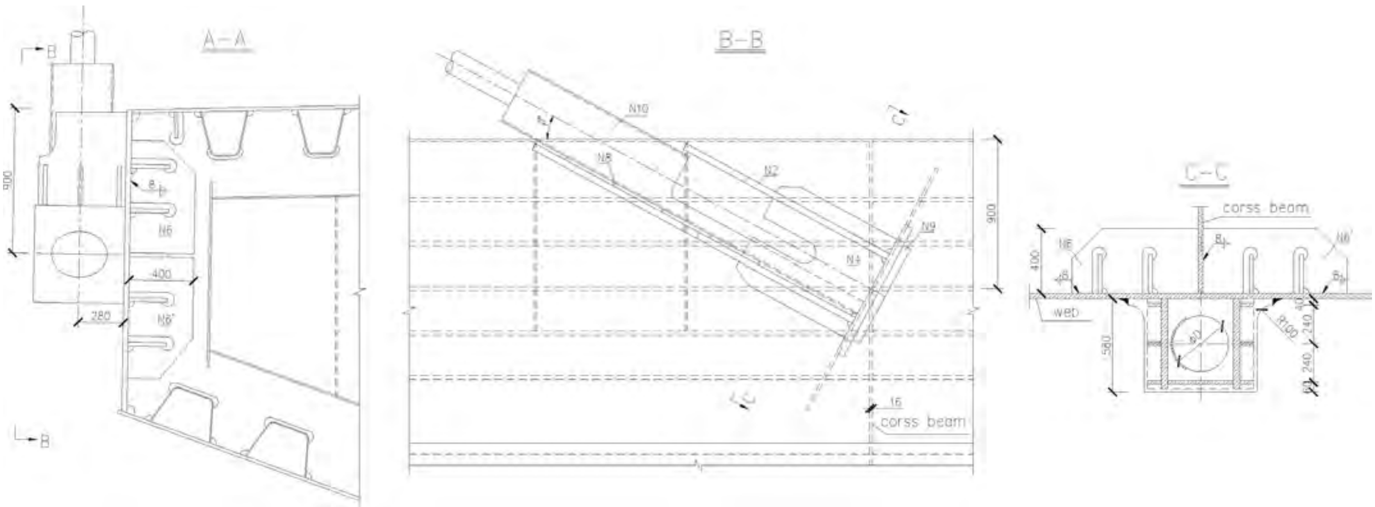

Fig. 8. Stay-cable anchorage at steel box girder

nel suspension bridge (Fig. 11). The height of the pylon above the pilecap is 226.14 m, and 160.45 m above the deck.

The concuk pylon shaft has a box cross-section with a 1.5 m arc chamber. The dimensions of the upper pylon shaft cross-section in the transverse and longitudinal directions are 5.5 and 8.5 m respectively, with thicknesses of 1.25 and 1.0 m. The lower pylon shaft dimension increases in the longitudinal direction from 8.5 to 11.5 m linearly, and the dimension in

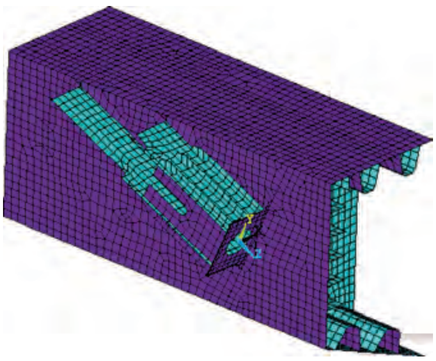

Fig. 10. Local FEM model of anchorage device

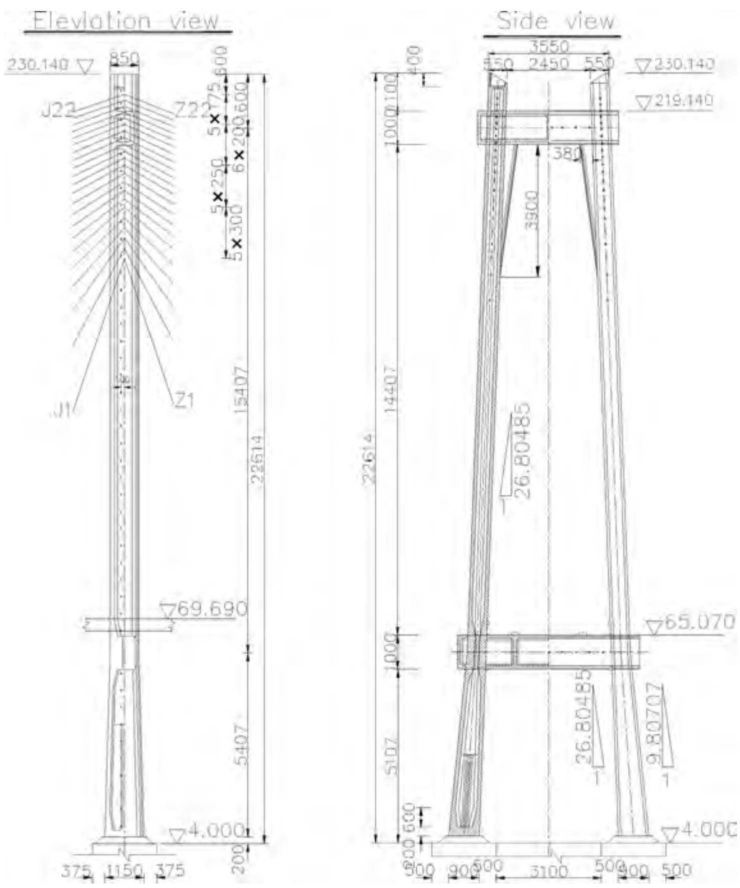

Fig. 11. Configuration of pylon (unit: cm)

the transverse direction from 5.5 to 9.0 m. Fig. 12 shows the cross-section of the upper pylon shaft.

There are two cross-beams between the pylon shafts which are in prestressed concrete with a box section and 10 m high. The parts of the cross-beam cantilevering beyond the pylon shaft are only for landscaping and decorative purposes.

The foundation beneath the pylon is in the form of 32 cast-on-site bored piles with a diameter of 250 cm, which are supported on hard rock.

Piers 38, 40, 41 and 42 have a solid rectangular cross-section with dimensions of 5 m in the transverse and 3.5 m in the longitudinal direction. The foundation is supported on cast-on-site bored piles.

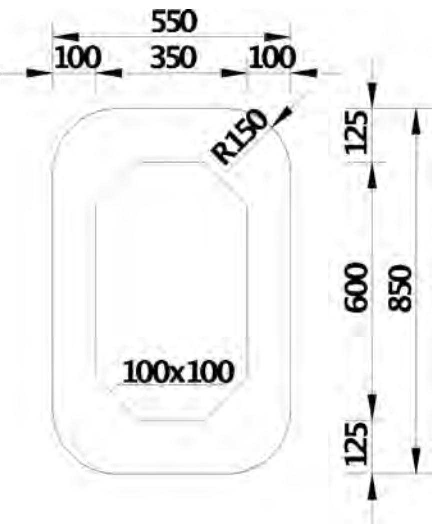

Fig. 12. Cross-section of upper pylon shaft (unit: cm)

4.5 Stay-cables

The stay-cables are prefabricated parallel wire cables with a diameter of 7 mm and an ultimate tensile strength of 1770 MPa. The numbers of steel wires are 121, 139, 163, 199, 223 and 253. They were tensioned by jacks on the pylon. The maximum length of the stay-cables is 391.2 m and the maximum weight is 28.4 t. Small dowels are fitted to the PE of the cable to prevent wind/rain-induced vibrations.

At the time of installation, the stay-cables were jacked to about 30~35 % of their ultimate capacity. The safety factor of the stay-cables at the ultimate limit state is about 2.5, which is about 15 % greater than the factor adopted in Eurocode 1993-1-11.

All cables were considered for the cable replacement load case. The objective of cable replacement is to satisfy the client that any such future scenario could be undertaken under sustainable conditions involving limited traffic disruption and without the need for full closures. The total weight of stay-cables is 1348 t.

5 Construction of main bridge

Since the pylon is located in shallow water, its foundation was constructed in a steel sheet pile cofferdam (Fig. 13). The pylon was constructed with hydraulic self-climbing formwork; the completed pylon is shown in Fig. 14.

The deck was constructed using the double cantilever method; traditional steel travellers were employed for the steel box girder installation. For each segment erection and site welding, the quickest cycle achieved was 4 days.

