Slovak University of Technology in Bratislava

Faculty of Civil Engineering

Department of Concrete Structures and Bridges

Annual Report

2012

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ANNUAL REPORT 2012
Department of Concrete Structures and Bridges
Faculty of Civil Engineering
Slovak University of Technology in Bratislava
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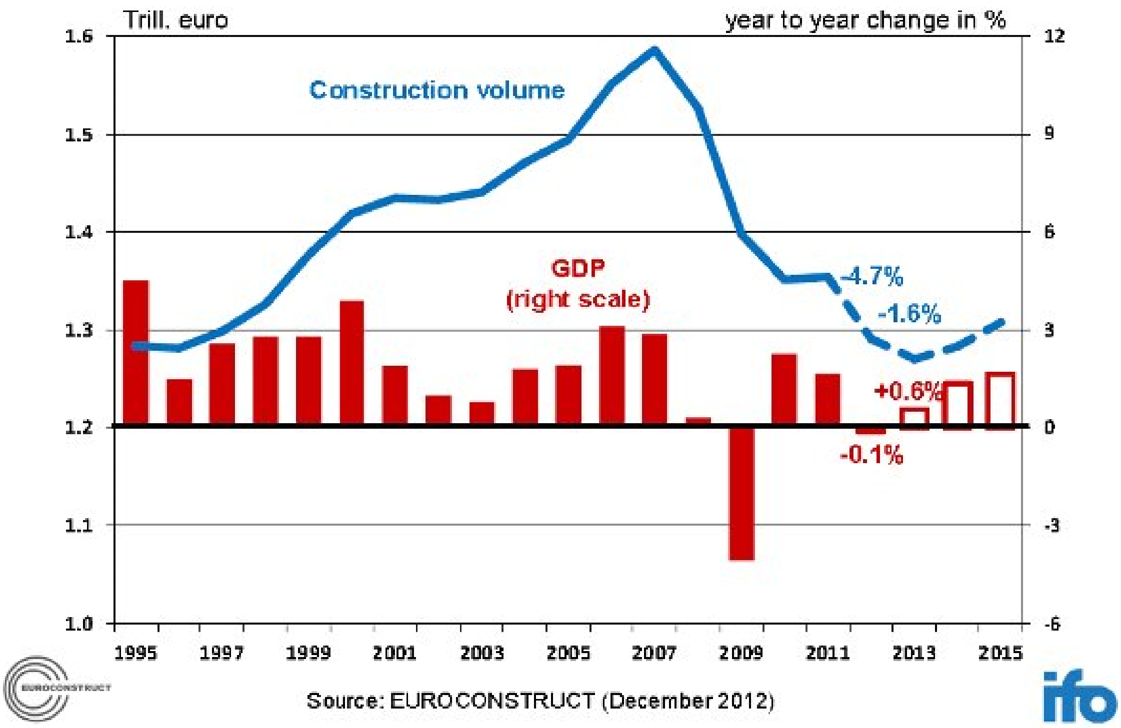
CONTENT

PREFACE

Hopes for a speedy recovery for the European construction industry have been dashed. In several of the 19 EUROCONSTRUCT countries, the euro crisis is continuing to hold construction activity in check. This trend is exacerbated by the situation of the world economy, which remains difficult. In June, construction experts from the EUROCONSTRUCT Group expected construction activity to decline by 2% during 2012, but they have now revised this forecast to a 4.7% decline. Contrary to their original forecasts, experts now predict further losses of around 1.5% for 2013. Any recovery by the end of the forecasting period in 2015 is only expected to be moderate. High unemployment, stagnant economic growth or even economic downturns in many places, as well as the strained financial situation of the public sector are curbing construction demand in all three construction segments. In 2012 civil engineering looks to be set the most heavily affected with a decline of around 7.5%. Downturns in the residential and non-residential construction segments of around 3.5% and a good 4% respectively are not quite as severe. All three sub-segments should also shrink in 2013 (Source: 74th EUROCONSTRUCT Conference - Munich, 2012).



