Slovak University of Technology in Bratislava

Faculty of Civil Engineering

Department of Concrete Structures and Bridges

Annual Report

2014

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ANNUAL REPORT 2014
Department of Concrete Structures and Bridges
Faculty of Civil Engineering
Slovak University of Technology in Bratislava
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CONTENT

PREFACE

Sustainable development of society is not possible without the development of environmental infrastructure, part of which is the demand for increasing the capacity of stored and treated water. The market for municipal wastewater treatment plants will continue to grow in the long term. Over the past 20 years, the number of municipal wastewater treatment plants in Europe has increased steadily while their technical standard improved. And the quantitative and qualitative development of wastewater treatment in Europe continues. The EU Urban Wastewater Treatment Directive and the EU Water Framework Directive still provide the strongest market stimuli – especially in Eastern and Southern Europe. By contrast, maintenance and renewals as well as measures for reducing operational costs (primarily in terms of energy) dominate in Central and Northern Europe.

The indicator presented in Fig. 1 refers to sewage treatment connection rates, i.e. the percentage of the national population connected to a wastewater treatment plant in OECD countries. The share of the population connected to a municipal wastewater treatment plant rose from about 50% in the early 1980s to about 60% in the early 1990s and has reached almost 80% today (in Slovak Republic 60 %).

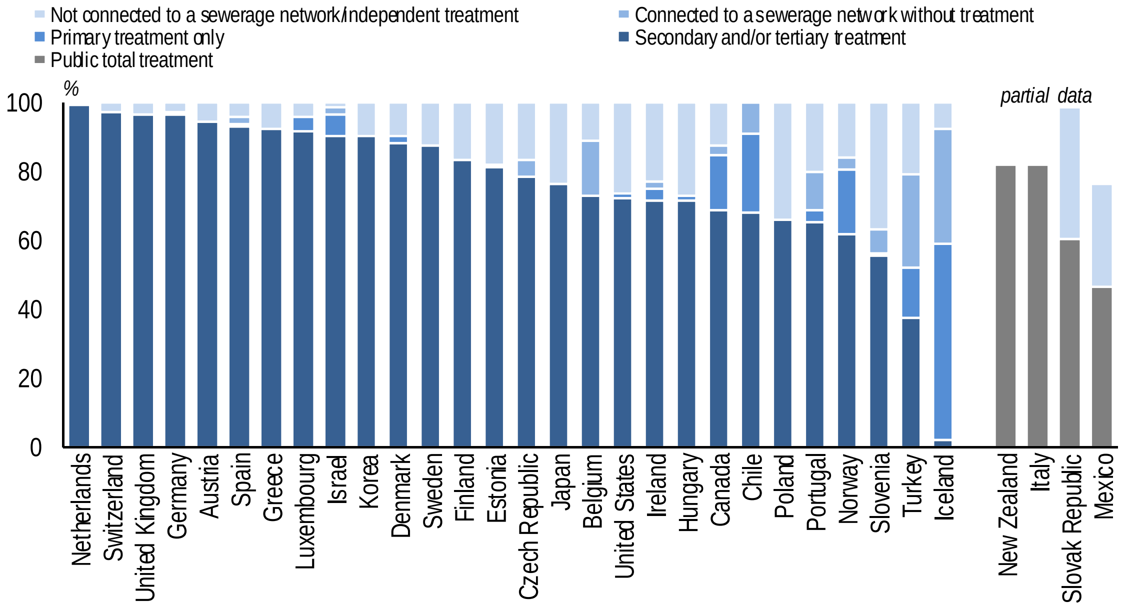

Figure 1 Sewage treatment connection rates % of national population connected to a wastewater treatment plant (OECD Environmental statistics Dec. 2013)

The Slovak Government is committed and has taken measures that may pertain to sewage and wastewater treatment in Slovakia. By 2015 municipalities which have a population of over 2 000 inhabitants shall be resolved. The financial analysis has proved that the costs to build the shared sewer system with wastewater treatment plant for each municipality would represent more than 4 milliard euro. These processes take place in rectangular or circular concrete tanks (Fig. 2).

With respect to their origin can be **distinguished** direct and indirect actions. At present design of concrete structures is mainly focused on the effects of direct actions.

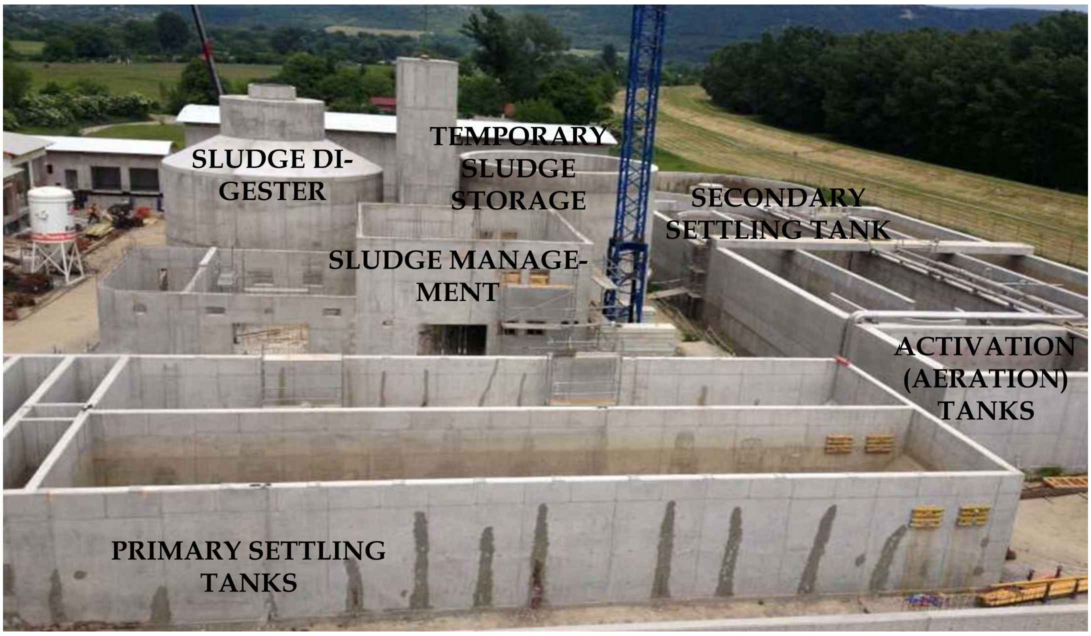

Figure 2 Wastewater treatment plant is a set of rectangular and circular tanks. Vertical separation cracks in the wall of the primary settling tank.

The effects of indirect actions are often neglected and the reinforcement of plane surface members in the direction perpendicular to the reinforcement for direct loads is designed according to the principles of secondary reinforcement. Misunderstandings of the indirect actions lead to under-dimensioning of these reinforcement and formation of separating cracks with uncontrolled cracks width. The area of the reinforcement for indirect actions is significantly greater than that of the secondary transverse reinforcement and often greater than the area of reinforcement for direct actions. This should be taken into account especially in watertight concrete structures.

Based on the above mentioned facts and a large number of failures caused by unconsidered indirect actions is threatening the serviceability and durability of concrete structures. It is obvious, that the design for indirect actions requires an evaluation of restrained deformations, as the main cause of **early-age** crack formation. To reduce the occurrence of cracks caused by volume changes, should be used a combination of constructive, technological and executional measures. The width of separating cracks can be controlled by the reinforcement design according to EN 1992-1-1.

In structures subjected to hydrostatic pressure the separation cracks, of any size, can form a water path, which may result in leakage or wet spots. It is the responsibility of the designer to limit crack widths to a predetermined size to restrict or prevent water from leaking through the concrete. The most effective method to control **crack widths due to** restrained volume changes is the provision of the required amount of reinforcement. The design approach for early-age thermal cracking adopted by STN EN 1992-1-1:2006 is broadly similar to that of DIN EN 1992-1-1/NA: 2010 but there are some significant and important differences. Read more in the paper on page 85.

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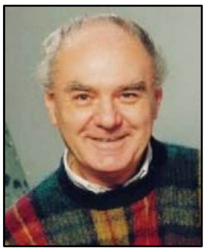

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I. NEW AT DeCoSaB

I.1 Defenses of the Doctoral Theses

HOLLÝ, I.: Effect of Steel Corrosion on the Bond Between a Reinforcement and the Concrete
Supervisor: Bilčík, J.

KIŠAC, M.: Nonlinear Analysis of the Safety of Slender Concrete Columns
Supervisor: Benko, V.

LACO, J.: Bonds of Prestressing Units Coated with Corrosion Protection Agents
Supervisor: Halvoník, J.

I.2 Awards Members of the DeCoSaB

On the session of SNK *fib* **prof. Eng. Juraj Bilčík, PhD.** has been awarded the award of SNK *fib* for the lifetime achievement in the development of the concrete structures and bridges in Slovakia.

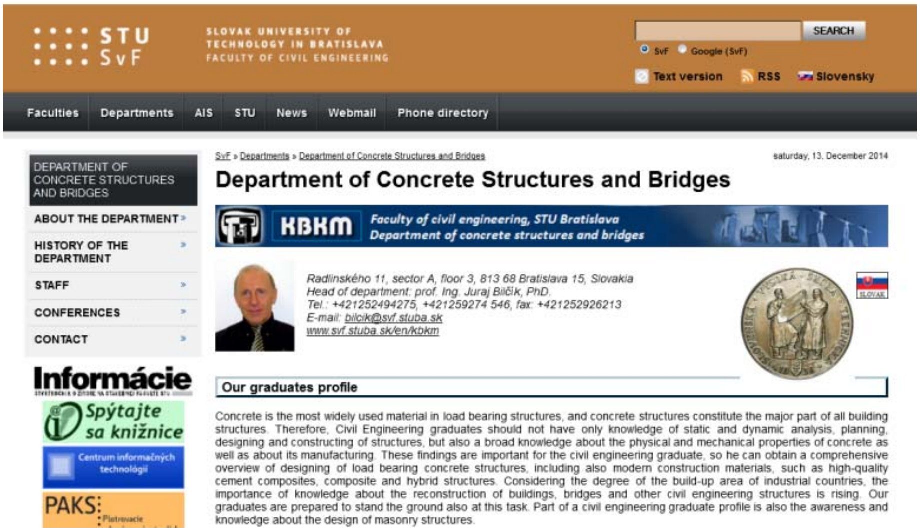
Prof. Eng. Jaroslav Halvonik, PhD. has been awarded the award of Academic Havelka by the dean prof. Eng. Alojz Kopáčik, PhD. of the Faculty of Civil Engineering on the occasion of his 50 birthday anniversary.

On the opening of Concrete Days 2014 **Assoc. prof. Eng. Ivan Harvan, PhD.** has been awarded the award of Academic Havelka by the dean prof. Eng. Alojz Kopáčik, PhD. of Faculty of Civil Engineering on the occasion of his 70 birthday anniversary.

I.3 Updated Webpage of the DeCoSaB

In 2014 English version of our department web page has been updated. Among other things the brief history of the department could be found there dated back to 1938 as well as the information about the famous concrete ruler developed by prof. Havelka.

Our English upgraded web page is www.svf.stuba.sk/en/kbkm Executive editor of the web page is Eng. Peter Paulík, PhD. and the translations from slovak language were done by Eng. Andrea Hrušovská and Eng. Miroslav Ignačák.



II. RESEARCH TARGETS

The research activities of the Department are focused on new design methods for reinforced, prestressed and composite structures, ultimate limit design of concrete structures for durability, methods of repair and strengthening of building structures and bridges and utilization of high-performance and fiber concrete for concrete structures and precast elements.

III. RESEARCH PROJECTS

1.) VEGA 1/0784/12, Holistic Design of Concrete Constructions (2012-2014, BILČÍK, J.)

2.) VEGA 1/0690/13, Diagnosis of Oldest Reinforced Concrete Bridges in Slovakia (2013-2015, HALVONÍK, J.)

3.) APVV 0442-12, Historical Experience and Current Requirements for the Design of Concrete Bridges and Knowledge Transfer of Information Acquired in Technical Practice, (2013-2016, HALVONÍK, J.)

4.) VEGA 1/0696/14, Reliability and Resistance of Concrete and Composite Steel-Concrete Structures (2014-2016, FILLO, Ľ.)

IV. EVENTS

Excursion of Our Students

LEIER, Bratislava, Slovakia
28-th April, 2014

The excursion is included into compulsory subjects for Structures of Buildings and Structural and Transportation Engineering fields of study. Excursion was organized to a company Leier in Hungary. Students visited in the company Leier the Blockyard.

Conferences we visited

SMSB-XI 2014 Conference, Calgary, Canada, 15-18-th July, 2014

The Calgary bridge engineering community organized the Canadian Society for Civil Engineering's 9th International Conference on Short and Medium Span Bridges, SMSB-IX. Since 1982 the International Conferences on Short and Medium Span Bridges have been held in Canada to provide a worldwide state-of-the-art forum on all aspects of short and medium span bridges (i.e. spans less than 150 m). Bridge engineers, researchers, contractors, and owners from around the world were again enjoyed this unique opportunity to discuss recent practices and new research and development concerning all types of short and medium span bridges.

Bridge Days, Visegrad, Hungary 26-27-th November, 2014

Eng. Peter Paulík, PhD. had an invited lecture at the conference: Bridge Days (Hidász napok), which was held in Visegrad (Hungary) on 26th November 2014. The topic of his lecture was the reconstruction of the oldest reinforced concrete bridge in Slovakia. Our department has participated on the reconstruction and performed numerous scientific measurements on the structure in cooperation with T.U. Žilina and TSUS Bratislava (Building testing and research institute). This two span structure constructed as a "monier arch" was built in 1891 and served without any repair for more than 124 years.

Czech Concrete Days, Hradec Králové, Czech Republic, 26-27-th November, 2014

The 21st Czech concrete days organised by Czech concrete society offered a comprehensive overview of current developments in the construction sector. Czech concrete days are a communication and innovation platform for experts in the field of concrete and construction technology.

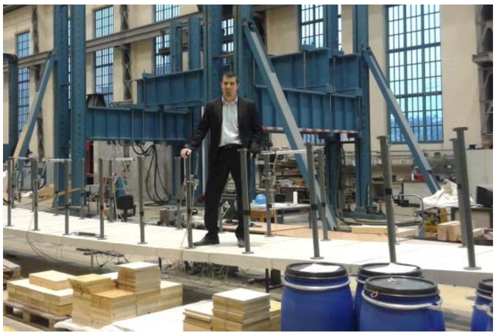
PRB-Workshop, Berlin, Germany, 4-5-th December, 2014

Eng. Peter Paulík, PhD had attended the First PRB-Workshop on Contributions for the Ease of Use of the EUROCODES, which took place at hotel Berlin in Germany on 4th and 5th December 2014. After the workshop he visited the office of the SBP company (Schlaich Bergermann und Partner), where he had a short presentation about the most interesting bridges in Slovakia. Later on that afternoon accompanied by Dr. Ing. MSc. Boris Reyher (SHP employee), he also visited the nearby laboratory of. Photo: Eng. Peter Paulík, PhD. on experimental bridge in the laboratory of T.U. Berlin.


CSCE
CGC
www.smsb2014.ca


KONEČNÁ POZVÁNKA
Konference
s mezinárodní účastí
21. BETONÁŘSKÉ
DNY 2014
spojené s výstavou
BETON 2014


PRB
FORUM
Proceedings of the
First PRB-Workshop
on Contributions
for the Ease of Use
of the Eurocodes
Beuth



Fourth International *fib* Congress 2014

The Fourth International *fib* Congress took place at Mumbai (India) between 10 – 13th February 2014 (*fib* - Fédération internationale du béton / International Federation for Structural Concrete). At 4-day congress and exhibition experts from 42 countries had an opportunity to meet each other, discuss and attend some of the 50 specialized sections in which 250 professional lectures were given by engineers and scientists.

Slovakia became a permanent member of the organization in 1994, when also the Slovak National Committee (SNK fib) has been founded. As at previous *fib* congresses: Washington 1994, Amsterdam 1998, Osaka 2002, Naples 2006 and Washington 2010, we have prepared the national report "Concrete in Slovakia 2010-2014", which was published as a special issue 06/2013 of Civil Engineering magazine (Inžinierske stavby). This national report was distributed at the congress for free of charge among the "national reports" and was presented in 30 minute lecture given by assoc. prof. M. Chandoga (President of SNK *fib*) and Eng. P. Paulík, PhD. (secretary and deputy of SNK *fib*).

In the lecture the most significant concrete structures and bridges built in the last 4 years in Slovakia were presented. Also other members of our department had professional lectures at the congress concerning various themes. Namely, they were prof. J. Halvoník, prof. Ľ. Fillo, PhD., assoc. prof. Š. Gramblička, assoc. prof. J. Šoltész and assoc. prof. V. Borzovič. The SNK *fib* and our department, in cooperation with SATUR travel agency, have also prepared a short study and sightseeing program for Slovak participants. Within this program, we visited the University of Technology in New Delhi, where we had the opportunity to see their methods of teaching and their research in the field of concrete structures.

Concrete Days 2014

10th International Conference

The 10th anniversary Conference Concrete Days 2014 took place from October 23 – 24th in Bratislava, Slovakia. The novelty of the jubilee conference was the connection with the 5. Post-congress colloquium of the Slovak National Committee *fib* (fédération internationale du béton). The participants praised the idea to join two professional events (they received two shares for the price of one). It is a trend that will be certainly followed in the future. Conference and colloquium was organized by the Department of Concrete Structures and Bridges.

The tradition of organizing this event was founded in 1994 and the conference is organized regularly in two-year period.

Concrete Days 2014 (CD) offered a comprehensive overview of current developments in the concrete construction sector. Recognized experts and specialists in design and execution of concrete structures from Slovak and Czech Republic participated, thus offering a unique opportunity to share their experience and acquired knowledge. Totally, the meeting took place in 17 sessions.

From 57 orally presented papers particular attention was offered to the contributions in the section Keynote Speakers. Prof. Hugo Corres from Technical University of Madrid presented the analysis of recent achievements in tall building design.

Dr. Miguel F. Ruiz introduced the elliptical concrete shell structure with the dimensions 95 x 52 x 22 m of the shopping centre in Chiasso (Switzerland).

The lecture of Prof. Jiří Stráský from Brno was devoted to the architecture and design of new flat arch bridges. Multispan arch bridges were built under his guidance in Oregon (USA) and the Czech Republic.

Eng. Pavel Čížek is considered as the doyen of prefabricated constructions in Czech and Slovak Republic. He pointed out the undeniable advantages of precast constructions and reported on the construction of industrial and sports halls built in recent years in Czech Republic.

High-rise buildings present unique challenges in terms of both design and construction. There were built in Bratislava more than 30 buildings with 18 up to 33 storeys. Based on the examples of high-rise buildings Millennium Tower II, Central and Panorama City in Bratislava indicated Eng. Daniel Bukov the key issues and possibilities for the verification of their reliability. Finally, at the end of the section Keynote Speakers President of the Slovak Chamber of Civil Engineers, prof. Vladimír Benko presented an analysis of the problems associated with the proposal of the new Slovak Building Act.

This information should not be a formal summary of contributions presented at the conference and that is why I would like to highlight the contribution of a young man with "passionate about concrete" despite (or just because?), that he is not a graduate of civil engineering. He got to concrete through skateboarding and building of skateparks. In his contribution states: “Concrete is my favourite material and work with him makes me happy. I didn't get my knowledge from books, **I have developed my skills through** many years of practical experience. Regarding the skateparks, concrete is an ideal material due to several reasons: - **the noise level during skating is the smallest** of all materials used in the construction of skateparks. **The longest service life** – a wooden surface will last for two years, iron surface rusts. Concrete skateparks will last about thirty years. Also the maintenance is simple. When the concrete is properly placed, consolidated, and cured, the only thing to do is to sweep debris from the surface."

The discussion continued during a social evening organized on 23rd October varied with a performance of folk music of ensemble "TECHNIK".

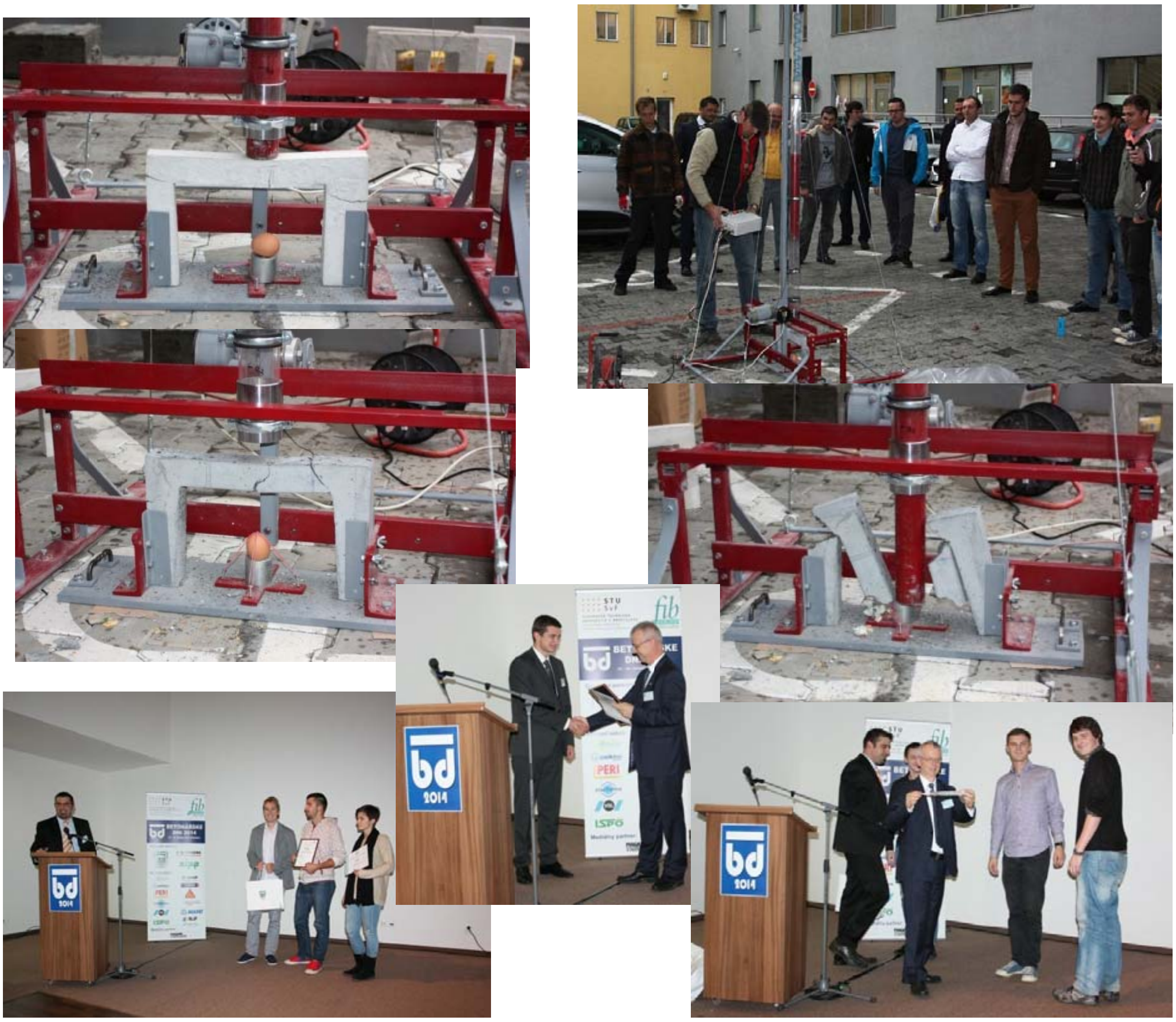
On the balcony of conference room, the accompanying exhibition of well-known companies from construction industry from Slovak and Czech Republic took place.

At the end of the second conference day, an excursion to Panorama City - building under construction was organised. The twin towers are located near the river Danube, with 34 storeys and to height 135 m they dominate the skyline of Bratislava.

Egg Protection Device Competition

As a part of the conference “Concrete days 2014” our department organized the student competition named “Save the Egg with Concrete” also known as the “Egg Protection Device Competition”. The main aim of the competition is to design a reinforced concrete frame according to prespecified criteria. Then the egg is placed under the frame and the puncher of a given weight is being released on it from an increasing height. The winner frame is the one, which can withstand the impact from the greatest height ..., plainly, this structure could best protect the egg placed underneath the frame. The frames may vary in shape and in reinforcement layout within the given constraints. There were two main categories: reinforced concrete and fibre reinforced concrete. Winner teams were awarded by financial prize, diploma and books. The winner teams were: in the category of reinforced concrete: “Betonári” from STU Bratislava, Slovakia (Eng Peter Kendický, prof. Dipl.-Ing. Dr. Vladimír Benko, PhD., Mária Kendická) and in the category of fibre reinforced concrete: “Míchačky“ from CTU Praque , Czech Republic (Bc. Michal Mára, Eng. Radoslav Sovják, PhD., Eng. Petr Máca, Eng. Jindřich Fornůsek, Bc. Michal Tvarog). The general sponsor of the event was the Slovak Chamber of Civil Engineers (SKSI) and the main organizers were Eng. Peter Paulík, PhD. and Eng. Peter Pažma from STU Bratislava in cooperation with TU Žilina.

Webpage of the event:
http://betonarskedni.sk/zachran-vajce-betonom



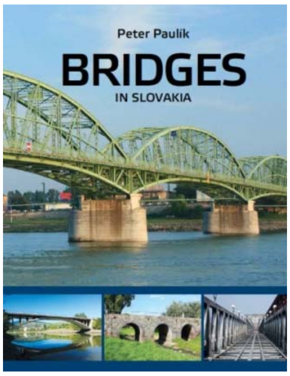
Saying goodbye to year 2014

Christmas session

At the end of year 2014, the members of our department as traditionally met all together around a small Christmas tree just few days before Christmas. Once again, we spent one afternoon together. Talking about every event and memory we spent with colleagues. At the beginning, the head of the department, professor Bilcik, gave a speech with the consideration "What is the happiness?"

Interesting publications of 2014

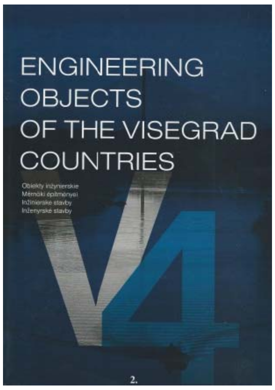
Bridges in Slovakia



In Slovakia, there are many remarkable bridges which represented the peak engineering ability of their era. However, many of them were gradually forgotten even though some of them were unique even on European, or global level. One of the main tasks of the book Bridges in Slovakia is to show them to the general public and point out, that although Slovakia is a relatively small country it has many bridges which we can be proud of them. The book contains information about approximately 250 bridges in Slovakia, which were selected because of their aesthetic, technical, or historical value. On the opening pages there are some basic information about the fundamental statical behaviour of different kind of bridges and the basic construction methods. The second, significantly larger part of the book is then devoted to the selected bridges. By every bridge you can always find its GPS coordinates as well as a short history of the bridge and its basic parameters. The book has 271 pages with 494 color photographs of the current state of bridges, as well as 175 archival photographs, diagrams and drawings. PDF of the book is available for free on this website:
www.mostynaslovensku.sk/en

Former Slovak president Rudolf Schuster gifted by prof. Vladimír Benko with the Slovak version of the book.

Engineering Structures of V4 (2nd Part)



In 2014 the second book concerning the most interesting civil engineering structures of the Visegrád countries (Slovakia, Czech Republic, Hungary and Poland) has been published (1. book has been published in 2012). Each country from the Visegrád Group has chosen 6 representative civil engineering structures (bridges, tunnels, dams .... etc.), which were built in their country in the last 24 years. Book is written in 4 languages (Slovak/Czech, Hungarian, Polish and English). Our department has been also involved in the preparation of this book since Prof. Eng. Vladimír Benko, PhD. is the head of the Slovak Chamber of Civil Engineers and Eng. Peter Paulík, PhD. was the editor of the Slovak section of the book.

V. COOPERATION

V.1 International Cooperation

1.) Klokner Institute ČVUT Prague, Czech Republic
2.) ETH - Laboratory for Building Materials, ETH Zurich, Switzerland
3.) Institut für Baustatik und Konstruktion, ETH Zurich, Switzerland
4.) Baustoffinstitut, TU Munich, Germany
5.) Department of Civil and Materials Engineering, University of Illinois at Chicago, USA
6.) RIB Bausoftware, Stuttgart, Germany
7.) Betosan, s.r.o., Prague, Czech Republic
8.) European Commission, DG Research, Brussels, Belgium
9.) Imperial College for Science, Technology and Medicine, London, U.K.
10.) St. Paul University, Brussels, Belgium
11.) Fachhochschule Braunschweig – Wolfenbütel, Germany
12.) Institut für Massivbau, TU Darmstadt, Germany
13.) Fachhochschule Coburg, Germany

V.2 Membership in International Professional Organizations

1.) BENKO, V.: Austrian Standard Institut. Member of Standard Commitees – ON-AG 01301, ON-AG 1011 01, ON-AG 010 01, ON-AG 176 02. ON-K 176, ON-K 010
2.) BENKO, V.: ECEC European Council of Engineers Chamber – Delegate
3.) BENKO, V.: ECCE European Council of Civil Engineers – Delegate
4.) BENKO,V.: ASCE American Society of Civil Engineers
5.) BILČÍK, J.: American Concrete Institute
6.) FILLO, Ľ.: Representative of the Slovak Republic in CEN TC 250 – SC2 Eurocodes – Design of Concrete Structures
7.) FILLO, Ľ.: Member of Task Group *fib* TG 1.1 Design Application
8.) FILLO, Ľ.: Honorary Member of Czech Concrete Society, Hradec Králové, Czech Republic, 25.11.2009
9.) HALVONÍK, J: Representative of the Slovak Republic on CEN TC 250 – SC1 Eurocodes – Actions on Structures
10.)HALVONÍK, J: Representative of the Slovak Republic on CEN TC 250 Eurocodes

VI. PUBLICATIONS

VI.1 Books and Textbooks

Books

1.) BENKO, V.: Erdbebenlasten Eurocode 8: Praxisbeispiel Brücke aus Stahlbeton. 2. updated edition, Vienna: Austrian Standards Plus, 2014, 83 pp. ISBN 978-3-85402-296-1 (in German)

2.) BILČÍK, J. – GAJDOŠOVÁ, K.: Design of Concrete Members, 1st ed. Bratislava: Slovak University of Technology in Bratislava 2014, 134 pp. ISBN 978-80-227-4125-5 (in English)

3.) HARVAN, I.: Reinforced Concrete Structures: Design According to European Codes. 3. corrected ed. Bratislava: Slovak University of Technology in Bratislava 2014. 292 pp. ISBN 978-80-227-4142-2 (in Slovak)

4.) PAULÍK, P.: Bridges in Slovakia. 1st ed. Bratislava: Jaga Group, 2014. 259 pp. ISBN 978-80-8076-111-0 (in English)

VI.2 Journals

Scientific Papers Abroad

1.) BELLOVÁ, M.: Determination of the Compressive Strength of Masonry According to the Eurocodes. In: Materiály pro stavbu. Vol. 20, No. 8 (2014), pp. 54-56. ISSN 1213-0311 (in Slovak)

2.) BELLOVÁ, M.: Design Resistance of Masonry Members in Agreement with the Eurocodes. In: Materiály pro stavbu. Vol. 20, No. 9 (2014), pp. 44-46. ISSN 1213-0311 (in Slovak)

3.) BENKO, V. – KIŠAC, M. – KENDICKÝ, P. – STRAUSS, A. – ŠALÁT, T. – LAŠÁN, Ľ.: Prediction of Resistance of Slender Concrete Columns at Stability Failure. In: Beton. Technologie - Konstrukce - Sanace. Vol. 14, No. 1 (2014), pp. 75-79. ISSN 1213-3116 (in Slovak)

4.) BENKO, V.: Statics Control in the Slovak Republic. In: Beton. Technologie - Konstrukce - Sanace. Vol. 14, No. 2 (2014), p. 2, ISSN 1213-3116 (in Slovak)

5.) BILČÍK, J. – PRIECHODSKÝ, V.: Assessment and Diagnostics of Concrete Structures. In: Beton. Technologie - Konstrukce - Sanace. Vol. 14, No. 3 (2014), pp. 3-8. ISSN 1213-3116 (in Slovak)

6.) BILČÍK, J. – GAJDOŠOVÁ, K.: Principles of Strengthening Concrete Structures with Bonded CFRP Reinforcements. In: Beton. Technologie - Konstrukce - Sanace. Vol. 14, No. 3 (2014), pp. 68-72. ISSN 1213-3116 (in Slovak)

7.) FILLO, Ľ. – HALVONÍK, J. – BORZOVIČ, V.: Punching of Flat Concrete and Foundation Slabs. In: Proceedings of Scientific Works of Mining School of University of Technology in Ostrava. Vol. 14, No. 1 (2014), pp. 17-24. ISSN 1213-1962 (in Slovak)

8.) GRAMBLIČKA, Š. – FRÓLO, J.: Composite Steel and Reinforced Concrete Columns with Full Steel Cores. In: Konstrukce. Vol. 13, No. 3 (2014), pp. 37-42. ISSN 1803-8433 (in Slovak)

9.) GRAMBLIČKA, Š. – ŽIVNER, T.: Structural Steel Construction for an Industrial Building in a Chemical Production Environment. In: Stavebnictví. Vol. 8, No. 11-12 (2014), pp. 51-56. ISSN 1802-2030 (in Slovak)

10.) HALVONÍK, J. – FILLO, Ľ.: Punching: the Reasons for Failure in a Complex Trinity. In: Proceedings of Scientific Works of Mining School of University of Technology in Ostrava. Vol. 14, No. 1 (2014), pp. 25-32. ISSN 1213-1962 (in Slovak)

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LOCAL STRESS OF REINFORCED CONCRETE ELEMENTS ACCORDING TO EN 1992-1-1

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Abstract

The goal of the article is to apply the European standard EN 1992-1-1 when appraising the element's bearing capacity in concentrated pressure under the anchoring slabs of pre-stressed reinforcement units. Local compressive load distribution model under the surface of the reinforced concrete element. Lateral tension in the local compressive load's distribution area. Reinforcement against tearing of the surface of elements. Application in the expertise of the class of concrete in foundations.

1 Strain under the anchors

In the areas below the anchoring slabs of the pre-stressed units, the concrete is under the effect of spatial state of stress, with the concrete's compressive strength being higher then under a single-axis strain in pressed elements. Vertical and horizontal stresses σ_c under the anchoring slab are illustrated in the fig. 1. The horizontal tensile stresses $^{(+)}\sigma_{c1}$ are those the reinforcement must endure. In the vertical direction, there is the danger of the force F_d crushing the concrete, as it generates concentrated compression on the examined area A_{c1}.