Fig. 13. Steel sheet pile cofferdam

Fig. 14. Completed pylon

The deck construction was monitored continuously by the on-site technical team to ensure that the required final design vertical and horizontal profiles were achieve. The segment erection level was continuously adjusted to take into account the difference between theoretical analysis deflection and site measurements.

The construction of the bridge began on 14 February 2005. Deck closure stitches were carried out in the required sequence by the traveller. The deck was finished on 28 March 2008 and opened to traffic on 16 December 2008. The total deck construction duration was 47 months. Fig. 15 shows the completed bridge.

6 Conclusion

The design and construction of the Pearl River Northern Channel Bridge, Guangzhou, were completed on time and to budget as a result of the excellent cooperation between client, designer, contractor, sub-contractors and site supervision consultants.

The progress to a successful completion of the project has benefited from having a viable design concept and solutions that changed very little during construction. The considerable investment made by the contractor in additional soil investigations and scientific research and tests enabled the development of a safe and efficient

Fig. 15. The completed main bridge

substructure and superstructure design.

The bridge has been well received by the Guangdong Province government and the Ministry of Communication of China, who regard the construction of the first major cable-stayed bridge in southern China as a symbol of progress and prosperity. It is destined to become a significant new landmark in this important city.

Acknowledgements

The authors would like to acknowledge the cooperation and assistance of the client, vice-general director Mr. *Shaojin Zhang*, the client's site representatives, Mrs. *Taike Zhang* and Mrs. *Hong Cheng*, and all the colleagues of the design team, the independent checking team and the construction team.

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News

Leicht France opens in Nantes

Four years after the founding of the company, LEICHT is pleased to announce the birth of a French sister company: LEICHT FRANCE SAS saw the light of day in Nantes and is assuming all the commitments of its German sister on the French market.

With the establishment of LEICHT FRANCE as a French enterprise, LEICHT intends to bring the successful cooperation with French customers and partners to a new level: the legal basis provided by this type of stock corporation (Société par actions simplifiée /SAS) not only simplifies many business procedures but, in particular, matters related to insurance coverage. Besides barrier-free communication, the introduction of a French engineering team also promises the enrichment of LEICHT's know-how with country-specific expertise.

According to the company, the goal of LEICHT FRANCE is to establish itself in the medium term among France's top experts with respect to consultation and support structure design for projects involving ETFE film, lightweight construction and façades.

As one hundred percent, equal sisters LEICHT FRANCE SAS and LEICHT Structural engineering and specialist consulting GmbH share both the same shareholder structure and range of services. General Manager of LEICHT FRANCE is Graduate Engineer *Marcel Enzweiler*, President is Dr. Ing. *Lutz Schoene*. Chief engineers and contact persons in Nantes are the membrane experts *Sylvain Bernard* and *Jochen Arndt*.

For more information please visit www.leichtonline.com.

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Awaiting the 2012 European Football Championship: some features of the reconstruction of the stadium of the "Olympic" National Sports Centre in Kiev

Aleksandr V. Shimanovsky

The article presents results of reconstruction and new construction works at the National Sports Center "Olympic" stadium in the city of Kyiv, which are being performed in Ukraine as preparations for the final tournament of the UEFA European Football Championship 2012. It describes conceptual designs and layouts involved. Totaling data are presented concerning the number of spectators, the built-up areas, the infrastructure. Structural designs of steel and ferroconcrete constructions are described, results of their computational analysis are included The article also presents data regarding the manufacturing and erection of structures.

1 Introduction

The reconstruction of the "Olympic" National Sports Centre (NSC "Olympic") in Kiev is now underway within the framework of a programme being undertaken in Ukraine to prepare for the finals of the 2012 European Football Championship. The UEFA decided to include the stadium at the NSC among the four locations in the Ukraine where the group, quarter-final and final matches of the championship are to be played. Unlike some other stadiums, NSC "Olympic" will still remain a venue for other sports too; facilities for track and field events are to be created there in accordance with the requirements of the International Association of Athletics Federations. This will make it possible to host both football matches and international athletics competitions on the highest level.

The reconstruction of the stadium included almost complete dismantling of its structures and floodlighting towers, except for a second tier of stands, in order to reduce the extent and duration of construction works. The whole façade of the stadium is covered by a glass shell, and the stands are covered by a transparent, long-span, suspended membrane/fabric system. The 13-storey "Olympic Star" building will be built next to the stadium (Fig. 1).

Considerable excavation and construction work has to be carried out around the stadium, including stabilizing adjacent slopes to the south and west and constructing an encircling concrete gallery to form an approach for spectators. A vast "underground city" is to be built beneath the stadium and around it, including rooms for various purposes and an underground car park.

The following is a description of the basic structural and engineering concepts adopted in the design of the reconstructed NSC "Olympic" stadium in order to turn it into a first-class location in accordance with UEFA requirements.

2 Conceptual design

The stadium sports a modern architecture: it looks quite organic within its natural and urban surroundings, and also has a structurally rational and technologically advanced layout that ensures high levels of comfort and safety for its visitors. The stadium can accommodate 69 000 spectators, has an area of about 60 000 m^2 and its football pitch measures 104 m long × 72 m wide. In addition, the total built-up area of the stadium covers about 160 000 m^2, the VIP area occupies 45 000 m^2 and the land improvement area extends about 1500 m around the stadium itself (Fig. 2).

a)

b)

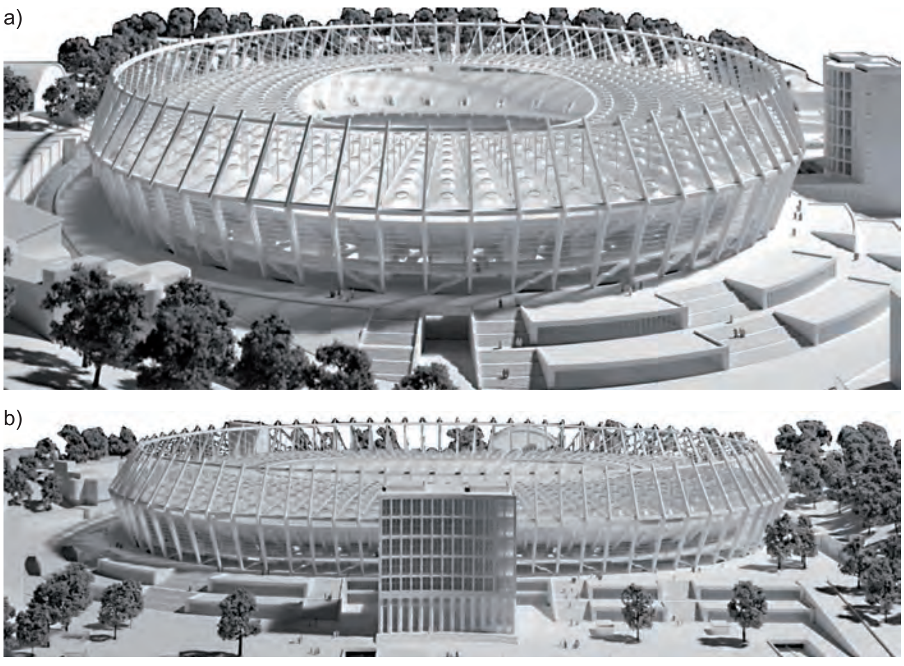

Fig. 1. A general view of the NSC "Olympic" stadium from a) north and b) west

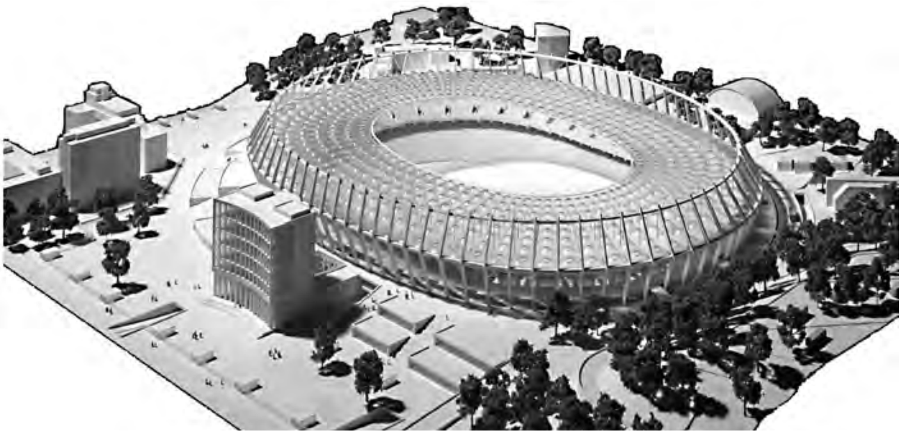

Fig. 2. A general view of the NSC “Olympic” stadium

The general contract for the reconstruction of the stadium was concluded with JSC Kievgorstroy (Ukraine), and the contract for the construction of the steel roof structure over the stands was awarded to LLC Master-Profi Ukraine Plant. The cables and their anchorages for the suspended roof structure over the stadium stands were fabricated by Bridon International Ltd. (UK), the fabric membrane system by Hightex GmbH (Germany).