Construction activity and economic growth in Europe at 2011 prices

The existing status can be used as an opportunity to get better – increase the range and quality of knowledge. The study, published in the journal Science, calculates the amount of data stored in the world by 2007 as 295 exabytes. That is the equivalent of 1.2 billion average hard drives. The researchers calculated the figure by estimating the amount of data held on 60 technologies from PCs and DVDs to paper adverts and books. Computer storage has traditionally been measured in kilobytes, then megabytes, and now usually gigabytes. After that terabytes, petabytes, then exabytes come. One exabyte is a billion gigabytes. According to the researchers, the same information stored digitally on CDs would create a stack of discs that would reach beyond the moon. Of course, the data on concrete structures are only a fraction of this information. What you can basically do with information is transmit it through space, and we call that a communication. You can transmit it through time; we call that a storage. Or you can transform it, manipulate it, change the meaning of it, and we call that a computation.

With the help of Annual Report, we are trying to convey and receive information about activities of a small academic community in Central Europe. Our Annual Report 2012 contains only 6,7 megabytes printed information. It is only a matter of a short time, when digital edition will replace the printed one. With my age, I belong to the category of dinosaurs and therefore it is not surprising that I am more excited when I hold in my hands a printed version of AR, as when it appears on the screen.

2011 was the starting year for the reform of the Eurocodes. Many experts believe that the time has come for an amendment of the design standards in order to simplify them and to meet requirements of practice. This decision is justified when one considers around 7300 pages of standard regulations. On the other hand, technical progress cannot be stopped. The development of new computational methods and procedures at the execution raise the pressure for their inclusion in standards. In addition to the normative rules civil engineers are governed also by regulations and guidelines. To these ones, we added in 2012 the Slovak Guideline for Watertight Concrete Structures. The authors hope that the guideline will contribute to the reduction of the large number of mistakes and failures that are manifested in this type of underground structures in Slovakia.

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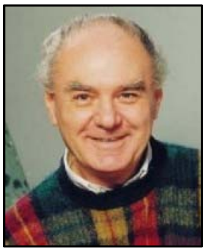

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I. NEW AT DeCoSaB

I.1 Defenses of the Doctoral Theses

PRÍTULA, A.: Cambers of Precast Bridge Girders - Technological and Static Influences
Supervisor: Halvoník, J.

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 1/0857/11, Resistance Analysis of Concrete, Masonry and Composite Steel-Concrete Structures (2011-2013, FILLO, Ľ.),

2.) VEGA 1/0458/11, Factors Affecting the Effectiveness of Utilization of High-Strength Concrete in Load-Bearing Elements and Structures (2011-2013, HUDOBA, I.),

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

IV. EVENTS

Concrete Days 2012

Ninth International Conference

On 25th and 26th October 2012 the ninth international conference Concrete Days 2012 was organized by our department led by professor Bilčík. The conference took place at Saffron Hotel Bratislava, Slovakia. The tradition of organizing this event was founded in 1994 and the conference is organized regularly in two-year period.

Concrete Days offered a comprehensive overview of current developments in the construction sector. Recognized experts and specialists in design of concrete structure from Slovak and Czech Republic participated, thus offering a unique opportunity to share their experience and acquired knowledge.

Keynote lectures were offered by Martin Haferl (about Donau City Tower 1, Vienna, Austria) and Peter Kasper (about Expansion of the Panama Canal). Totally, the meeting took place in ten sessions.

The discussion continued during a social evening organized on 25th 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 Forum Business Center - building under construction was organised. Building is located at Prievozská – Bajkalská Street [Bratislava II], with its 16 floors it is a 65 m high building with a build-up area of 1346 m^2.

Total of 150 participants expressed their enjoinment with the conference and we look forward to meet again in two years – at the tenth year anniversary!