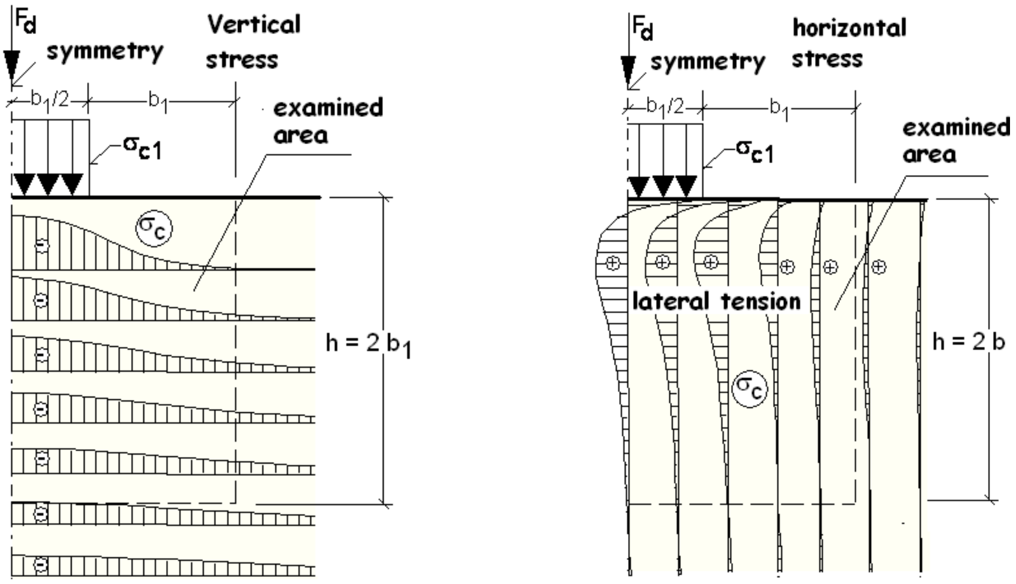

Fig. 1 Course of vertical and horizontal stresses σ_c under the anchoring slab in the examined design distribution area of stresses σ_c and the force F_d

$^{(-)}\sigma_{c1} = {}^{(-)}F_d / A_{c1}$. Design distribution of vertical compressive stresses σ_c we assume to have a pyramidal shape, as shown in the fig. 2. The concentrated effect of the force F_d is continually distributed from the examined area onto the rectangular area A_{c2}, which is in the depth h

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below the A_{c1} area. In case the area A_{c1} is circular in shape, the distribution of stresses σ_c is assumed to have a conical shape. On the distribution area A_{c2} we observe a single-axis effect of F_d with compression $^{(-)}\sigma_{c2} = {}^{(-)}F_d / A_{c2}$ When designing the distribution of compressive stresses σ_c caused by F_d, the following principles must be obeyed:

- The design distribution areas A_{c1}, A_{c2} must be similar in shape. The centers of both areas must lie on line below the point of effect of the force F_d.
- Widths b_2, d_2 of the inner distribution area A_{c2} must not exceed the real edges of the concrete element's section. With full (unlimited) strain distribution σ_c the measurements of A_{c2} cannot be bigger than $b_2 = 3\ b_1$ and $d_2 = 3\ d_1$. With partial strain distribution σ_c (limited by the edges of the element's section) the maximum values of the measurements can be $b_2 = 2\ b_r + b_1$ and $d_2 = 2\ d_r + d_1$.
- Height h of the designed distribution of F_d needs to meet all the conditions illustrated on the fig. 2. Overhang of the inner A_{c2} area's borders from the surface contact area A_{c1} is usually in 1:2 ratio to the examined area's height h. With rectangular areas we must determine h_b separately for the overhanging in the direction of b_1, b_2 and separately h_d for the overhanging in the direction of d_1, d_2. The design height of the vertical stress distribution σ_c is then defined as the lower value h_b or h_d.
- If multiple forces F_d act on the surface, their distribution areas A_{c1}, A_{c2} must not overlap.

We will follow these principles when creating the **computation model of the design distribution** σ_c below the anchoring slab. In the analysis of the concentrated effect of the force F_d we will use two models, one for calculating the lateral design tensile force T_{Ed} the direction b (fig. 3) and one for calculating the lateral design tensile force T_{Ed} in the direction d (principles are demonstrated on the fig. 3, we just have to switch the indexes b for d).

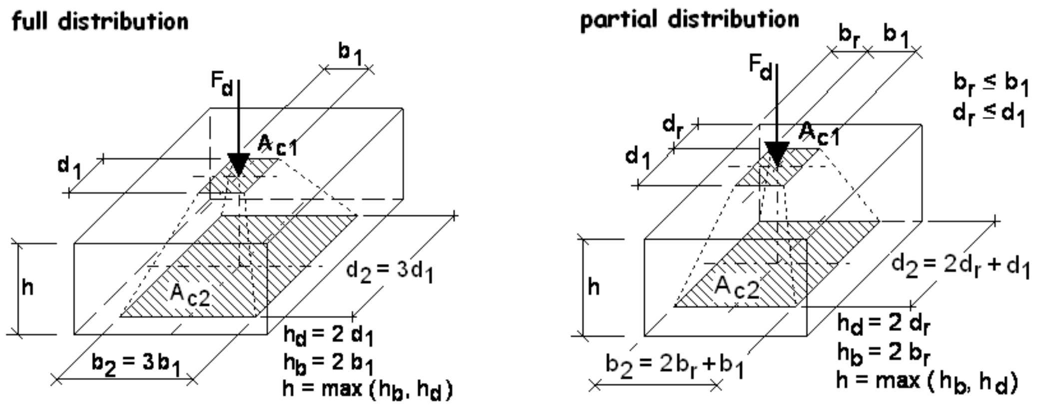

Fig. 2 Design distribution of vertical compressive stresses σ_c caused by F_d in pyramid shape

The design compressive strength of concrete under concentrated load is higher than the strength f_{cd} in a single-axis compression only if we manage to successfully deal with the lateral tension $^{(+)}\sigma_c$, using reinforcement in the distribution area. The increase in the strength of concrete can be defined by parameter ω_c. The area A_{c1} must meet the following condition

$$^{(-)}\sigma_{c1} = {}^{(-)}F_d / A_{c1} \geq \omega_c\ {}^{(-)}f_{cd} \qquad \omega_c = \sqrt{A_{c2}/A_{c1}} \quad \max\omega_c = 3{,}0 \qquad (1)$$

Compressive stress σ_{c1} from the element's surface is distributed in two directions b and d to the depth h (fig. 3) onto the surface A_{c2}, reducing the value of compressive stress σ_{c2}, where it acts as a compression of single direction. The area A_{c2} must meet the following condition

$$^{(-)}\sigma_{c2} = {}^{(-)}F_d / A_{c2} \geq {}^{(-)}f_{cd} \qquad (2)$$

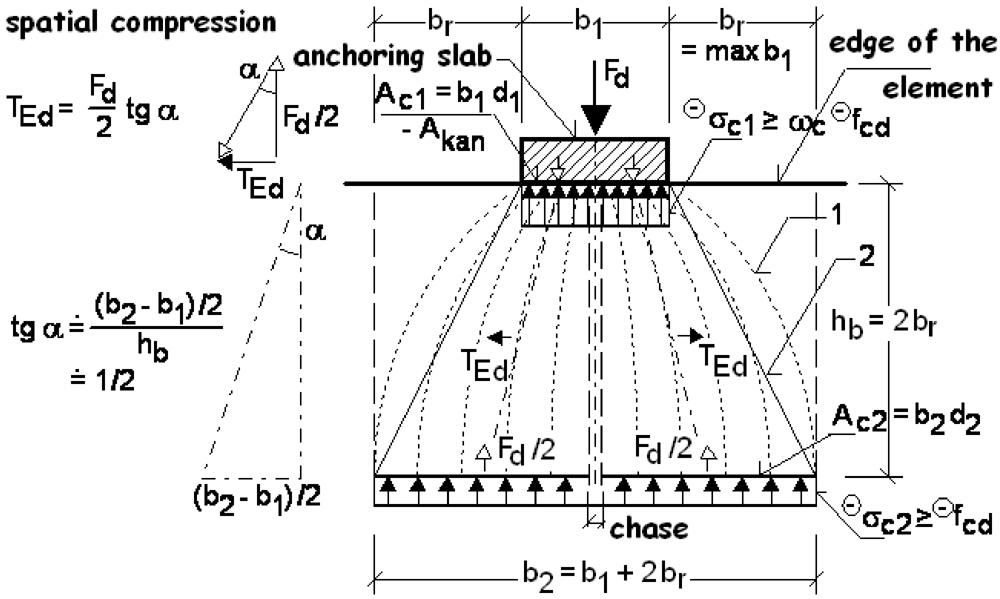

Fig. 3 Design model of the examined area of the vertical compressive stress σ_c distribution below the anchoring slab, the tensile force T_{Ed}, 1 – real distribution of the compressive stress σ_c, 2 - design distribution of the compressive stress σ_c

2 Method of concentrated pressure analysis

1. For concrete with design compressive strength f_{cd}, we first determine the necessary strength increase by means of the coefficient $\omega_c = {}^{(-)}F_d / (A_{c1}\ f_{cd})$ while the force F_d acts on the area A_{c1}. If the required value of $\omega_c > 3$, it is necessary to increase the class of concrete, or enlarge the contact area A_{c1}. If the required value of $\omega_c \leq 1$, it is not necessary to make an expertise of concentrated pressure.
2. We determine the distribution area $A_{c2} = A_{c1}\ \omega_c^2$, with its minimum value being $A_{c2} = {}^{(-)}F_d / {}^{(-)}f_{cd}$.
3. The shape of the area A_{c2} must be the same as the shape of the area A_{c1}. If the area A_{c1} matches the criteria $\beta = b_1 / d_1$, the sides of the distribution area $A_{c2} = b_2\ d_2$ must meet the same criteria, i.e. $\beta = b_2 / d_2$, or $b_2 = \beta\ d_2$. Thus we can state the following formulas for calculating the measurements of A_{c2} in such a way, that the lateral stress distribution σ_c is equal in the directions of b and d..
$A_{c2} = b_2 d_2 = \beta d_2 d_2 = \beta d_2^{\ 2}$ with $d_2 = \sqrt{A_{c2}/\beta}$ $b_2 = \beta d_2$
4. We confirm that the measurements b_2, d_2 meet the criteria of A_{c1} and A_{c2} placement above each other, i.e. if the stress distribution of σ_c from the force F_d is full, in the shape of the modeled pyramid (fig. 2 on the left). Full distribution occurs if the borders of the area A_{c1} are further from the edges of the element than the sizes b_1, d_1. If the borders of A_{c1} are closer to the edges of the element, only partial distribution of stresses σ_c occurs, in the shape of modeled pyramid, but limited by one or two edges of the element (fig. 2 on

the right. In that case, the designed distribution in the directions of b and d will not be equal and we have to correct the shape of A_{c2} according to the principles of partial distribution, so that we can achieve the required value of coefficient ω_c .

5. In order to analyze the effects of design lateral tensile forces T_{Ed} (or their characteristic values T_{Ek}) we determine the height h of design distribution of concentrated pressure below the contact area A_{c1} while taking directions b and d, into account. In the following calculations, we will use the value of $h = \max(h_b, h_d)$, with the partial heights in the directions of b, d will be $h_b = b_2 - b_1$, $h_d = d_2 - d_1$.

3 Method of performing expertise of the effects of lateral tensile forces T_{Ed}, T_{Ek}

The effect of lateral tensile forces we analyze separately for each of the directions b, d of the design distribution of σ_c. When considering the possibility of plasticized zones in concrete occurring, as well as the positive effects of shear stress in concrete right under the contact area A_{c1}, we can determine the overall designed and characteristic values of lateral tensile stresses in the zone of σ_c distribution using the following formulas:

for the direction b / for the direction d

$$T_{Ed,b} = \frac{1}{4}\left(1 - 0{,}7\frac{b_1}{h}\right)F_d \qquad T_{Ed,d} = \frac{1}{4}\left(1 - 0{,}7\frac{d_1}{h}\right)F_d$$

$$T_{Ek,b} = \frac{1}{4}\left(1 - 0{,}7\frac{b_1}{h}\right)F_k \qquad T_{Ek,d} = \frac{1}{4}\left(1 - 0{,}7\frac{d_1}{h}\right)F_k \qquad (3)$$

The basic condition, which grants the occurrence of spatial state of stress in the zone below A_{c1} (and related higher compressive strength, which equals $\omega_c f_{cd}$) is reliable transfer of tensile forces, which cause the appearance of cracks, within concrete. For these lateral tensile forces below the area A_{c1}, reinforcement A_{st} must be calculated, separately for both directions of σ_c distribution. The minimum section area A_{st} of the weaker reinforcement must be 25% of the opposite direction. Using the design value of reinforcement's yield strength f_{yd} we can determine the required reinforcement that can secure the lateral tensile strength. The area A_{st} has to be bigger than the minimum $A_{st,min}$. This means, that

for the direction b / for the direction d

$$A_{st,b} = T_{Ed,b} / f_{yd} \qquad A_{st,d} = T_{Ed,d} / f_{yd}$$

$$A_{st,b} \geq A_{st,min,b} \qquad A_{st,d} \geq A_{st,min,d} \qquad (4)$$

Minimum reinforcement $A_{st,min}$ can be obtained from the condition, which states maximum width of cracks w_{max} in the distribution area. It is

for the direction b / for the direction d

$$A_{st,min,b} = T_{Ek,b} / \sigma_s \qquad A_{st,min,d} = T_{Ek,d} / \sigma_s \qquad (5)$$

The highest strain σ_s in the reinforcement is stated in the chart. 1 and it depends on maximum crack width w_{max} in the design distribution area of σ_c. When designing the load-bearing constructions of civil structures, we take the value

w_{max} = 0,2 mm ... in the analysis of concentrated pressure in foundations and under the anchoring slabs of adhesive pre-stressed reinforcement,

w_{max} = 0,3 mm ... in the analysis of concentrated pressure in other cases.

The width of cracks depends on multiple factors. One of the most important factors is the biggest real diameter ϕ_s of the reinforcement rods used for elimination of tensile forces $T_{Ek,b}$, $T_{Ek,d}$ within the concrete. From the real diameter ϕ_s we can obtain the chart value $\phi_{s,tab}$ for checking the cracks width, with the coefficient k_{ref} being 2,0 at most.

$$\phi_{s,tab} = \phi_s \frac{2{,}9\,\text{MPa}}{f_{ctm}} k_{ref} \quad (\text{mm}) \qquad k_{ref} = \frac{8\,a_{s,min}}{0{,}7\,h} \qquad k_{ref} \leq 2{,}0 \tag{6}$$

Parameters in these formulas

f_{ctm} ... medium value of tensile strength of concrete,

$a_{s,min}$ ... lowest distance between the edge of reinforcement and the contact area A_{c1},

h ... height of the design distribution model of σ_c in the examined zone.

The highest acceptable strain σ_s in the reinforcement depending on the chart diameter $\phi_{s,tab}$ we can state using the chart 1. For the intermediate reinforcement diameters $\phi_{s,tab}$, we use linear interpolation. The lowest acceptable strain σ_s equals 160 MPa.

Chart 1 The values of acceptable strain σ_s in the reinforcement

Maximum diameter of reinforcement $\phi_{s,tab}$ (mm) for specific maximum crack width		Strain σ_s in the reinforcement
w_{max} = 0,3 mm	w_{max} = 0,2 mm	(MPa)
32	25	160
25	16	200
16	12	240
12	8	280
10	6	320
8	5	360
6	4	400
5	-	450

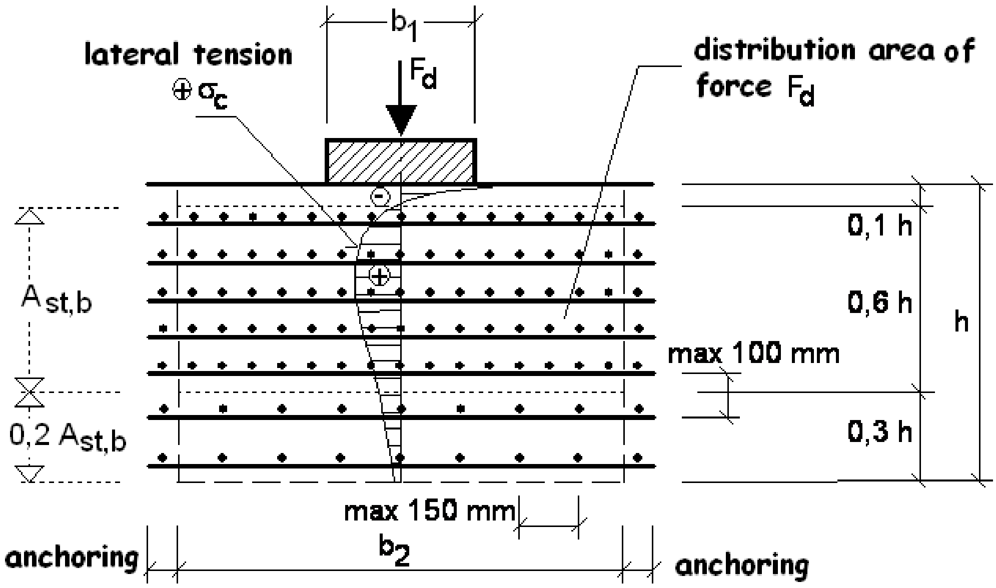

Fig. 4 Arranging the lateral reinforcement $A_{st,b}$ below the contact area A_{c1}

4 Design of reinforcement in the F_d force's distribution area

The reinforcement we usually design as welded trellis, or multiple enclosed stirrups placed in parallel with the contact area A_{c1} according to fig. 4. We arrange the main lateral reinforcement from 0,1 h to 0,7 h below the contact area A_{c1} closely enough to cover the whole lateral tensile stress $^{(+)}\sigma_c$ schema.

In the depth 0,7 h to h we place reinforcement according to the designing standards . It is necessary to anchor the lateral reinforcement over the side of A_{c2} contour, using one lateral rod of welded trellis or an enclosed stirrup. We can achieve a favorable envelopment of concrete within the examined design zone by doing so.

Failure caused by tearing the surface of concrete element occurs in case of eccentric location of A_{c1} away from the centre t_c of the concrete section according to the fig. 5. As for compressive strain in concrete, it is convenient if the distance b_r between the edge of anchoring slab and the edge of the element is bigger than the width of the anchoring slab b_1. If the force F_d acts with the eccentricity e greater than 0,1 h_c, cracks can appear on the surface of the element and we need to eliminate them by additional reinforcement with overall area A_{se}. The additional reinforcement can be added to the overall area of lateral reinforcement $A_{st,b}$. The value of A_{se} can be calculated from F_{se} using the additional reinforcement's design yield strength f_{yd}.

$$F_{se} = \frac{0{,}015}{1-\sqrt{(e-h)}} F_d \qquad \max F_{se} = 0{,}3F_d \qquad A_{se} = F_{se}/f_{yd} \tag{7}$$

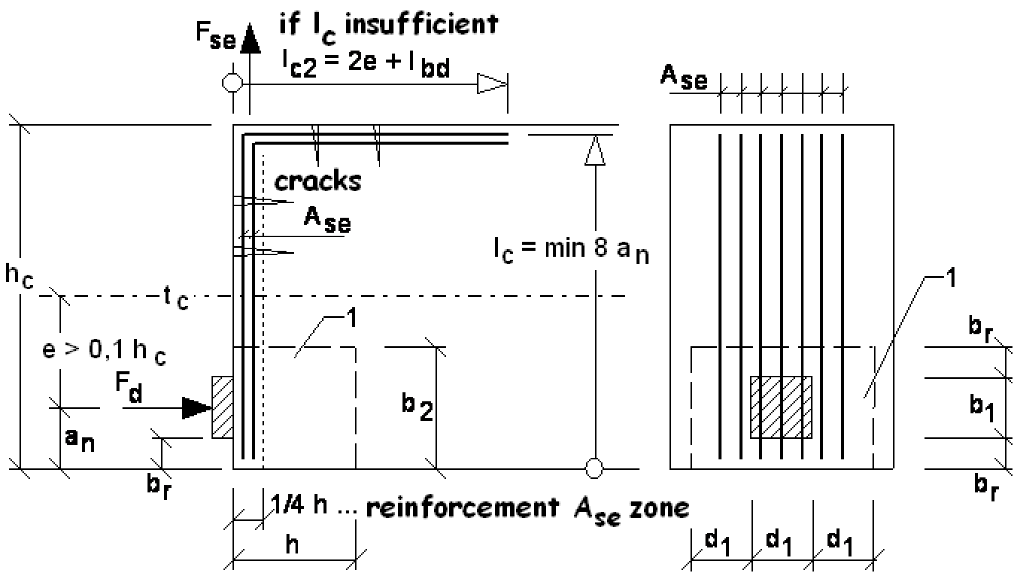

Fig. 5 Additional reinforcement A_{se} in case of eccentric placement of A_{c1}, 1 – distribution area of the F_d force

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Overall Reliability of the Concrete Columns in Case of Stability Loss

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Abstract

The European standard EN1992-1-1 shows a significant lack of an overall reliability in cases when loss of stability of concrete columns precedes reaching of resistance in a critical cross-section. The columns, subjected to axial force and bending moment, experimentally tested in the laboratory of Faculty of Civil Engineering SUT in Bratislava, were designed so that the loss of stability occurs at the concrete strain about 1.5 ‰ on the compressed edge of a critical cross-section.

Introduction

A semi - probabilistic method of reliability estimation of bearing elements and structures according to the European standards is based on using of partial factors of reliability. A required reliability index and hence also a probability of failure occurrence, is ensured by the partial safety factors. For ultimate limit states (ULS), loads effects are being increased by a partial safety factor γ_F, and resistances are being reduced by partial safety factors of material γ_M.

$$\gamma_F E_k \leq \frac{R_k}{\gamma_M} \tag{1}$$

At slender concrete columns, an increasing of axial compressive force causes an increasing of lateral deformations of a column, whereby a total eccentricity of axial force on critical cross-sections grows. Second order effect can be very large due to eccentricity increasing, so the loss of the stability of a compression concrete element can occur inside design interaction diagram *M-N* of a cross-section much earlier than capacity and structural materials strain limiting values are reached. In these cases, partial safety factor γ_M can be lost from the reliability format of the European standards. Therefore it would be appropriate to create new partial safety factor for the stability loss in order to ensure overall reliability in European standard.

Using of a nonlinear analysis, as defined in [1] clause 5.7 (4) P, demands to use mean values of the material properties, however the partial factors of reliability for the cases of stability loss does not define. The definition used in standard [1] "... but take account of the uncertainties of failure shall be used..." is insufficient and practically unusable.

A practical application of the general method is possible also according to [1] clause 5.8.6 (3) with using design values of the material properties. It is questionable, if also in this case the overall reliability of a compression structural elements is sufficient.

The goal of this article is to quantify the partial factors of reliability, as well as the overall reliability, in the case of use of nonlinear methods in accordance with [1].

Experimental verification

The aim of the experiments was to prepare set of RC columns with an initial eccentricity of axial force where the loss of stability occurs at the strain of about $\varepsilon_{c1} \cong 1.5$ ‰ in concrete in the critical cross-section.

The tested concrete columns (Fig. 1) have the rectangular cross-section with the dimensions of 150 x 240 mm. The total length of the columns is 3840 mm. The columns are reinforced by four bars with diameter of Ø14 mm. The transverse reinforcement consists of two leg stirrups with diameter of Ø 6 mm. With regard to the additional longitudinal reinforcement at the column's ends, the four-leg stirrups were used for an enhancement of the confinement effect. Concrete class C45/55 and steel B500B were used in RC columns. The production of the columns was ensured in cooperation with ZIPP Bratislava Inc.

Fig. 1 Tests of the columns in the laboratory FCE SUT in Bratislava

28 days after casting, the tests of the concrete were performed. The second set of the tests was carried out at the time of testing the columns. Compressive strength and the elastic modulus were tested in the laboratory of Testing and Research Institute in Bratislava.

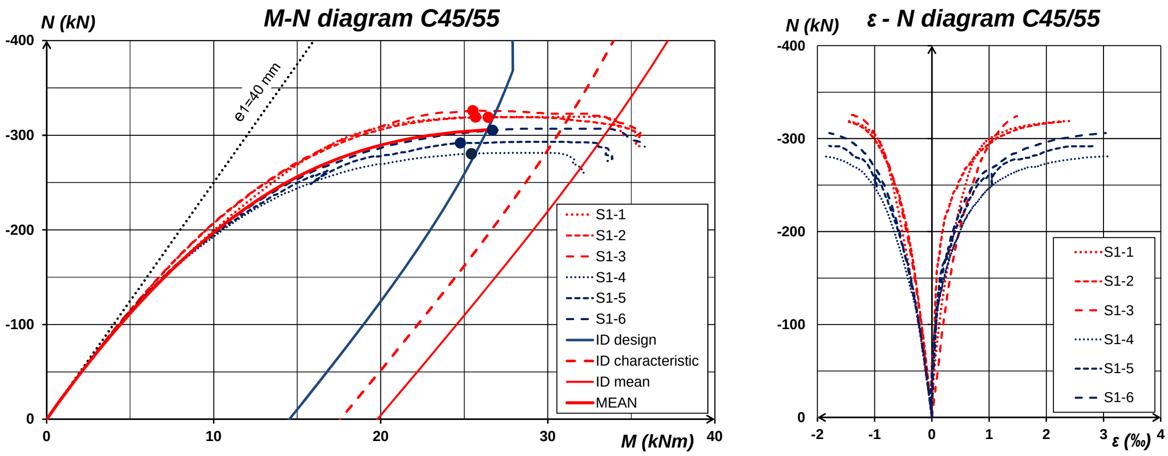

Fig. 2 Course of the tests of the columns S1 - 1 to S1 - 6 a) in Moment vs Axial Force diagrams b) in the Strain vs Axial Force diagrams

The columns were cast in the horizontal position. The loading by axial force was imposed with the initial eccentricity of 40 mm in the weaker inertial plane of the column cross-section. The columns S1 - 1 to S1 - 3 were loaded by axial force on the down cast side and the columns S1 - 4 to S1 – 6 on the upper cast side of the cross-section.

Overall reliability

Overall reliability can be also verified by further experimental studies [3, 4, 5, 6, 7]. The results of the experimental measurements are shown in Fig. 2a and Fig.2b. In Fig. 2a are points where the failure due to the loss of stability was recorded during the experiments. In Fig.2b is visualizing of relative deformations on compressed and tensioned edge of columns, when the loss of stability was reached. Results demonstrate that the loss of stability occurred inside of interaction diagram *M-N* at compressive strains between 1.5 – 1.8 ‰. These results point out that the stability loss occurred much earlier than the final resistance of critical cross-section has been reached; it means that the overall reliability calculated by formula (1) does not contain partial safety factor for material γ_M.

In Fig.3a is drawn curve representing a mean value of the experimental results. This representative curve has been used for calibration of non-linear models with mean values of material properties. Using reliability format as it is written in formula (1), overall reliability equals to partial safety factor γ_F only. So, the overall reliability is reduced to the reliability on the loads side $\gamma_O = \gamma_F = 1.40$.

In Fig.3b are shown results of nonlinear calculations with use of design values of material properties according to [1] clause 5.8.6 (3). The overall reliability has been increased by influence of difference between mean and design values, and equals to 1.28 in this case.

The overall reliability is defined with the partial safety factor γ_F on the loads side and the difference between the critical load for stability obtained with mean and design values of material properties. In our study it was 1.28 times more, what is evidently not enough for substitution of the partial factor on the side of materials and for ensuring of the reliability demanded by EN 1990, especially for brittle failure.

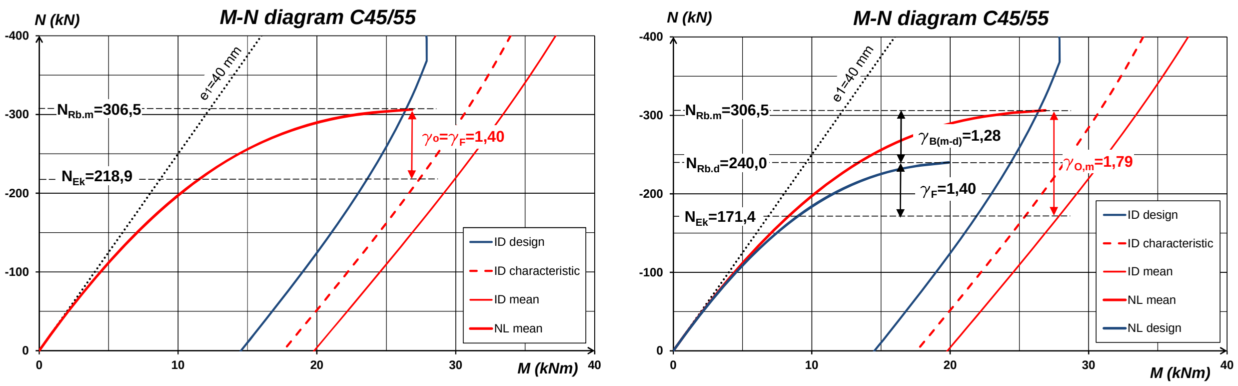

Fig. 3 Nonlinear analysis and the overall reliability of the column according to [1] – a) mean values, b) design values of materials

Summary

The executed experimental tests have verified the predicted results of the authors and the behaviour of the slender columns. The columns failed by the loss of stability before reaching the resistance of the cross-sections. I.e. the stability failure occurred inside design interaction diagram *M-N* of the column's cross-section at the strain of the compressed part of the column's critical cross-sections in the range of 1.5 – 2.0 ‰. For these cases, authors consider it as a necessity to define a partial safety factor for the stability loss failures of compression members. The importance of correct definition is enhanced by the fact of brittle failure without warning, which requires higher overall reliability than the ductile failures. The recommended partial safety factor for the stability loss failures of the compression elements can be found in the Austrian national annex [2]. Authors recommend additional tests of overall reliability of concrete columns in case of stability loss, verification of reliability index and index of probability of failure and parametrical studies which would help to define a missing partial safety factor for stability loss in nonlinear method calculations according to Chap. 5.7 a Chap. 5.8.

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Design of the Height of Flat Slabs

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Keywords: flat slab, punching, shear resistance

Introduction

Results of latest experiments have revealed that the maximum punching resistance defined from crushing of concrete struts at the perimeter of a column is an insufficient criterion for limitation of maximum shear forces at the vicinity of the columns. Therefore further limitation has been recommended by CEN TC250 SC2 and introduced in STN EN 1992-1-1/NA [1]. The paper will deal with the new requirements concerning the maximum punching shear resistance, based on the $k_{\max}$ factor and design of min flat slabs height.

Limits of Punching Resistance

There are two possible ways of structural failure due to the punching. The first one is strut diagonal failure (crushing of concrete) at control perimeter u_0 of the column. The second one is the shear-tension failure of concrete or transverse reinforcement in area surrounded by the basic control perimeter u_1.

The maximum shear force was limited by compressive capacity of the struts at the column perimeter up to now. The new limit is based on the punching resistance of a member without shear reinforcement $v_{\mathrm{Rd,c}}$, so the maximum punching resistance including effect of shear reinforcement at the basic control perimeter shall be less than value $k_{\max}\nu_{\mathrm{Rd,c}}$, even if $v_{\mathrm{Rd,cs}} > k_{\max}\nu_{\mathrm{Rd,c}}$ Eq.1.

$$v_{\mathrm{Rd,cs}} = 0{,}75 v_{\mathrm{Rd,c}} + \left(\frac{1{,}5.d}{s_{\mathrm{r}}}\right)\frac{A_{\mathrm{sw}} \cdot f_{\mathrm{ywd,ef}}}{u_1.d} \leq k_{max}.v_{\mathrm{Rd,c}}. \tag{1}$$

The latest experiments have shown that the failure depends on many factors. The first important factor is rotation ψ of a slab around the supporting area. Thicker flat slabs require smaller rotation for development of main reinforcement tension capacity and therefore there is a possibility to increase the value of $k_{\max}$ [2]. The most important factor is type of shear reinforcement and particularly conditions for their anchoring [3]. The best performance of shear reinforcement is ensured by double headed studs. Studs allow for full development of their tensile capacity just behind the heads. The $k_{\max}$ value is 1.9 for this type of studs with diameter of heads larger than three times the bar diameter. For the other types of shear reinforcement the $k_{\max}$ value depends on the effective depth of a slab d. The minimum value is 1.4 if $d \leq 200$ mm and maximum value 1.7 if $d \geq 700$ mm. For intermediate values of d a linear interpolation can be used.

Punching failure depends also on a position of column in plan of a building [4]. There are several types of column - slab contacts illustrated on typical flat slab plan - Fig.1. By taking into account the larger one "quarter" of a loading area, there is a possibility to substitute the influence of imbalanced bending moments, which also means applying the coefficient β =1. These loading areas for column and wall contacts with flat slabs is possible determine by global analysis of building. The limits for the loading areas on Fig.1 are defined by coordinates x_{yc},x_{zc},x_y and x_z. By increasing the slab height decreases value of these coordinates and the loading area of C column increases. The same increase of C loading area arises if the focus of analysis is the roof flat slab. For this decisive loading area, column C, graphs of relation between the design uniform distributed load and the heights of flat slab were created.

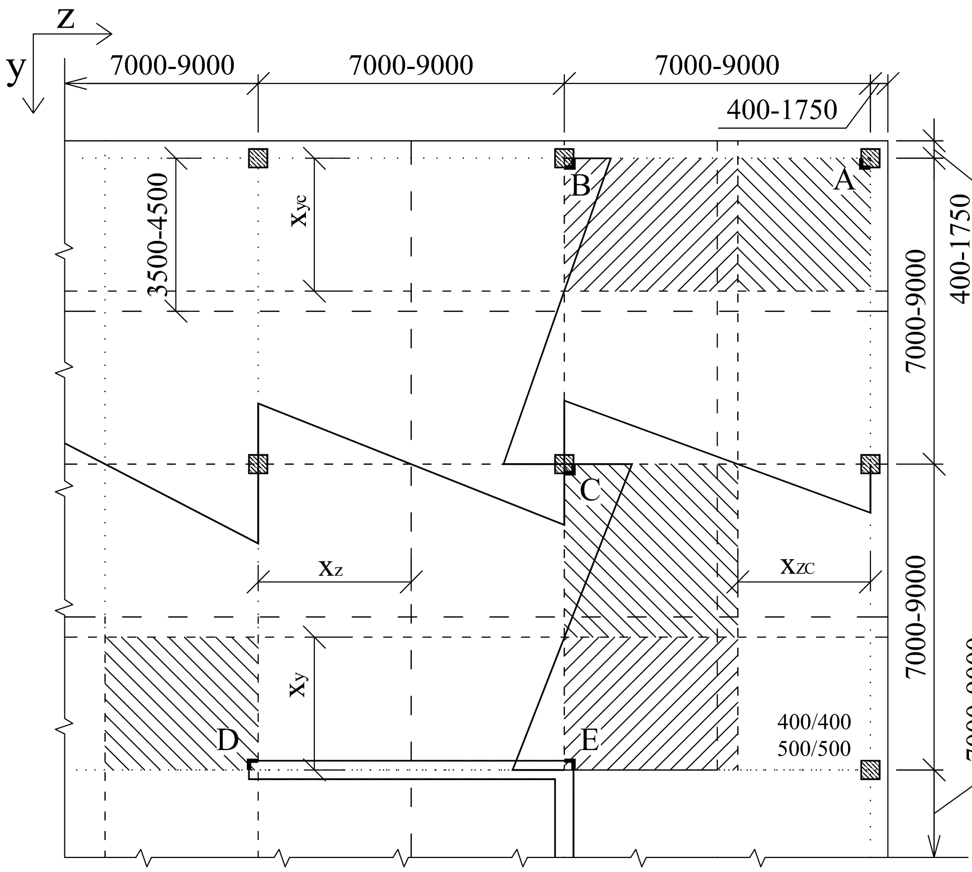

Fig.1 Analyzed plan of flat slabs

Analysis of min flat slab height

Coming from above mentioned limits for maximum punching resistance of a flat slab and surroundings of loading areas for different contacts of column slab there was a possibility to determine a relation between the uniform distributed load and the flat slab height. In our example it was a place of the column C (Fig.1) and possible real variables: the column distance (7 and 9 m), an intensity of uniform distributed load, the column perimeter (400 and 500 mm), the concrete class C30/37 and the amount of main reinforcement (reinforcement ratio $\rho = 0{,}01$ a $\rho = 0{,}005$).