The design of the reconstruction of the NSC “Olympic” was carried out by GMP von Gerkan, Marg und Partner (Germany), taking into account the UEFA standards that apply to structures of this kind. The V. Shimanovsky Ukrainian Institute of Steel Construction was asked to perform an expert appraisal of the design decisions for the steel roof structure over the stands, to devise an erection procedure, to develop a design and to fabricate racks for the assembly of columns and various erection rigging.

In addition to the reconstruction of its bowl, the reconstruction of the stadium included rebuilding the central grandstand to create a four-level VIP building with presidential, VIP and corporate boxes, two-tier sky boxes, bars, restaurants, a mix area, a press box, rooms for television, photographers, commentators, security, two stationary and one small intermediate tier on the stand itself, rooms for players and officials, plus many other plant/ancillary rooms. The “Olympic Star” building will house various functional rooms for official, sporting and media (a press centre) purposes; there will be a helicopter landing pad on its roof (Fig. 2).

The infrastructure around the stadium is to be fully renovated. Parking will be created for football teams, officials, VIP visitors, media representatives and spectators; separate and secure passages for evacuation will be arranged. A broad commercial, trading and entertainment area will be laid out in the stadium’s surroundings. An “Olympic” electric substation (110 MW) will be constructed nearby.

The old lighting system (based on four separate free-standing towers) has been dismantled and a new system installed beneath the roof. It provides the necessary illumination for the playing field. Two LED indicator boards have been set up in the stadium.

The element involving the greatest responsibility and complexity on the NSC “Olympic” stadium is the roof structure over its stands, with an outreach of 65–69 m and an area of 48 500 m^2. It is one of the world’s biggest roofs over a sports arena, and at the same time it is a fine combination of architectural aesthetics and operational reliability. The features of the roof structure include the following:

- The steel columns supporting the roof are placed outside the stadium bowl, are not carried on its stands and thus do not transfer any additional loads onto them (Fig. 3a).
- The roof is an oval on plan; it is a long-span, suspended, two-layer radial annular structure consisting of upper and lower radial cables of 55–85 mm dia., suspenders and an inner ring (Fig. 3b).
- The roof covering is a fabric membrane made of a synthetic material with glass-fibre reinforcement and a PTFE coating on both sides; it has a high strength, high fire resistance and high light transparency (Fig. 3c).
- The surface of the roof may experience unavoidable changes in its geometry depending on the environmental conditions; the process of shape adaptation is controlled by a

a)

b)

c)

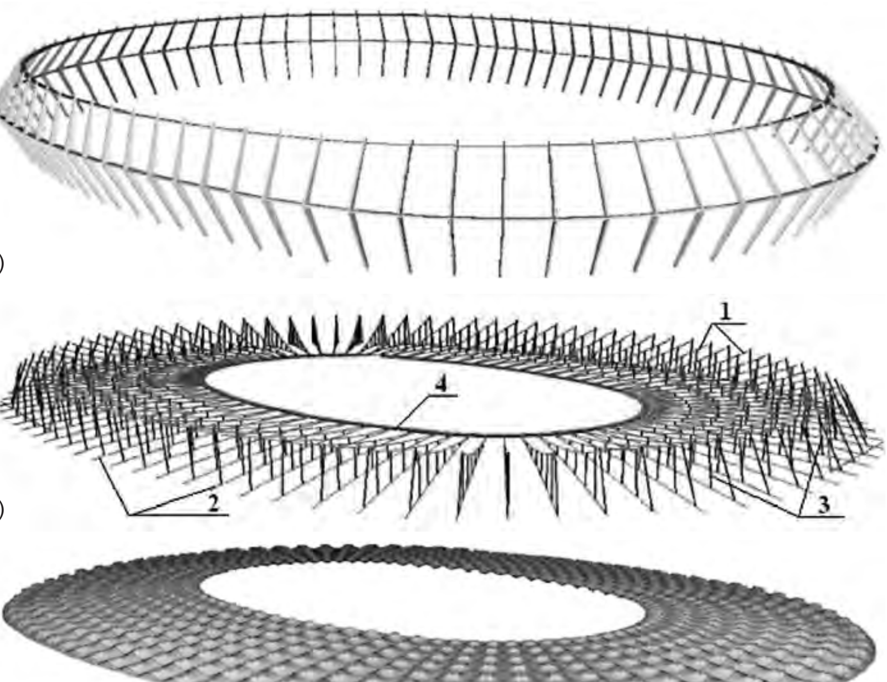

Fig. 3. The roof structure to the NSC “Olympic” stadium: a) loadbearing frames, b) cable system, c) fabric membrane roof; 1) upper radial cables, 2) lower radial cables, 3) suspenders, 4) internal ring

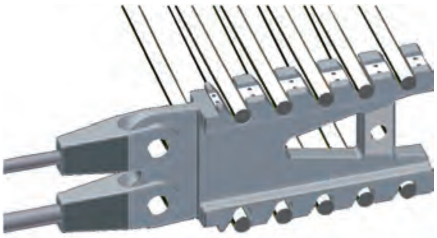

Fig. 4. A cast node for joining the upper and lower radial cables to the inner ring

computer system which ensures good ventilation and prolonged natural illumination of the playing field, maintains proper temperature and humidity levels, and eliminates the possibility of a “hothouse effect”.

There are about 3000 cables (13–115 mm dia.) in the suspended roof structure – a total length of 40 km which weighs 765 t. More than 1000 clamps are required to connect the separate cables together. Uniting the cables into a single roof structure requires about 22 000 high-precision cast nodes (Fig. 4).

3 Structural design

The long-span, suspended, two-layer radial annular roof system over the stadium’s grandstands hangs on the upper outer loadbearing ring supported by 80 steel columns, each 49 m long and weighing about 50 t (Fig. 5).

The upper radial cables support the inner ring of the suspended roof system, which is made up of a bundle of cables 90 mm in diameter, and hangs on the upper outer ring on columns at a height of about 40 m. The lower radial cables, attached to the inner ring and located directly above the stands, hang on the lower outer loadbearing ring supported on columns at a height of about 22 m. The outer loadbearing rings are made of rolled steel box sections (800 × 1200 mm, wall thickness 30–70 mm).

According to the structural design of the roof, the columns are spaced about 10.5 m apart and supported via hinges on the reinforced concrete structures of the spectator gallery that encircles the stadium. The upper loadbearing ring hangs on the tops of the columns, attached to those via hinges, in the plane of the cable trusses, and the lower loadbearing ring is attached rigidly to the columns. The columns, together with the lower loadbearing ring clamped to them, make up a rigid spatial frame that ensures the stability of the whole structure, being united into a whole by the prestressed suspended roofing system.

The outer loadbearing rings (upper and lower) undergo compression because of the thrust in the cables transferred to them; the magnitude of the compression can reach 5000 t. The inner ring is in tension – about 5500 t. The maximum tensile force that the ring is capable of resisting is 13 500 t. The steel columns that support the suspended roof have been designed for the height of the polygonal configuration; the knuckle of its profile is at the level of the lower outer loadbearing ring (Fig. 5). The height of the lower part of the columns is about 22 m, that of the upper part 18 m, which made it possible to reduce the ellipse-like form of the upper outer loadbearing ring in order to decrease the bending moments that occur there under external loads.