2012
BETONÁRSKE DNI 2012
SKANSKA


BETONÁRSKE DNI




fischer
software pro statiky



FORUM BUSINESS CENTRUM

Conferences we visited

Concrete Day 2012, Vienna, Austria,
19-20 April 2012

This year, more than 90 exhibitors came to the Austria Center; over 50 specialist lectures underlined the congress's significance as one of the most important meetings in the sector. In 2012 the exhibition in the Austrian capital greeted more than 1700 specialists from 12 countries. The Concrete Day has taken place under its traditional name for the last time. When the field's experts meet again in Vienna in April 2014, they will do so under the rather more illustrious-sounding name "Construction Conference 2014".

CICE 2012, Rome, Italy,
13-15 June 2012

CICE 2012 was the 6^{th} International Conference on FRP Composites in Civil Engineering. The aim of CICE 2012 was to provide an international forum for all concerned with the application of FRP composites in civil engineering to exchange recent advances in both research and practice.

Design of Concrete Structures Using Eurocodes, Vienna, Austria,
20-21 September 2012

The Third International Workshop focuses on application of all EN 1992, since these standards have been introduced and accepted as the national standard in many European countries and there already are practical experiences with its use.


BETONTAG 2012
ACV


CICE 2012
Rome
Registration

IALCCE 2012, Vienna, Austria,
3-6 October 2012

The objective of the International Association for Life-Cycle Civil Engineering (IALCCE) is to promote international cooperation in the fields of life-cycle civil engineering for the purpose of enhancing the welfare of society (http://www.ialcce.org). For this reason, it was deemed appropriate to bring together all the very best work that has been undertaken in the field of life-cycle civil engineering at the Third International Symposium on Life-Cycle Civil Engineering (IALCCE 2012) held in one of Vienna´s most famous venues, the Hofburg Palace, October 3-6, 2012.

Recent Advances in Concrete Technology and Sustainability Issues,
Prague, Czech Republic,
31 October - 2 November 2012

12th International Conference on Recent Advances in Concrete Technology and Sustainability Issues was sponsored by the Committee for the Organization of International Conferences; cosponsored by ACI.

Czech Concrete Days, Hradec Králové, Czech Republic,
21-22 November 2012

The 19th 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.


Recent Advances in Concrete Technology and Sustainability Issues

Proceedings
Twelfth International Conference
Prague, Czech Republic

October 2012

Terence C. Holland
Pawan R. Gupta
V.M. Malhotra

aci
American Concrete Institute
SP-289

Excursion of our students

FORUM BUSINESS CENTER, Bratislava, Slovakia
30 April 2012

The excursion is included into compulsory subjects for Structures of Buildings and Structural and Transportation Engineering fields of study. Excursion was organised to Forum Business Center - building under construction. Building is located at Prievozská – Bajkalská Street (Bratislava II) and it has a reinforced concrete load-bearing structure. The main contractor of a structure is SkyBau, member of Skanska SK group.

Saying goodbye to year 2012

Christmas session

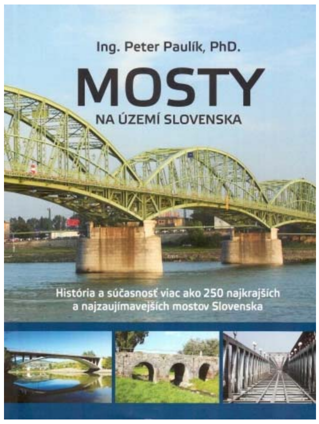
At the end of year 2012, the members of our department as traditionally met all together around a small Christmas tree just few days before Christmas. Once again, we spend one afternoon together. Talking about every event and memory we spent with colleagues.

At the beginning, the head of the department, professor Bilcik, gave a speech on angelic tax. All good things require their toll. Despite of this, we will try to be better next year!