The graph on Fig.2 was created for typical floor of the building Fig.1 with flat slabs and column distance of 7 m. To explain the chart is to say that e.g. by design of flat slab height 240 mm the shear-tension failure arises by load 17 kN/m^2 - $\rho = 0{,}005$ and 21,5 kN/m^2 - $\rho = 0{,}01$. The shear-tension failure precedes the failure of diagonal struts, which should arises by 25 kN/m^2. Diameter of main slab bars was ϕ18 and concrete cover 28 mm.

The graph on Fig.3 was created for roof top floor of the building Fig.1 with flat slabs and column distance of 7 m. To explain the chart is to say that e.g. by design of flat slab height 240 mm the shear-tension failure arises by load 15,5 kN/m^2 - $\rho = 0{,}005$ and 19,5 kN/m^2 - $\rho = 0{,}01$. The shear-tension failure precedes the failure of diagonal struts, which should arises by 23 kN/m^2. Diameter of main slab bars was ϕ18 and concrete cover 28 mm.

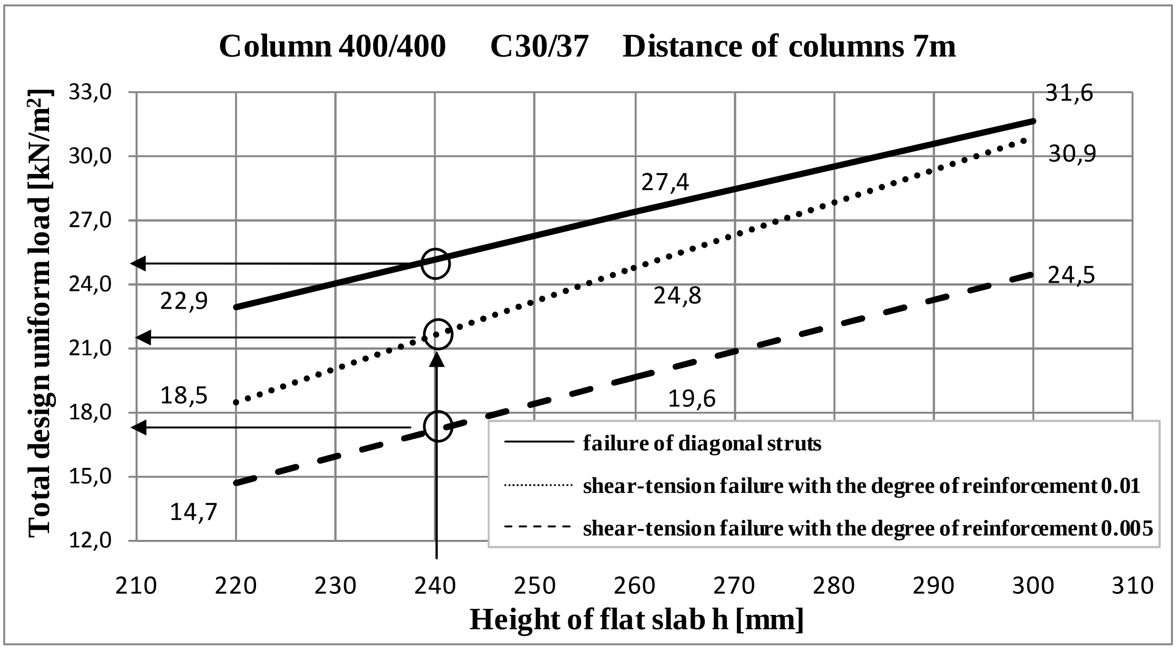

Fig.2 Minimal flat slab heights for distance of columns 7 m, typical floor (column C-Fig.1), C30/37, main horizontal reinforcement ϕ18, ρ = 0,01 a 0,005; k_{max} = 1,9.

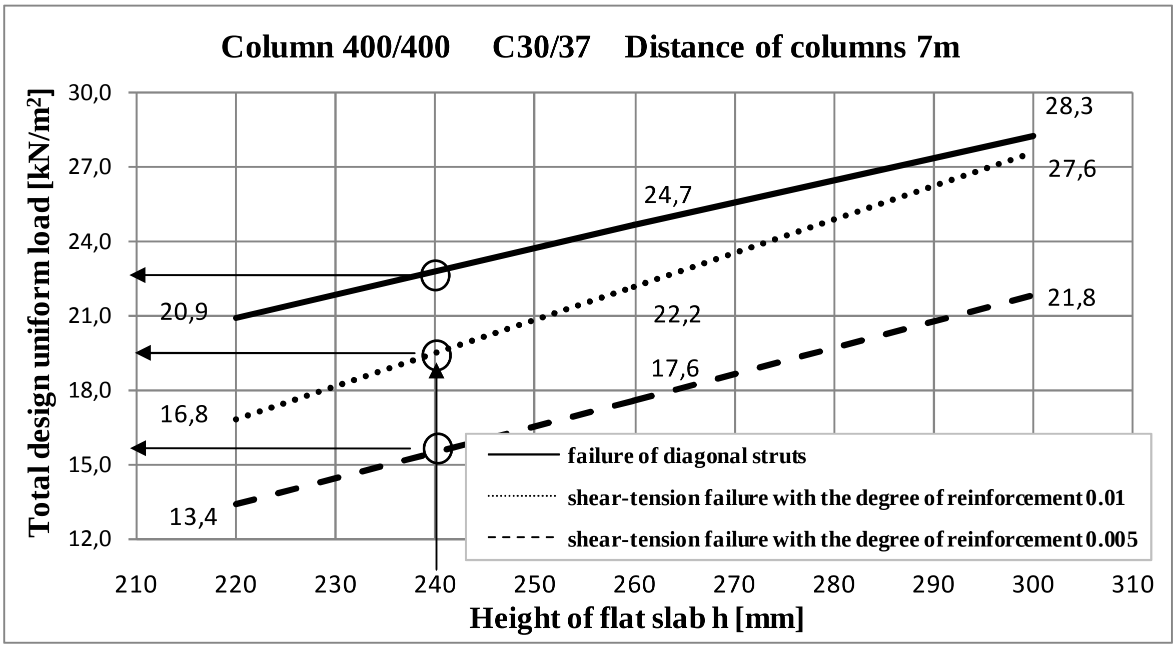

Fig.3 Minimal flat slab heights for distance of columns 7 m, roof floor (column C-Fig.1), C30/37, main horizontal reinforcement ϕ18, ρ = 0,01 a 0,005; k_{max} = 1,9.

The graph on Fig.4 was created for typical floor of building with flat slabs and column distance of 9 m. To explain the chart is to say that e.g. by design of flat slab height 280 mm the shear-tension failure arises by load 14 kN/m^2 - ρ = 0,005 and 17,5 kN/m^2 - ρ = 0,01. The shear-tension failure precedes the failure of diagonal struts, which should arises by 22 kN/m^2. Diameter of main slab bars was ϕ22 and concrete cover 32 mm.

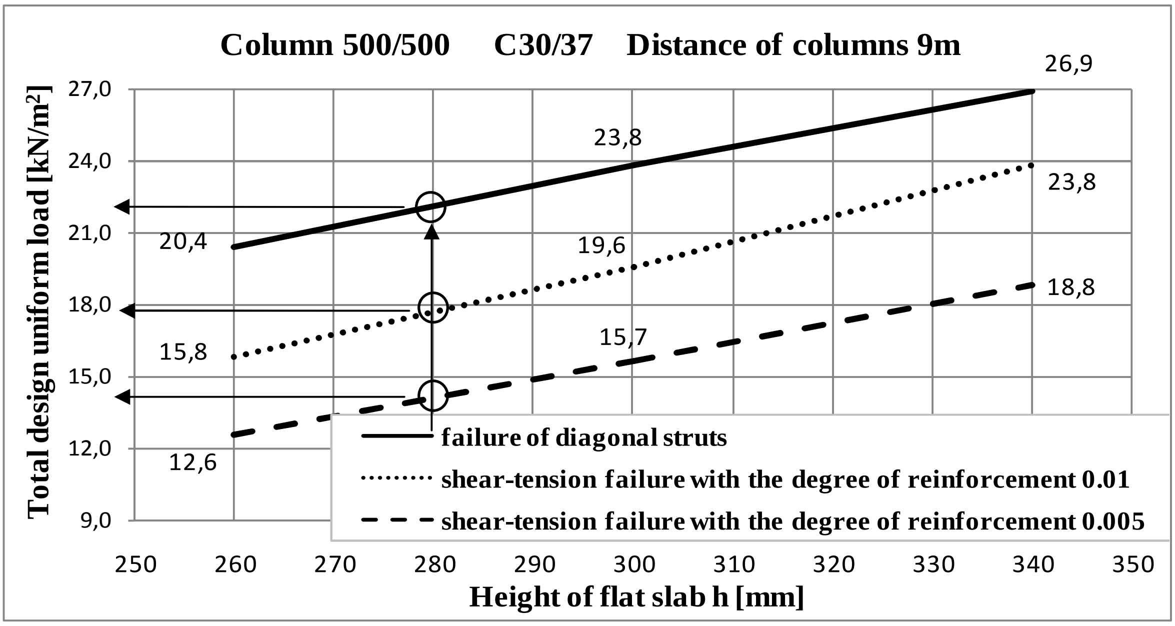

Fig.4 Minimal flat slab heights for distance of columns 9 m, (column C-Fig.1), C30/37, main horizontal reinforcement ϕ22, ρ = 0,01 a 0,005; k_{max} = 1,9.

Conclusions

The problem of flat slab punching is presented in the paper. Coming from limits of shear resistance defined in EN 1992-1-1/NA and recommended by CEN TC250 SC2 there was an option to create graphs for control of minimal necessary height of flat slabs depending on uniform distributed load, distance of columns, reinforcement ratios and both ways of shear failure. In the paper the typical plan of building with flat slabs for distance of columns 7 and 9 m is analysed - Fig.1. For two grades of reinforcement ratio 0,01 and 0,005 the surrounding of central column C was analysed. The presented analyses were carried out with k_{max} value of 1.9 where shear reinforcement consists of double headed studs. By all analysed cases the shear-tension failure precedes the failure of diagonal struts and so decides about the height of flat slabs .

Acknowledgement

Authors gratefully acknowledge Scientific Grant Agency of the Ministry of Education of Slovak Republic and the Slovak Academy of Sciences VEGA č. 1/0696/14.

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COMPOSITE STEEL - REINFORCED CONCRETE (SRC) COLUMNS – ANALYSES OF SLENDERNESS

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Abstract

Composite steel - reinforced concrete (SRC) columns are popular in tall buildings due to combining the rigidity of reinforced concrete with structural steel sections. There are a lot of types of SRC columns. We analyzed columns, which are completely or partially concrete-encased steel members. In practice, most composite SRC columns are relatively slender and in design the second - order effects will usually need to be included.

According to the results of the experiments, which were made in the Department of Concrete Structures and Bridges, Slovak Technical University of Technology in Bratislava (total of 18 columns were tested in two series), we analyzed the effects of the second - order theory. The experimental results were compared with theoretical from the model developed in the non-linear software Atena 3D.

The evaluation of the results is also shown in comparison with the general design method according to Eurocode 4, Design of composite steel and concrete structures - Part 1.1 : General rules and rules for buildings (EN 1994-1-1).

Keywords: Column, Composite, Experiment, Reinforced concrete, Second order theory, Steel

1 Introduction

Composite steel - reinforced concrete (SRC) column is defined as a composite member with components of concrete (better reinforced concrete) and structural steel. These two components act together to resist external forces. A composite column is mainly subjected to compression or to compression and bending. There is a wide variety of types of columns with various types of cross-sections. The most commonly used and studied are the two main types of typical cross-sections of composite columns (Fig. 1):

- completely or partially concrete-encased steel sections,
- concrete-filled rectangular and circular steel tubes.

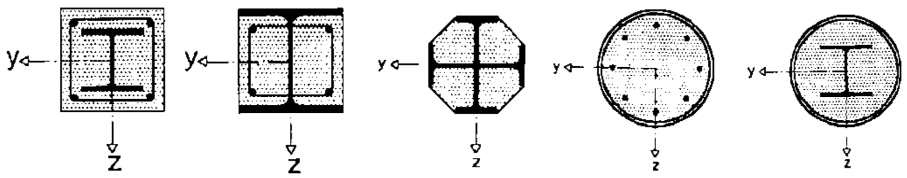


Fig. 1 Types of cross-sections of SRC columns

The research work was directed at an analysis of the design of composite steel-reinforced concrete columns. It was based on contemporary European codes, which use the latest knowledge gained from science and investigations. According to these standards columns can be designed only which are from normal weight concrete of strength classes C20/25 to C50/60 and from steel grades

S235 to S460. If the high strength concrete (HSC) is used in the composite column, the resistance will be greater than the resistance of the column with the use of normal strength concrete; respectively, we achieve a smaller size of the cross-section. In the slender composite column it is necessary to take into account the increasing of bending moments according to the second-order theory.

2 Resistance of composite SRC columns loaded by normal compressive force and bending moment

In order to verify the methodology of determining the resistance of the cross-section and take into account a second order theory, the experimental investigation was created. The objective of this investigation was:

- to verify the theoretical background of the design and check the composite steel-reinforced concrete columns according to EN 1994-1-1
- to analyze the effect of the second-order theory and to generate an interaction curve with the effect of slenderness.

In the EN 1994-1-1 code the second-order effects may be allowed for by using the factor *k*. We used the following relations for this experimental verification.

$$\left.\begin{aligned} M&=M^{I}+M^{II} \\ M&=k\cdot M^{I} \end{aligned}\right\} k = M/(M - M^{II}) \tag{1}$$

where

M is the failure bending moment,

M^{I} is the primary bending moment,

M^{II} is the increment of the bending moment through the effect of the second –order.

The value M^{II}, which presents increments of the bending moment according to the effect of the second order theory, may be calculated for constituent steps of the load according to the following relation:

$$M_n^{II} = N_n \cdot \sum_{i=1}^{n} \Delta w_i = \left(N_1 + \sum_{i=1}^{n-1} \Delta N_i \right) \cdot \sum_{i=1}^{n} \Delta w_i \, , \tag{2}$$

where

n is the step of the load,

N_n is the normal compressive load at the n –th step of the load,

Δw_i is the increment of deflection for the i-th step of the load,

ΔN_i is the increment of the normal compressive load for the i-th step of the load.

3 Experimental analysis of composite steel-reinforced concrete columns

At the Department of Concrete Structures and Bridges, Slovak University of Technology in Bratislava, were tested partially encased composite steel-reinforced concrete columns. The structural steel I section was HEA 280 and HEA 200, the longitudinal reinforcement was 4ϕ16 or 4ϕ14 (Fig.3, Tab.1). Total 18 columns with the lengths of 3 m and 4 m were tested. In the tests the normal compression forces were brought to the columns with the eccentricities from 30 to the 80 mm (Tab.1).

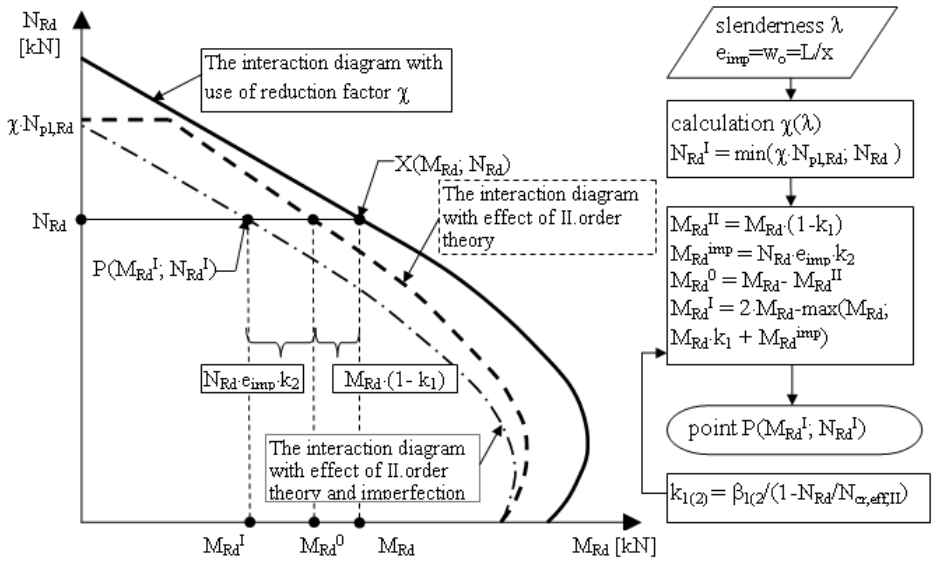


Fig. 2 The procedure of generating an interaction diagram with the effect of the second order theory

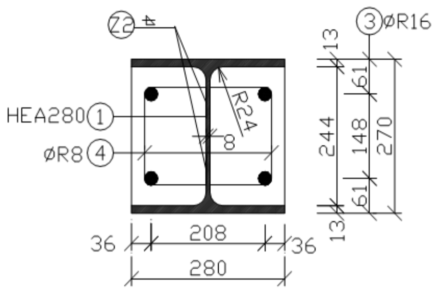


Fig. 3 The cross-section of tested composite Sx series columns and failure of the SRC column

Table 1

Specimen dimensions and material properties of tested composite SRC columns

Test	Dimensions			Steel section	e	Num. of columns	Material properties			Ref.
	b_c (mm)	h_c (mm)	L (mm)	Number and diameter of reinforcement	(mm)		f_{cd} (MPa)	f_{sd} (MPa)	f_{yd} (MPa)	
S1-40	200	190	3000	HEA200 - 4Φ14	40	3	65,45	566,25	305,46	[2]
S2-40	200	190	4000	HEA200 - 4Φ14	40	3	65,45	566,25	305,46	[2]
Sx1-30	280	270	3000	HEA280 - 4Φ16	30	3	36,34	569,4	395,8	[1]
Sx1-80	280	270	3000	HEA280 - 4Φ16	80	3	36,34	569,4	395,8	[1]
Sx2-40	280	270	4000	HEA280 - 4Φ16	40	3	36,34	569,4	395,8	[1]
Sx2-60	280	270	4000	HEA280 - 4Φ16	60	3	36,34	569,4	395,8	[1]

4 Analysis of composite SRC columns in software Athena 3D

The results from the non-linear calculation in computational program Athena 3D were compared with the experiments. With this accurate modelling in Athena 3D is possible to determine the real resistance of members. The width of cracks, deformation and crushing of concrete and the creep of concrete, respectively, can be considered in the analysis. The program is designed for non-linear analysis of structures and members by using the finite elements method.

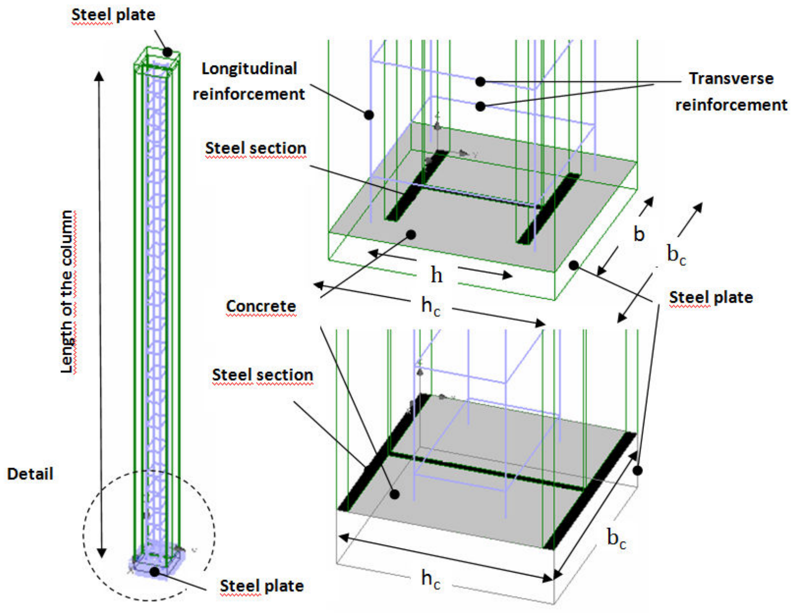


Fig. 4 Model of the composite SRC columns in software Athena 3D

The arrangement of the models of composite SRC column and the detail of the column end are shown in Fig.4. The column was modeled with hinged endings on both sides just as it was made in the experiments.

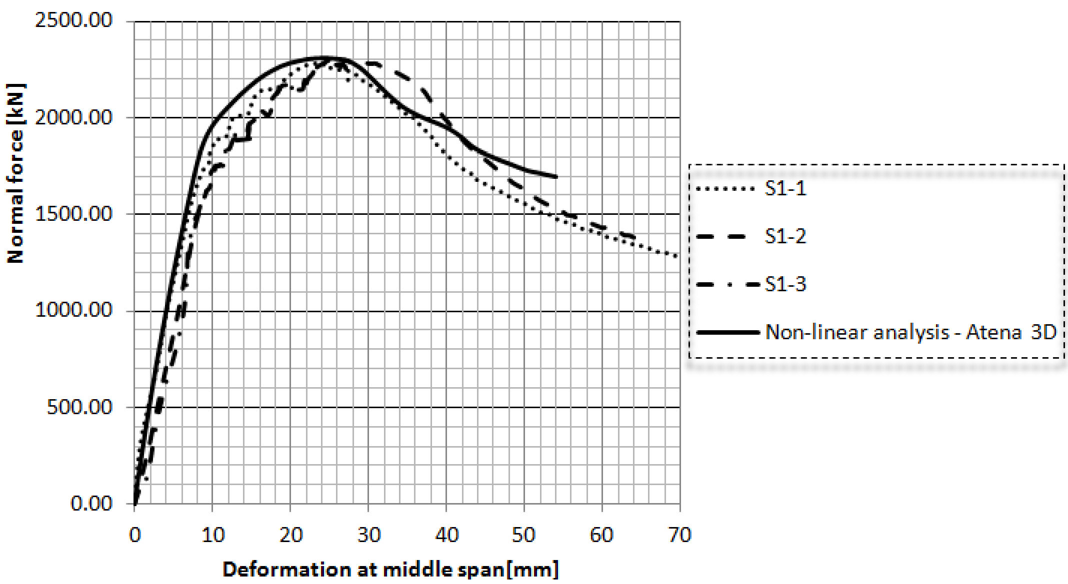
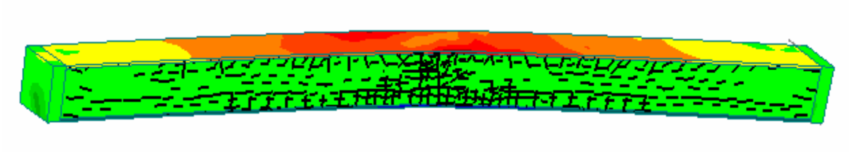


Fig. 5 Comparison of the relationships force/deformation of composite S1 series columns - the length of columns is 3 m



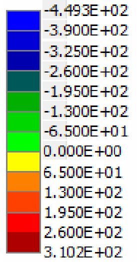


Fig. 6 Stresses in the deformed column

The exact values of the resistance of the column were determined from the force – deformation diagram and the maximum resistance was reached, when the deformation grows even if there is no load added. The force – deflection relationship of the composite SRC columns of series S1 with the length of 3 m obtained from the nonlinear analysis and the measured deflection are in the Fig.5. Fig. 6 presents stresses in the deformed column according the non-linear calculation in the program Athena 3D. The experimental obtained resistances of the tested composite SRC columns were compared to the calculated resistances from the non-linear analysis. The interaction diagrams were calculated with the design values of the material properties and also with the real measured values of the material properties (Fig.7 and Fig.8).

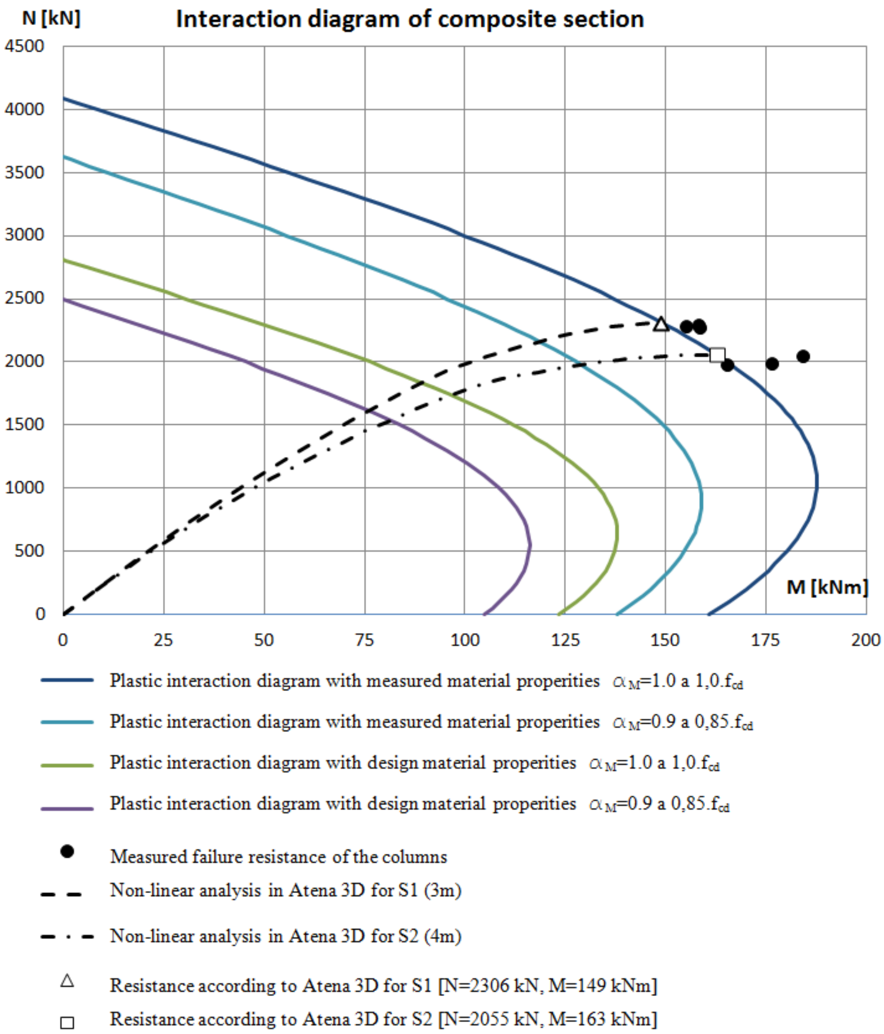


Fig.7 The interaction diagram (in the direction of the web of the cross-section) for the series of experiments S1 a S2 and comparison with the non-linear analysis

Both relationships (force – deflection and interaction diagram) showed a very good match of the non-linear analysis of the ATENA with the experimental measurements.

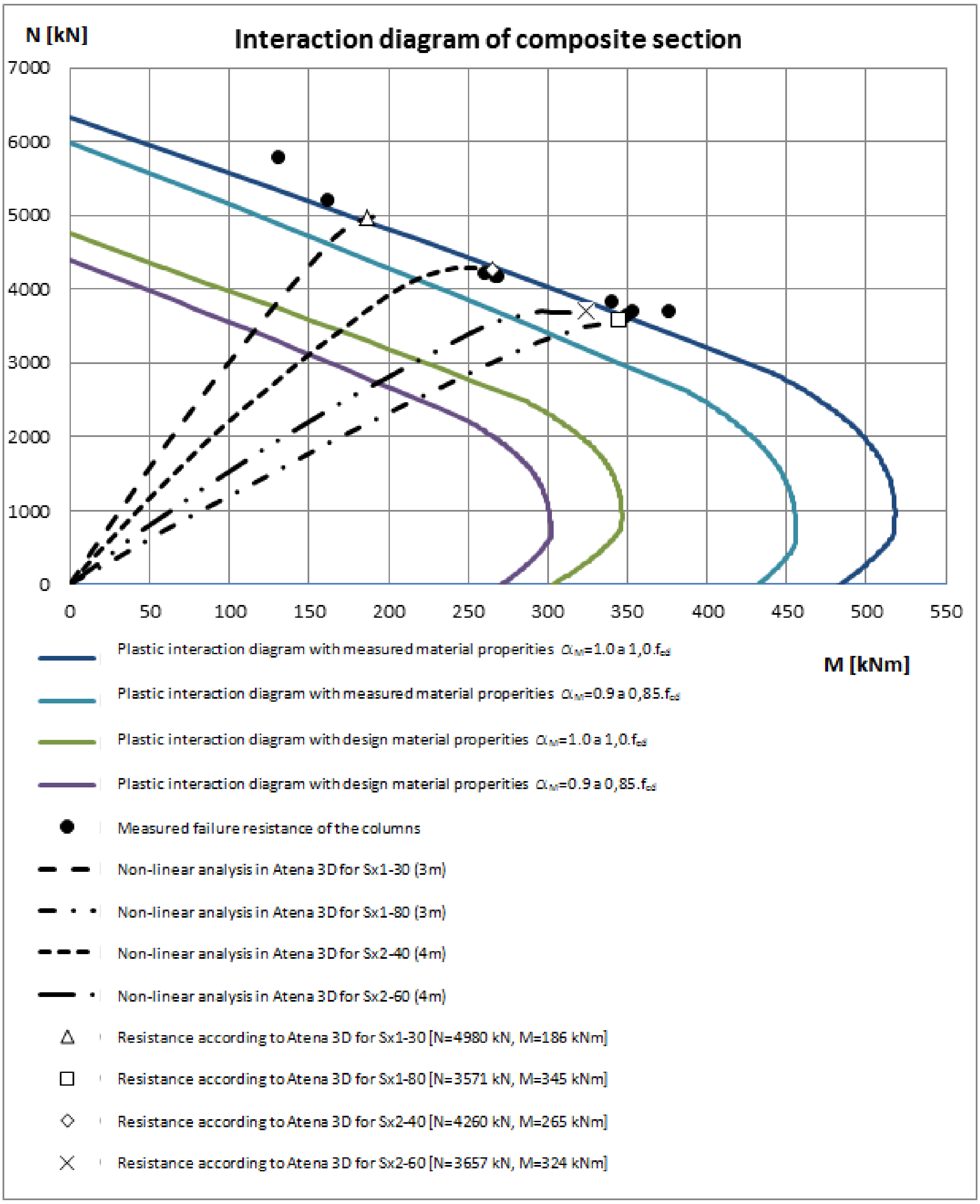


Fig.8 The interaction diagram (in the direction of the web of the cross-section) for the series of experiments Sx1 a Sx2 and comparison with the non-linear analysis

5 Analysis of slenderness and flexural stiffness

The effect of the slenderness and flexural stiffness of cross section of composite SRC columns of two types of calculated and tested types of cross sections by using of the high-strength concrete (C80/95) were also analyzed.

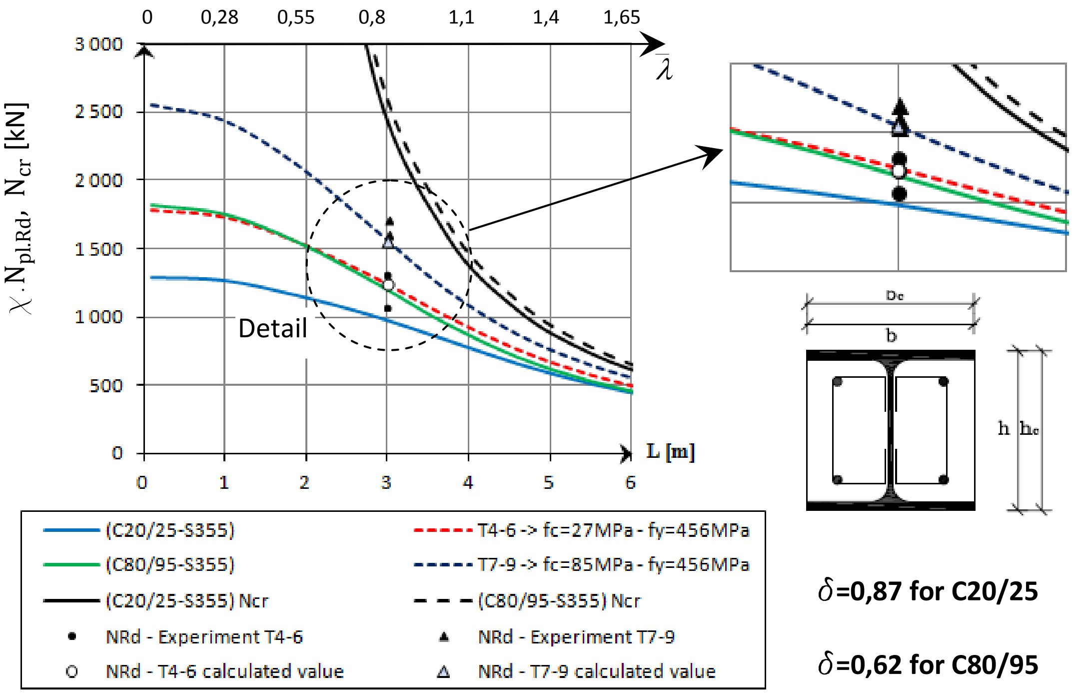


Fig. 9 Relationships N_{Rd} - L for partially encased composite SRC section of the column with concrete C20/25 and C80/95

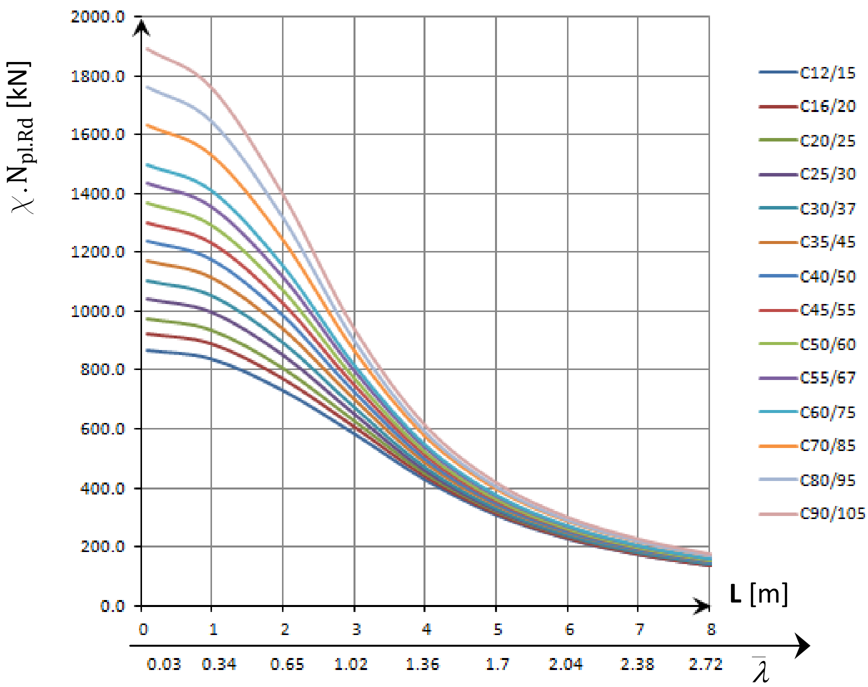


Fig.10 Comparison of the resistance of partially encased composite steel-reinforced concrete column in the compression with the different classes of concrete

In Fig. 9 are shown the results of partially encased composite SRC cross sections. The results shown a good match of the calculated values of resistance, where were used the measured material properties with the results of experiments. For the concrete with the f_c = 85 MPa the difference was 4%. For the slenderness 0,28 was the increase of resistance in compression by using the concrete of the class C 80/95 significant, but for the slenderness 1,5 the increase of resistance is slight.