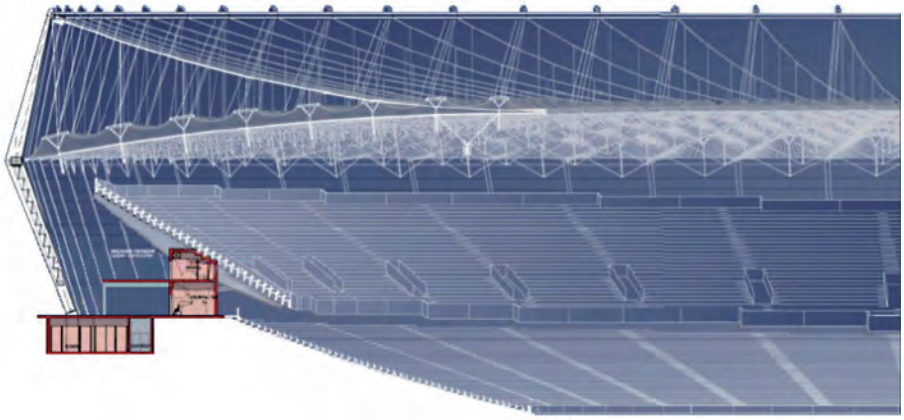

Fig. 5. Section through and structural design of the suspended roof over the stadium stands

The fabric membrane system is positioned above the lower radial cables of the roof. The system is designed as an “awning” structure with knee-braced props to raise the level of the surface: this helps to create a prestress that stabilizes the shape and allows rainwater to drain in a natural way. The centres of the sections where the props are installed will have dome skylights with a transparent covering of polycarbonate sheets.

The principal structural members of the roof framework are made of grade C355 steel. Their factory connections are welded, their on-site connections use high-strength bolts.

4 Fabrication and assembly of steel structures

The contract for the fabrication and assembly of the steel roof structure was awarded to LLC Master-Profi Ukraine Plant. The installation of the principal loadbearing members of the roof (loadbearing columns and outer upper/lower loadbearing rings) has been carried out by JSC Krivorozhstalkonstruktsia (Ukraine). The installation of the long-span suspended fabric membrane system was carried out by Fagioli S.p.A. (Italy), appointed to ship, set up and check hoisting equipment to erect the roof structure to its design position, and by Hightex GmbH (Germany), appointed to install the actual fabric membrane.

One of the main project requirements for the roofing over the stands was to ensure very high precision in the fabrication and erection of the main loadbearing members. The whole loadbearing structure for the roof – a total length of nearly 1 km – had to be assembled consecutively – element by element – with minimum tolerances (in some cases set to values lower than those in the national standards) in order to guarantee that the closure point coincided with the starting point of the assembly.

Therefore, the fabricator of the loadbearing structure for the roof, LLC Master-Profi Ukraine Plant, provided the production facility with additional hi-tech metal-processing equipment and developed specialized technologies for fabricating and supervising the manufacturing and assembly processes to make sure all project requirements were met. In addition, special rigging and assembly benches were set up to achieve the proper quality of the construction, assembly and erection. For example, the production facility performed a check assembly of all large structural members to form larger sections, then eliminated the discrepancies thus found. This satisfied all the project requirements regarding precision in fabrication and erection of the structure. Therefore, the project implementation procedure included little or no correction of the loadbearing members of the roof after fabrication.

The design of the roof paid special attention to the technological convenience of fabrication and the ability to assemble the loadbearing steel structure to form larger units in order to make construction simpler and faster. All steel columns were pre-assembled on the ground to form larger units. The erection of the loadbearing structure consisted of the following phases: five columns; five sections of the lower outer loadbearing ring; a section of the upper outer loadbearing ring of the first span; and so on in this order, taking into account the retardation of the erection of sections of the upper outer loadbearing ring by four spans, particularly: first and second columns with a section of the lower outer loadbearing ring between them; third column and a section of the lower outer loadbearing ring of the second span; fourth column and a section of the lower outer loadbearing ring of the third span; fifth column and a section of the lower outer loadbearing ring of the fourth span, together with a section of the upper outer loadbearing ring of the first span, and so on in this order until the whole structure was finished.

Fig. 7. A travelling tower for installing columns at their design positions and regulating their geometry

Special erection rigging was designed for the purpose of installation and for regulating the geometry of the loadbearing structure of the roof. The works to be performed on site included assembling columns from two shipped items into a single structural member; four benches were designed and built for this task (Fig. 6).

The assembled columns were placed in the design position one by one using special travelling towers about 10.5 m high which, firstly, functioned as temporary/erection supports for the columns and sections of the lower outer loadbearing ring and, secondly, permitted regulation of the column geometry using special lifting devices installed on them (Fig. 7). After the geometry of another group of five columns had been regulated, the travelling towers were moved into a new position to support the next group. Props were added under the columns already installed and regulated until completion of the whole loadbearing structure of the roof (Fig. 8).

After the installation was over and the loadbearing support structure for the roofing was in place, and its shape stabilized by a prestress, the cable net of the suspended part of the roofing were erected. For this purpose, each column had a lifting/tensioning device attached to it; the devices were controlled by a special computer system.

The installation of the members of the roof's loadbearing structure was performed on the side of the central (inner) area of the stadium using various lifting plant, including a unique crane, a Demag CC 2800-1, with a capacity of 600 t and a 114 m jib (Fig. 9).

5 Structural analysis

The complexity of the design of the roof over the NSC "Olympic" stadium and strict safety requirements made it necessary to perform a sophisticated analysis of the behaviour of the roof structure under most unfavourable

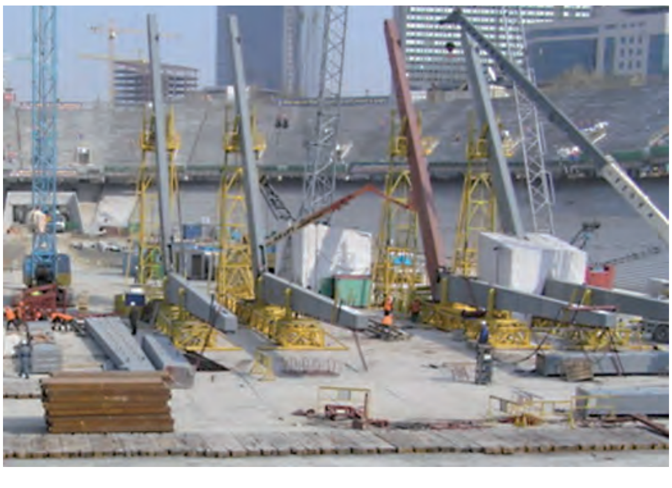

Fig. 6. A general view of the benches for assembling the column segments into larger sections on site

Fig. 8. Temporary props under the columns during erection

Fig. 9. Installing a column with the Demag CC 2800-1 and SKG-100 cranes

combinations of actions. A design model was adopted that included the principal loadbearing structures of the roof separated into structural parts. The first of the parts was a loadbearing roof framework that consisted of the steel columns and the upper and lower outer loadbearing rings (see Fig. 3a). The second was its cable system, which consisted of the inner ring, the upper and lower radial cables, and suspenders between those (see Fig. 3b).

The finite element model of the roofing was built by representing nearly all of the loadbearing structures as finite bar elements that resist the tensile/compressive, bending and torsion forces. Certain parts of the columns and the loadbearing rings (as a rule, in a vicinity of joints) were represented as finite plate elements. The model took into account hinged supports to the loadbearing columns; the hinges were assumed to have two degrees of freedom in order to permit rotation in two vertical planes. The total number of finite elements in the model was 5468, with 4880 nodes and 20 928 unknowns.

The particular feature of the behaviour of the roof is that it can resist external loads only when the cable system is prestressed. Therefore, the whole analysis consisted of two phases. Phase one included determining a design configuration for the prestressed roofing over the stadium using its given basic geometry parameters. The initial state of the roofing was assumed to be unstressed (both load and prestress set to zero), and the design state obtained by a step-by-step application of the load and an appropriate reduction in the lengths of the upper and lower radial cables in proportion to a certain parameter α so that the design values were reached at $\alpha = 1$. Phase two included an analysis of the roof in accordance with the load cases established: the dead load of the roof structure, an operational load caused by auxiliary equipment (lights, electric cables, video screens and their installation rigging, acoustic actions), a prestress load, a wind pressure, a snow/ice load, temperature and seismic actions. Four analyses of the roof were performed: distributions of the stresses and strains caused by the prestress and by static loads with a symmetric/asymmetric application of the snow load were found; the general stability of the structure under the static loads was verified; natural frequencies and modes of oscillation were determined for the prestressed structure of the roof with and without taking into account the apparent masses of the auxiliary equipment and snow.

The results of the analysis performed for the roof over the stadium show that the structure does possess a high loadbearing ability and general stability. It also has sufficient reserves of strength because the maximum stress in the finite bar elements, even under a most unfavourable combination of loads, is 225 MPa (yield point = 355 MPa), the stresses in the cables remain positive and their values are nearly two times lower than the permissible limits. Secondly, the structure has proved to be hardly sensitive to uniformly distributed temperature actions which have little noticeable effect on the distribution of stresses and strains in the structural members of the roof. Thirdly, the system possesses a sufficient dynamic stiffness because the frequencies of three lowest natural oscillation modes of the roofing – with prestress, dead load of structural members and snow – have the relatively high values of 0.25, 0.27 and 0.29 Hz and are hardly dependent on an actual distribution of snow over the roof.