Interesting publications of 2012

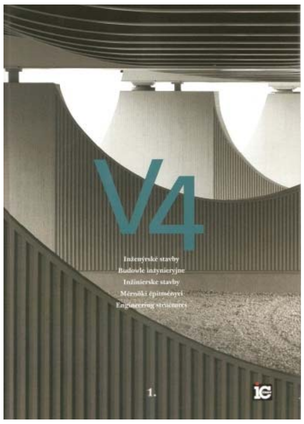
Bridges of Slovakia

In Slovakia we have a number of remarkable bridges, but only a few people know them. Many of these bridges represent great construction skills of our ancestors and we can also find bridges, which were European and World record holders at the time of their execution.
One of the main tasks of this book, written by the member of our department Ing. Peter Paulík, PhD., is to show our most interesting bridges to the public and experts and to point out, that even though Slovakia is a relatively small country, it has a number of bridges which we can be proud of. The book contains information about more than 250 bridges, which were selected from among 21000 bridges built on the territory of Slovakia. First few pages of the book are devoted to the general information about how bridges work and how they are constructed. The second, much larger part is then devoted to the selected bridges, which could be found in our country.
Preview of the book and the geographic location of selected bridges with photographs are available on the website www.mostynaslovensku.sk.



Engineering Structures of V4

Our department also cooperated on another interesting book published in 2012. Ing. Peter Paulík, PhD. and prof. Ing. Vladimír Benko, PhD. were the editors of the Slovak part of the book *Engineering structures of V4* (Visegrad Four group), which has been published in cooperation with the Chamber of Civil Engineers of Slovakia, Czech Republic, Hungary and Poland. This book is the first in the planned series devoted to the most interesting engineering structures of V4. Every 2 years a book containing 6 engineering structures from each country of V4 (together 24 structures) built in a given period will be published. In the first book, structures built between 1990 and 2012 were selected.
The book is written in 4 languages: Slovak (Czech), Hungarian, Poland and English. The chief editor was the Svatopluk Zídek from Czech Republic.



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 Visitors to the Department

1.) prof. Eng. Jan Vítek, CSc. – Metrostav Praha, Czech Republic
2.) Eng. Vlastimil Šrůma, CSc. – director of ČBS, Czech Republic
3.) doc. Eng. Zdeněk Bažant, CSc. – VUT Brno, Czech Republic
4.) prof. Eng. Radim Čejka, CSc. – VŠB – TU Ostrava, Czech Republic
5.) doc. Dr. Eng. Luboš Podolka – ČVUT Prague, Czech Republic

V.3 Visits of Staff Members and Postgraduate Students to Foreign Institutions

1.) GRAMBLIČKA, Š. : Editorial board of journal "Stavebníctví", Prague, Czech Republic, 13.01.2012, 18.05.2012, 21.08.2012, 18.10.2012
2.) BENKO, V.: Seminar on "Lehrgang für Baudynamik und Erdbebeningenieurwesen für die Praxis", Vienna; Austria, 26.01.2012
3.) BILČÍK, J.: Seminar: Repair of Concrete Structures, Prague, Czech Republic, 20.02.2012
4.) BENKO, V.: TU Vienna "Institut für Tragkonstruktionen – Betonbau, Vorlesung-Betonbau 3", Vienna, Austria, 08.3, 15.3, 22.3 29.3, 19.4, 26.4, 3.5, 28.06 2012
5.) BILČÍK, J. – FILLO, Ľ. – HOLLÝ, I. – SONNENSCHEIN, R.: Concrete Days 2012, Vienna Austria 19. – 20.04.2012
6.) BILČÍK, J.: XXII International Symposium Repair of Concrete Structures, Brno, Czech Republic, 23.-25.05.2012

7.) GAJDOŠOVÁ, K.: CICE 2012, 6th International Conference on FRP, Composites in Civil Engineering. Rome, Italy, 13-15 June 2012

8.) FILLO, Ľ.: 32th Session CEN TC250 SC 2, Brussels, Belgium,28.-29.6.2012

9.) FILLO, Ľ. - HALVONÍK, J. - BENKO, V. – GAJDOŠOVÁ, K.: Active participation in the 3rd international workshop: Design of Concrete Structures Using Eurocodes. 20. and 21.9.2012 TU Vienna