The same results are shown in Fig. 10, where we compared calculated resistances in compression of the partially encased composite SRC columns with the different classes of concrete.

6 Analysis of the effects of the second order theory

The effect of the second order theory is very important in the design of slender columns. European standards for the design of composite columns give us a simplified relationship to calculate the effects of second order theory. This relationship can be applied only for columns made from concrete class up to C50/60. The steel grades can be from S235 up to S460.

The effects of the second order theory can be also calculated by a method according to reference [4], (general method). The main advantage of this method is that it can be applied in case if high-strength concrete is used. In this chapter we compared the results calculated according to simplified and the general method.

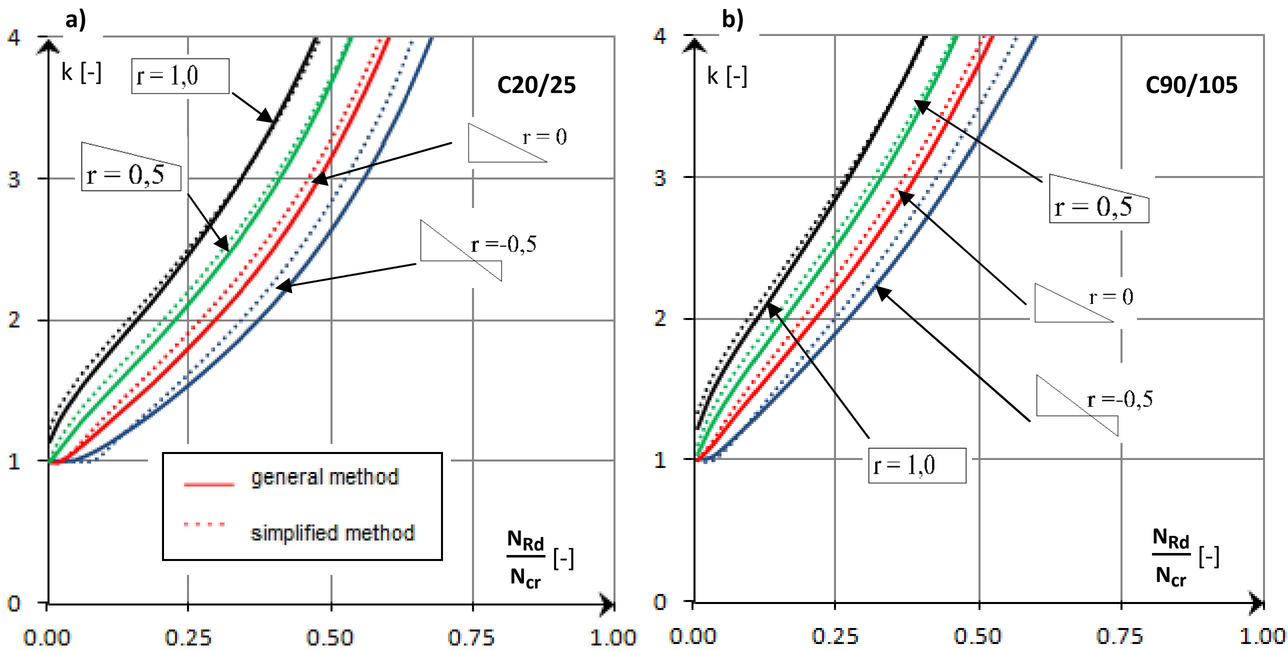


Fig.11 Simplified and general method of second order effects for encased composite SRC section a) with the use of concrete class C20/25, b) with the use of concrete class C90/105

These two methods were compared by the help of factor “k”. In case of slender columns, the factor “k” is used to calculate the second order moment by multiplying the first order moment with the factor “k”.

$$k = M_{max,II} / M_{max,I} \tag{3}$$

The second order moments were calculated for fully encased and for partially encased composite SRC columns. In Fig. 11 are shown the results of encased composite SRC cross sections of columns.

Ratio N_{Ed}/N_{cr} depends on the columns buckling length. Results of the comparison are presented in Fig.11 a) in the case of using the concrete class C20/25 and in Fig. 11 b) when concrete class C90/105 is used. The comparison was made also with the different types of moment distribution.

The results according to the general method show a good agreement with the results calculated according to the simplified method (the difference was 3% in average). We can allege that the simplified method for the second order effects according to the European standards shows results on the safe side in comparison with the general method according to the reference [4]. Moreover it can be also used in cases when HSC is used.

7 Conclusions

The experimental and theoretical analysis of the composite SRC columns also with the use of high strength concrete provides the following conclusions:

- a very good match of the resistances and the force-deflection relationship of the of the tested composite SRC columns and values calculated in non-linear program ATENA 3D was found,
- the increase of resistance in compression of the composite SRC columns with the use of high strength concrete and with the slenderness 1,5 is slight,
- the simplified method, which is recommended by code [3] for calculation of the effects of the second order theory provides the results with a good agreement with the general method according to reference [4]. The small differences were on the safe side.

Acknowledgement

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Experimental analysis of Reinforcement corrosion on bond behaviour

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Keywords: Reinforcement, corrosion, bond, chlorides.

Abstract. The corrosion of reinforcement is the major cause of damage of reinforced concrete structures. This has an impact on safety, serviceability and durability of the structure. The corrosion of steel in concrete reduces the cross sectional area of the reinforcement and decreases the bond between reinforcement and concrete. Corrosion products have a higher volume than steel, which produces internal stresses that lead to the cracking and spalling of the concrete cover. The paper analyses the effect of the chloride-initiated corrosion of reinforcement on bond behaviour.

Introduction

The corrosion of reinforcement in the construction of a transport infrastructure (especially bridges), parking areas, etc., is primarily initiated by chlorides from de-icing salts. The reinforcing steel embedded in concrete is generally protected against corrosion by the high alkalinity (pH = 12.5 to 13.5) of the concrete pore solution. The stability of the passivation film depends on the presence of chloride ions in the ratio of the chloride and hydroxide ions of Cl^- /OH^- [1]. Once the ratio exceeds 0.6, the passivating film does not protect the reinforcement against corrosion even at a pH greater than 11.5. Higher corrosion rates can lead to the cracking and spalling of the concrete cover [2, 3]. Continued corrosion of reinforcement causes a reduction or total loss of bond between concrete and reinforcement.

Experimental programme

The aim of the experimental programme [4] was to investigate the effect of different steel corrosion rates on bond. For this purpose, a total number of 48 reinforced concrete specimens (200x200x130 mm) were manufactured for the anchorage bond/corrosion investigations. Each specimen contains one reinforcing steel bar in each corner. The specimens were divided into 6 groups (types A to F), depending on the diameter of the reinforcement (Ø8 - types A, C, E, and Ø10 - types B, D, F), the thickness of concrete cover (30 and 40 mm respectively), and with or without stirrups. The bars were weighed before the concreting to determine the weight loss due to corrosion.

When the required concrete strength is reached, the process of intense corrosion process can be established. To accelerate the reinforcement corrosion, the impressed current technique was used. The set-up for accelerating reinforcement corrosion is schematically illustrated in Fig. 1. The set-up used for inducing reinforcement corrosion through the impressed current consisted of a direct current (DC) power source, a counter electrode, and an electrolyte. The positive terminal of the DC power source was connected to the steel bars (anode) and the negative terminal was connected to the counter electrode (cathode). The specimens were immersed in a 5 % NaCl solution by the weight of the water. The impressed current's direction was adjusted so that the reinforcing steel served as the anode, while a cooper plate was positioned in the tank to act as a cathode. The acceleration corrosion test was finished when cracks were observed on the surface of the concrete or when the electrolyte seeped through the cracks.

After the acceleration tests were completed, each sample was analysed for the presence of cracks. Their shape, width and position were recorded. Subsequently, a pull-out test was performed.

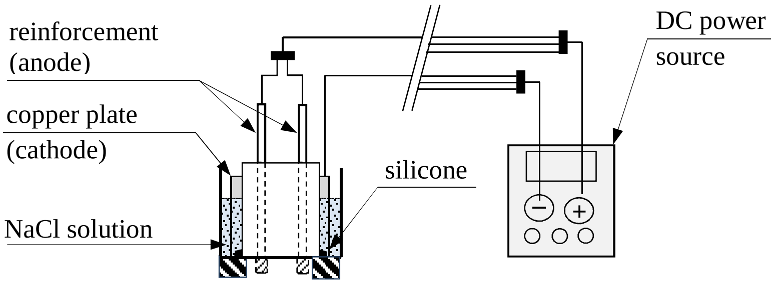

Figure 1: Accelerated corrosion test set-up.

The geometric parameters of the specimens with stirrups and the principle of the pull-out test for the determination of rebar to the concrete bond are shown in Fig. 2. The specimen was placed in the diagonal of the frame. The pulling end of the bar reinforcement was joined through the eyelet with the pulling unit. At the free end of the bar a dial test indicator was placed, which measured any slipping of the bar, depending of the increased force. After performing the pull-out tests, the actual corrosion rate of each bar was measured.

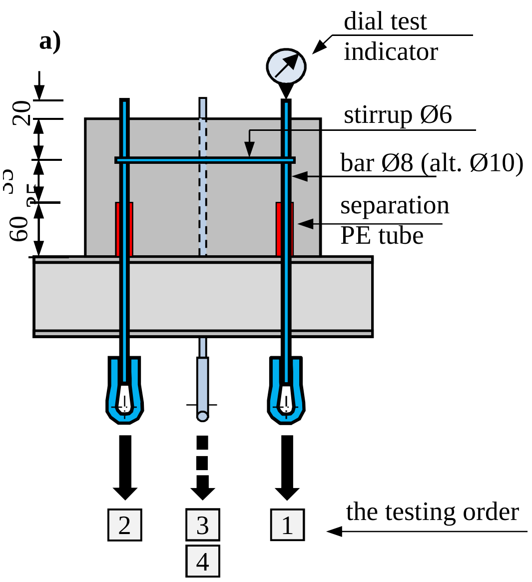

Figure 2: Geometric parameters of the specimens with stirrups and the principle of the pull-out test a) front view, b) side view

The results of experiments

Depending on the degree of the reinforcement corrosion rate $C_{L,}$ the values of the bond strength for specimens reinforced with Ø8mm are shown in Fig. 3. On this basis, it can be concluded that an increased corrosion rate leads to an expected decrease of bond strength. The diagram also shows that the specimens of set C in relation to set E, at the same degree of the reinforcement corrosion rate, have about 4 MPa higher values of bond strength.

From Fig. 4 it can be observed that concerning the specimens without stirrups (set F), there is a 50 % decrease in the bond strength at a much earlier stage of the reinforcement corrosion rate than in the specimens with stirrups (set D). In particular for the specimens without stirrups, a 50 % decrease in bond was observed at a corrosion rate of approximately 8 %, while for samples with stirrups, a 50 % decrease in the bond occurred at a corrosion rate of approximately 22 %.

The experimental results reveal that the presence of the stirrups in specimens contributes to an increase in the bond strength. The positive influence of the transverse reinforcement could also be observed at higher corrosion rates: the specimens with stirrups showed, in comparison with the specimens without stirrups, higher bond strength. The increase in the mean bond strength in the specimens was 37 % of set C compared to set E (Fig. 3) and 45 % of set D compared to set E (Fig. 4).

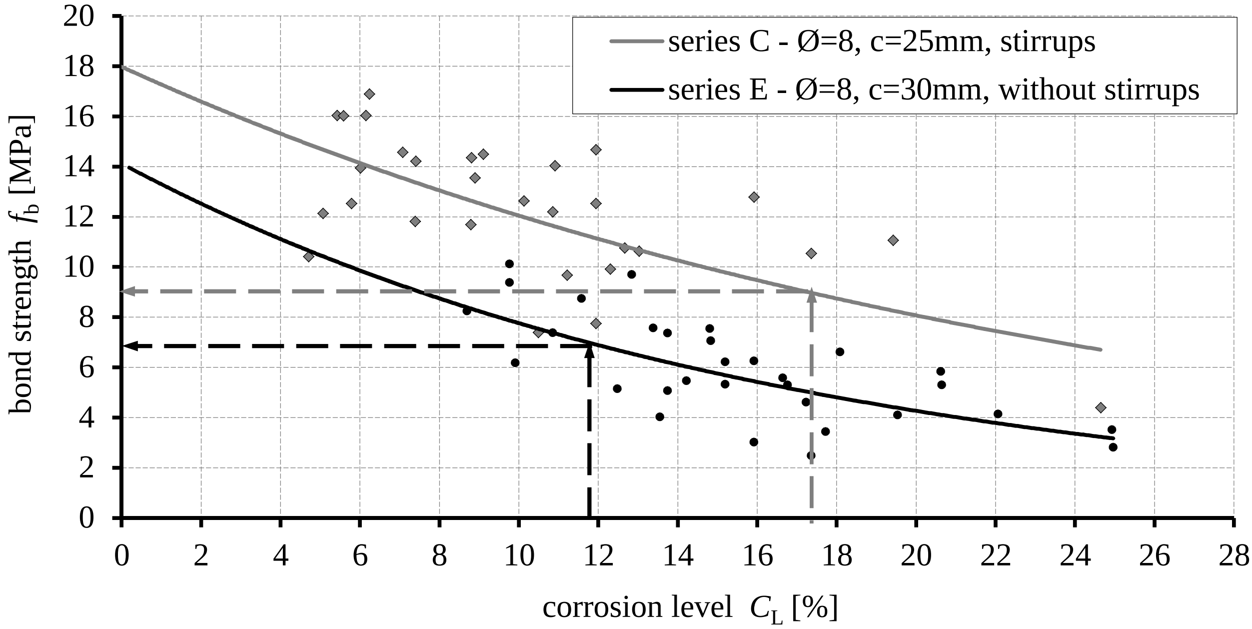

Figure 3 The impact of corrosion rate on the bond strength, Ø8 mm reinforcing bar

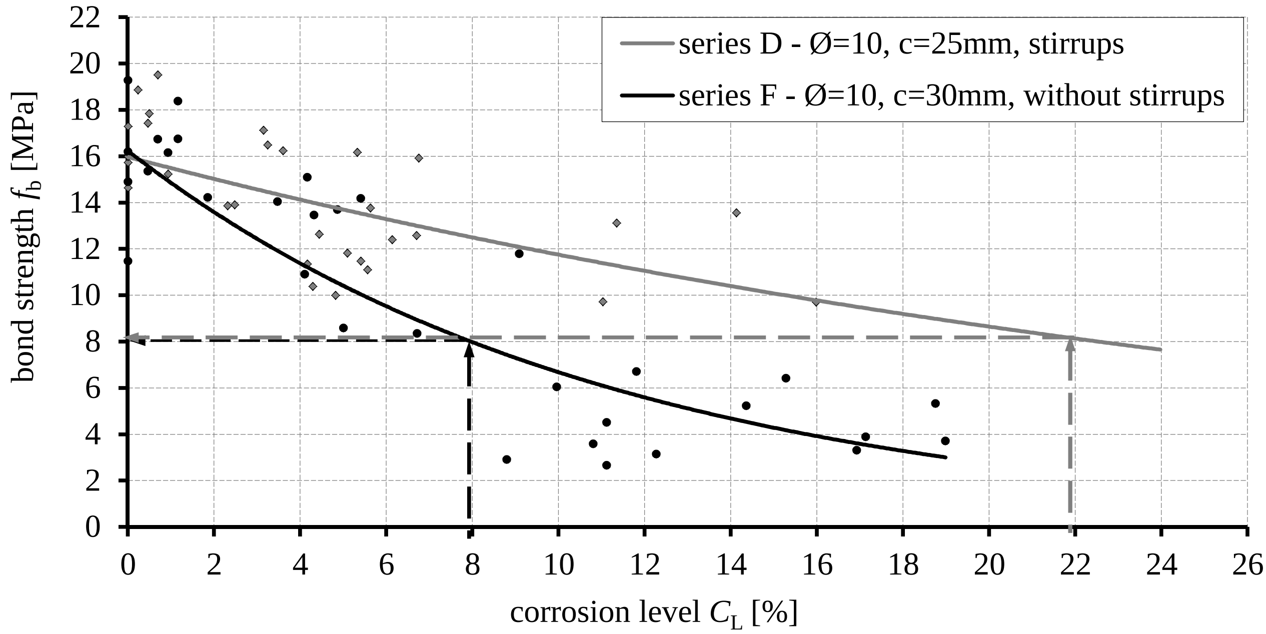

Figure 4 The impact of corrosion rate on the bond strength, Ø10 mm reinforcing bar

Discussion

The results obtained provide relevant information on the effect of the corrosion of a reinforcement on bond strength:

1. An increased corrosion rate resulted in a reduction of bond strength. The experiments indicate that the larger diameter reinforcement is more sensitive than the smaller with regard to the bond strength.
2. The use of transverse reinforcement (stirrups) in the specimens led, in comparison with the specimens without a transverse reinforcement, at the same corrosion rate to smaller cracks widths and an increased bond strength by an average of:
- 37 % for specimens with Ø8 mm bars,
- 47 % for specimens with Ø10 mm bars.

Conclusions

The paper analyses the effect of steel corrosion on bond behaviour between a reinforcement and concrete. The experimental results confirmed that:
1. The corrosion of the reinforcing steel adversely affects the bond strength. The loss of bond strength is potentially more severe than the loss of the bar's cross section. The results demonstrated that due to the reinforcement corrosion, the bond strength can be reduced by 50% while the loss of the bar section is only 12%.
2. The bond behaviour of the specimens during the pull-out tests was significantly affected by the presence of a transverse reinforcement in the form of stirrups. The stirrups provide smaller crack widths and a milder decline in the bond strength at all the corrosion rates.
3. For the given degrees of corrosion rate, the risk for cracking, spalling and decrease of bond strength mainly depends on the geometry of the cross section (concrete cover) and the confinement (transverse reinforcement).

Acknowledgement

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Punching Resistance of Flat Slabs without Shear Reinforcement

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Keywords: reinforced concrete, flat slab, punching, shear.

Abstract. RC flat slabs are frequently used structural members in building construction. Safety verification and avoidance of failure due to punching in the vicinity of a column is currently performed using empirical model which is introduced in Eurocode 2. However extensive discussions are held about replacement of the EC2 model by more refined mechanical model which is presented in Model Code 2010. The paper deals with statistical evaluation of the safety level above mentioned models for punching resistance without shear reinforcement. Evaluation is supplemented by third model from already cancelled Czechoslovak national standard ČSN 731201. Database which includes results of more than 400 experimental tests of flat slab specimens has been used for the statistical evaluation.

Introduction

Punching is one of the most dangerous form of structural failure of RC slabs due to its brittleness. Failure at one support may lead to the overloading of neighbouring areas and then spreads over the whole structure with consequence of progressive collapse. Therefore, design models for punching shall be conservative on one side but also adequately accurate on the other side in order to avoid useless growth of the construction costs. Design model for punching introduced in EC2 is based on the model which was originally used in Model Code 1990. This model is more empirical than physical because parameters having influence on punching resistance were statistically determined using results of some experiments. All experiments used for calibration were carried out on slab specimens with inner column. Therefore question was raised if calibrated model is applicable also for slab areas with edge or corner columns or for foundation slabs and footings. A new model for punching based on the Critical Shear Crack Theory (CSCT) has been developed in 90s. The principles of CSCT were published by Muttoni and Schwartz [4] in 1991 for the first time. The theory was later refined by prof. Muttoni [5] and finally has become the fundamental design model for calculation of punching resistance in Model Code 2010 [6].

Model of Critical Shear Crack Theory (CSCT)

CSCT is mechanical model for calculation of punching resistance. The model was verified by results of 99 experiments. The principles of the theory came out from the assumption of critical crack development at vicinity of the column. Punching resistance is ensured by aggregates interlocking in the critical crack and by tensile strength of the concrete. Shear resistance $V_{\mathrm{Rd,c}}$ then depends on friction in the critical crack and on the crack width. Friction is mainly influenced by the maximum aggregate size d_{g} and crack width is proportional to the slab rotation (ψ), see fig.1.

$$V_{\mathrm{Rd,c}} = k_{\psi} \frac{\sqrt{f_{\mathrm{ck}}}}{\gamma_{\mathrm{C}}} b_0 \cdot d_{\mathrm{v}} \tag{1}$$

$$\psi = 1.5 \cdot \frac{r_s}{d} \frac{f_{yd}}{E_s} \cdot \left(\frac{m_{Ed}}{m_{Rd}} \right)^{1.5} \tag{2}$$

Where: d_v is effective depth of the slab for shear, usually $d_v = d$
b_0 – length of control perimeter at distance $d_v/2$ from the face of a column
k_ψ – factor depending on the maximum aggregate size d_g and slab rotation (ψ)
r_s – distance from the column axis to the line of contraflexure of radial bending moments
m_{Ed}, m_{Rd} – are the average design moments per unit length effect of action/ bending resistance

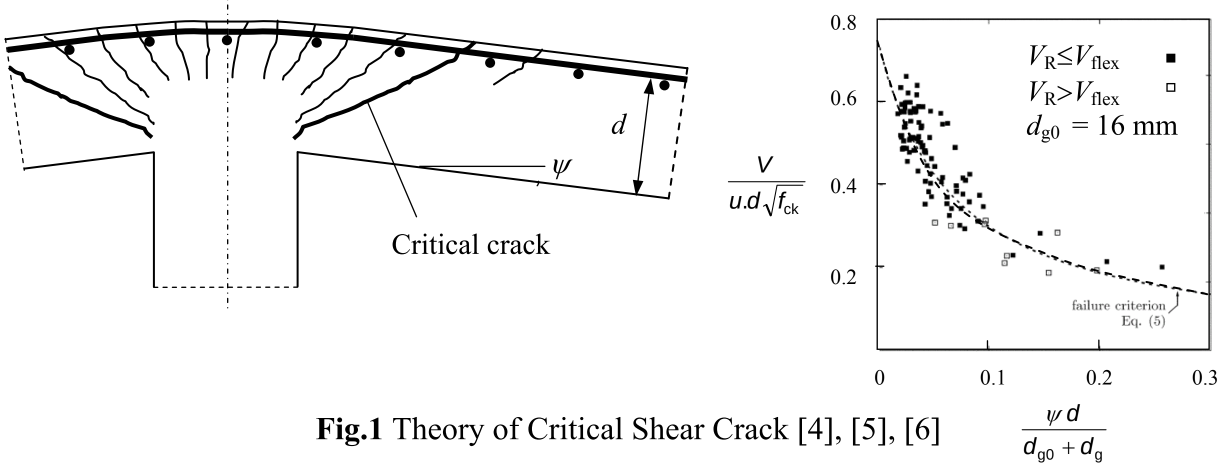

Fig.1 Theory of Critical Shear Crack [4], [5], [6]

Slab rotation "ψ" depends on the stresses in main reinforcement as well as geometry conditions concerning of radial bending moments distribution eq. (2). Punching resistance in theory CCST does not depend directly on amount of main reinforcement "ρ" as in EC2 model.

Statistical evaluation of the models for punching resistance

Statistical evaluation of the models for punching resistance without shear reinforcement has been carried out for three different models MC 2010, EC2 and ČSN 731201 with partial safety factor γ_C = 1,0. For strength f_{ck} cylinder strength of concrete introduced by authors of the experiments has been used. Tensile strength of concrete R_{btn} in ČSN model was determined based on f_{ck}, see formula in [1]. Control perimeters were assumed at distance 2d from the face of column for EC2 model, d/2 for MC 2010 model and h/2 for ČSN model, where h is a slab thickness. Main statistical variable in the evaluation was ratio $P_i = (V_{R,test}/V_{Rd,c})_i$, where "i" is number of a test, $V_{R,test}$ is a resistance obtained from an experimental test and $V_{Rd,c}$ (Q_{bu}) is punching resistance obtained from theoretical model. Only variables P_i which satisfy condition $0.5 < P_i < 2.0$ have been used in statistical evaluation. Mean value P_m was calculated using formula $P_m = (\sum P_i)/n$ where n is a number of assumed tests. Characteristic value was determined as 5% fractile for Gaussian distribution $P_{k,0.05} = P_m(1\text{-}1{,}645.V_p)$, where V_P is coefficient of variation $V_P = \sigma_P/P_m$ and σ_P is standard deviation $\sigma_P^2 = [\sum(P_i\text{-}P_m)^2]/(n\text{-}1)$. If characteristic value $P_{k,0.05} \geq 1.0$ a resistance model can be assumed reliable and opposite if $P_{k,0.05} < 1.0$ the design model does not fully meet reliability requirements of EN 1990.

Model Code 2010 model. Two input data were usually missing for evaluation of the model, first, stresses in main reinforcement and secondly maximum aggregate size d_g. Because influence of d_g can be significant, three statistical analyses were performed. First one with a set of tests, where for missing d_g value of 16 mm has been used and stresses in main reinforcement of yield strength f_{yk} were applied, (S1) analysis. The second and the third analysis (more accurate) were performed only with tests where d_g is known. Stresses in reinforcement of 100% f_{yk} $(m_{Ed}/m_{Rd})^{1,5}$ =1 were used in the first and second analysis (S2) and 80% of f_{yk} $(m_{Ed}/m_{Rd})^{1,5}$ = 0.8 in the third analysis (S3). Obtained results are introduced in Table 1.

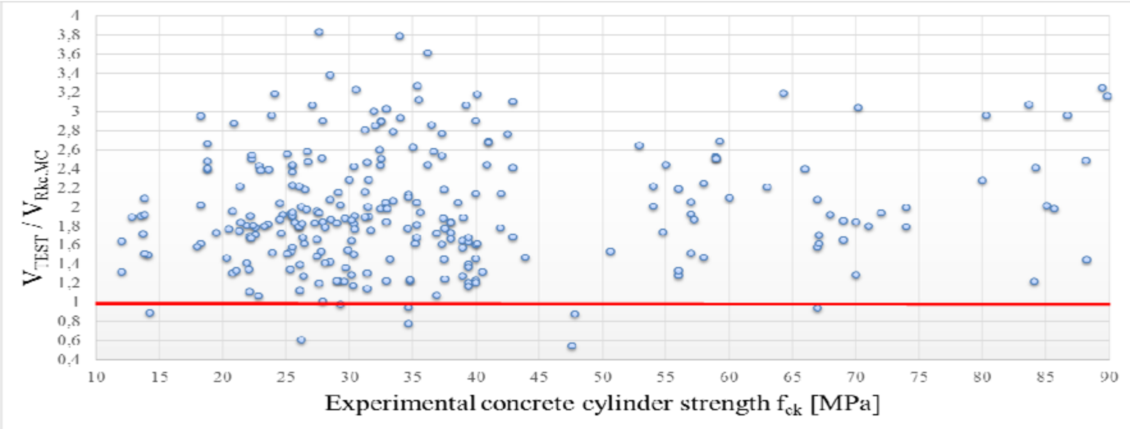

Fig. 2 - Ratio of tested and calculated punching resistance vs. concrete strength, MC 2010 model

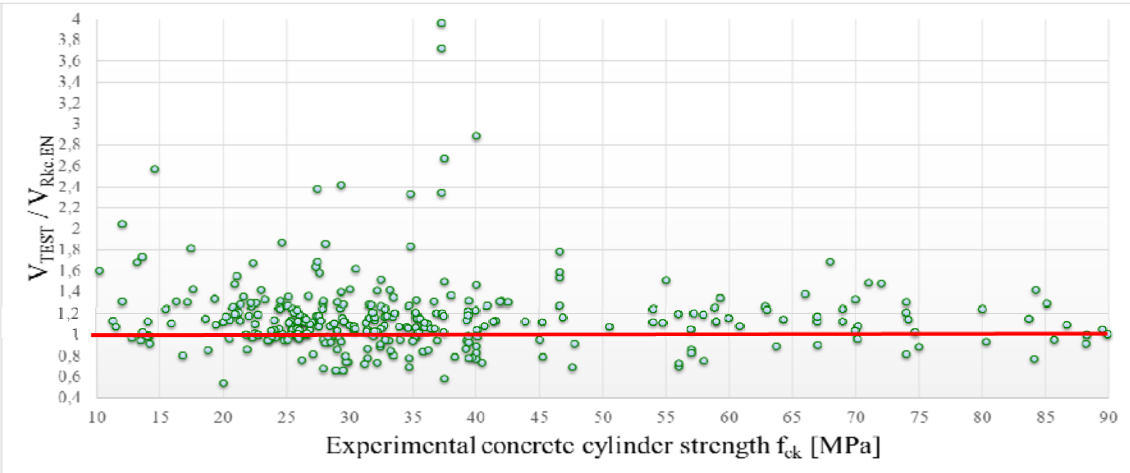

Fig. 3 - Ratio of tested and calculated punching resistance vs. concrete strength, EC2 model

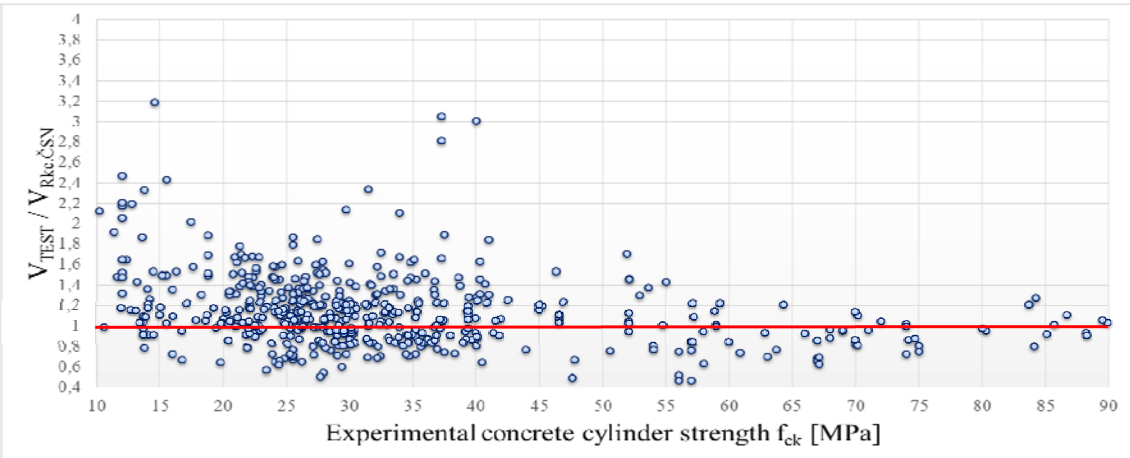

Fig. 4 - Ratio of tested and calculated punching resistance vs. concrete strength CSN model

EC2 and ČSN Models. Statistical evaluation of the EC2 and ČSN models has been carried out with full database for all tests which satisfy condition $0.5 < P_i < 2.0$.

Discussion of obtained results

Statistical evaluation of the tests for punching resistance of axially loaded slabs has shown that the most reliable is model introduced in MC 2010. For reduced database, where all input data have been known, characteristic value of model reliability $P_{k,0.05}$ exceeds value 1 by more than 10%. So the model seems to be too safe and it would be suitable make a little recalibration of it. Lower reliability indicates EC2 model with $P_{k,0.05} \approx 0.78$ with mean value $P_m \approx 1.17$. So, some corrections are needed in this case. The lowest safety provides ČSN model, where low mean value of $P_m = 1.12$ and large scatter of test results caused that the characteristic value is only between $P_{k,0.05} = 0.69$.

Relation between concrete strength and ratio $V_{R,test}/V_{Rk}$ is plotted in fig.2, fig.3 and fig.4 for all assumed models with full database without restriction of $0.5 < P_i < 2.0$. In fig.4, it can be seen e.g. that the ČSN model does not suit well for high performance concrete, because nearly all results are below 1. Opposite, model MC 2010 is well calibrated for all strength classes. Similar relations can be plotted e.g. with effective depth "d" or with reinforcement ratio "ρ" etc..

Table 1 Statistical evaluation of reliability of the models for punching resistance

Set	Model	Number of specimens [n]	Average value [P_m]	Variation coef. [V_P]	Characteristic value $P_{k,0.05}$
	ČSN	406	1.122	0.234	**0.689**
	EC 2	408	1.168	0.201	**0.782**
(S1)	MC 2010	259	1.565	0.196	**1.061**
(S2)	MC 2010	127	1.629	0.161	**1.199**
(S3)	MC 2010	162	1.544	0.168	**1.118**
(S1), (S2): stresses in main reinforcement 100% f_{yk} , (S3) stresses in main reinforcement 80% of f_{yk}					

Conclusion

Technical committee CEN TC250/SC2/WG1/TG4 holds extensive discussions how to proceed with model for punching resistance in Eurocode 2 in connection with works on the second generation of Eurocodes. Should be used original model with some corrections or should be adapted new mechanical model which is based e.g. on CSCT theory? Statistical evaluation of the model′s reliability is suitable tool which can help experts to make right decision.

Acknowledgement

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Construction of Concrete Bridges using Reinforcement made of Low Level Radioactive Steel

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Abstract

Nowadays, many nuclear power plants are approaching the end of their designed lifetime and the question of their decommissioning is being increasingly discussed. In connection with the dismantling of the nuclear facilities large quantities of typical decommissioning waste material are produced. Among this waste material, there is a significant amount of radioactive steel. Its level of radioactivity just slightly exceeds the regulatory limits set for unconditional release into the environment and, moreover, the steel scrap mostly contains radionuclides with relatively short half-life. Disposal of all decommissioning steel into specialized repositories would require considerable financial investments. Therefore, melting and reuse of this steel in the construction of bridges seems like an advantageous alternative. The article deals particularly with the possibilities of using slightly radioactive steel in concrete bridges construction. The object of this paper is to present the possibilities of the utilization of very low level reinforcement steel in the bridge building process. Two models of common widespread bridges and their construction methods were transformed into VISIPLAN 3D ALARA computational tool to calculate radiation exposure of workers constructing the bridge. The calculated exposure was then confronted with the exposure limits stated in legislation. These two bridges and their construction methods were compared in order to point out which one is more suitable for the use of radioactive reinforcement.

Keywords

low level radioactive steel, bridge reinforcement, recycling, concrete bridges,

1 Introduction

During the operation of nuclear power plants (NPP) around the world the radioactive steel is produced by the contact of solid metallic materials with active media (contamination process) or by the direct impact of radiation after which solid metallic materials become sources of radiation (activation process). In the process of NPP decommissioning at the end of its lifetime a significant amount of materials arises, including dismantled metal parts of technological units. The levels of radioactivity bound in these metal parts vary and depend on the concentration of individual radionuclides contained in the material [1]. A considerable number of procedures exist for the reduction of this concentration – beginning with various decontamination techniques ending with the melting process of metallic materials. If the level of concentration of single radionuclides is lower than the legislation limit (an example is the limit of specific activity 300 Bq/kg for ^{60}Co, valid in the Slovak legislation [2]) then it is possible to unconditionally release such metallic materials into the environment, mostly for the interim storage, whence the metal is transported to the scrap recycling facility. If the conditions for the unconditional release of radioactive materials are not met, these materials are classified as a radioactive waste. Consequently, significant financial resources for treatment, conditioning and the disposal of this material in the respective type of radioactive waste repository have to be invested [3].