Keywords: sports facilities; stadiums; space strcutures; numerical 3D modelling; design; manufacturing; assembly

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Conferences

Steel Bridges 2012

The Steel Bridges 2012 – "Sustainably bridging the future: the steel way Design and best practices" will be held in Paris, France on 18–20 April and is organized by SCMF – Syndicat de la Construction Metallique de France. The Conference will also host the European Steelbridges Design Awards ceremony.

There will be 10 sessions during the conference in which the following topics will be discussed:

1st Session – The architects and owners point of view
2nd Session – Global cost – Life Cycle
3rd Session – Design for sustainability
4th Session – Design and calculation
5th Session – Steel: the future raw material
6th Session – Steel bridges – some examples
7th Session – Monitoring and sustainability

Bridges Design Awards

8th Session – Maintenance and repairs
9th Session – High Speed Railways 25 years experience
10th Session – Bridges of the future

Do not miss this international event in the heart of the capital Paris and with the best European specialists in steel bridges.

VIII CMM – Portuguese Conference on Steelwork

On the 24th and 25th of November 2011, the 8th edition of the Conference on Steel and Composite Construction took place in Guimarães, Portugal in "Centro Cultural de Vila Flor". This event was organized by CMM, the Portuguese Steelwork Association and had over 200 participants.

Several sessions were held during the conference: "Living with Steel" (Architect Neven Sidor from Grimshaw); "Cold-Formed Steel Structures in Seismic Area: Research and Applications" (Professor *Raffaele Landolfo* from the University of Naples); "Composite Steel-Glass Fins for the Lobby Façade of Iberdrola Tower" (Eng. *Francesc Arbós Bellapart*, from Bellapart, S.A.U.); "Construção Metálica: Novas Fronteiras" (Professor *António Reis* from IST); "Best Use of Eurocode 3 for Member Design in Frame-Structures" (Professor *Richard Greiner*, from Graz University of Technology); "Mantos Metálicos" (Architect *Francisco Vieira de Campos*).

Facing the present need of companies to be introduced to CE Marking for Steel products and components so as to fulfill the EN 1090-1 requirements, we emphasize the conference "CE Marking of Structural Steelwork – Implications for Manufacturers and Specifiers" presented by Dr. *Roger Pope*, Consultant for the British Constructional Steelwork Association and Tata Steel Europe), which took place during the Conference on Steel and Composite Construction.

Some seminars also took place during the conference: DiSTEEL (Displacement Based Seismic Design of STEEL Moment Resisting Frame Structures) and Semi-Comp+ (Valorization action of plastic member capacity of semi-compact steel sections – a more economic design).

In this 8th edition of the conference we also gave the sponsors the opportunity to present their products and services in commercial sessions.

Due to the number of papers submitted triple sessions were necessary which demonstrates not only the quality of the event but also the high number of participants from the both technical and scientific steel community.

The conference also had a technical exhibition in the foyer of Centro Cultural Vila Flor, with the presence of 17 companies.

As tradition in the CMM Conferences, a Gala Dinner was held on the first day of the event. This dinner took place in "Pousada de Santa Marinha". Lunch was offered to the participants in the restaurant "Café-Concerto".

Announcements

International events

STEELBRIDGES 2012 – International Conference on Steel Bridges
18–20 April 2012, Paris, France

NASCC: The Steel Conference 2012
18–21 April, 2012, Gaylord Texan Convention CenterGrapevine, Texas

ECCS European Steel Day and ECCS Annual Meeting 2012
19–22 September, Lisbon, Portugal

**NSCC 2012 – Nordic Steel Construction Conference
5–7 September of 2012,
Oslo, Norway
www.nordicsteel2012.com
post@stalforbund.com

**ICSA 2013 – 2nd International Conference on Structures and Architecture
24–26 July 2013, Guimarães, Portugal
www.icsa2013.arquitectura.uminho.pt

6th International Conference on Composites in Construction Engineering (CICE)
13–15 June 2012
conference.cice2012.it

National events

Connections VII – 7th INTERNATIONAL WORKSHOP ON CONNECTIONS IN STEEL STRUCTURES
30 May to 2 June 2012
www.ct.upt.ro/connections/index.htm

1º Congresso Luso – Africano de Construção Metálica Sustentável
Organized by CMM
27 July 2012, Luanda, Angola

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Technical Committees (TC) activities

Agenda TC Meetings

TMB – Technical Management Board
Chairperson: Prof. *L. Simões da Silva*
Vice-Chairperson: Prof. *M. Veljkovic*

PMB – Promotional Management Board
Chairperson: Mr. *Bertrand Lemoine*

TC3 – Fire Safety
Chairperson: Prof. *P. Schaumann*
Secretary: Prof. Paulo Vila Real
Date: 17–18 September 2012, Helsinki, Finland (planned)

TC6 – Fatigue and Fracture
Chairperson: Dr. *M. Lukic*
Date: 08–09 March 2012, Berlin, Germany

TC7 – Cold-formed thin-walled Sheet Steel in Buildings
Chairperson: Prof. *J. Lange*
Date: 28–29 June 2012, (planned)

TWG 7.5 – Practical Improvement of Design Procedures
Chairperson: Prof. *Bettina Brune*
Date: 28–29 June 2012, (planned)

TWG 7.9 – Sandwich Panels & Related Subjects
Chairperson: Mr. *Paavo Hassinen*
Date: 28–29 June 2012, (planned)

TC8 – Structural Stability
Chairperson: Prof. *H. H. Snijder*
Secretary: Dr. *Markus Knobloch*
Date: 15 June 2012, Dresden, Germany

TWG 8.3 – Plate Buckling
Chairperson: Prof. *U. Kuhlmann*
Secretary: Dr. *B. Braun*
Date: 2 March 2012, Stuttgart, Germany
Date: 26 October 2012, Liège, Belgium

TWG 8.4 – Buckling of Shells
Chairperson: Prof. *J. M. Rotter*
Secretary: Prof. *S. Karamanos*

TC9 – Execution and Quality Management
Chairperson: Mr. *Kjetil Myrhe*
Date: 27 February 2012, Brussels, Belgium

TC10 – Structural Connections
Chairperson: Prof. *F. S. K. Bijlaard*
Secretaries: Mr. *Edwin Belder*
Date: 29 May 2012
Date: 4–5 April 2013

TC11 – Composite
Chairperson: Prof. *R. Zandonini*
Secretary: Prof. *Graziano Leoni*
Date: 4 May 2012 Lahti, Finland

TC13 – Seismic Design
Chairperson: Prof. *R. Landolfo*
Secretary: Dr. *A. Stratan*
Date: 11 May 2012 Budapest, Hungary
Date: 23 November 2012, Luxembourg (Planned)

TC14 – Sustainability & Eco-Efficiency of Steel Construction
Chairperson: Prof. *Luís Bragança*
Secretary: Ms. *Heli Koukkari*

TC15 – Architectural & Structural Design
Chairperson: Prof. *P. Cruz*

TC News

TWG 7.5 – Practical Improvements of Design Guidelines

Chairperson: Prof. *Bettina Brune*

The last meeting of TWG 7.5 held in Ithaca (Cornell University, USA), in cooperation with the European Racking Federation ERF, the American Rack Manufacturers Institute RMI and guests from Japan. It was an interesting meeting dealing with all kind of cold-formed structures (housing, earthquake design, rack design etc.).

During the meeting there were five sessions where the following topics were presented and discussed:

Session 1 – TWG 7.5/REF Meeting (RMI)
- Research on Cold-Formed Steel in Salford, UK
- Low and midrise cold-formed steel building systems in the USA
- Theoretical and numerical modelling of mode-coupling and the yield-arc/yield-eye mechanism
- Benchmark example of rack uprights
- Benchmark example of box columns made of stainless steel
- Calibration of an FEM for evaluation of CFS bolted joints in pitch-roof portal frames and MS frames

Session 4 – Joint TWG 7.5/ERF/RMI Meeting
- Australian rack specifications and research
- American research on rack systems
- Earthquake evaluations from a probabilistic point of view
- Seismic design of racks in Japan
- ERF-items on Rack Systems
- Investigation on the test method for distortional buckling of compressed pallet rack members
- An Investigation on the design of steel storage rack columns using the DSM
- Fatigue of rack systems
- Building supervision provisions for rack structures
- Column base tests
- Earthquake design according to the RMI specification

Session 5 – Joint TWG 7.5/ERF Meeting
- Seismic resisting cold-formed steel wall systems
- A beginner's overview of Eurocode – possible tasks for WG 7.5 to help its use easier
- National Annexes to EN 1993-1-3
- Evolution Groups for Eurocodes
- Lipped channel column in axial compression – Eigenmode imperfection and strength

The next meeting is expected to be held on 28–29 June 2012 along with the joint TC7 – meeting.