10.) HOLLÝ, I.: Conference: Testing and Quality in Civil Engineering 2012, Brno, Czech Republic, 2.-3.10.2012

11.) BILČÍK, J. – HOLLÝ, I.: Life-Cycle and Sustainability of Civil Infrastructure Systems, Vienna, Austria 3. – 6.10.2012

12.) BENKO, V.: TU Wien, NDU St. PÖLTEN , Event Materials, Internationals Symposium, Vienna, Austria, 18.10.2012

13.) GRAMBLIČKA, Š. : The Day of Civil Engineers 2012, Brno, Czech Republic, 22.10.2012

14.) FILLO, Ľ.: Chairman of the Commission for the Execution of Concrete Structures in Czech Republic. Czech Concrete Society, Prague, Czech Republic, 30.10.2012

15.) BENKO, V. - BORZOVIČ, V. - FILLO, Ľ. - HALVONÍK, J. - GAJDOŠOVÁ, K. – ŠOLTÉSZ, J. – IGNAČÁK, M.: Conference Concrete Days 2012, Hradec Králové, Czech Republic, 20.-22.11.2012

16.) BELLOVÁ, M.: Participation in the international workshop, organized by European Committee: EUROCODES – Background and Applications: STRUCTURAL FIRE DESIGN, Brussels, Belgium, 27.-28.11.2012

17.) BENKO, V.: SCIA USER Meeting, Mondsee, Austria, 30.11 2012

18.) FILLO, Ľ. - HALVONÍK, J.: Invited lectures at the Faculty of Civil Engineering VUB Ostrava within the "Young Researchers" project on 11. and 12.12.2012

V.4 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.) BILČÍK, J.: American Concrete Institute

5.) FILLO, Ľ.: Representative of the Slovak Republic in CEN TC 250 – SC2 Eurocodes – Design of Concrete Structures

6.) FILLO, Ľ.: Member of Task Group *fib* TG 1.1 Design Application

7.) FILLO, Ľ.: Honorary Member of Czech Concrete Society, Hradec Králové, Czech Republic, 25.11.2009

8.) HALVONÍK, J: Representative of the Slovak Republic on CEN TC 250 – SC1 Eurocodes – Actions on Structures

9.) HALVONÍK, J: Representative of the Slovak Republic on CEN TC 250 Eurocodes

VI. ACTIVITIES

Commercial Activities for Firms and Institutions

1.) BELLOVÁ, M.: Determination of Time for Fire Resistance Classification of Masonry Walls Made from Moulded Masonry Units of LIAS Vintířov Company. Report for LIAS Vintířov, Light-Weight Building Product, Czech Republic, 2012

2.) BILČÍK, J.: Evaluation of the film thickness of a coating system for Cooling Towers in NPP Mochovce. Report for Cooling Towers Praha, 2012

3.) GAJDOŠOVÁ, K.: Translation of EN 1317-5: 2007+A2:2011 - Road Restraint Systems — Part 5: Product Requirements and Evaluation of Conformity for Vehicle Restraint Systems, 2012

4.) GRAMBLIČKA, Š.: Statical Assessment of Structural System of Trinity Multifunctional Complex of Buildings Trinity in Bratislava, 2012

VII. PUBLICATIONS

VII.1 Books and Textbooks

Books

1.) BILČÍK, J. – et al.: Guidelines for Watertight Concrete Structures – White Tanks. Bratislava: SKSI, 2012, ISBN 978-80-891-89113-90-3, 67 pp. (in Slovak)

2.) PAULÍK, P.: Bridges in Slovakia. Bratislava: Jaga Group, s.r.o., 2012, ISBN 978-80-8076-103-5, 260 pp. (in Slovak)

Textbooks

1.) ABRAHOIM, I. – BORZOVIČ, V.: Reinforced Concrete Supporting Structures. Instructions for Exercises, Slovak University of Technology in Bratislava, 2012, ISBN 978-80-227-3794-4, 181 pp. (in Slovak)