However, a certain amount of these funds could be saved by using the concept of conditional release of materials and their reuse for a specific industrial purpose in case the concentration of some radionuclides in the material exceeds the legislative limit values [2] but it is clearly proven that the radiation impact of the proposed specific utilization on the workers and population will be negligible.

The basic principles of conditional release are as follows:

1. Conditionally released material contains solid radioactive materials, mostly metals.
2. Only materials with bound radionuclides with a relatively short half-life are considered.
3. Long-term boundedness of the released materials at one place is considered, which allows the sufficient decrease in the concentration of radionuclides due to the natural radioactive decay.
4. International recommendations that define the limits of the annual individual effective dose (tens of µSv) are used.
5. Requirements of the Slovak Statutory Order 345/2006 Coll. (based on EU Directive 96/29/EURATOM) are obeyed [4].

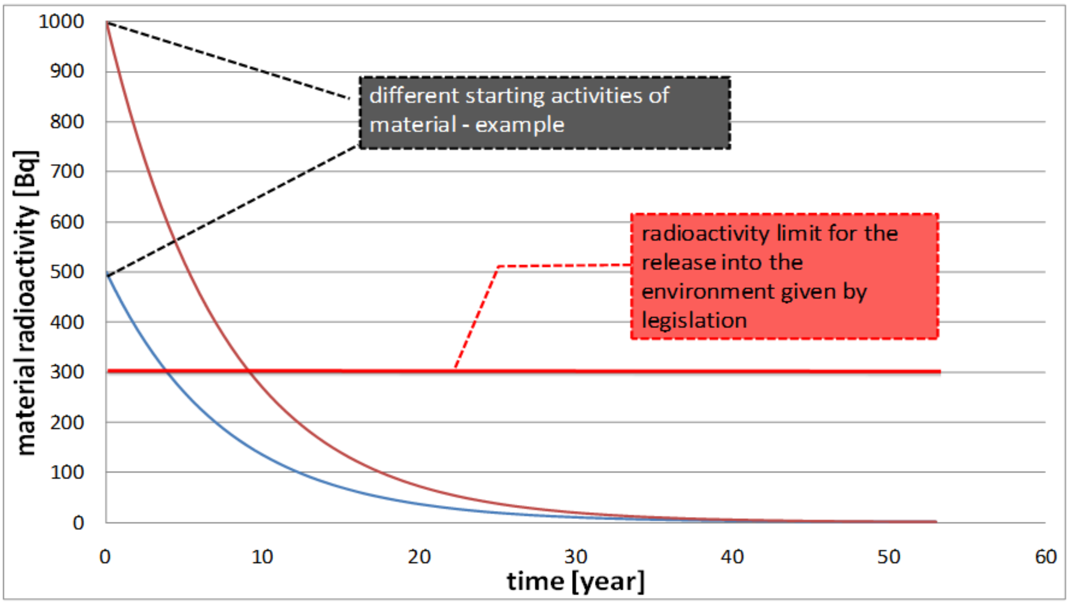

Fig. 1 Decrease of radioactivity of conditionally released steel due to natural decay (valid for the typical radionuclide ^{60}Co with half-life 5.27 year)

The considered scenarios of utilizing very low level radioactive steel in the construction presume melting of the contaminated metal scrap arising from decommissioning that results in the production of ingots. Ingots will be applied in the production of various reinforcing elements (reinforcing rods, reinforcing steel grids) suitable for the use in many civil engineering projects such as bridges [5].

2 Material and Method

The potential for the use of very low level radioactive steel in bridge construction stems mainly from the characteristics of bridge structures such as the geographical location of many bridges (non-occupied territory), relatively large reinforcement cover, high quality concrete with low permeability and the required service life of bridge structures which significantly exceeds the requirements imposed on the natural decline of the radioactivity down to the legislation limits (Figure 1).

2.1 ***Irradiation of workers during bridge building process***

In any case, when we talk about the irradiation of workers, it is always just the minimum dose that is legally allowed and does not exceed 5% of the typical natural radiation background. The level of radiation received from the environment, i.e., the natural radiation background, varies depending on many factors and differs in various locations. The radiation dose from the conditionally released material is many times lower than the dose absorbed by staff of nuclear power plants.
Workers are exposed to the radiation notably only during reinforcement works, the manipulation with reinforcement cages and during moulding and concreting. Once the reinforcement is covered by concrete, coating provides partial radiation shielding. Then only uncovered connection rods remain stronger sources of radiation. The thickness of concrete ensuring a 50% decrease of gamma radiation is about 10 cm. In a typical coating thickness of 4 cm, radiation intensity decreases by about 25%.

The reduction of exposure of the workers during the construction as well as the reduction of doses received from already completed constructions can be achieved by using the so-called mixed reinforcement cages. The mixed reinforcement cage is made of ordinary reinforcing steel, next to which rods of radioactive steel are incorporated just before concreting. In this way it is possible to utilize this kind of steel very effectively since complicated shapes can be manufactured from ordinary steel and the whole reinforcement cage can be then finished using simple rods made of radioactive steel. This reduces the duration of handling processes with radioactive steel and also the time required for bending complicated shapes.

The study was focused on various parts of bridge structures that were first examined separately and then as a complex bridge construction. The complex bridge was split into three basic parts - foundations, piers and superstructure.

2.1.1 ***Foundations***

In the study it was assumed that the utilization of very low level radioactive steel could be possible in the construction of the bridge foundations, particularly as the reinforcement of large-diameter piles. Simple reinforcement and a relatively short time required for its incorporation were considered as one of the crucial factors.

2.1.2 ***Piers***

Another option is to use radioactive reinforcement in piers, where the simple shaped piers are of course the most suitable ones.

The decisive criterion for the design of the thickness of concrete cover and its quality is still determined by the required lifetime of the structure in the specified environment but here is necessary to pay more attention to the quality of works. The transport of radionuclides into the environment must be prevented until their activities do not fall below a specified value.

2.1.3 ***Superstructure***

Searching for a suitable technology for the construction of concrete bridges that supports the incorporation of the largest possible amount of radioactive reinforcement is based mainly on the factor limiting the maximum radiation dose received by workers during the construction. Therefore only the technologies that allow a relatively fast construction speed for larger lengths were considered. The technology using prefabricated I-shaped girders and incremental launching technology were finally chosen as the base for computational models. The design of the foundation was adapted to requirements of chosen construction technologies. The amount of reinforcement steel was estimated on the basis of several previous realized construction projects (radioactive reinforcement is also encountered in load bearing capacity). The length of both model bridges was chosen as 1650 m. In cooperation with construction practices the durations of working procedures and numbers of construction workers needed were estimated. Fictional working groups performing individual procedures were created. In models these groups left the construction site after finishing their work. Subsequently, calculated radiation doses were evaluated for each group and they were compared with the legislation limits. Description of the calculation and modeling is described in the respective chapter.

2.2 ***Description of Modeled Bridges***

Dimensions of individual parts of bridges are based on experience with designing bridges and they are resulting from the preliminary structural design [6].

2.2.1 ***Prefabricated alternative with I-Shaped girders***

The first alternative model (Figure 3) comprises the bridge consisting of longitudinal prestressed prefabricated girders with a field range of 31 m. The foundation consists of large-diameter piles with a diameter of 1.2 m and an average length of 14 m. At each foundation a pair of piers stood with a diameter of 1.6 m which held the prefabricated crossbeam on which 1.4 m high longitudinally prestressed I-shaped girders were then placed. Basic dimensions of the bridge are shown on Figure 2. The final step in modeling the construction is the completion of the crossbeam and the upper deck. Finally, the dose received by the workers finishing superstructure parts was assessed. The production of prefabricated girders was considered in a separate production hall with an interim storage next to it. The radiation dose assessment comprises also the interim storage of reinforcement steel and the mobility of workers in its vicinity [7].

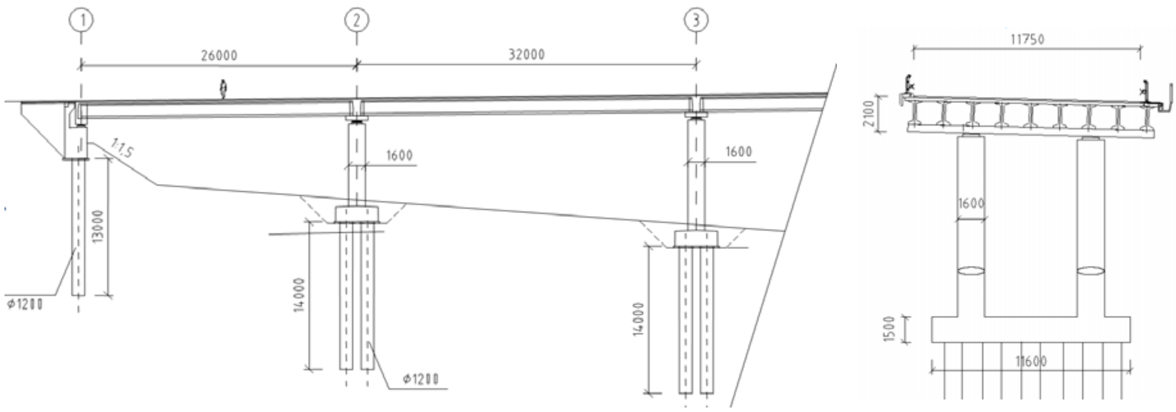

Fig. 2 Scheme of the bridge made of prefabricated I-shaped girders (longitudinal and cross section)

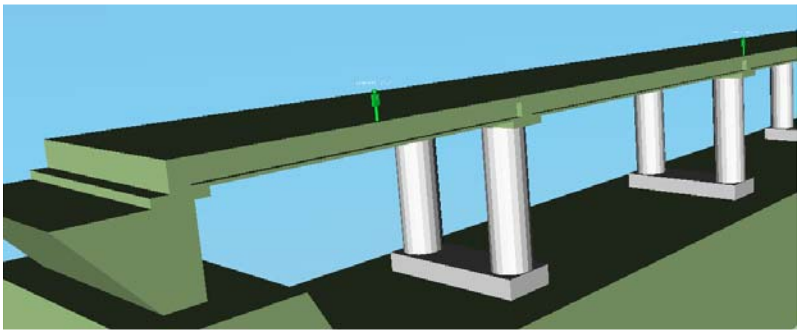

Fig. 3 Software model in VISIPLAN 3D ALARA.

In connection with the manufacture of prestressed concrete girders an issue can occur whether the production line becomes radioactive over time. This phenomenon is negligible since the radioactive nuclides contained in the steel are mostly gamma emitters, thus activating the device is very unlikely. The only way of the pollution of the equipment may be its contamination with the dust and debris from the radioactive material. However, this can be decontaminated quickly and inexpensively.

*2.2.2 **Longitudinal launching alternative***

Another alternative under consideration was construction of the bridge by using the incremental launching technology. Large-diameter 1.2 m high piles of average length of 14 m were used as the foundation. Consequently, a pillar with a head adapted to the launching technology was modeled. Basic dimensions of the modeled bridge are shown on Figure 4. The bridge superstructure was constructed gradually in the workplace situated behind the abutment. The necessity of bearings replacement and finishing works on the bridge superstructure, as well as on the first alternative, were assessed after the completion of the bridge.

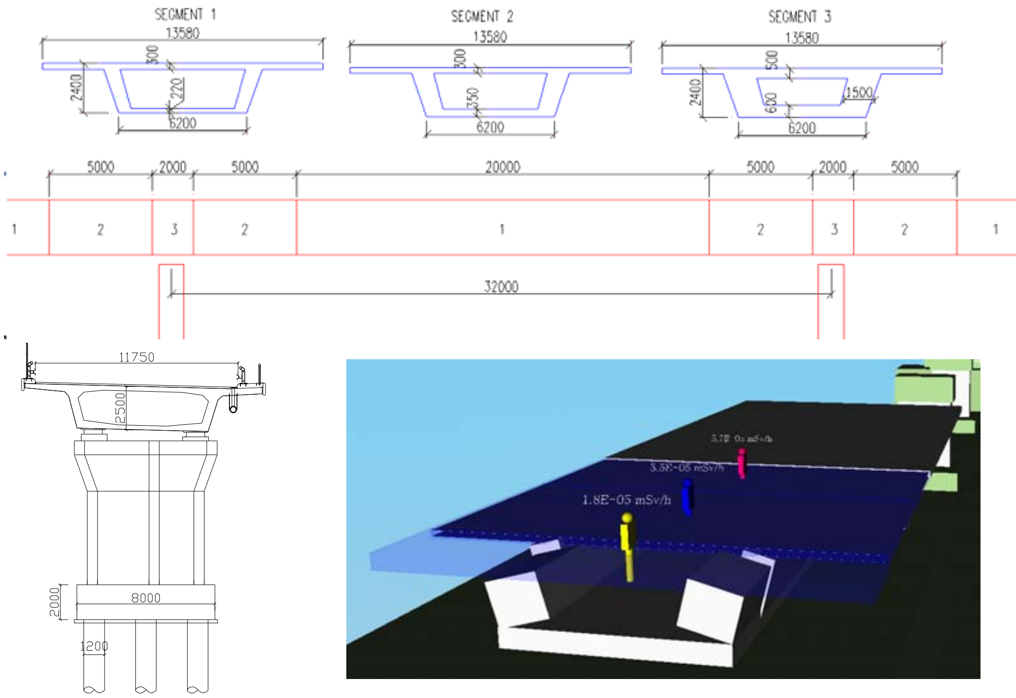

Fig. 4 Scheme of the bridge built using the launching technology and its software model in VISIPLAN 3D ALARA.

4 Calculation

4.1 ***Applied computational tools***

The exposure of workers handling radioactive material can be calculated using the simulation in a variety of computational tools which include, e.g., VISIPLAN 3D ALARA [11]., or a simpler alternative MICROSHIELD [12] in the case of the external irradiation and GOLDSIM [13] designed to calculate the internal exposure.
VISIPLAN 3D ALARA software allows the calculation of the absorbed individual effective dose in complex environments. It allows creation of a simplified geometry model of the working environment based on the technical documentation. Radiation sources identified by measurement are placed in this geometry. A useful option of the program is creation of a `trajectory`. Trajectories describe the movement of the person in the environment with radiation sources. They consist of several points. Each point contains information about the movement of the person in the environment, on the duration and type of his/her activity. The calculated trajectory contains a record of the dose absorbed by the worker. This calculated dose can be then compared with the limits in legislation, standards or internal regulations. The computational tool VISIPLAN has been chosen for the calculations of the external exposure of workers during the construction of the bridge.
Program GOLDSIM simulates the transport of radionuclides in the environment and subsequent intake of these radionuclides by humans (inhalation, ingestion ...). The software can calculate the received dose, depending on the time or concentration of radionuclides in the monitored environment. This tool is therefore designed to estimate long-term effects of contaminants present in the environment. Its outputs will be used to determine long-term impacts of the completed bridge construction.

From the point of view of the exposure of construction workers the outputs are therefore important of the computational tool VISIPLAN. Therefore attention will be focused on the model scenarios developed in this program. The VISIPLAN software has been verified and validated, thus its outputs can be considered accurate and reliable.

4.2 ***Description of model scenarios and calculation results***

For the modeling purposes two typical bridge constructions were selected which in various respects meet requirements for the utilization of conditionally released very low level radioactive steel. The first typical bridge uses prefabricated parts and the second one, a monolithic bridge, was built by the launching technology. These two model bridges were described in Sections 3.1 and 3.2.

Generally, it is possible to divide the concrete bridge into 3 parts:

- Bridge foundation
- Piers and abutments
- Bridge superstructure

This division allows splitting the dose absorbed during the construction of each part and then to assess if different parts of the bridge are suitable for the incorporation of conditionally released steel. It is possible to determine the value of the mass activities for each part separately. ^{60}Co radionuclide was used as a radiation source for the purpose of modeling. ^{60}Co remains fixed in the steel during the melting process besides being one of the most prevalent contaminants in the operation of nuclear power plants.

4.2.1 ***Longitudinal prefabricates alternative***

The chosen working procedures were transformed into trajectories in VISIPLAN software after the cooperation with the construction practice. Each construction procedure was modeled with real required construction time proposed by the construction companies. Figure 5, 6 and 7 shows some of the working procedures which were modeled and evaluated.

The construction of abutments was processed and modeled similar to the construction of piers and foundations with modified times.

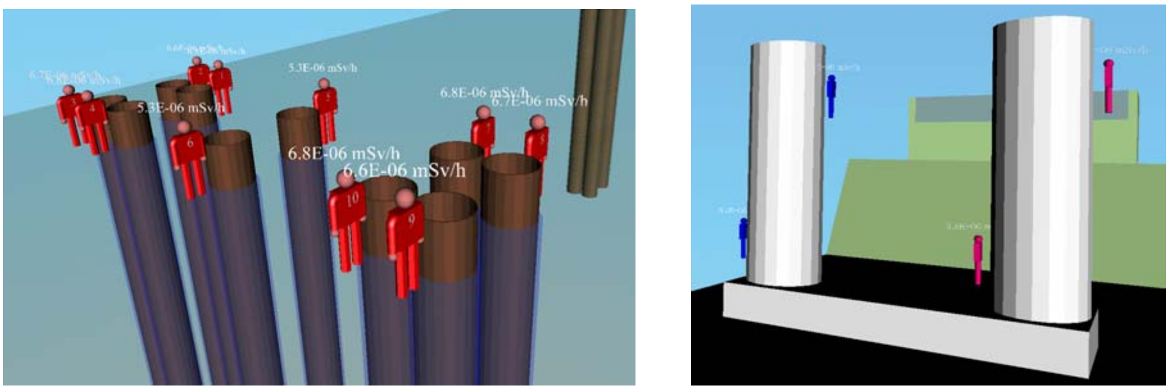

Fig. 5 Working procedures connected with building the foundations and the constructing the piers processed in VISIPLAN software

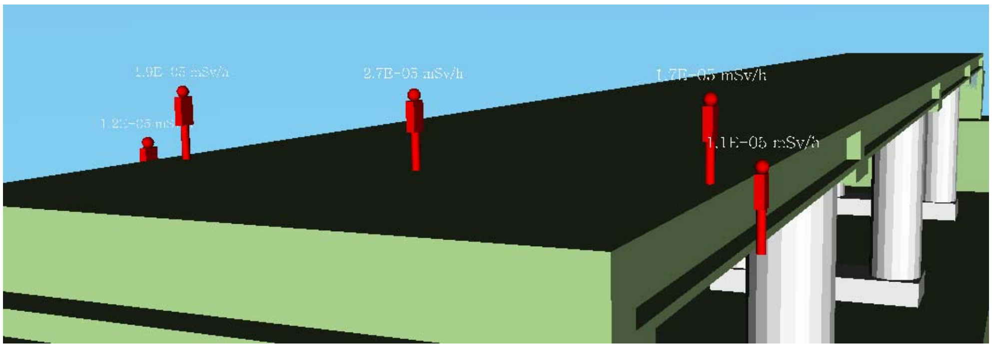

Fig. 6 Working procedures connected with building the superstructure processed in VISIPLAN software

Other working procedures, e.g., road construction, assembly of rails, ground works, etc. were processed and modeled similarly.

4.2.2 ***Launching alternative***

The alternative describing the bridge built using the incremental launching technology was modeled in analogy to the prefabricated bridge. The foundation and piers were modeled to be built similarly to the alternative of prefabricates, although the real dimensions of them are adapted to the changed concept of the superstructure and the construction technique. The modeled technique of the construction of large-diameter piles was the same as in Alternative 1.

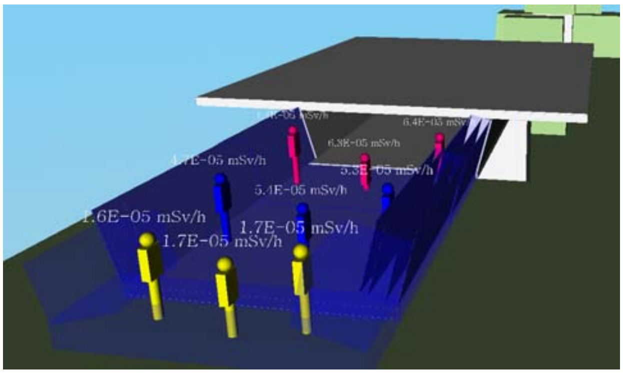

Fig. 7 Working procedures connected with building the superstructure segment in VISIPLAN software

Similarly were processed and modeled also other working procedures, e.g., dismantling of temporary constructions, road construction, assembly of bridge equipment, etc.

5 Results of calculations

*5.1 **Longitudinal prefabricates alternative***

The utilization of conditionally released steel is theoretically possible for this construction technology in each of the three main parts of the bridge. Somewhere, however, steel incorporation must be accompanied by certain restrictions to ensure that the legislation exposure limits are not exceeded. The group of workers that receives the biggest radiation dose was identified for each part of the bridge, in other words, `Critical group` was recognized. The dose received by this group was compared with the exposure limit values. These limits are specified by the Slovak legislation [2] at a level of 10 µSv/year, or 50 µSv/year under special conditions (µSv unit represents absorbed energy from the source of radiation). The construction of the bridge using very low level radioactive steel falls within the definition of special conditions, therefore, the limit of 50 µSv/year was considered as well. At this point it is appropriate to remind that natural background radiation typically reaches a level of more than 3 mSv/year (this means that the radiation dose received from the natural background is about 60 times greater than that received from working with this radioactive steel). The calculated values of radiation doses with resulting possible proportions of the very low level radioactive waste in respective bridge parts are shown in Table1.

Table 1 Calculated radiation doses and resulting possible shares of very low level radioactive waste in respective bridge parts

Bridge part	Calculated dose absorbed by the critical group of workers* [µSv/year]	Legislation dose limit [µSv/year]	Permitted proportion of very low level radioactive steel in bridge part* [%]
Piles	8.3	10	100
		50	100
Piers	37.8	10	26.4
		50	100
Superstructure	37.9	10	26.4
		50	100

* value is valid for the ^{60}Co radiation source with specific activity 300 Bq/kg (the legislation limit for the unconditional release into the environment)

The results of calculations show that in the case of the 50 μSv/year dose limit the conditionally released steel with the specific activity of 300 Bq/kg can be applied to all main components of the bridge without any additional restrictions. The level of specific activity can even be increased while keeping safe doses of the radiation, which is a welcome outcome, particularly for the nuclear industry.

*5.2 **Launching alternative***

Likewise the prefabricated bridge alternative also in the case of the bridge built by launching the critical group for every bridge part was identified. The results of calculations are stated in Table 2.

Table 2 Calculated radiation doses and resulting possible proportions of very low level radioactive waste in respective bridge parts

Bridge part	Calculated dose absorbed by the critical group of workers* [μSv/year]	Legislation dose limit [μSv/year]	Permitted proportion of very low level radioactive steel in bridge part* [%]
Piles	8.3	10	100
		50	100
Piers	129.4	10	7.7
		50	38.6
Superstructure	85.4	10	11.7
		50	58.5

* value is valid for the ^{60}Co radiation source with specific activity 300 Bq/kg (legislation limit for the unconditional release into the environment)

From the results of calculations it is possible to see that the application of conditionally released steel in the construction of the bridge built by launching is appropriate only in piles reinforcement. In this case, there is a possibility to increase the specific activity, while maintaining the required level of safety. For other applications very low level radioactive steel can be used only in limited quantities. An example is the construction of the superstructure, where the proportion of very low level radioactive steel cannot exceed 32% of the whole reinforcement. In this case, the utilization of this kind of steel for straight reinforcement rods that would be topped up with common steel rods could be taken into consideration as an alternative.

6 Discussion and Conclusion

The concept of conditional release of materials has received an increased attention from major world and international organizations working in the field of nuclear energy (IAEA, USNRC) in recent years. The quantity of materials produced during decommissioning of nuclear facilities ceases to be negligible. Its price as of potentially recyclable products is increasing along with the cost of its disposal in the case of not using the concept of its release into the environment. In the past, a number of studies were developed that directly or partially address the concept of conditional release, recycling and the reuse of very low level radioactive materials [8-10].

The aim of this paper was to present, based on calculations using the VISIPLAN 3D ALARA, that this material may be, keeping several criteria, subject to the economical utilization for the construction of bridges without exceeding permitted radiation doses to workers. Further follow-up studies will be devoted to a long-term impact of utilizing very low level radioactive reinforcement of bridges on the environment and people living near such structures. Preliminary results of these long-term impact simulations show that this effect can be considered insignificant.

By recycling and reuse of very low level radioactive steel in structural engineering it would be possible to save considerable financial funds on both nuclear and construction sides and contribute to the sustainable development of society in the modern era of nuclear energy. For example in the case of prefabricated alternative more than 3 000 t of reinforcement could be made of very low level radioactive steel. It is expected that price of this material will not exceed 10% of the price of common reinforcing steel so its use could save huge amounts of investments for construction industry.

This study also aims to point out the new material the presence of which we begin to notice increasingly in practice and in the foreseeable future it will be necessary to deal with this material as effectively as possible. Therefore it is important to devote a heightened attention to this area in the near future and to try to find a globally acceptable solution. One solution could represent the alternative of utilizing very low level radioactive reinforcement in the construction of bridges that was described and presented in the article.

Acknowledgements

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Analysis of Secondary Effects and Bond Behavior due Prestressing

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Keywords: post-tensioning, bond behavior, secondary effects, statically indeterminate system.

Abstract. Subject of this article is an introduction of experimental research realized by department of a concrete structures and bridges at Slovak University of Technology in Bratislava. This research was focused on analysis of secondary effects due prestressing by changing statically indeterminate system to a kinematical mechanism. Other part of this research was observing of bond behaviour of a strands coated with corrosion protection agents.

Introduction

Prestressing tendons are usually necessary solution for long spans concrete structures and bridges, which are characteristically with their shape and geometry of tendons along construction. Due the prestressing is implemented active compressing stress to the concrete structure. This effect leads to the increasing of a bending stiffness in comparison to the general reinforced concrete structures. After crack initiation by bonded tendons is possible to calculate with their tension strength on total load capacity of the structure. Disadvantage of prestressing strands is their predisposition to notched corrosion. This is the reason why is necessary to protect prestressing steel before final grouting in construction. For that purpose are usually used various oil based agents as a strand coating, which can have also influence on their bond with a cement grout.

Influence of corrosion protection agents on bond

Bond is a basic material characteristic expressing rigid connection of two or more materials. For a prestressing strands can be bond between them and surrounding material generally defined on two levels which are adhesion and friction. Mechanical interlocking has a minor influence on bond in case of seven wire strand. Both of primary mechanism, adhesion and friction, can be significantly affected by using of oil based compounds as a corrosion protection.

Experimental research

For the experimental research were produced seven post-tensioned beams with cross sectional dimensions 0,25 x 0,4 m with length 10,5 m and concrete grade C35/45. Beams were reinforced with general concrete reinforcement and also with two post-tensioned seven wire strands ϕLs15,7 mm/1860 MPa. One of the strands was leaded by bottom surface of the beam and other one was deviated for minimalizing his secondary effects due prestressing (concordant tendon). Direct strand on other side caused maximal secondary effect due prestressing. Scheme of longitudinal leading of strands is shown on fig. 1. Bonded strands were leaded in HDPE ducts and anchored on both sides of the beam. Strands were prestressed by force P_0 = 200 kN applied from one side.

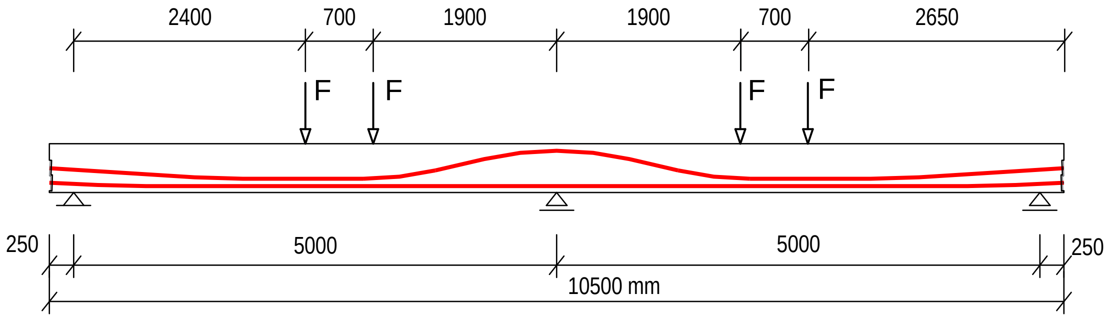

Fig 1. Scheme of longitudinal leading of strands

Tested beams were supported in three points which provides boundary conditions of continues two span girder. Each of spans was loaded with two increasing nodal forces. Ascending of load acting on a statically indeterminate systems leads to plastic hinge development by middle support cross section. This effect cause change from statically indeterminate system to statically determinate two simple beams.

The objective of the experimental research was an observation of effect by decreased bond between tendons and injection material due oil based strand coating and also changes of statically scheme from indeterminate to determinate system. Therefore were post-tensioned beams tested to their collapse. Prestressing force in strands was recorded with elasto-magnetic gauges, which were placed on all of strands in each beam. Fig. 2 shows EM gauge placed on tendon duct. For observation of differences between beams with various grade of bond, two of them were prestressed with unbonded tendons, other two with tendons with decreased bond due corrosion coating and last three were prestressed with fully bonded tendons. One after another beams were placed on testing set up, post-tensioned, injected with cement grout and after reaching of seven days strength of the grout they were tested. Testing load was entered as two nodal forces acting on each span. Force was ascending symmetrically on both of the spans.

Fig. 2. Detail of EM gauge placing and detail of a front of the beam with prestressing anchor heads.

Experiment results

Experiment results were compared with plastic analysis. This method of analysis assumes plastic hinge development after reaching of maximal carrying capacity of a cross section. That method also allows additional load acting on static system were the plastic hinges are already developed. Failure criterion is assumed after multiple plastic hinges are developed or kinematical mechanism development. First plastic hinge development was awaited near middle support and other one in mid-span area. Theoretical resistance of a cross section was based on equilibrium state of acting inner forces till reaching maximal strain in concrete with value $\varepsilon cu = 0{,}0035$.

Tab. 1 Comparison of theoretical and measured values by failure

Type of beam prestressing	Theoretical acting force	Measured acting force	gap
	[kN]	[kN]	[%]
Bonded tendons	4×121,5	4×129,5	6,0
Tendon with decreased bond	4×121,5	4×129,5	6,0
Unbonded tendons	4×90,0	4×125,6	28,3

No differences were observed on beams behaviour during serviceability limit state till first crack has developed as it is show on fig. 3. First cracks all of the beams have started to develop by load 4x49,1 kN. Corresponding bending moment near middle support has value 90,8 kNm. Assuming bending moment by self-weight and prestressing the total bending moment has value 58,7 kNm.

First cracks development has indicated differences in bond behaviour. Bonded tendons lead to many cracks development in most exposed areas and close to the most exposed areas as it is shown on fig. 4a. On other site unbonded tendons lead to the one central crack development with bigger width and few other cracks near the central one with significantly smaller width as it is shown on fig. 4b. Central crack in higher load levels leads to general reinforcement breaking. As seen by cracks tendons with decreased bond had a similar behaviour to the bonded tendons.

Tab. 2 Theoretical values of beams resistant

Type of beam prestressing	Cross section	Linear-elastic analysis			Plastic analysis		Total acting load on static system	Total resistance
		Initial bending moment	Acting load	Bending moment due acting load	Acting load	Bending moment due acting load		
		[kN.m]	[kN]	[kN.m]	[kN]	[kN.m]	[kN]	[kN.m]
Bonded tendons	Middle support	-	4×88	-154,1	4×33,5	-	4×121,5	-154,1
	Mid-span	-		115		71,6		186,6
Tendons with decreased bond	Middle support	32,1	4×82,5	-152,1	4×7,5	-	4×90,0	-120
	Mid-span	-19,6		107,7		11,9		100

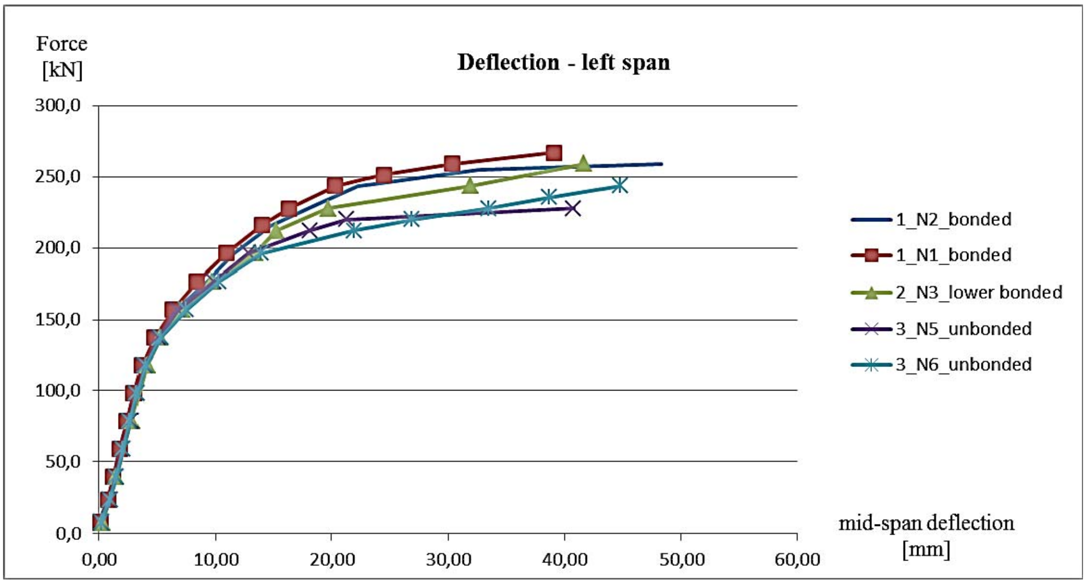

Fig. 3. Relationship between mid-span deflection and total acting force.

Fig. 4. Cracks development (plastic hinge development): a) bonded tendons; b) unbonded tendons

Conclusion

All measurements show that the tendons with decreased bond have similar static deformation behaviour to the bonded tendons. There are no significantly differences in beams behavior on serviceability limit state with regards to the various bond levels of the tendons. Different behaviour of the beams prestressed with tendons with decreased bond was observed by ultimate limit state close to the beam failure. At this level the behaviour starts to resemble to the unbonded tendons.

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DESIGN OF THE FOUNDATION STRUCTURES AND INTERACTION WITH THE SUBSOIL

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ABSTRACT

The main objective of this paper is to present the design concepts and methods of modeling of the soil-structure interaction with focus on foundation slab in combination with bored piles systems. The concept of linear global and local interaction will be described and discussed. The impact of relative subsoil rigidity on soil-structure interaction phenomena will be discussed as well. The authors will present the analysis and construction of foundations of spatial structures as: Bridge structures foundation, Deposit of used nuclear fuel rods, Bridge pier foundation, High rise building foundation, Radar tower foundation.