TWG 7.9 – Sandwich Panels & Related Subjects

Chairperson: Mr. *Paavo Hassinen*

The European Joint Committee consists of the Technical Working Group ECCS TWG 7.9 and of Working Commission CIB W056. The preliminary date of the next meeting of the Joint Committee is 28–29 June 2012. The meeting place is at the moment open.

The Joint Committee has completed a new European Recommendation for design of sandwich panels with openings. The manuscript is now in linguistic check. The Joint Committee has started to draft two new documents. The first one will introduce the possibilities to stabilize steel structures such as columns and purlins against buckling and lateral buckling by utilizing sandwich panels. The document will be an important guideline in optimizing the frames and substructures of industrial buildings. The second new document will study the effect of claddings which are typically fixed in the external face of the wall or roof panels. The document will evaluate risks and benefits and further, a good practice of cladding of sandwich panels.

TWG 8.3 – Plate Buckling

Chairperson: Prof. *Ulrike Kuhlmann*
Secretary: Dr. *Benjamin Braun*

The work of TWG 8.3 concentrates on plate buckling issues in steel plated structures.

The most recent meeting took place on 11 November 2011 in Stuttgart, Ger-

many, with presentations and discussions on the following subjects:
- Proposal of amendments to EN 1993-1-5
- Cross-section classification of angles (*D. Beg*)
- Modified determination of the critical column-buckling stress (*G. Lener*)
- Comparison of the effective width method and the reduced stress method (*J. Simon-Talero*)
- Numerical and experimental analysis of intermediate transverse stiffener in stiffened plate girders (*F. Sinur*)
- Resistance of steel plate girders subjected to patch loading (*R. Chacon, B. Braun*)
- Stress distribution in the flanges of corrugated web girders (*B. Kövesdi*)
- Numerical and experimental analysis of longitudinally stiffened plated girders subjected to bending-shear interaction (*F. Sinur*)

Short time outcome
A series of papers on recent topics is prepared for Steel Construction Journal 1/2012. They also reflect some planned amendments for EN 1993-1-5 which will partly be presented for discussion during the meeting of CEN/TC250/SC3 in April 2012.

Next meetings
The next meetings will be held on 2 March 2012 in Stuttgart, Germany, and on 26 October 2012 in Liège, Belgium.

TC 9 – Manufacturing and Erection Standards

Chairperson: Mr. *Kjetil Myhre*

TC9 is the technical committee within ECCS responsible for making recommendations concerning technical specifications for manufacturing and erection. The committee has prepared the publication 'Guide to the CE Marking of Structural Steelwork' to provide practical guidance on the CE Marking of structural steelwork in accordance with the Construction Products Directive (CPD), the Construction Products Regulation (CPR) and the transition period between CPD and CPR. ECCS acknowledges the advice provided by the British Constructional Steelwork Association Ltd and access to its publication 'Guide to the CE Marking of Structural Steelwork' on which this publication is based.

The CE Marking standard for fabricated structural steelwork, EN 1090-1: 2009 'Execution of steel structures and aluminum structures: Part 1: Requirements for conformity assessment of structural components' has been cited in the European Commission's Official Journal (OJ). The Applicability Date is 1st of January 2011, followed by an 3.5 years co-existence period. This means that CE Marking of fabricated steelwork can start and will become mandatory in all European countries from 1st of July 2014.

It is hoped that this publication will assist the steelwork contractors, their purchasing clients and supply chain including stockholders, part fabricated products (curved steel), proprietary products (purlins, cellular beams etc.) designers, specifiers and construction managers.

The TC9 is also preparing a 'Checklist for initial inspection and surveillance of the factory production control FPC according to EN 1090-1'. Furthermore, the Committee will develop a website with a list of certified fabricators, and prepare a handbook for FPC (Factory Production Control).

The next meeting is on the 27th February 2012 in ECCS offices in Brussels. I welcome more members to join the committee, please contact your national member association or send an e-mail to post(at)stalforbund.com

TC 10 – Structural Connections

Chairperson: Prof. *F. S. K. Bijlaard*
Secretary: *Edwin Belder*

The next meeting will be combined with the next International Connections Workshop to be held in Timisoara. The meeting after that will take place on the 4 and 5 of April 2013 (location not specified).

In the 99th meeting held in Karlsruhe the technical committee has been working on the corrigenda and amendments for the EN 1993-1-8.

The following topics were also presented and discussed:
Amendments Eurocode 3 part 1-8
- Discussion about corrigenda and amendments on Chapter 7 of EN 1993-1-8 "Hollow Section Joints
- Approval of the Proposed amendments EN 1993-1-8 Comments Actions Decisions

Joints

Presentation of design of joints with fabricated box sections according to Eurocode 3.
Bjørn Aasen

The introduction of steel grades S420 and higher have encouraged Norwegian engineers to use these steels in fabricated box sections in various structures. However, practical design rules for structural joints seem to be missing, and FEM-analyses very time consuming. Therefore *Bjørn Aasen* wants to show the committee that EN 1993-1-8 can be applied for the design of fabricated box section joints.

New software for the design of hollow section joints. This is provided for free by Vallourec & Mannesmann, including book presenting design tables for standardized joints.
Klaus Weynand, Ralf Oerder

During the last ECCS-meeting in Munich design software was presented for composite joints. Now a new tool has been developed for the design of hollow section joints under fatigue loading (see Cidect Design Guide 8

Many tools are available within Europe for open section joint design; however this is not the case for hollow sections. The objective is to provide design tools for standardized joints –?design book & individual joint layouts – design software.

The first phase will have a limited scope (most common configurations). This will be approved by German building authorities. The scope shall include the design rules according EN 1993-1-8, static loading, only RHS sections in material S355 & S460. The Design Book shall include design tables and background information for various configurations (K, N, X, Y, T) in S355 and S460.

A copy of the design book will be provided to the members at next ECCS TC-10 meeting.

NA to EN 1993-1-8, some comments about National Annexes regarding practitioners.

Work of CEN TC 185WG6 in particular the difficulties encountered to update the EN 14399-1 standard for preloaded "Structural Bolting" on the evolution of the EN bolt standards, bolts. (*Ivor Ryan & Maël Couchaux*)
The standard series EN 14399; Bolts for Preloading has been expanded up to 10 parts since 2005. Part 1 covers the CE marking (requirements). The EN 14399-1 needs modification in order to extend the scope of CE marking to parts 7–10. However, this encounters problems with the CEN expert. *Ivor Ryan* made some remarks about the performance requirements of part 2 and the actual behavior under tightening. Information from a manufacturer indicated that type HV had difficulties meeting $\Delta\theta 1$ requirements and sometimes $\Delta\theta 2$ requirements. *Ivor Ryan* concluded that it is questionable whether a HV product (as described in part 4) is suitable for tightening by the combined method unless it meets the now "optional" requirement on $\Delta\theta 1min$ during the tightening test.

Composite joints

The experimental behavior of heated composite joints, subject to variable bending moments. (*Aldina Santiago*)
Within the European RFCS Robustfire Project research is done to obtain new design criteria of car parks with sufficient robustness under localized fire.

The tested sub-structure (experimental test) was a part of an open car park building. Seven experimental tests were done with various high temperatures. Ceramic pad heating elements were used to reach these temperatures. Bending moments were applied and tested with/without axial restraint. Failure mode was concrete crushing and deformation of the steel end-plate at centre and bottom part. Other failure modes were local buckling of the bottom flange and fractured bolts (M30 bolts-10.9).

Connectors

Tests on connections with injection bolts with high bearing stresses (*Nol Gresnigt*)
In Amsterdam, many M27 injection bolts are applied in a the glass roof of an extension of the Central Railway Station. Because of the glass, rather tight limits were put on the deformations of the supporting steel structure. High strength friction grip bolts were not a good solution because of the necessity to take away the paint coating of the steel at the contact surfaces and the danger of corrosion pollution later on (the steel remains visible for the public).