2.) ŠOLTÉSZ, J. – BARTÓK, A.: Digital Modelling of Concrete Structures Using the CAD-FEM Structural Analysis System STRAP: Slovak University of Technology in Bratislava, 2012, ISBN 978-80-227-3670-1, 141 pp. (in Slovak)

VII.2 Journals

Scientific Papers Abroad

1.) BILČÍK, J. - HOLLÝ, I.: Effect of the Corrosion of Reinforcement on the Reliability of Concrete Structures. In: Beton: Technologie - Konstrukce - Sanace, ISSN 1213-3116, Vol.12, No. 3 (2012), pp. 53-57 (in Slovak)

2.) GRAMBLIČKA, Š.: Reinforcement of Columns and Walls of Cast-in-Situ Reinforced Concrete Structures. In: Konstrukce, ISSN 1213-8762, Vol. 11, No. 3 (2012), pp. 14-21 (in Slovak)

3.) HARVAN, I.: Assessment of Prefabricated Panel Buildings According to the Requirements of the Applicable European Standards. In: Stavebnictví, ISSN 1802-2030, Vol. 6, No. 2 (2012), pp. 50-54 (in Slovak)

4.) HUDOBA, I.: Technical Tools for Sufficient Reinforcement Cover and Related Reliability Aspects of Reinforced Concrete Structures. In: Beton: Technologie - Konstrukce - Sanace, ISSN 1213-3116, Vol.12, No. 5 (2012), pp. 67-71 (in Slovak)

5.) ŠOLTÉSZ, J. - ABRAHOIM, I.: Vinohradis Residental Complex in Bratislava. In: Beton: Technologie - Konstrukce - Sanace, ISSN 1213-3116, Vol.12, No.1 (2012), pp. 34-39 (in Slovak)

Scientific papers in Slovak Journals

1.) BILČÍK, J. - CHANDOGA, M.: Strengthening Concrete Structures by External Prestressing. In: Stavebné materiály, ISSN 1336-7617, Vol. 8, No. 7 (2012), pp. 52-54 (in Slovak)

VII.3 Conferences

Contributions to Proceedings Abroad

1.) BENKO, V. - FILLO, Ľ.: Resistance of Columns Subjected to Axial Load. In: Proceedings of Design of Concrete Structures and Bridges Using Eurocodes, International Workshop, Vienna, Austria, 20.-21.09.2012, Technische Universität Wien, ISBN 978-3-902749-03-1, pp. 211-214 (in English)

2.) BILČÍK, J.: Additional Sealing of Watertight Concrete Tanks. In: Repair of Concrete Structures 2012, Proceedings of the 22th International Symposium Brno, Czech Republic, 23.-25.05.2012, Brno, The Concrete Structures Repair Association, pp. 40-45 (in Slovak)

3.) BILČÍK, J. - HOLLÝ, I.: Design of Concrete Structures for Durability. In: Life-Cycle and Sustainability of Civil Infrastructure Systems. Proceedings of the Third International Symposium on Life-Cycle Civil Engineering, Vienna, Austria, 3.-6.10.2012, London, Taylor & Francis Group, ISBN 978-0-415-62126-7, pp. 1330-1334 (in English)

4.) BORZOVIČ, V. - HALVONÍK, J.: Flat Slab Reinforcement with Regard to the Distribution and Redistribution of Internal Forces In: Proceedings of Design of Concrete Structures and Bridges Using Eurocodes, International Workshop, Vienna, Austria, 20.-21.09.2012, Technische Universität Wien, ISBN 978-3-902749-03-1, pp. 223-228 (in English)

5.) GAJDOŠOVÁ, K.: Slender Rectangular Concrete Columns Strengthened by CFRP Confinement and NSMR. In: Proceedings of CICE 2012, Rome, June 2012 (in English)

6.) GAJDOŠOVÁ, K.: Stress and Crack Width Control According to EN 1992 In: Proceedings of Design of Concrete Structures and Bridges Using Eurocodes, International Workshop, Vienna, Austria, 20.-21.09.2012, Technische Universität Wien, ISBN 978-3-902749-03-1, pp.199-202 (in English)