Keywords: subsoil and bored piles, soil-structure interaction, continuous and non-continuous interaction, FEM modeling

INTRODUCTION

All the existing approaches towards the solution of the issue of interaction of an engineering structure with the subsoil have been to a large extent affected by the interdisciplinary nature and complexity of the problem. The respective solutions are determined by the level of knowledge and practical experience based on longtime practice. The ability to abstract the required background materials from the experimental measuring, observation and to correctly synthesize them into a complex analysis is also significant. From the perspective of the soil mechanics and foundation engineering, the scientific literature ([Cytovič (1951)], [Terzaghi and Peck (1956)], [Boswell (1976)], [Bolteus (1984)], [Myslivec et al. (1970)], [Bažant, Z. (1973)], [Rojík et al. (1981)], [Kolář and Nemec (1989)], [Jesenák (1994)], [Kuzma (1987)] (and others) defines the interaction as the mutual influence and synergy between the building object (foundations and the entire structure) and subsoil.

[Bolteus (1984)] specifies a very pregnant and maximalistic but at the present already feasible problem of interaction as particularly complex and very complicated, feasible under the condition that all the decisive factors affecting the correct result are considered. He considers as necessary to develop such solution procedures which only take account of the essential characteristics of the problem, while he emphasizes that the decision on what is and what is not necessary is a very complex one. The author specifies the important characteristics which must be considered in a correct analysis of the issue of interaction as follows:

- the spatial effects of the system;
- change in the rigidity of the structure during the construction (with gradual increase of loading of the structure – incremental phasing);
- non-linearity of the characteristics of the subsoil materials involving time-dependent behavior.

A structural engineer who reads the works of [Rojík et al. (1981, 1988)], where the author states that the “redistribution forces resulting from the subsoil deformation may be as large as the forces resulting from the permanent and imposed loads” must verify the impact of the “interaction” phenomenon. In this material we present four examples from our designing work where we have taken the maximum reasonable account of the impact of the object/subsoil interaction.

LINEAR GLOBAL AND LOCAL INTERACTION

The numerical methods represent a very effective and relatively universal tool for the modeling of more complex interaction tasks. The mathematical apparatus used within the Final Elements Method (FEM) enables to consider also the complex marginal conditions (geometric and material non-linearity, non-homogeneity, the overall complexity of the structure and its 3D impact, and if necessary, also the phasing of the structure system in time), also for most demanding tasks solving the issue of interaction of an engineering structure and subsoil. At the present, the FEM belongs to the most universal and exploited methods used for the complex solution of various interdisciplinary problems in the engineering practice. Due to the above-mentioned reasons we use the FEM for static and dynamic model calculations, taking account of the global or local interaction of the object and subsoil. In [1], we have presented an illustration of a seismic analysis, where the interaction of the object and subsoil has been represented both by the global and local interaction. See Figure 1 for the schematic drawing of the model. We have established and we have used new terms such as "global" and „local interaction“.

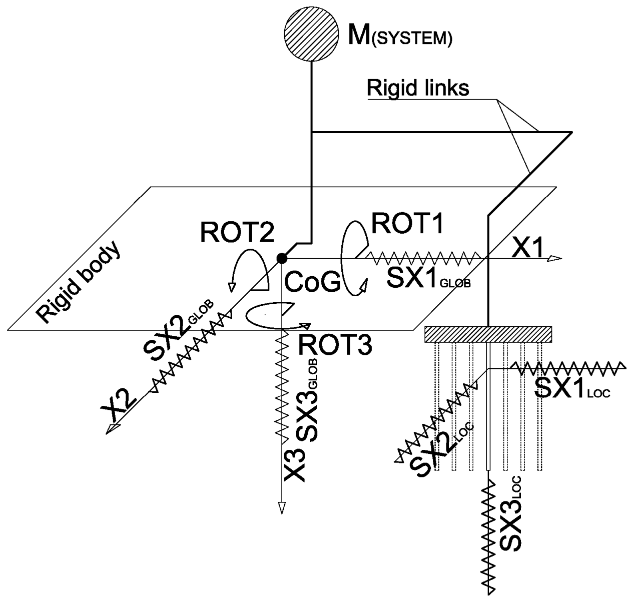

Figure 1. Diagram of the Interaction of the Structure ($M_{(system)})$ and Subsoil

The global interaction means equivalent springs (3 translational and 3 rotary) by which we simulate in the FEM model of the object its interaction with the subsoil. The foundation plate or structure is defined as a solid body and the global springs are defined in its centre of gravity. Such approach is suitable for solution and check of SLS (Service Limit State, check of deformations) and seismic analysis. It enables to eliminate the hard-to-solve phenomenon of the unsticking of the foundation plate from the subsoil in the dynamic analysis. It is only possible to compute the seismic response with the interaction of the subsoil, using the technique of the "bi-directional springs not active in tension" (which would eliminate the problem) by means of a quasi-static method or complex non-linear time analysis. Certain disadvantage is that such model does not allow the design of the foundation plate reinforcement and the part of the structure defined as a solid body.
The local interaction means equivalent springs (2 or 3 translational) defined at each calculation point of the foundation plate or structure being in contact with the subsoil. Such approach is suitable to solve the ULS (Ultimate Limit State), i.e. the design of the reinforcement in the foundation structures and piles. We also use the local interaction for the solutions for foundation structures with a limited width of cracking, i.e. the water proof concrete structures.

SEPARATE MODEL SOLUTION FOR THE INTERACTION

We provide an example from the designing practice (Figure 7), where we solve a bridge object with piers based on pile foundations. We examine the interaction on a model representing the foundation structure and subsoil, while the design already complies with the bearing capacity and serviceability limiting states.

BRIDGE PILE FOUNDATION

We have used the modeling of interaction of the foundation structure and subsoil to solve the task of designing the founding of the bridge object. All decisive facts concerning the geological relations in the territory of interest and constructional-technical solution of the bridge object (material, geometric and static) have been considered in the definition of the marginal conditions of the calculation model. The geo-technical design has resulted in the 32 piles, with the diameter of 1.2 m, 7.5 m long. The group of piles has been evenly placed in a 4 x 8 (width x length) grid, in an axial pitch of 1.7 m (along the width) and 1.9 m. The interaction of the piles is ensured by means of a foundation plate with the dimensions 2.0 x 7.0 x 15.5 m (thickness x width x length). The subsoil consists of silty clay (within the meaning of the STN 73 1001 Class F6-Ci) and Paleogen rocks (mostly claystones) with various degree of decay. The pile foot has been fixed (0.5 m) into the rocky subsoil of the R4 (STN 73 1001) qualitative class. The silty clays of the F6-CI class have been of stiff consistence and their deformation module has been determined at E = 3.0 MPa, i.e. relatively very amenable (soft) subsoil. For a more complex examination of the impact of the subsoil-foundation structure interaction, we have selected the two following calculation models:

a) Continuous model of interaction of piles and subsoil (the so-called “bi-directional bond” – the possibility of transfer both of the pressure and tension forces)
b) Discontinuous model of interaction of piles and subsoil (the so-called “one-directional bond” – the possibility of transfer of the pressure forces only).

Deformation characteristics (deformation module "E") of the clay subsoil (F6-CI) have been simulated for two alternatives:

1. Alternative: E = 3.0 MPa (relatively soft subsoil – real subsoil model);
2. Alternative: E = 30.0 MPa (relatively stiff subsoil – fictitious subsoil model);

The chosen calculation models will allow for a more complex quantitative and qualitative assessment of the foundation-subsoil interaction. A force load of F = 1.0 MN has been applied to the foundation structure taking effect at the eccentricity of e = 3.5 m (at the edge of the foundation), i.e. a torque load of M = 3.5 MNm. The task has been solved as planar (along the width of the foundation) within the meaning of theoretical assumptions of a linearly flexible semi-space. The computer program GEO-5 (FEM) has been used for the solution. See Figure 2 for the schematic drawing of the solved calculation models and defined marginal conditions.

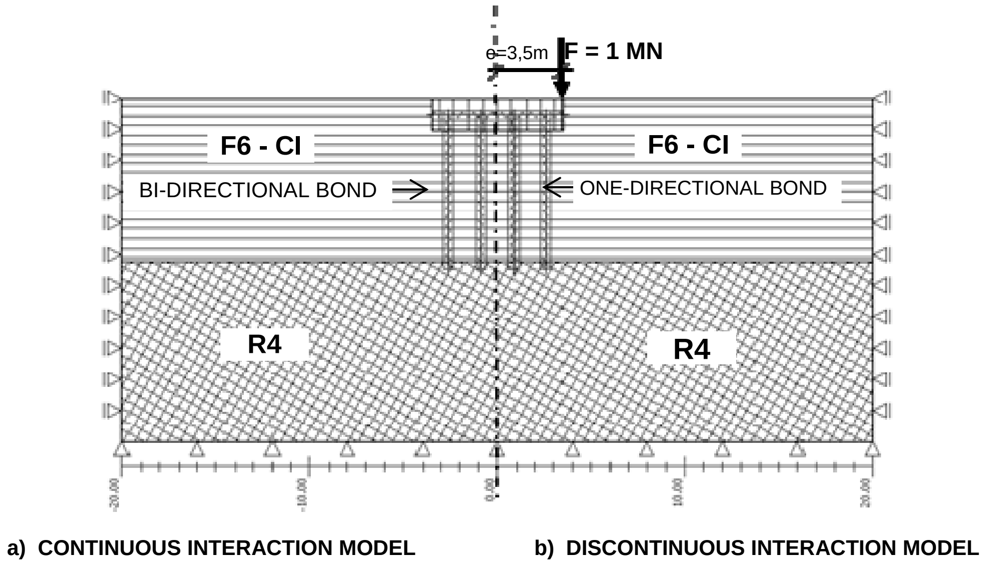

Figure 2. Calculation model (continuous and discontinuous) of the interaction of the subsoil with the group of piles

Results of the numerical calculations (iso-surface of horizontal deformations Δx) for the chosen calculation models and alternatives are specified in Figure 3 to 6. The following partial findings follow from the mentioned results:

- the impact of the linkage at the contact between the piles and subsoil will show by an increase in the horizontal deformation of the models with a one-sided linkage. In the case of the 1st Alternative (soft subsoil with E = 3.0 MPa), the maximum deformations in the case of the continuous model reach the size of 13.9 mm and in the case of the discontinuous model the size of 18.7 mm, which represents, in relative comparison, (discontinuous versus continuous model) a 34.5% increase in deformation. In the case of the 2nd Alternative (stiff subsoil with E = 30.0 MPa) the maximum deformations in the case of the continuous model reach the size of 4.4 mm and in the case of the discontinuous model the size of 6.4 mm, which represents, in relative expression, deformations are higher by 45.5% in the case of the discontinuous model.

- the impact of the subsoil interaction is more distinctive and shows in a larger extent in a subsoil with higher relative stiffness (higher impact of the pile-subsoil interaction in the tension areas of the model).

Based on the above-mentioned findings, we can say that correct determination of the foundation-subsoil interaction substantially affects the definition of the size of the design values of reinforced concrete elements of the foundation structure (internal forces and bending moments). Correct determination of the design characteristics is decisive for a reliable design of the foundation (from the view of the evaluation of the ULS and SLS). At the present, a growing emphasis is also placed on the cost-effectiveness of the design (in particular by investors). Given that the interaction of the foundation structure and subsoil (in particular with regard to the respecting of the physical essence of the problem solved) is adequately and correctly taken into account, it is possible to obtain an optimum design, while maintaining the priority requirement for the reliability of the structure and also with regard to the required cost-effectiveness.

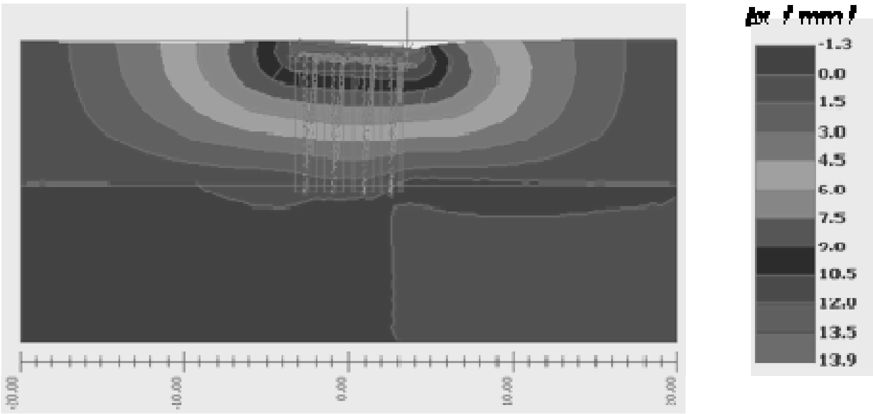

Figure 3. Iso-surfaces of Horizontal Deformations for the Continuous Interaction Model

(1. Alternative: for the F6-CI deformation module E = 3.0 MPa)

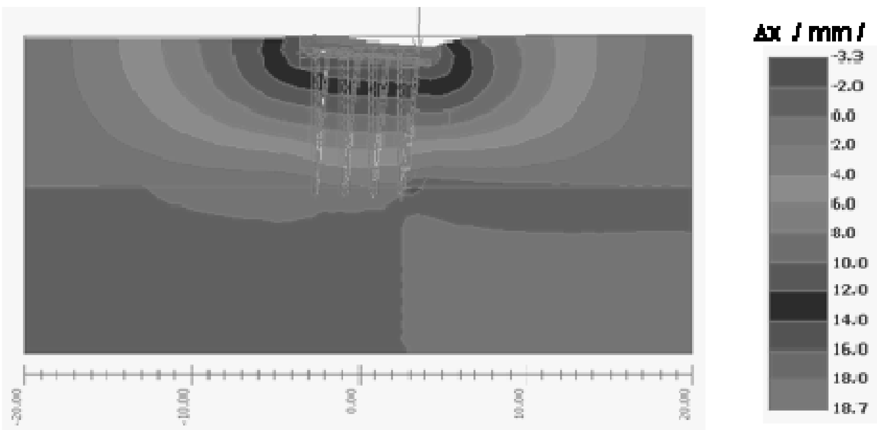

Figure 4. Iso-surfaces of Horizontal Deformations for the Discontinuous Interaction Model (1. Alternative: for the F6-CI deformation module E = 3.0 MPa)

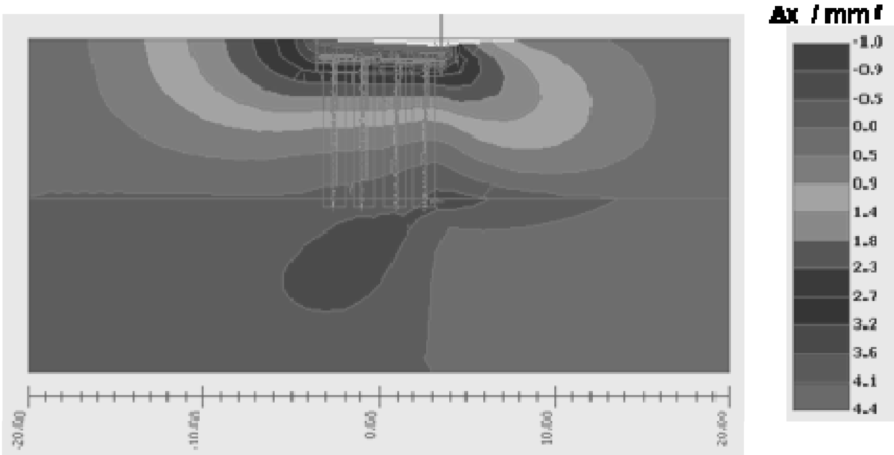

Figure 5. Iso-surfaces of Horizontal Deformations for the Continuous Interaction Model (2. Alternative: for the F6-CI deformation module E = 30.0 MPa)

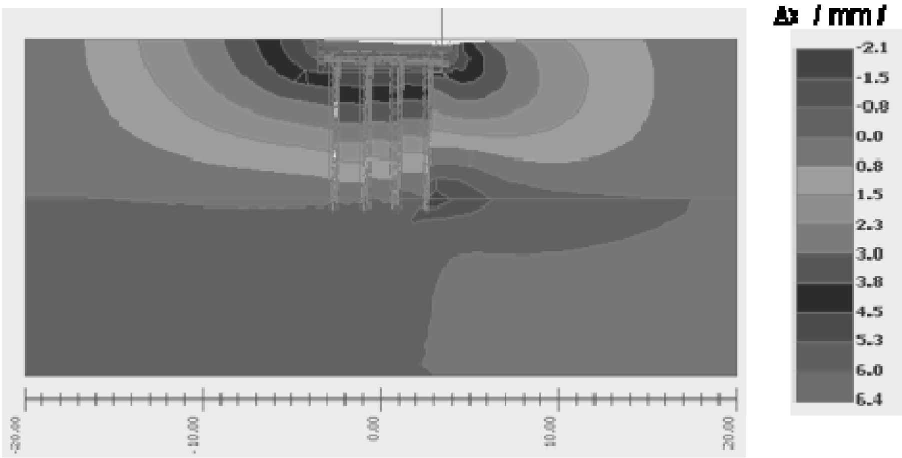

Figure 6. Iso-surfaces of Horizontal Deformations for the Discontinuous Interaction Model (2. Alternative: for the F6-CI deformation module E = 30.0 MPa)

EXAMPLES

Bridge Pier on the Motorway M8 on the River Danube, near Dunaújváros, Hungary

With the complex check calculation of the bridge structure of extraordinary importance, we have also solved the impact of the pier/subsoil interaction. We have simulated the body of the pier, foundation plate and the lenticular side arm lining in the pier head alternatively by means of the 3D shell elements and cubic elements. We have simulated the large-diameter piles (diameter of 1.5 m and length of 33 m) with the use of the 3D beam elements. We have simulated the local interaction of the pier and subsoil with the use of constant springs not active in tension, which have simulated the interaction at each contact point/calculation nod. We have examined the response to the design horizontal loads. The seismic response has been solved by the quasi-static method**.** We have examined 15 model interpretations of the subsoil in a parametric study.

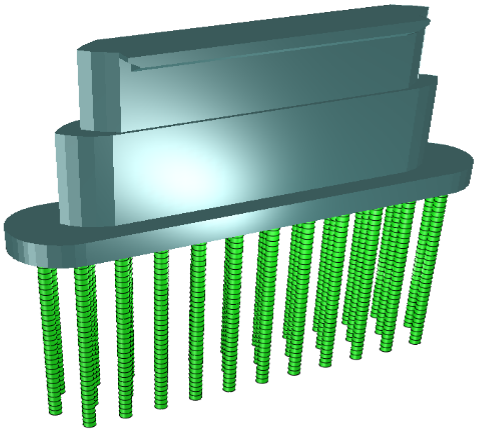

Figure 7. FEM Model of the Pier of the Bridge over the River Danube on the Motorway M8

Interim Storage of the Used Nuclear Fuel (ISUNF) at the Jaslovske Bohunice Nuclear Power Plant

The long-term ISUNF monitoring tracks the irregular settlement of the object which represents a typical load caused by the interaction. We have set the irregular settlement so that we have entered into the respective nodes of the calculation FEM model the deformation pulses representing the vertical measured/calculated displacement of the structure at the relevant point. We have used the measured deformations at 13 locations from which we have interpolated/extrapolated the load of the system.

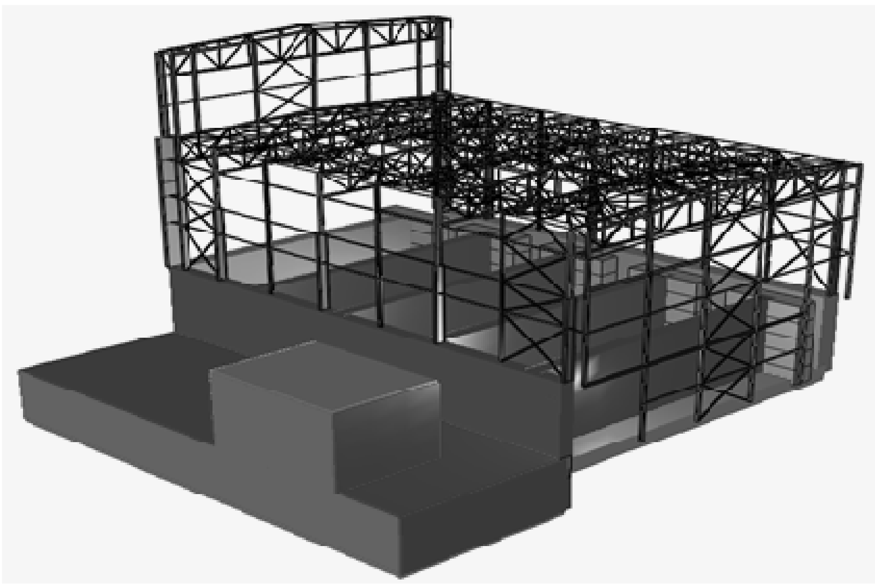

Figure 8. FEM Model of the Foundations, Pond Area and Steel Hall

Bridge on the Motorway D3, Žilina – Strážov, Slovakia

Within the static calculations on the CD (Contract Drawings) degree, we have made both the static and dynamic calculations. For the first time, we have solved the seismic response with the use of the time analysis with the linear local interaction model.

 

Figure 9. Bridge on the Motorway D3 and the Construction Process

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REQUIRED REINFORCEMENT AREA FOR THE CONTROL OF CRACK WIDTHS IN CONCRETE STRUCTURES

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Abstract: Visible cracking occurs when the tensile stresses exceed the tensile strength of the material. Visible cracking is frequently a concern since these cracks provide easy access for the infiltration of aggressive solutions into the concrete and reach the reinforcing steel or, other components of the structure leading to deterioration. The design of a structure with reduced width of the cracks can be done using a variety of standards and guidelines. The individual guidelines introduce different procedures for design of the amount of the reinforcement for the cracks width limitation. This paper deals with calculation of the required reinforcement area for the crack widths limitation according to national annexes of the standard EN 1992-1-1, Model Code 2010 and other standards and their differences in the design of the structures.

Keywords: watertight concrete, cracks width, minimum reinforcement area, cracks spacing

1 Introduction

In concrete, mortar and cement paste shrinkage takes place from the very beginning of the life of the material. In early age volume change can be both swelling and shrinking, but later shrinkage is relevant, which is caused by water movement in the porous and rigid body. During the hydration of cement (in the first 2 to 8 hours), while the cement paste is plastic, fresh concrete and cement mortar undergoes a volumetric contraction (plastic shrinkage) and free water content is moving toward the external surface of the specimen. After compaction and subsidence of particles due to its surface tension water is absorbed from the capillary pores towards the external surface and evaporated. Volume reduction of the outer layer is inhibited by the inner part of the material, and this can result map-like wide cracks. During the hydration of cement paste also a volume change occurs (autogenous shrinkage), due to the hydration products (cement matrix) volume is less than the volume of the raw materials (cement + water). However, the extent of hydration prior to setting is small, and once a certain stiffness of the system has developed, the contraction induced by the loss of water due hydration is greatly restrained. Withdrawal of water from concrete in unsaturated air causes drying shrinkage. A part of this drying shrinkage is irreversible and should be distinguished from the reversible moisture movement caused by alternating storage under wet and dry condition. Plastic, autogenous and drying shrinkage together are called early age shrinkage. [3], [6]

Influencing factors of early age shrinkage in mix design:

- cement content of the paste; specific surface area of cement
- fine aggregate content (under 0.125 mm particle size); specific surface area of fine aggregate
- water-cement ratio
- total aggregate content
- type of aggregate; water absorption capacity/water content of aggregate
- applied admixtures
- compacting rate of paste
- porosity
- other added components e.g. fibres.

Shrinkage of concrete depends on the temperature of concrete and its surroundings, on relative humidity and on the velocity of air movement as well as the curing and composition of the concrete. To fulfil the requirements of crack-free structures is often a problem during the design and construction of concrete and reinforced concrete structures, e.g. exposed concretes, hydraulic engineering works, gas- and water- tight concretes. Crack formation is also disadvantageous from the point of view of durability. [3], [6]

2 Design of reinforcement area according to EN 1992-1-1

2.1 The control of cracks width with the direct calculation

This control is based on the conception shown in Fig. 1.

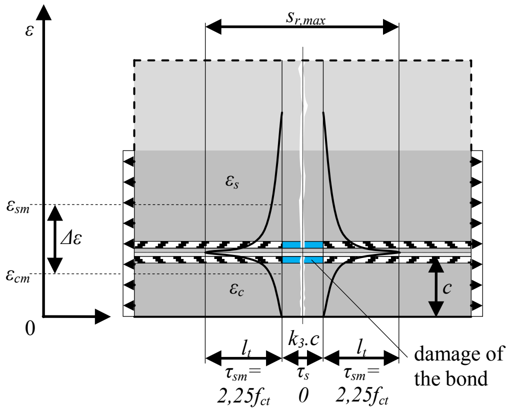

Fig. 1 Conception of calculation of crack width according to STN EN 1992-1-1 [4]

The crack width may be calculated from the expression:

$$w_{s,\max} = s_{r,\max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \qquad (2.01)$$

where

$s_{r,\max}$ - is the maximum crack spacing [mm]

ε_{sm} - is the mean strain in the reinforcement under the relevant combination of loads

ε_{cm} - is the mean strain in the concrete between cracks

Difference of the mean strains may be calculated from equation:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s}{E_s} - k_t \cdot \frac{f_{ct,eff}}{\rho_{p,eff} . E_s} . (1 + \alpha_e . \rho_{p,eff}) \geq 0.6 \frac{\sigma_s}{E_s} \qquad (2.02)$$

where

σ_s - is the stress in the tension reinforcement assuming a cracked section [kPa]

E_s - design value of modulus of elasticity of reinforcing steel [kPa]

k_t - is a factor dependent on the duration of the load [-]

$f_{ct,eff}$ - is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur: $f_{ct,eff} = f_{ctm}$ or lower $(f_{ctm}(t))$, if cracking is expected earlier than 28 days [kPa]

$\rho_{p,ef}$ - is the effective reinforcement ratio [%] $= A_s / A_{c,eff}$

A_s - is the area of reinforcing steel within the tensile zone [m2]

$A_{c,eff}$ - is the effective tension area [m2]

α_e - is the ration E_s / E_{cm} [-]

E_{cm} - is the secant modulus of elasticity of concrete [kPa]

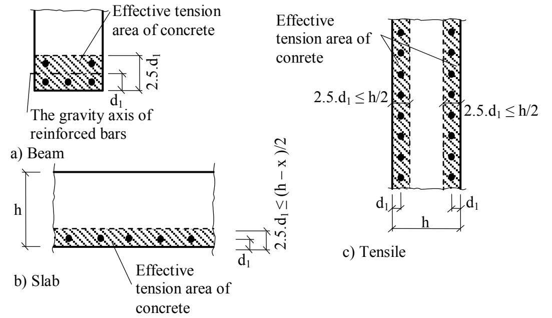

Fig. 2 Effective tension area

The maximum crack spacing calculated from the expression:

$$s_{r,\max} = \begin{cases} k_3.c + k_1.k_2.k_4.\dfrac{d_s}{\rho_{p,eff}} & s \le 5.(c + \dfrac{d_s}{2}) \\ 1.3 \cdot (h - x) & s > 5.(c + \dfrac{d_s}{2}) \end{cases} \qquad (2.03)$$

where

c - is the cover to the longitudinal reinforcement [m]

d_s - is the bar diameter [m]

k_1 - is a coefficient which takes account of the bond properties of the bonded reinforcement:
= 0.8 for high bond bars
= 1.6 for bars with an effectively plain surface (e.g. prestressing tendons)

k_2 - is a coefficient which takes account the distribution of strain:
= 0.5 for bending
= 1.0 for pure tension

k_3 - is the recommended value
= 3.4

k_4 - is the recommended value
= 0.425

s - is the spacing of bars [m]

h - is the overall thickness of a cross-section [m]

x - is the neutral axis depth in the stage II [m]

2.1 Minimum reinforcement areas according to EN 1992-1-1

$$A_{s,\min} = k_c.k.f_{ct,eff}.\frac{A_{ct}}{\sigma_s} \qquad (2.04)$$

where

$A_{s,\min}$ - is the minimum area of reinforcing steel within the tensile zone [m^2]

k_c - is a coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm:
For pure tension $k_c = 1.0$
For bending or bending combined with axial forces
- For rectangular section and webs of box sections and T-sections:

$$k_c = 0.4 \cdot \left[1 - \frac{\sigma_c}{k_1 \cdot \left(h/h^* \right) \cdot f_{ct,eff}} \right] \le 1$$

where

σ_c - is the mean stress of the concrete acting on the part of the section under consideration

h^* $\quad h^* = h$ for $h < 1.0m$
$\quad h^* = 1.0m$ for $h \ge 1.0m$

k_1 - is a coefficient considering the effects of axial forces on the stress distribution:
$k_1 = 1.5$ if N_{Ed} is a compressive force
$k_1 = \dfrac{2 \cdot h^*}{3 \cdot h}$ if N_{Ed} is a tensile force

k - is the coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces
= 1.0 for webs with $h \le 300mm$ or flanges with widths less than 300 mm
= 0.65 for webs with $h \ge 800mm$ or flanges with widths greater than 800 mm
Intermediate values may be interpolated.

A_{ct} - is the area of concrete within tensile zone. The t ensile zone is that part of the section which is calculated to be in tension just before formation of the first crack.

3 Adjustment of calculation according to DIN EN 1992-1-1/NA

DIN EN 1992-1-1/NA introduces the following changes in the calculation:

1.) Changes in the equation for the calculation of the cracks spacing:
- coefficient $k_3 = 0$, because DIN EN 1992-1-1/NA is not considering the loss of bond of reinforcement and concrete near the crack
- conjunction of coefficients $k_1 \cdot k_2 = 1$; allowed only high-bond bars
- coefficient $k_4 = 1/3.6$, which corresponds to the stress in bond $\tau_{sm} = 1.8 \cdot f_{ctm}$ for high-bond reinforcement

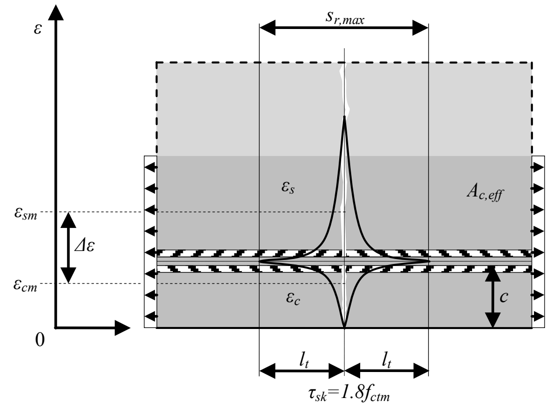

Fig. 3 Conception of calculation of crack width according to DIN EN 1992-1-1/NA [4]

Taking into account the above assumptions of DIN EN 1992-1-1/NA applied for maximum cracks spacing:

$$s_{r,\max} = k_3.c + k_1.k_2.k_4.\frac{d_s}{\rho_{p,eff}} = \frac{d_s}{3.6 \cdot \rho_{p,eff}} \le \frac{\sigma_s \cdot d_s}{3.6 \cdot f_{ct,eff}} \qquad (3.01)$$

2.) Effective mean value of the tensile strength of the concrete $f_{ct,eff}$ [kPa]
- if is possible to determine with certainty the formation of cracks in the first 28 days
$= 0.5 \cdot f_{ctm}$ if a crack is created at the time 3 to 5 days
$= \max\left(f_{ctm}; 3.0MPa\right)$ - hardening concrete
$= \max(0.5 \cdot f_{ctm}; 1.5MPa)$ - green concrete
- if is not possible to determine with certainty the formation of cracks in the first 28 days
$f_{ctm} \ge 3.0MPa$ - ordinary concrete
$f_{ctm} \ge 2.5MPa$ - lightweight concrete

3.) Effective tension area $A_{c,eff}$ is determined based on the Fig. 2. However, it is necessary to take into account the extra diagram in Fig. 4, taking into account the effect of the thickness of the element on the effective thickness of the tension area.

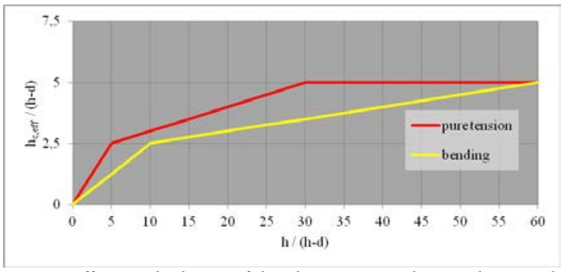

Fig. 4 Effective thickness of the element according to the actual thickness of the element [1]

4.) Changes in the equation for minimum reinforcement area

k - is the coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces,

- in the presence of tensile stresses induced due the Eigen-stresses from restraints (e.g. a decrease in heat of hydration) 0.8

  $k = 0.8$ for $h \leq 300$mm

  $k = 0.5$ for $h \geq 800$mm

  Intermediate values may be interpolated.
- when the tensile stress is induced by external stress (e.g. slump supports) applies: k = 1.0

4 Calculation according to Model Code 2010

$$w_d = 2.l_{s,\max}.(\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) \quad (4.01)$$

For the length $l_{s,\max}$ the following expression applies:

$$l_{s,\max} = k.c + \frac{1}{4}.\frac{f_{ctm}}{\tau_{bms}}.\frac{\varphi_s}{\rho_{s,ef}} \quad (4.02)$$

where:

k - is an empirical parameter to take the influence of the concrete cover into consideration. As a simplification $k = 1.0$ can be assumed.

c - is the concrete cover

τ_{bm} - is mean bond strength between steel and concrete (Table 1)

The relative mean strain follows from:

$$(\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) = \frac{\sigma_s - \beta.\sigma_{sr}}{E_s} + \eta_r.\varepsilon_{sh} \quad (4.03)$$

where

σ_s - is the steel stress in a crack

σ_{sr} - is the maximum steel stress in a crack in the crack formation stage, which, for pure tension, is:

$$\sigma_{sr} = \frac{f_{ctm}}{\rho_{s,ef}}.(1 + \alpha_e.\rho_{s,ef}) \quad (4.04)$$

where

$$\rho_{s,ef} = \frac{A_s}{A_{c,ef}}$$

with $A_{c,ef}$ = effective area of concrete in tension

α_e - is the modular ratio $= E_s / E_{cm}$

β - is an empirical coefficient to assess the mean strain over $l_{s,\max}$ depending on the type of loading (Table 1)

η_r - is a coefficient for considering the shrinkage contribution

ε_{sh} - is the shrinkage strain

Table 1: Values for τ_{bms}, β a η_r for deformed reinforcing bars [5]

	Crack formation stage	Stabilized cracking stage
Short term, instantaneous loading	$\tau_{bms} = 1.8.f_{ctm}(t)$ $\beta = 0.6$ $\eta_r = 0$	$\tau_{bms} = 1.8.f_{ctm}(t)$ $\beta = 0.6$ $\eta_r = 0$
Long term, repeated loading	$\tau_{bms} = 1.35.f_{ctm}(t)$ $\beta = 0.6$ $\eta_r = 0$	$\tau_{bms} = 1.8.f_{ctm}(t)$ $\beta = 0.4$ $\eta_r = 1$

5 Comparisons of standards

The comparisons of standards were made using the program MS Excel. The comparison of the overall required reinforcement area for crack width limitation under all the above standards shows the Fig. 8. Then, the comparisons were made influence of an individual parameters, which focus on differences in the standards STN EN 1992-1-1, DIN EN 1992-1-1/NA, Model Code 2010 (Fig. 5 to Fig. 7). In Fig. 9 is shown the comparison of the eight selected standards.

All comparisons were based on the following assumptions:

- concrete class C25/30 and cement class S,
- reinforcing bars grade B 500B with diameter = 16 mm,
- structural class S3, exposure class XC2, XC3,
  => the cover to the longitudinal reinforcement c_{nom} = 30 mm,
- maximum crack width $w_{k,max}$ = 0.2 mm,
- age of the concrete t = 5 days.