It was asked whether for the load case with wind in the ultimate limit state, higher bearing stresses could be allowed than the recommended values that are based on long duration creep tests. The bearing stresses in this ULS load case could be more than twice the recommended values.

Recent test results by Prof *Darko Beg* indicated that indeed for short duration loads higher bearing stresses can be allowed. In Delft additional short duration cyclic tests were carried out, some with more than double the recommended bearing stresses. The test results together with test results from research in Ljubljana were sufficient to allow the much higher bearing stresses for 90 % of the glass roof in Amsterdam. Only at the edges of the roof where wind gusts are most severe, additional strengthening was necessary.

A. Load indicated washers.
Frans Bijlaard informs the committee about the latest developments on DTI washers. According EN 1090-2 no more than 10 % of the indicators are allowed to be fully impressed. However, the fabricators want to change this rule (check on over-tightening) by making a new rule in EN 14399, and declare it binding.

This could very well endanger the safety of the connection according EN 1090-2.

Bjørn Aasen, also member of TC135, will make attempts to discuss about this issue within TC135 once again.

Bolts

Document Tragverhalten von Scher-Lochleibungsverbindungen mit dem Beginn des Gewindes in der Nähe der Scherfuge.
Part of document:
In bolted connections the shear load is calculated with the cross section of the bolt. It is important whether the shank or the thread is in the shear plane. In the new code for bolts DIN EN 14399-4 the dimension of the bolts are modified. Compared to the previous code, the DIN 6914, in many situations the thread is closer to the shear plane or even in the shear plane. In the current standards there is no requirement about how long the shank has to protrude into the shear plane.

The question is if such a rule should be added to the code. Should engineers check bolt lengths on drawing or is it the responsibility of the steel construction fabricator?

Klaus Weynand feels that this is the responsibility of the fabricator to act according the design of the joints.

High strength steel

A. Current research focuses at KIT
Thomas Ummenhofer presented various research projects which are currently in progress at KIT.

These projects include the optimization of supporting structures for offshore wind energy convertors using HIFIT and the development of a joining technique for adhesive bonding of hollow sections.

B. Effects of post treatment on the fatigue life of ultra high strength steels
PhilippWeidner presented research concerning Domex 960 steel treated with HIFIT. Especially in crane design weights must be as low as possible. Therefore higher strength steels are used. The test results show that the fatigue life in the LCF range can be extended 5–6 times using HIFIT-treatment. All specimen failed at the weld toe. *Philipp Weidner* concluded that ultra high strength steels offer more potential with regard to fatigue compared to lower yield steels.

Publications

ECCS book "Design of joints", to be considered a background-document to EN 1993-1-8 (new book under development) (*Klaus Weynand, Jean-Pierre Jaspart, Jürgen Kuck*)

Request for information from *Klaus Weynand*, regarding books for the design of connections according EC. This could also be documents or books with design tables. Books like BCSA-simple joints (green book). Work on the book is well underway. Inform KWor JPJ about status of their books and contents, so they can refer to these books and documents also. Please reply to them before end of May.

These meetings were a good opportunity for delegates to present their research, recent projects and make suggestions to improve the EN-code based on research or practice.

TC 11 – Composite Structures

Chairperson: Prof. *Riccardo Zandonini*
Secretary: Prof. *Graziano Leoni*

Before the last meeting held in Timişoara on November 4th, 2011, Prof. *Jiri Studnicka* has resigned as Full Member in the TC 11. He will continue his involvement in the TC activities as a Correspondent Member. In this way the TC will not loose his important input as he continues his active participation, similarly to other Emeritus Professors such as *Jean Marie Aribert, Roger Johnson*, and *Joël Raoul*, who enhance the activities of the TC with their experience and expertise.

Two new members joined the TC 11: *Jakub Dolejs* from Czech Republic, who replaces Prof. *Jiri Studnicka*, and *Manuel Romero* from Spain. Nineteen Full Members and seven Corresponding Members, from thirteen European countries, Australia and New Zeeland, currently constitute the TC 11.

Besides the ongoing activities aimed at preparing documents that should complement Eurocode 4 with references to issues on shear connections, flooring systems and composite frames, other topics are currently being discussed within TC 11.

In the last meeting, Prof. *Jean-Paul Lebet*, from the Ecole Polytechnique Fédérale de Lausanne, and Prof. *Dennis Lam*, from the University of Bradford, reported about advances in research on composite bridges and elliptical hollow columns, respectively.

Due to its valuable members, TC 11 represent a permanent and useful source of design information for composite construction in Europe.

The next meeting of ECCS-TC 11 will be held in Lahti (Finland) on 4th May 2012.

TC 13 – Seismic Design

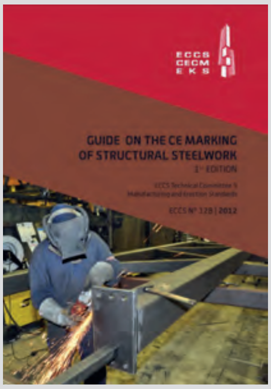
Chairperson: Prof. *R. Landolfo*
Secretary: Prof. *A. Stratan*

In 2011 TC 13 organized two meetings: the former in Rennes (France) on 20th May, the second in Ljubljana (Slovenia) 18 November. The main activities of TC 13 concentrated on the seismic design of steel structures with special regard to the following aspects:

1. **TC 13 commentary and integration to EN 1998-1: 2004:** on the basis of a work document developed in the last year within TC 13 activity, it is under preparation the following ECCS publication: "Assessment of EC 8 provisions for seismic design of steel structures". In this book the main issues in EN 1998-1: 2004 that need clarification and/or further development are described and discussed. The document is organized in twelve Sections and one Annex.
2. **TC 13 dissemination activity:** Within the "STESSA 2012" conference ("Behaviour of Steel Structures in Seismic Areas"), held in Santiago (Chile) on the last January 9–11 and also supported by ECCS, a special session dedicated to TC 13 activity was organized. In that occasion 8 papers concerning the main topics dealt with each TC13 TWGs were presented.
3. **Next meeting:** The next TC 13 meeting is fixed for 11 May in 2012 in Budapest. The following one is planned for 23 November 2012 in Luxembourg.

New Publications

Guide on the CE Marking of Structural Steelwork



The CE Marking standard for fabricated structural steelwork, EN 1090-1: 2009 'Execution of steel structures and aluminum structures: Part 1: Requirements for conformity assessment of structural components' has been cited in the European Commission's Official Journal (OJ). The Applicability Date is 1st of January 2011, followed by an 18 months co-existence period. This means that CE Marking of fabricated steelwork can start and will become mandatory in most European countries from 1st of July 2012. CE Marking is optional in the UK, Republic of Ireland, Finland, Sweden and Norway for one more year, until 1st of July, when the Construction Products Directive (CPD) is replaced by the full Construction Products Regulations (CPR).

TC 9 is the technical committee within ECCS responsible for making recommendations concerning technical specifications for manufacturing and erection. The committee has prepared this publication to provide practical guidance on the CE Marking of structural steelwork in accordance with the Construction Products Directive (CPD), the Construction Products Regulation (CPR) and the transition period between CPD and CPR.

It is hoped that this publication will assist the steelwork contractors, their purchasing clients and supply chain including stockholders, part fabricated products (curved steel), proprietary products (purlins, cellular beams etc.) designers, specifiers and construction managers.

Thin Walled Structures – Recent Research Advancences and Trends (2 volumes)

The Proceedings of ICTWS 2011 – RECENT RESEARCH ADVANCES AND TRENDS, organized in two volumes, contain 9 keynote lectures presented by outstanding scientists and 115 papers, by authors from 33 countries of Africa, Asia, Australia, Europe, North America and South Africa. These papers are grouped in 9 main topics i.e.:

Session 1: Buckling;
Session 2: Post-bucking analysis and failure modes;
Session 3: Behaviour of thin-walled structures under extreme loadings;
Session 4: Connection on thin-walled structures;
Session 5: Cold-formed steel structures;
Session 6: Composite structures;
Session 7: Storage racking;
Session 8: Shell and space structures;
Session 9: Plated Structures.

Full price: 95 €
Available at ECCS online Bookstore (www.steelconstruct.com). Limited to existing stock.

Projects

SKILLS – Steel Construction Industry Live Long Learning Support

Skills is a new project funded by the LEONARDO DA VINCI Lifelong learning programme in France and is being led by CTICM (Centre Technique Industriel de la Construction Mátallique), in collaboration with PIKS and ASCEM, the steel fabricator associations in Poland and Spain, Warsaw University of Technology, the Universitat Politècnica de Catalunya and ConstruirAcier, the steel construction promotional organization in France.