5.1 Comparison influence of the individual parameters

The approaches of individual standards are nearly identical. The biggest difference between the calculations is the determination of the cracks spacing.

When comparing of the individual parameters to the standards DIN 1992-1-1 and STN EN 1992-1-1 were observed for the slab thickness 2.0 m following the effects:

1. the coefficient k: 16.8% increase of the reinforcement area according to STN EN 1992-1-1,
2. the effective area of the tensile concrete $A_{c,eff}$: 18.8% increase of the reinforcement area according to DIN EN 1992-1-1,
3. the equation for the calculation of the cracks spacing: 35.4% increase of the reinforcement area according to STN EN 1992-1-1.

These differences can be observed for the various slab thicknesses in the following comparisons.

The influence of coefficient *k*

Fig. 5 shows the differences of required reinforcement area due to different values of the coefficient *k*. For comparison, the procedure selected in the required reinforcement according to EN 1992-1-1, the values of the coefficient were taken from the standards DIN EN 1992-1-1/NA and STN EN 1992-1-1

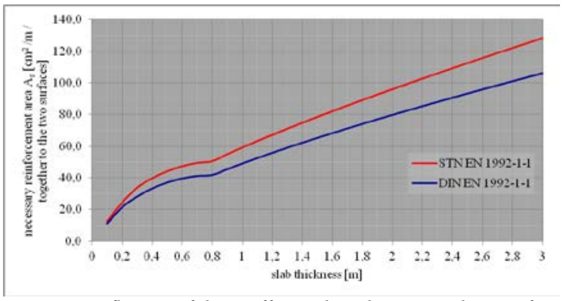

Fig. 5 Influence of the coefficient k to the required area of reinforcement

The influence of the effective area of tensile concrete $A_{c,eff}$

In contrast to other influence has the determination of the effective area on the basis of DIN EN 1992-1-1/NA the opposite effect than any other adjustments that standard, i.e. causes an increase in the required area of the reinforcement. This phenomenon can be observed in Fig. 6. For the calculation of the required area of the reinforcement was used the approach of EN 1992-1-1. The effective area of reinforcement was determined on the basis of DIN EN 1992-1-1/NA and STN EN 1992-1-1/NA.

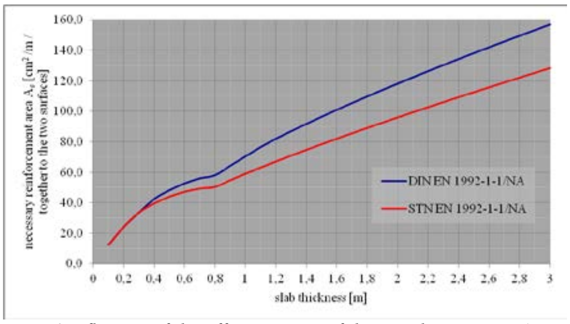

Fig. 6 Influence of the effective area of the tensile concrete $A_{c,eff}$

The influence of the coefficient k_1 to k_4

Given that the equation for calculating the distance between the cracks consists of several coefficients whose values differ depending upon the standard was made compare the influence of coefficients k_1 to k_4 to the required area of reinforcement shows Fig. 7. The values of the coefficients are selected on the basis of the individual standards. The calculation was made according to EN 1992-1-1.

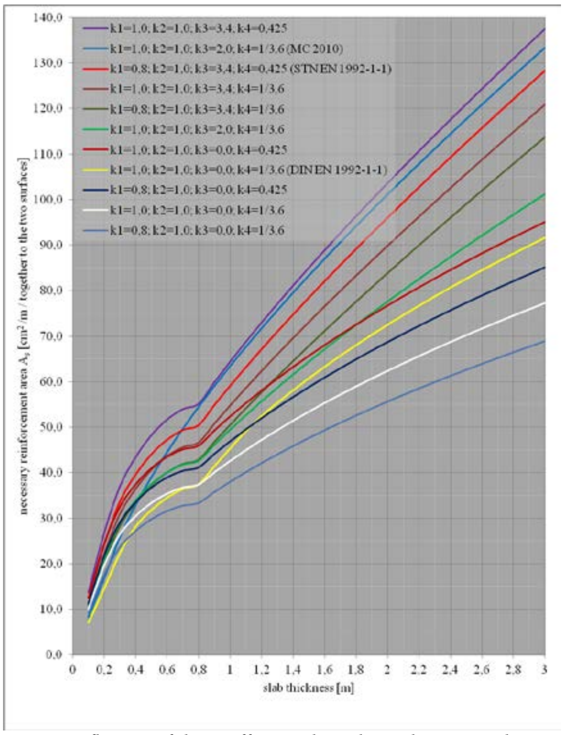

Fig. 7 Influence of the coefficients k_1 to k_4 to the required area of reinforcement

The influence of the equation for the calculation of cracks spacing

Fig. 8 highlights the differences caused by different approaches to calculate the cracks spacing. For comparison, the procedure selected in the draft reinforcement according to EN 1992-1-1, distance calculation cracks was performed using the coefficients k_1 to k_4 according to STN EN 1992-1-1, DIN EN 1992-1-1/NA and the Model Code 2010.

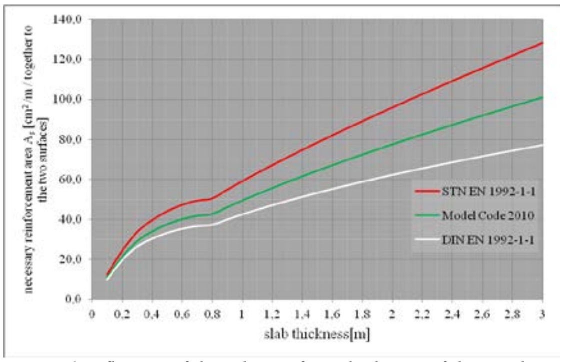

Fig. 8 Influence of the relation for calculating of the cracks spacing on the amount of a required reinforcement

The results of the comparison of the various standards

For the comparison (Fig. 9) of the required reinforcement area for the control of the crack widths were chosen the results of these standards: BS 8007, SIA 262, Model Code 1990, SS EN 1992-1-1, NF EN 1992-1-1, Model Code 2010, STN EN 1992-1-1, DIN EN 1992-1-1.

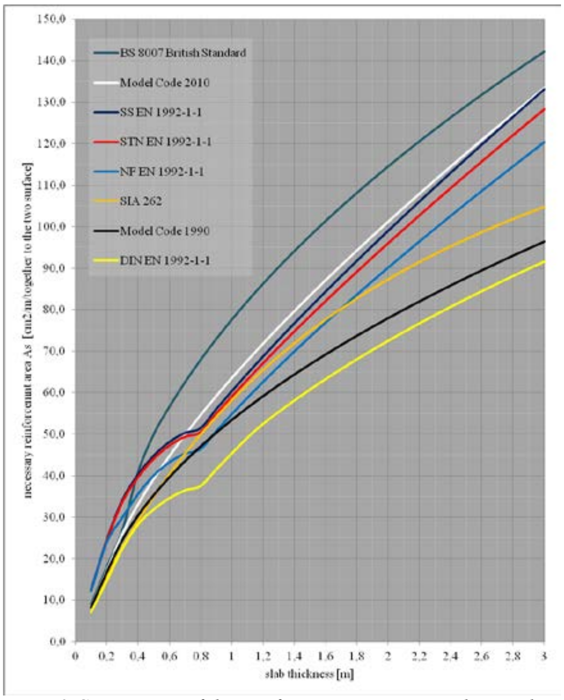

Fig. 9 Comparison of the reinforcement area according to the various standards

6 Conclusions

Cracking in concrete will occur in all but the simplest and smallest of structures. In the structure subjected to hydrostatic pressure a through-crack, of any size, can form a water path, which may result in leakage or wet patches occurring. It is the responsibility of the designer to limit design crack widths to a predetermined size to restrict or prevent water from leaking through the concrete into the basement. The principal and most effective method to control restrained shrinkage and thermal movement cracking is by the provision of sufficient reinforcement. The design approach for early-age thermal cracking adopted by STN EN 1992-1-1:2006 is broadly similar to that of DIN EN 1992-1-1/NA but there are some significant and important differences as follows:

1. the value of the coefficient k, which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces is according to DIN EN 1992-

1-1 lower by the 0.8 times than according to STN EN 1992-1-1,

2. the loss of bond of reinforcement and concrete near the crack in DIN EN 1992-1-1/NA is not taken into the consideration.

The analysis of different standards for the design of reinforcement required to control the crack width revealed, that significant savings in reinforcement area can be obtained using the DIN EN 1992-1-1/NA. Many parameters in STN EN 1992-1-1 vary according to DIN EN 1992-1-1 which causes an increase in steel reinforcement and a significant increase in cost. The main reason for the concentration of our research work to the mentioned topic is to find out the decisive parameters and try to derive their real values. The results demonstrate highly significant differences of the required reinforcement area according to the selected standards.

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Primary Paper Section: J

Secondary Paper Section: JM, JN

SLAB-ON-GIRDER BRIDGES IN SLOVAKIA

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ABSTRACT

This paper deals with experiences concerning of application and behaviour of precast slab-on-girder bridges in Slovakia. A development of motorways in Slovakia within last 15 years has brought on market wide variety of prestressed precast beams with in-situ topping and with spans ranging from 10 m to 42 m. From redundancy point of view, some solutions were simply supported structures and some solutions were hyperstatic systems where continuity over intermediate support is ensured by reinforced concrete diaphragms. A large number of completed bridges provide reach experiences with advantages and drawbacks of introduced systems.

1. INTRODUCTION

Precast technology in bridge construction has a very long tradition in Slovakia. The first bridges assembled from prestressed precast girders were completed in the late 50′s. The precast bridges were mostly designed as orthotropic slabs with shear concrete connectors in transverse direction. In a few cases there were used also post-tensioned diaphragms located in the spans. The first slab-on-girder bridges with composite behaviour were developed in the early 90′s.

Golden age of bridge construction in Slovakia is dated from 1996 to 2010 when program of motorways construction was hugely financially supported by government and partially by European Union. Mountainous terrain had brought need to built long elevated highways within a short time so prefabrication had become suitable solution of this problem. Well financing of infrastructure construction allowed to develop many types of precast bridge beams by Slovak construction companies.

2. STANDARDIZED PRECAST BEAMS FOR SLAB-ON-GIRDER BRIDGES

Precast bridge beams were usually developed as a set of beams allowing assembling single span or multi span bridges with a different span length and different width, without further structural calculation. Bridges were designed as isostatic structures even for multi-span systems. Elimination of the joints at intermediate supports was reached by continuity of RC deck slab where the slab was separated from the end diaphragms by soft membrane. It enables free rotation of the precast beams at the support. Type of prestressing depended on the maximum length of the precast beams. Pre-tensioning is used for sets of beams with maximum length of 32 m. If the set includes longer beams post-tensioning was the best solution how to cope with problem of beam transportation on public roads. Precast beams were usually cast from three segments. Each segment was transported separately on the construction site then they were prestressed together and erected on a final position in the bridge. The most frequently used precast bridge beams are introduced in the next part of the chapter 2. Besides, four additional sets of precast beams were developed in Slovakia, but they have not been used yet because of crises. They are post-tensioned beams Zipp 2009 (max. length 42 m), beams with hybrid prestressing DPS VP-T07 (max. length 45 m) and two sets of pre-tensioned beams (max. length 30 m).

2.1 Precast beams DPS I-96

DPS I-96 beam is the most used type of precast bridge beam in Slovakia. Posttensioned beams were designed in four length variants (24 – 27 – 30 and 42 m) with section depth of 1.10 - 1.25 - 1.4 and 2.0 m. These beams are composed of three segments, tensioned together on the construction site. The maximum transverse space between standardized beams is 1.50 m (wider spacing had to be confirmed by structural calculation). Precast beams were originally cast from concrete C35/45, later C45/55. Prestressing units are four strand tendons composed of low relaxation 7-wire strands ϕ15.5 mm/1800 MPa, e.g. the longest beam is prestressed by 10 tendons, the others by 7 tendons. The thickness of cast-in-situ slab is 0.20 m and concrete of C30/37.

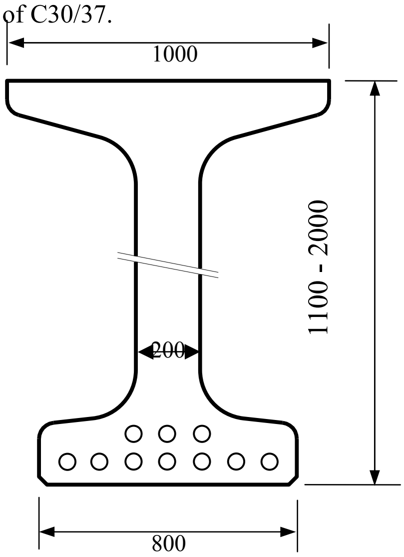
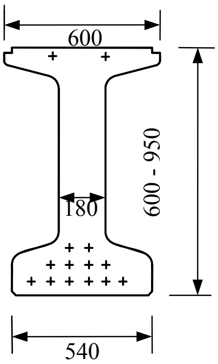


Figure 1 Cross-section DPS I-96 precast beam and view on beam forehead

2.2 Precast beams DPS VP I-97 and DPS VP I-04

DPS VP I-97 are pretensioned beams designed in four length variants (12 – 15 – 18 and 21 m) with section depth of 0.60 - 0.75 - 0.85 and 0.95 m. Concrete C45/55. The thickness of cast-in-situ slab is 0.20 m and concrete of C30/37. The maximum transverse space between standardized beams is 1.0 m.

DPS VP I-04 are pretensioned beams designed in three length variants (24 – 27 and 30 m) with section depth of 1.15 - 1.25 and 1.4 m. The thickness of deck slab is 0.16 m and concrete of C35/45. The maximum transverse space between standardized beams is 1.5 m. Concrete C55/67. Prestressing units are low relaxation 7-wire strands ϕ15.5 mm/1800 MPa in both sets of the beams.



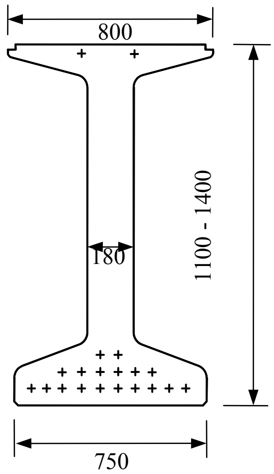


Figure 2 Cross-section DPS VP I-97 and DPS VP I-04 precast beams

2.3 Precast beams IST-97

IST-97 are pretensioned beams designed in eigth length variants from 9 to 30 m and section depth ranging from 0.70 to 1.40 m. Used concrete is C45/55. The thickness of cast-in-situ slab is 0.20 m and concrete of C30/37. The maximum transverse space between standardized beams is 1.25 m. Precast beams with lengths of up to 24 m have typical inverted T- cross section, while beams with length of 27 and 30 m were suplemented by top flange. Prestressing units are low relaxation 7-wire strands ϕ15.5 mm/1800 MPa.Inverted T crosssection has some advantages particularly at prestress transfer. Due to the low eccentricity the section is able absorb higher prestressing force, need of strand separation at the ends of beams and cambers are lower. Because of the small web width high shear stresses at inteface between in-situ slab and precast beam are the main drawback of this beam cross section.

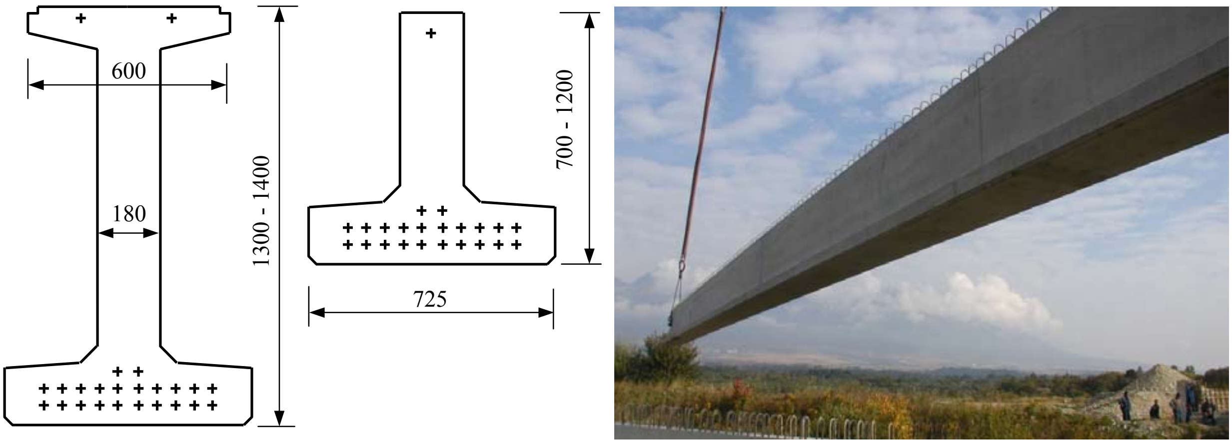


Figure 3 Cross-section of IST-97 precast beams

2.4 Precast beams DZ-97

The standardized precast beams are pretensioned members designed in eight length variants. Shorter beams have TT- shaped cross-section and lengths of 12 - 15 - 18 m, the depth varies from 0.76 to 0.93 m. Longer beams with lengths of 21 - 24 - 27 and 30 m have T-shaped cross section and depth is ranging from 1.00 m to 1.40 m. Concrete C45/55. Precast beams are spaced at maximum 1 m . The width of the upper flange was proposed with intention to remove usage of precast panels that cover gaps between precast beams during casting of the slab. The thickness of the deck slab is 0.16 m. Prestressing units are low relaxation 7-wire strands ϕ15.5 mm/1800 MPa.

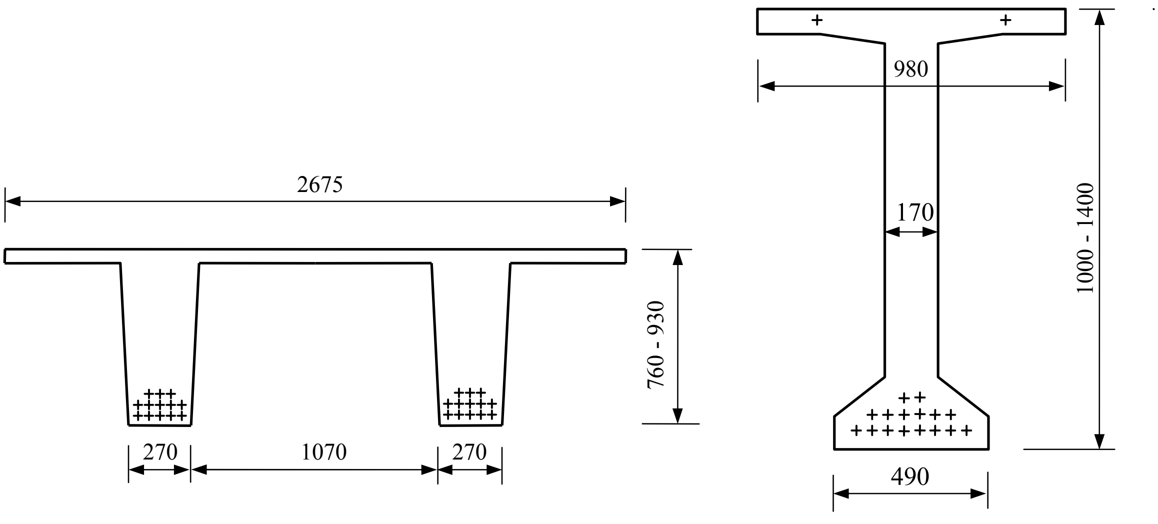


Figure 4 Cross-section of DZ-97 precast beams

2.5 Precast beams VPH PTMN - 2010

Pretensioned precast beams were designed in six length variants (18 – 21 – 24 – 27 – 30 and 32 m) with section depth of 1.20 and 1.40 m. Concrete C45/55. The thickness of cast-in-situ slab is 0.20 m and concrete of C30/37. Longer beams 38 and 42 m respectively consist of three segments and they have hybrid arrangement of prestressing (see fig.6). Central segment with a length of 24 m is partially pretensioned. The second part of prestressing, four strand tendons, is applied at a construction site to join segments. Prestressing units are low relaxation 7-wire strands ϕ15.5 mm/1800 MPa. The maximum transverse space between standardized beams is 1.5 m.

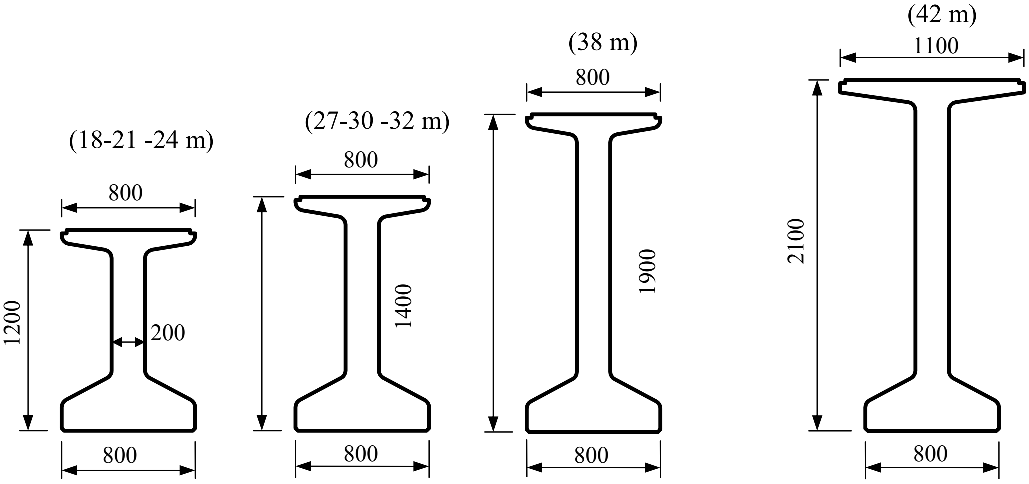


Figure 5 Cross-section of VPH PTMN - 2010 precast beams

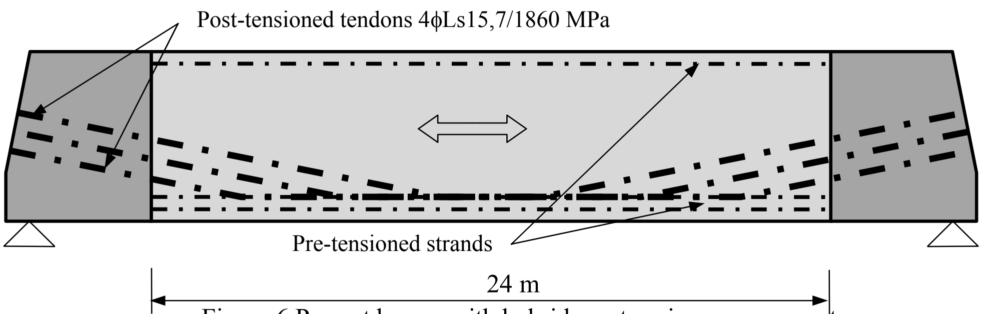


Figure 6 Precast beams with hybrid prestressing arrangement

3. MULTISPAN HYPERSRTATIC SLAB-ON-GIRDER BRIDGES

The standardized precast beams with a length of 30 m or more are frequently used for construction of long multi-span bridges. The first multispan bridges were completed as sequence of simply supported composite girders with continuous RC slab at intermediate supports. However this solution has two significant drawbacks. The first one is large amount of bearings, each precast beam requires two bearings, and the second one is lateral instability of precast beams during erection and casting of a deck slab. Both drawbacks accompanied construction of the first long viaduct at Beckov (Fig.7), finished in 1997. The viaduct consists of two expansion units having four and five spans, together 152 simply supported girders with a length of 42 m and 304 pot bearings. Precast beams of several spans slightly rotated around the longitudinal axis during the casting of deck slab. The rotation caused that the sealing steel rings of upper and lower part of bearings got contact each other on one side and big gap has arisen on the other side. This defect has threatened structural performance of the bearings in the bridge structure.

Figure 7 Viaduct at Beckov on Motorway D1 - built as sequence of isostatic girders

3.1 Construction method used for continuous slab-on-girder bridges

Bad experience with the viaduct at Beckov led to decision, re-design standardized precast systems into hyperstatic structures where continuity over intermediate support is ensured by reinforced concrete (non-prestressed) diaphragms. Diaphragms are always composed of two parts with composite behaviour. The first one is precast or cast-in-situ crossbeam (fig.8,9,10) which is also used as a platform for installation of precast girders. The second one is cast-in-situ part of a diaphragm. Precast crossbeams, having the width depending on a type and length of precast beam (measured in longitudinal direction of the bridge). For longer spans over 40 m the width of 2.8 m is used, for shorter spans (up to 33 m) the width varies from 1.8 m to 2.3 m. The depth of the precast crossbeams depends on used construction method.

Figure 8 Precast crossbeam temporally supported by 8 hydraulic jacks

Two construction methods are currently used. The first method is based on the precast crossbeams laying on system of hydraulic jacks that are placed on the top of the piers (Fig.8). Subsequently precast beams are placed on this crossbeam and the gap is filled by concrete (Fig.9). This method requires the higher depth of precast crossbeams (0.55 to 0.65 m) depending on the space between bearings and the length of cantilevers. Major advantage is in elimination of scaffolding during bridge construction. The lack of this method is in high consumption of reinforcement because precast crossbeams are highly stressed members that have to withstand beside self-weight, the weight of precast girders, cast-in-situ slab and cast-in-situ part of the crosshead. Therefore, this method can not be used for construction of bridges composed of longer precast girders e.g. 40 m long DPS I-96 beams.

Figure 9 Installed precast girders laying on the precast crossbeams

The second method requires scaffolding supporting the precast crossbeam (Fig.10,11). Columns of the scaffolding are usually spaced at 2 m and lay on the pier foundation. Because the precast crossbeam behaves a slab with the span length of 2 m, its depth can be lower (0.40 to 0.50 m).

Figure 10 Scafolding used to support precast crossbeam

Figure 11 Precast girders DPS I-96 placed on the precast crossbeams

3.2 Prestressing of precast girders in hyperstatic slab-on-girder bridges

As it is mentioned in chapter 2, standardized precast girders with a length up to 32 m were originally designed with pre-tensioning as simply supported members. Strands were straight and concentrated in the bottom flange. However, continuity created by RC diaphragms brought hogging bending moments at intermediate supports due to actions imposed on a completed structure (bridge furnishing, traffic load, thermal action, uneven settlement) and part of precast girders became exposed to tensile stresses in the top. Strands concentrated in the bottom made situation even worse here. Therefore prestressing has been updated and part of pretensioned strands was replaced by polygonal four strand tendons. Number of tendons depends on a type of precast beam, e.g. beam DPS VP I-04 (fig.12) with maximum spacing in transverse direction ranging from 1.5 to 1.8 m have three tendons while girders DZ-97 (fig.4) with spacing ranging from 1.0 to 1.25 m only two tendons. It is clear that originally standardized precast girders have to be individually designed for each project now. Hyperstatic solutions of slab-on-girder bridges allowed for saving of precast girders in bridge cross section, e.g. two girders can be removed from full motorway profile compare with a isostatic solution and length of span can be increased by width of the diaphragms. Application of hybrid system of prestressing may bring also a problem e.g. with camber.

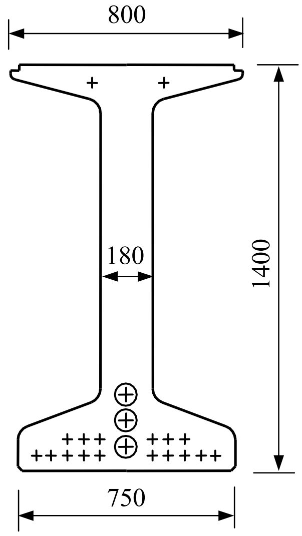


Figure 12 Precast girders DPS VP I-04 with hybrid arrangement of prestressing

3.3 Cambers of precast girder with hybrid prestressing

During construction of the 17 spans viaduct composed of DPS VP I-04 beams at city Poprad there were found out large differences between the cambers of erected beams. Cambers were ranging from 40 mm to 8 mm. Predicted value was 35 mm and the most frequently observed values 24 mm. Precast beams had a length of 32,1 m, depth 1,40 m and prestressing consisted of 18 pre-tensioned low relaxation strands ϕLS15,5/1800 MPa and three post-tensioned polygonal tendons, see Fig.12. Transfer of pre-tensioning strands was scheduled only 18 hours after casting and tendons were prestressed one month later. Running production of the beams for viaduct enabled to carry out extensive monitoring in order to determine reasons of these differences between predicted and measured values.

Figure 13 Elevated highway built from DPS VP I-04 beams at city Poprad

Monitoring consisted of in situ measurements and laboratory testing of material properties. In situ measurements included: measurements of actual cambers, measurements of the prestressing forces and concrete strains and finally measurements of the curvature due to uneven shrinkage on the beam segments with a length of 1 m (fig.14).

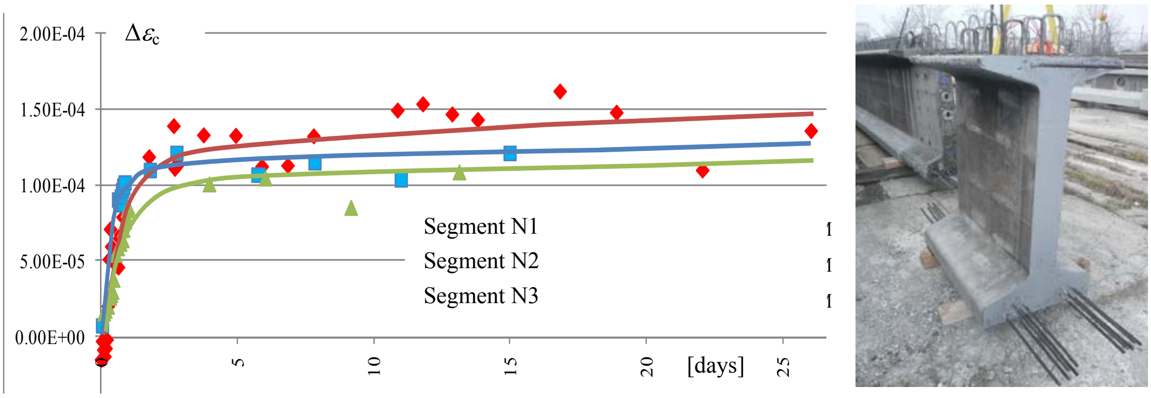


Figure 14 Development of strain differences $\Delta\varepsilon_c = \varepsilon_{top} - \varepsilon_{bottom}$

Monitoring revealed that the primary reason of low cambers was used technology of concrete placing and the way of concrete compaction which made beam cross-section non homogeneous, particularly when concrete was very young. Differences between shrinkage of upper concrete layers and lower ones (fig.14) led to development of constant curvature accompanied with deflection over 11 mm at mid span section. Because prestressing was divided into two stages, 18 hours old concrete was prestressed by only 57 % of all prestressing tendon embedded in the bottom flange, in the first stage introduced prestressing force was able to create camber of only several millimetres which was soon overcome by deflection due to the uneven shrinkage and the beam camber had changed into sag. Opposite, deformations due to the post-tensioning were perfectly matched with prediction. We believe if one stage prestressing is used beams are not so sensitive on such effects. High compressive stresses in the bottom create distinctive elastic camber and its further growth due to the creep of concrete. This effect is then much stronger than influence of cross-section non homogeneity.

4. CONCLUSION

Tens kilometres of elevated highways on slab-on-girder bridges built in Slovakia within last 15 years are good example of prefabrication efficiency in bridge construction. An application of precast beams has accelerated construction of motorways thanks to whole year production in precast plants and speedy execution.

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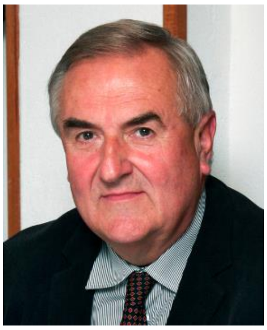

prof. Eng. Igor Hudoba, PhD.

The Application of UHSC for Load-Bearing Composite Elements and Structures

ABSTRACT

In last decade a great attention was paid for research and development of new generation of concrete. Existing knowledge in the field of high strength, resp. high performance concrete (HSC, resp. HPC) and ultra high strength concrete (UHSC), resp. ultra high performance concrete (UHPC) have been used for many successful projects of various kind of concrete structures all over the world. Codification of HSC classes in current standards created legal space and tools for its application in practical design and construction execution. Despite of certain resistance from the side of contractors and partly of ready mix concrete producers of HSC, respectively HPC is finding more and more space for its applications in concrete practice. At present time it is possible to observe also an increasing interest in research field of the UHPC in many countries. UHPC is gradually moving from the laboratory towards the concrete practice. As the matter of fact the design of elements, resp. constructions of UHPC is not jet reflected in valid standards. Only some regulations were published in the recent time.

In spite that a lot of information about successful UHPC applications have been presented and published recently.

Keywords: ultra high strength concrete (UHSC), composite CC (concrete-concrete) columns, failure capacity, failure mode

CHARACTERISTICS OF UHPC

In the recent time more attention is paid for research and practical utilization of UHPC. Looking into the detail UHPC can be characterized as a very high strength fine corn cementitous composite. Upper limit of the HSC compressive strength in accordance with valid standard is the concrete class C90/105. In accordance with generally accepted scale of HSC classification is its compressive strength defined till 150 MPa. Concrete with compressive strength over this limit is defined as UHSC. The main principles of UHSC is not only its strength parameters, but also its constituent composition, technological issues, procedures of dosing, mixing, casting and curing. Many of these specific technological issues are already well known. Therefore excellent mechanical and other physical properties of UHPC and specific technological approach is giving us the chance for its practical application. But it is necessary to emphasize that, because of many specific reasons, application of UHSC, resp. UHPC is appropriate mostly for special purposes at present time.

SPECIFIC MATERIAL ANG TECHNOLOGICAL ISSUES OF UHPC

Basic principle of ultra high strength level of UHPC is based on solving of four fundamental issues: There are: strength increasing of hardened cementitous matrix, using of high strength fine aggregates (max. size less than 1 mm), bond strengthening between cementitous matrix and corns of aggregate and finally utilizing of *confinement effect* based on 3-axis stress state of material [4]. Increasing of UHSC strength parameters is based on very low water-binder ratio (W/B). Binder value (B) represents the ratio between total amount of water to amount of cement including other active constituents like silica fume (SF), flying ash (FA) etc. In the case of fresh UHPC water-binder ratio use to be less than 0,2 . Such a low value can be reached by using of significantly increased amount of high effective super-plasticizers (SP), resp. high range water reducing admixtures (HRWRA). Important part of UHPC composition is also non-active fine aggregate, which has an important role of *filer* in hardened, very dense cementitous matrix. Very high strength and high density of hardened UHPC are the reason of its insufficient ductility. This problem can be solved by using of *steel micro-fibre reinforcement.* In general UHPC can be characterized as multi-constituent structural material. Hand in hand with above mentioned specific material requirements go technological process of UHSC composition design, dosing, mixing, casting and curing. High content of fine and ultra-fine solid particles in mix composition requires utilization of mixing equipment of very high efficiency. Therefore as the rule, smaller mixers with content of less than max.100 liters are used for UHPC mixing. The optimal and generally accepted way of UHPC dosing and mixing does not exist. Mixing of fresh concrete for UHSC and dosing of steel micro-fiber reinforcement show Fig.1. In the case of steel micro-fibers using in UHPC has to be paid great attention. High viscosity of fresh UHSC and absence of traditional way of fresh concrete compacting during its placing (by using of micro-fiber reinforcement) requires its rheological properties similar to self-compacting concrete. Specific property of fresh UHSC is its tendency to create „ *skin*“ on the surface practically in a few minutes after its casting into the mould, resp. formwork. Because of very low content of water it is necessary to start with curing of UHSC surface immediately after finishing of the casting process.

APPLICATION OF UHPC IN LOAD-BEARING COMPOSITE CONCRETE ELEMENTS

There are many arguments which are influencing the application of the UHPC for certain kind of elements, resp. structures. They depend especially on material basis, technology of fresh concrete production and many other processes of concreting in the local, resp. regional frame. The most important fact and condition in decision making process for application or not application of UHPC must be the effect of higher added value for the designed element, resp. concrete structures made of UHPC. As an example of such approach can be seen the research and development project realized at Department of Concrete Structure and Bridges , FCE, STU

Fig.1 Mixing of UHSC and dosing of steel micro-fibers

Bratislava (Prof.Hudoba, Ing.Mikuš) in collaboration with company STACHEMA Bratislava, Ltd. (Ing.Zemánek and Ing. Doválʼ). UHSC mix composition with 28-days compressive strength level of 140 MPa was developed. Using of local and regional raw materials for choice of constituents was applied as pre-condition for development of the UHSC. Informative mix composition and other basic parameters of UHPC are presented in Tab.1. Fig.2 shows the different types of UHSC reinforcing bars made of UHSC. Cylindrical UHPC bars with diameter of 72 to 80 mm and length of 1500 mm were used as the alternative reinforcement (instead of classical steel core reinforcement) in composite CC (Concrete-Concrete) columns.

Tab.1 Informative mix composition and basic parameters of UHPC

Mix constituent	
Cement CEM- A	831 kg
Silica fume	191 kg
Silica powder 0,9 μm	205 kg
Fine aggregate 0,1 – 0,4mm	914 kg
Superplasicizer	28 kg
Water	180 kg
Steel micro-fibres OL 612	144 kg
Water-binder ratio W/C+SF	0,21
Density of hardened UHPC	2385 [kg/m^3]
Compressive strength (28 days)	140 MPa
Tensile bending strength (28 days)	27,5 MPa
Flow test	650

EXPERIMENTAL INVESTIGATION OF COMPOSITE ***CC*** COLUMNS WITH UHPC CORE

Ultra high compressive strength of UHPC is possible to use e.g. in composite „*Concrete-Concrete"* columns (CC columns), where instead of traditional solid steel core the core made of UHPC will be applied. For that reason three types of UHSC cores were made. Fig.3 shows the cross section of such a composite column reinforcement. The core is centrically situated. Steel reinforcement consists of 4 Ø 12 mm longitudinal steel bars and links of 6 mm. Five different types of complete column reinforcement was prepared (Fig.4). From each type of reinforcement three columns were cast. All together 15 columns, resp. composite columns were made (Fig.5).

Fig.2 Different types of UHSC core bars

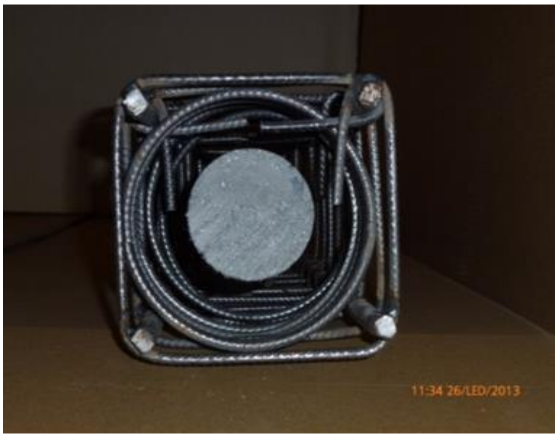

Fig.3 Cross section of CC-composite column reinforcement with UHSC core

Fig.4 View on the column reinforcement consisting of different types of UHSC cores

Fig.5 Set of columns prepared for laboratory tests

Fig.6 Preparation of the laboratory test

Table 2 Test results of composite columns failure capacity

Type of column	Column reinforcement	Average value of compressive failure capacity [kN]
A	4ØR12	1712
B	4ØR12 + solid steel core Ø70mm	2485
C	4ØR12 + smooth UHSC core Ø72mm	1973
D	4ØR12 + corrugated UHSC core Ø78mm	2050
E	4ØR12 + UHSC core Ø78mm in corrugated steel tube	2140

LABORATORY TEST RESULTS

Tests of column failure capacity were realized by using of laboratory test machine (Fig.6). Every column was exposed to pure compressive load. Column´s load have been increased step by step until its failure capacity. Table 2 shows the average values of columns failure capacity. Deformations of concrete on the surface of all four column sides were measured. Fig.7 shows the relations between pure compressive load and relative concrete compressive strain values (average values) of different type of columns. The failure mode of all type of tested columns was observed and identified. Tab.3 shows the characteristic failure modes of different type of tested columns.

CONCLUSIONS

Presented knowledge level and experience concerning the design and production of UHSC with compressive strength over 150 MPa gives a chance for its effective application in composite CC columns. By using of available domestic raw sources for UHSC design, common mixing equipment and processes, it is possible to made the UHSC with compressive strength level till 140 MPa.

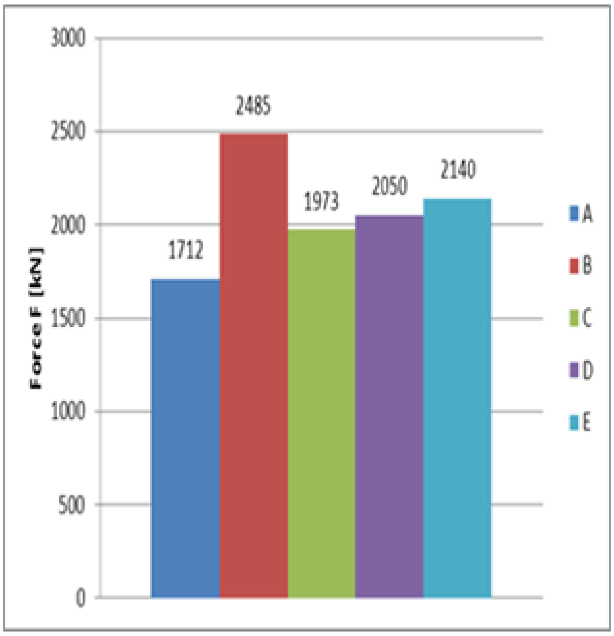

Fig.7 Comparison of load-bearin capacity of five different types of columns

Table 3 Characteristic failure mode of different type of columns

Column failure mode	 column type **A**	 column type **B**	 column type **C**	 column type **D**	 column type **E**

Production of UHSC with compressive strength level over 140 MPa requires specific equipment for mixing, using of thermal curing, resp. hardening of UHSC under compressive conditions. Substitution of solid steel core reinforcement by UHSC reinforcing bar brings certain advantages in the field of composite columns. Density of UHSC reinforcing bar is three times lower than steel core bar, its cost is much cheaper than steel and according to presented experience the technological and manufacturing processes of UHSC reinforcing bars production and its application in composite CC column can be well realized.

Acknowledgement

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Eng. Mária Bellová, PhD.

Some Remarks Towards Surface Reinforcement Used in Concrete Members

1 INTRODUCTION

In concrete members are there possible such special situations, when the cover of reinforcement of structural members exceeds 50 mm, and it may be appropriate to provide a surface reinforcement. Model Code 2010 makes a mention about such a reinforcement, but there are no data about its' type, cover and the area amount. However such a surface reinforcement is able to increase the bending and especially shear resistance of structural members, subjected to bending and shear. The cover reinforcement in addition to that – reduces an expansion and width of cracks.

According to Eurocode 2 Part 1-1 in Annex J – Detailing rules for particular situations – there are some special cases when such a surface reinforcement should be used. It is necessary to use this reinforcement type when:

- diameter of provided bars is greater than 32 mm, or equivalent diameter of bundled bar exceeds this limit;
- cover to reinforcement is greater than 70 mm (for enhanced durability);
- crack widths without using surface reinforcement exceeds allowed limit;
- reinforced concrete structure is subject to fire and there is a danger of falling off of a concrete in the latter stage of a fire exposure according to Part 1-2 of Eurocode 2.

The surface reinforcement should consist of *wire mesh* or *small diameter bars* and should be placed *outside of links.*

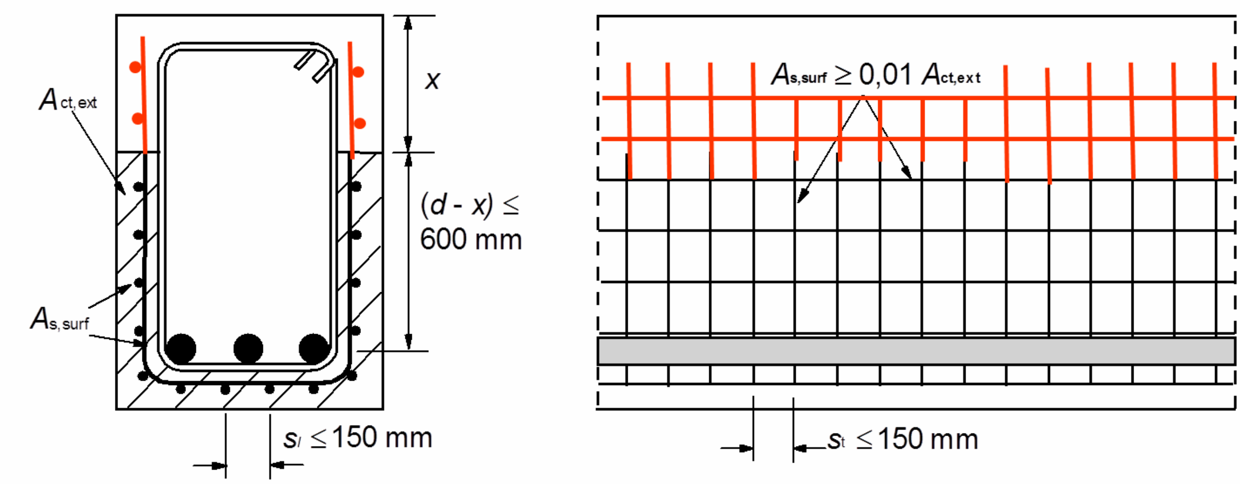
Figure 1 demonstrated the position and relevant terminology of the surface reinforcement, placed in two directions, **parallel** and **orthogonal** to the tension reinforcement in the beam. The values of $A_{s,surfmin}$ are also differentiated according to the *purpose of the surface reinforcement use.*

The *longitudinal bars of the surface reinforcement* may be taken into account as a contribution to the *longitudinal bending reinforcement* and the *transverse* bars as a contribution to provided *shear reinforcement.* The surface reinforcement shall however meet all the requirements for the arrangement and anchorage of these types of reinforcement. It is very important, that **the transverse bars with the shape of a letter "U" shall be properly anchored into the compression part of the reinforced concrete cross section.**

Reasons for using of the surface reinforcement:

- **Spalling of the concrete**
- **Cover to reinforcement cnom > 70 mm**
- **Crack width control**
- **Fire performance**

The summary of the minimal required areas of the surface reinforcement in various cases of its use shows Table 1.



x is the depth of the neutral axis at ULS

Figure 1 Position and required cross sectional area of the surface reinforcement

Table 1 The summary of the minimal required areas of the surface reinforcement

Area of the minimal surface reinforcement	Reasons for using the surface reinforcement			
	Spalling of the concrete	**Cover to the reinforcement c_{nom} > 70 mm**	**Crack widths control**	**Fire performance**
longitudinal $A_{s,surfmin,l}$	**0,01 $A_{ct,ext}$**	**0,005 $A_{ct,ext}$**	**0,02 $A_{ct,ext}$**	**1,256.10^{-4} m^2/m**
transverse $A_{s,surfmin,t}$	**0,01 $A_{ct,ext}$**	**0,005 $A_{ct,ext}$**	**0,01 $A_{ct,ext}$**	**1,256.10^{-4} m^2/m**

Eng. Juraj Frólo

Experimental Investigation of Composite Steel-Concrete Columns with Solid Steel Profile

1 INTRODUCTION

Composite steel-concrete columns with solid steel profile, called with steel core, are characterized by high axial stiffness by high slenderness. Designing these columns is limited in practice due to absence of simplified design method according STN EN 1994-1-1 for columns with solid steel profile. Reason is significant reduction of flexural stiffness caused by residual stresses in solid steel profile and strain limitation in concrete according to steel core dimension. In 2008 recommendations are given by Prof. Hanswille and Lipes to design composite columns according simplified design method STN EN 1994-1-1. Recommendations are given for column types of concrete filled steel tube with steel core only. These recommendations have not been verified in application for columns types of solid steel profile covered by reinforced concrete. Whereas experimental tests are the best way of verification, actual experimental investigation is running on composite columns with mentioned unverified cross-section.

2 DESCRIPTION OF EXPERIMENTAL INVESTIGATION

Six specimens of composite columns where designed. Its resistance depended on capacity of available load press at laboratory. All specimens were designed with same cross-sections. N-M diagram of ultimate plastic resistance of specimens is given in Fig.1. Strain gauges were fitted on surface of solid steel profiles before concreting. Concreting of columns was realized in vertical position (Fig.2). Specimens were fabricated in 3m and 3,85m lengths. Material specimens of concrete, reinforcement and structural steel were fabricated too. Columns will be tested by eccentrically pressure with eccentricity of 20mm (Fig.1). Geometrical imperfection of each column must be measured before test. Strain gauges will be fitted on concrete surface. Axial force, steel core strains, concrete strains and column deflections will be measured during tests. Columns will be loaded until its failure. Investigation will continue by numerical analysis.

3 OBJECTIVE OF RESEARCH

The aim of this investigation is verification or modification Prof. Hanswilles´ recommendations for columns with solid steel profile covered by reinforced concrete. It means determination correction factor of plastic bending resistance a_M, theoretical initial imperfection of column w_0 and corresponding buckling curve for save and economical designing.

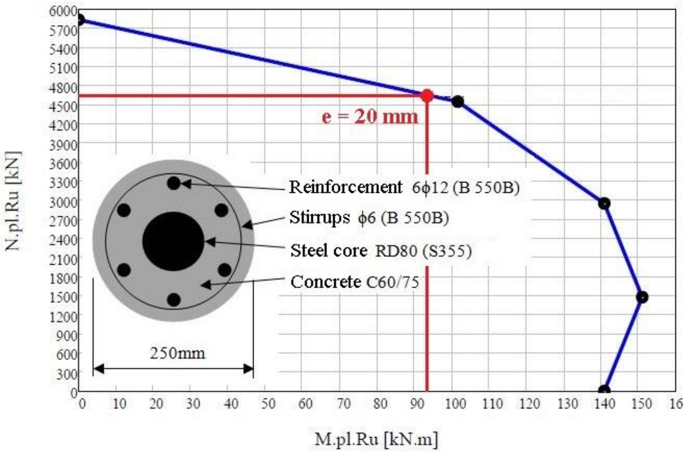


Fig.1: Ultimate resistance N-M diagram of column specimens

Fig.2: Reinforcement of columns (left) and columns in formworks after concreting (right)

Fig.3: Column specimens, material specimens of concrete, reinforcement and structural steel

Ing. Andrea Hrušovská

Comparison of Longitudinal Shear Resistance of Composite Slabs containing ArcelorMittal´s Sheets

1 INTRODUCTION

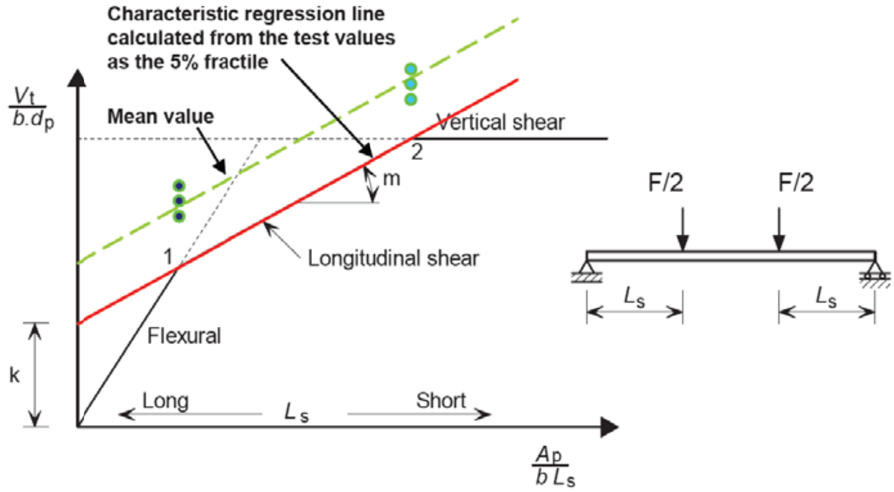
A composite flooring system consists of profiled steel sheeting and a monolithic concrete slab. The effectiveness of shear connection is studied by means of leading tests on simply supported composite full-scale slabs by using one of two methods – "m-k" method or partial shear connection method. The m-k method is based on establishing the gradient and intercept of a linear relationship by plotting the results from the composite slab tests in terms of the vertical shear against shear bond. The shear bond characteristic is rated by two empirical parameters *m* for the mechanical interlocking between steel and concrete and *k* for friction between them. (*Fig.1*)

2 COMPARISON

Cofraplus sheets are made up of two open-rib trapezoidal and nestable decks with embossments for easy storage and transportation. Cofraplus60 (C60) is manufactured from 0.75mm gauge steel and is designed for medium spans without props over 2 continuous bays, and slab thicknesses of 10 to 28 cm. Dove-tail shaped re-entrant ribs of Cofrastra sheets can provide anchor lines for suspending ceilings and technical networks using clips fixed in place by hand and create a very strong bond with the concrete. Cofrastra40 (C40) is used to lay very thin (8 cm) or thick (20 cm) floors as it covers practically the entire field of construction with light and heavy loads and has a very good fire resistance. Cofrastra70 (C70) is especially adapted to unpropped medium spans and can be used with slabs of 11 to 30 cm thickness and can bear very heavy loads. Its aim is to lighten structures with heavy dead loads. This type of sheet enables suspension of ceilings using special clips fitted into the closed ribs. (*Fig.2*, *Fig.3*)

3 CONCLUSION

Effects of re-entrant profile on resistance in the longitudinal shear are significant and visible when comparing the effects of C40 and C60 sheets. Although it was a sheet of the same thickness with the same hight of concrete layer, the critical factor was different cross section area and especially the parameters of shear connection, whereas the re-entrant profile reaches third higher values than open rib sheet. In the case of specially shaped sheet C70, the increase in resistance is caused by the combination of re-entrant effect and embossments fitted into the closed ribs. Although the parameter describing friction (k) was lower than for sheet C40, a mechanical interlocking (m) was greater than the value of both sheets which was a decisive factor. (*Fig.4*)



Design shear resistance $V_{l,Rd} = \frac{bd_p}{\gamma_{VS}}\left(\frac{mA_p}{bL_s}+k\right)$

Fig.1 "m-k" method.

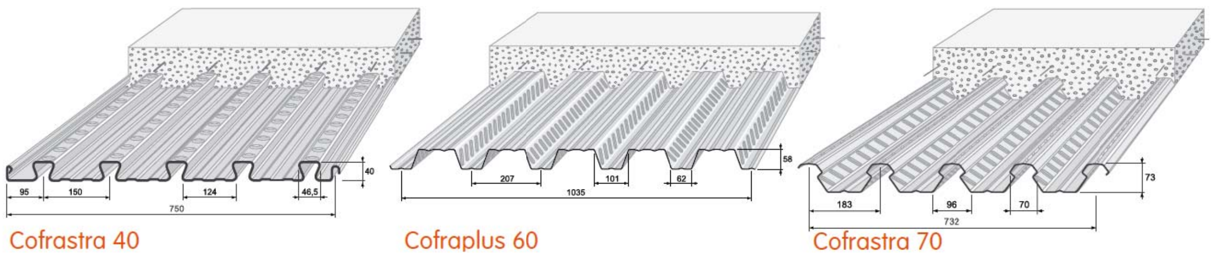


Fig.2 Different types of ArecorMittal sheets.

Cofrastra40			*Cofraplus60*			*Cofrastra70*		
m [MPa]	**k [MPa]**	**d_p [mm]**	**m [MPa]**	**k [MPa]**	**d_p [mm]**	**m [MPa]**	**k [MPa]**	**d_p [mm]**
129,37	0,185	125,8	92,50	0,056	124,7	254,16	0,090	141,6

Fig.3 Parameters of ArecorMittal sheets.

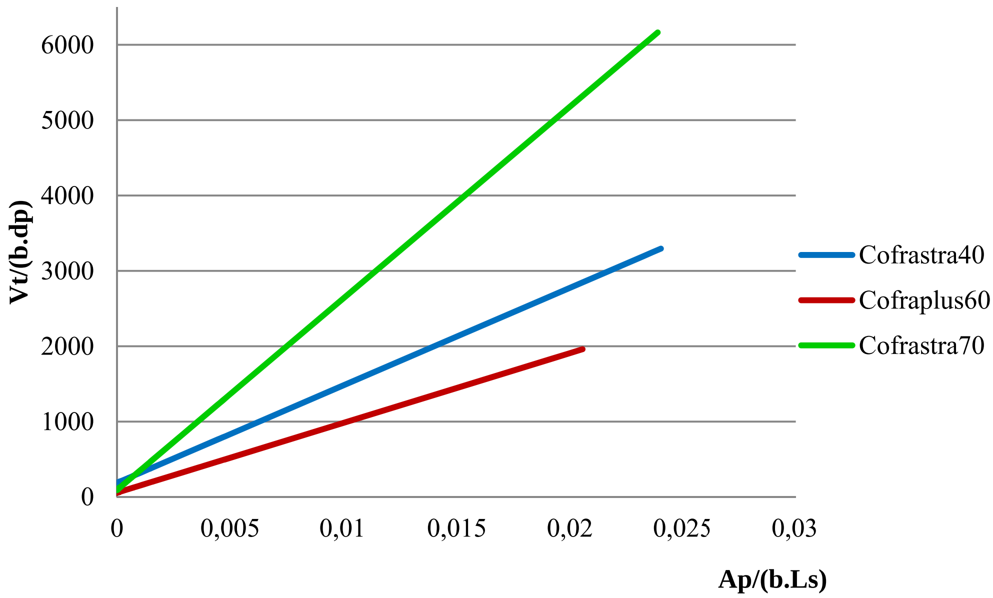


Fig.4 Parameters of ArecorMittal sheets.

VII. TEACHING

VII.1 Graduate Study

Obligatory subjects

Bachelor's degree study	Semester	Hours per Week Lectures Seminars	Lecturer
Design of Concrete and Masonry Members	4.	2 – 2	Ľ. Fillo
Design of Concrete Members (Slovak and Englisch)	4.	3 – 2	J. Bilčík
Design of Concrete Members	5.	3 – 2	J. Halvoník, V. Benko, Š. Gramblička
Reinforced and Prestressed Members	5.	2 – 2	Ľ. Fillo
Design of Concrete Structures I	5.	3 – 2	J. Bilčík
Design of Concrete Structures I	6.	2 – 2	I. Hudoba
Design of Concrete Structures II	6.	2 – 2	I. Harvan, V. Borzovič

Master's degree study	Semester	Hours per Week Lectures Seminars	Lecturer
Masonry Structures of Buildings	1.	2 – 2	M. Čabrák
Design of Concrete Structures II	1.	2 – 2	I. Hudoba
Prestressed Concrete	1.	2 – 2	I. Harvan
Design of Environmental Constructions	1.	2 – 2	J. Bilčík
Design of Concrete Bridges I	2.	2 – 2	J. Halvoník
High – Rise and Span Structures	2.	2 – 2	I. Gramblička, J. Šoltész
Modelling of Civil Engineering Works	2.	2 – 2	J. Šoltész
Design of Composite Structures	3.	2 – 2	Š. Gramblička
Design of Concrete Bridges II	3.	2 – 2	J. Halvoník
Experimental Testing of Concrete Structures	3.	0 – 3	V. Priechodský
Special Problems of Concrete Structures	3.	2 – 2	Ľ. Fillo
Repair of Concrete Constructions	4.	2 – 1	J. Bilčík
Masonry Structures	4.	2 – 1	M. Čabrák
Execution of Concrete Structures	4.	2 – 1	I. Hudoba

Optional Subjects

Bachelor's degree study	Semester	Hours per Week Lectures Seminars	Lecturer
Design of Concrete Structures	8.	2 – 2	I. Harvan

Master's degree study	Semester	Hours per Week Lectures Seminars	Lecturer
Modelling of RC Structural Systems I	1.	2 – 2	I. Harvan
Modelling of RC Structural Systems II	2.	2 – 2	I. Harvan
Modeling of RC 2D Structures	1.	2 - 2	J. Šoltész
Modeling of RC 3D Structures	3.	2 – 2	J. Šoltész

Postgraduate courses

Modelling of Concrete Structures
Experimental Testing of Concrete Structures

VIII. THESES

VIII.1 Bachelor Theses

ČERVENKA, T.: Design of a Reinforced Concrete Two-Way Floor Slab

Supervisor: Halvoník, J.

ČORBA, A.: Apartment Building: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Gajdošová, K.

DEKRÉT, N.: Design of a Floor Structure Using Filigran Panels

Supervisor: Halvoník, J.

HLÁDEK, S.: Proposed Concrete Ceiling Construction Above a Basement

Supervisor: Fillo, Ľ.

HŐRMANN, T.: Ceiling Construction of an Administrative Building

Supervisor: Borzovič, V.

KRAMČÍK, A.: Flat Floor Construction of a Residential Building

Supervisor: Borzovič, V.

LIDAY, D.: Static Analysis of the Ceiling Board of the CBC Administrative Center

Supervisor: Paulík, P.

MAJERČÍK, J.: Design of the Foundation Slab of a Concrete Building

Supervisor: Fillo, Ľ.

PETROVIČ, M.: Reinforced Concrete Floor Construction of a Family House

Supervisor: Grambličká, Š.

RUSKO, M.: Punching Analysis of a Two-Way Reinforced Concrete Slab

Supervisor: Fillo, Ľ.

SMETANOVÁ, I.: Column Analysis of a Concrete Skeleton Structure

Supervisor: Fillo, Ľ.

SOLÁR, M.: Multifunctional Building: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Gajdošová, K.

STOKLASOVÁ, M.: Design of a Floor Structure Designed as a Cast-in-Situ One-Way Reinforced Concrete Slab

Supervisor: Halvoník, J.

TEKÁČ, P.: Floor Slab Construction of a Residential House

Supervisor: Borzovič, V.

TURÁN, Ľ.: Structural Analysis of Cast-in-Situ Concrete Floor Slabs of a Parking Garage

Supervisor: Pažma, P.

VARGA, G.: Structural Analysis of Cast-in-Situ Concrete Floor Slabs of a Parking Garage

Supervisor: Pažma, P.

VIII.2 Graduate Theses

ANTOL, J.: Road Bridge: Post-Tensioned Cast-in-Situ Concrete Structure

Supervisor: Borzovič, V.

AUGUSTÍN, T.: High-Rise Apartment House: Bearing Structure

Supervisor: Gramblička, Š.

BUDZÁK, J.: High-Rise Residential Building with an Atypical Shape: Cast-in-Situ RC Bearing Structure

Supervisor: Bartók, A.

CIFRA, R.: Multifunctional Building: Cast-in-Situ RC Bearing Structure

Supervisor: Bartók, A.

ČAJOVÁ, S.: Bottova Centre: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Gajdošová, K.

ČERNAJ, R.: Staré Grunty - Bratislava Hotel: Combined Cast-in-Situ Reinforced Concrete Structure with Stiffening Cores

Supervisor: Abrahoim, I.

DOVIČIČ, J.: Railway Bridge in Trenčín: Design and Assessment of the Lower Structure

Supervisor: Paulík, P.

DUDŽÁKOVÁ, Z.: Hotel "TULA", Banská Bystrica: Cast-in-Situ RC Slab-Wall Structure on a Skeleton Pedestal with a Stiffening Core

Supervisor: Harvan, I.

DVORANOVÁ, V.: Austria Trend Hotel: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Gajdošová, K.

FILO, T.: High-Rise Building with a Polygonal Shape: Cast-in-Situ RC Bearing Structure

Supervisor: Bartók, A.

GAŽOVIČOVÁ, N.: Letná Multifunctional Building: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Gajdošová, K.

GONDA, M.: Central Police Office in Bratislava: Multi-Storey RC Skeleton Structure with Stiffening Cores

Supervisor: Harvan, I.

HEVEŠI, A.: Multifunctional Building: 15-Storey Cast-in-Situ RC Slab-Wall Structure on a Skeleton Pedestal

Supervisor: Harvan, I.

JÁVOR, P.: Motorway Bridge over the River Váh on the D1 Motorway Between Dubná Skala and Turany

Supervisor: Borzovič, V.

JURČOVIČ, R.: Underground Parking Garage: Watertight Diaphragm Wall

Supervisor: Šoltész, J.

KELEMENOVÁ, L.: Office Block: Cast-in-Situ RC Bearing Structure

Supervisor: Bartók, A.

KESELI, O.: Large-Diameter Silo for the Storage of Corn

Supervisor: Šoltész, J.

KRAJČI, M.: Prestressed Cast-in-Situ Bridge on D1 Motorway over the River Váh

Supervisor: Halvoník, J.

KRIVÝ, M.: Reinforced Concrete Underground Collector for Cooling Water Pipes

Supervisor: Hudoba, I.

KRUĽÁK, J.: Incremental Launching Bridge in Nitra: Static Analysis of Upper Structure

Supervisor: Paulík, P.

KUBIŠ, J.: Prestressed Cast-in-Situ Bridge on D1 Motorway Built by Cantilever Balanced Method

Supervisor: Halvoník, J.

LACO, K.: Office Building with Water Tank: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Borzovič, V.

LÁSKOVÁ, M.: Load - Bearing Structure of a High-Rise Building with an Atypical Floor Plan

Supervisor: Gramblička, Š.

LÍŠKA, M.: "Uzbecká - Dolné Hony – Bratislava" Apartment Block: Cast-in-Situ RC Combined Bearing Structure with Stiffening Cores

Supervisor: Abrahoim, I.

MAGDOLEN, R.: 25 - Storey High-Rise Building of an Administration Centre. Cast-in-Situ RC Structure with Flat Slabs and a Stiffening Core

Supervisor: Harvan, I.

MARKOCSY, E.: Multifunctional Residential Building: Cast-in-Situ Concrete Structure

Supervisor: Borzovič, V.

MARTIŠ, J.: Load-Bearing Structure of a Multifunctional High-Rise Building

Supervisor: Gramblička, Š.

MATÚŠKA, D.: Load-Bearing Structure of a Multifunctional Building

Supervisor: Gramblička, Š.

MIŠKOV, M.: Bearing Structure of a Part of a Residential Complex with Underground Garages

Supervisor: Gramblička, Š.

MRÁZIK, R.: Design Calculations of a High-Rise Building Made from Structural Concrete

Supervisor: Fillo, Ľ.

PETKOVÁ, V.: Prestressed Cast-in-Situ Bridge on R1 Expressway in Town of Nitra

Supervisor: Halvoník, J.

PTÁK, M.: Design Calculations of an Office Block: Cast-in-Situ RC Structure

Supervisor: Bilčík, J.

RAJCSÁNYI, V.: Concrete High-Rise Building

Supervisor: Benko, V.

RASZGYŐRGYOVÁ, A.: Office Building of an Automobile Centre: Bratislava Rožňavská: Cast-in-Situ Reinforced Concrete Combined Bearing Structure with a Stiffening Core

Supervisor: Abrahoim, I.

STAŠ, P.: Office Building - Generali Slovakia Insurance Company, Bratislava: Cast-in-Situ Reinforced Concrete with Stiffening Cores

Supervisor: Abrahoim, I.

STEFANKOVICS, A.: Design of a Load-Bearing Structure of a Concrete High-Rise Building

Supervisor: Fillo, Ľ.

SZALAI, T.: Administration Building – Hanging RC Structure

Supervisor: Benko, V.

ŠENKÁR, M.: Static and Dynamic Analysis of a High-Rise Building

Supervisor: Šoltész, J.

ŠINKOVÁ, J.: Static and Dynamic Analysis of a High-Rise Building

Supervisor: Šoltész, J.

ŠKRIPUTA, Š.: Office Building: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Borzovič, V.

ŠLESARIKOVÁ, L.: "GREEN PARK – Part A, Podkolibská, Bratislava" Apartment House – Cast-in-Situ RC Frame Structure with Stiffening Cores

Supervisor: Abrahoim, I.

ŠTEFANČÍK, I.: Motorway Bridge over the River Váh on the D1 Motorway at Hubová - Ivachnová

Supervisor: Borzovič, V.

ŠTEFULA, M.: Design Calculations of Multi-Storey Garage: Cast-in-Situ RC Structure

Supervisor: Bilčík, J.

TUŽINSKÝ, M.: Design Calculations of a Multi-Storey Garage: Precast Concrete Structure

Supervisor: Bilčík, J.

VIDA, R.: Road Bridge over the Danube River at Komárno - Komárom

Supervisor: Halvoník, J.

VYŽINKÁR, P.: Multifunctional Building: Cast-in-Situ Reinforced Concrete Structure

Supervisor: Gajdošová, K.

ŽIDEKOVÁ, R.: Office Block in Komárno: Cast-in-Situ RC Bearing Structure

Supervisor: Bartók, A.

VIII.3 Student's Scientific Conference Theses

ANTOL, J.: Stability Analysis of a Balanced Cantilever on the Behaviour of a Bridge Structure

Supervisor: Borzovič, V.

AUGUSTÍN, T.: Effect of the Vertical Load Redistribution in a RC High-Rise Buiding

Supervisor: Gramblička, Š.

BUDZÁK, J.: Analysis, Design and Use of High-Efficiency Column Concreting through Centrifugation

Supervisor: Bartók, A.

DUDŽÁKOVÁ, Z.: A Time Action of Sub-Soil on Reinforced Concrete

Supervisor: Harvan, I.

GONDA, M.: Approximate Analysis of the Effects of Wind and Seismicity on a Multi-Storey Building

Supervisor: Harvan, I.

HEVEŠI, A.: Assessment of Crack Widths on Concrete Walls

Supervisor: Harvan, I.

KESELI, O.: Design Optimization of a Silo Structure for the Storage of Corn

Supervisor: Šoltész, J.

LACO, K.: Impact of the Phasing of Construction on the Stress of Structural Elements

Supervisor: Borzovič, A.

MAGDOLEN, R.: Accidental Actions Caused by the Impact of a Highway Vehicle into the Columns of a Garage Area

Supervisor: Harvan, I.

MARTIŠ, J.: Analysis of Changes in the Structural Bearing System of a High-Rise Building

Supervisor: Gramblička, Š.

MATÚŠKA, D.: Composite Steel and Concrete Columns with Steel Circular Cross-Sections Used in the Bearing Structure of a Multifunctional Object

Supervisor: Gramblička, Š.

VIDA, R.: Fatigue Resistance of Extradosed Tendons

Supervisor: Halvoník, J.