The aim of SKILLS is to assist training providers in France, Poland and Spain meet the substantial needs of the steel construction industries in these countries, arising from the changes in working practices due to implementation of the Eurocodes. The project will deliver practical guidance which will demonstrate Eurocode application in steel building design.

The tangible outcomes will be training materials, in tried and tested formats, aligned to national priorities and in national languages, for delivery by national teachers and trainers.

The international journal “Steel Construction – Design and Research” publishes peer-reviewed papers covering the entire field of steel construction research and engineering practice, focusing on the areas of composite construction, bridges, buildings, cable and membrane structures, façades, glass and lightweight constructions, also cranes, masts, towers, hydraulic structures, vessels, tanks and chimneys plus fire protection. “Steel Construction – Design and Research” is the engineering science journal for structural steelwork systems, which embraces the following areas of activity: new theories and testing, design, analysis and calculations, fabrication and erection, usage and conversion, preserving and maintaining the building stock, recycling and disposal. “Steel Construction – Design and Research” is therefore aimed not only at academics, but in particular at consulting structural engineers, and also other engineers active in the relevant industries and authorities.

“Steel Construction – Design and Research” is published four times a year.

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If required, offprints or run-ons can be made of single articles. Requests should be sent to the publisher.

Current prices
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personal	256 sFr	295 sFr	73,60 sFr
personal	243 $	279 $	69,86 $
institutional	520 €	598 €	149 €
institutional	865 sFr	995 sFr	248,69 sFr
institutional	818 $	941 $	235,18 $

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Steel Construction 2/2012

Markku Heinisuo, Henri Perttola, Hilkka Ronni
Component method for end plate joints, modeling of 3D frames: literature review

Literature of modelling steel frames in 3D using beam finite elements is presented. The development of member modelling is described first followed by joint modelling. The review ends with a brief introduction of the 3D component model for end plate joints. The next part includes three examples of the 3D component method: base bolt joint, beam-to-column joint and member splice joint.

Johan Maljaars, Frank van Dooren, Henk Kolstein
Fatigue assessment for deck plates in orthotropic bridge decks

Various bridges in highways suffer from fatigue cracks in the deck plate of orthotropic steel bridges. This article provides an assessment procedure for the fatigue lifetime of the deck plate. The lifetime determined with the assessment procedure agrees well with the lifetime observed in practice. The required deck plate thickness for new bridges is determined with the assessment procedure. A method is developed to quantitatively account for results of inspections in the design.

Matthias Wieschollek, Nicole Schillo, Markus Feldmann, Gerhard Sedlacek
Lateral torsional buckling checks of steel frames using 2nd order analysis

This article summarizes rules from EN 1993-1-1 to perform a 2nd order analysis for the flexural and lateral torsional buckling check of members and systems that are loaded in the strong plane of their cross-section and shall not fail by out-of-plane instability.
The rules relevant in EN 1993-1-1 are rules for equivalent geometrical imperfections resulting from the elastic buckling modes with amplitudes derived from column buckling tests according to EN 1990 – Annex D and rules for the linear superposition of utilization rates in the strong plane and the weak plane as used for the evaluation of the tests to determine the amplitude of imperfections and the partial factors.
To determine the elastic buckling modes in general computer-assistance is required; it gives both modal out of plane displacements but also modal out of plane bending moments for the relevant flanges of the profiles. The use of out of plane bending moments makes the assessment clear and simple. It leads to a magnification factor for the in-plane-utilization rate that allows to detect the design point x = xd along the length of the member or system where the cross-sectional check is relevant. It also shows that the results of the 2nd order analysis for this design point and the use of buckling curves for member checks at this design point are identical.
The assessment procedure with 2nd order analysis is generic and applicable to all design cases whereas the member checks with buckling curves as specified in EN 1993-1-1 are intended to be valid for specific loadings and boundary conditions only (object oriented rules). They should be consistent with the generic rules. In this respect this article is a contribution for the further evolution of EN 1993-1-1.

(subject to change without notice)

Literature for bridge building by Ernst & Sohn

■ Bauingenieure mögen Fußwegbrücken mäßiger Breite und Traglasten weniger beachten als Straßen- und Eisenbahnbrücken von spektakulärer Spannweite und Konstruktion. Städtebauer und Landschaftsplaner hingegen erachten innerstädtische Rad- und Gehwegbrücken für weitaus entscheidender und veranlassen in der Regel Wettbewerbe. Da es keine einheitlichen Entwurfsrichtlinien gibt, bilden die Erfahrungen aus realisierten Brücken eine wichtige Informationsquelle für Planer.

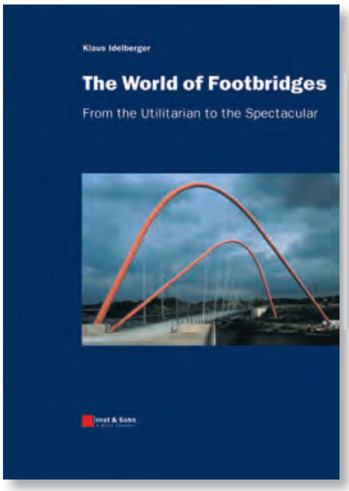
Das vorliegende Buch enthält über 100 Beispiele, die in den letzten zehn Jahren weltweit gebaut wurden: offene Fuß- und Radwegbrücken, Viehtrieb- und Medienbrücken sowie einige geschlossene Verbindungsstege.

Die Beispielsammlung ist nach Tragwerkstypen und Spannweiten gegliedert. Zu jeder Brücke gibt es eine kurze Darstellung der Randbedingungen und eine Bauwerksbeschreibung, illustriert anhand von Fotos, Grund- und Aufrissen, besonderen Konstruktionsdetails. Somit bildet es eine Fundgrube für Bauingenieure.

KLAUS IDELBERGER

Fuß- und Radwegbrücken
Beispielsammlung

2011.
182 S., 351 Abb., Br.
€ 49,90*
ISBN 978-3-433-02937-4



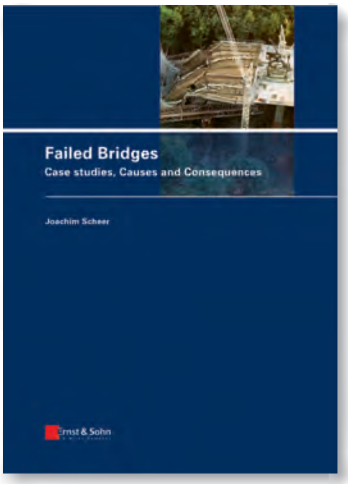
■ Structural engineers are often more interested in road and rail bridges of spectacular construction and enormous spans than in relatively narrow footbridges built for modest loads. City planners and landscape architects, on the other hand, see inner-city pedestrian and cycle bridges as important architectural elements and generally invite bridge builders to compete to find the winning design. As there are no official guidelines for the design of footbridges, the building techniques and performance of existing bridges are an important source of information for planners.

This book contains over 100 examples of bridges built worldwide over the last ten years: open pedestrian and cycle bridges, cattle and utility bridges and some enclosed skyways. The collection is arranged according to load-bearing structure. For each bridge there is a brief description of the location and the structural system, including special construction details, and illustrated by photographs, plans and elevations. "Footbridges" is a treasure trove for structural engineers.

KLAUS IDELBERGER

The World of Footbridges
From the Utilitarian to the Spectacular

2011.
183 pages, 351 figures. Softcover.
€ 69,–*
ISBN 978-3-433-02943-5



■ When bridges fail, often with loss of human life, those involved may be unwilling to speak openly about the cause. Yet it is possible to learn from mistakes. The lessons gained lead to greater safety and are a source of innovation.

This book contains a systematic, unprecedented overview of more than 500 bridge failures assigned to the time of their occurrence in the bridges' life cycle and to the releasing events. Primary causes are identified. Many of the cases investigated are published here for the first time and previous interpretations are shown to be incomplete or incorrect. A catalogue of rules that can help to avoid future mistakes in design analysis, planning and erection is included.

A lifetime's work brilliantly compiled and courageously presented – a wealth of knowledge and experience for every structural engineer.

JOACHIM SCHEER

Failed Bridges
Case Studies, Causes and Consequences

2010.
321 pages. 120 fig. 15 tab.
Hardcover.
€ 79,–*
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