7.) GRAMBLIČKA, Š. - VERÓNY, P.: Analysis of Influence of Transverse Reinforcement in Reinforced Concrete Columns. In: Proceedings of Design of Concrete Structures and Bridges Using Eurocodes, International Workshop, Vienna, Austria, 20.-21.09.2012, Technische Universität Wien, ISBN 978-3-902749-03-1, pp. 215-222 (in English)

8.) HALVONÍK, J.: Design of Foundation Slabs and Footings for Shear. In: Proceedings of Design of Concrete Structures and Bridges Using Eurocodes, International Workshop, Vienna, Austria, 20.-21.09.2012, Technische Universität Wien, ISBN 978-3-902749-03-1, pp. 85-92 (in English)

9.) HOLLÝ, I. - BILČÍK, J.: Application of Probabilistic Degradation Models for Increasing the Quality of Concrete Structures. In: Proceedings of Conference on Investigation and Quality in Civil Engineering 2012, Brno, Czech Republic, 2.-3.10.2012,University of Technology Brno, ISBN 978-80-214-4578-9, pp. 37-45 (in Slovak),

10.) LELKES, A.: Theoretical Analyses of Composite Steel-Concrete Columns. In: Juniorstav 2012, Annotations, 14th PhD. Technical Conference, Brno, Czech Republic, 26.1.2012, University of Technology Brno, ISBN 978-80-214-4393-8, 187 pp. (in Slovak

11.) PAULÍK, P. – PÁNIK, M. - NEČAS, V.: Use of Radioactive Reinforcement in the Construction of Concrete Bridges. In: Proceedings of Recent Advances in Concrete Technology and Sustainability Issues: 12th International Conference. Prague, Czech Republic,30.10.-1.11.2012. - Farmington Hills, American Concrete Institute, 2012, ISBN 978-0-87031-780-4, pp. 425-434 (in English)

12.) VERÓNY, P.: Reinforcement of Concrete Columns-Analysis of the Influence of Transverse Reinforcements. In: Juniorstav 2012, Annotations, 14th PhD. Technical Conference, Brno, Czech Republic, 26.1.2012, University of Technology Brno, ISBN 978-80-214-4393-8, pp. 171 (in Slovak)

Contributions to Proceedings in Slovak Republic

1.) ABRAHOIM, I.: Anchorage Regions of Post-Tensioned Concrete Members. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, 2012, ISBN 978-80-8076-104-2, pp. 299-304 (in Slovak)

2.) ABRAHOIM, I.: Assessment of Cracking of Pre-Tensioned Concrete Members. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 277-282 (in Slovak)

3.) BELLOVÁ, M.: Determination of Time for Fire Resistance Classification of Masonry Walls According to Eurocode 6. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 499-506 (in Slovak)

4.) BELLOVÁ, M.: Influence of Different Kinds of Mortar on the Resulting Strength of Masonry. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 67-72 (in Slovak)

5.) BENKO, V. - VYSKOČ, A. - ĎURAČKA, J.: Verification of Damage Caused by Neglecting Crack Widths During Design of WWTP Tanks. In: Proceedings of 17th Conference on Statics of Buildings 2012, Piešťany, Slovak Republic, 14.-15.03.2012, ISBN 978-80-970037-3-9 (in Slovak)

6.) BILČÍK, J. - HALAŠA, I.: Applying Guidelines for Watertight Concrete Structures-White Box. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 35-42 (in Slovak)

7.) BILČÍK, J. - BAJZA, A. - WEISSMANN, Ľ.: Granulation Tower Repair at Duslo Šaľa. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 337-342 (in Slovak)

8.) BORZOVIČ, V. - VALACH, P.: Distribution and Redistribution of Internalforced in RC Flat Slab. In: 17th Conference on Statics of Buildings 2012, Piešťany, Slovak Republic, 14.-15.03.2012, ISBN 978-80-970037-3-9, pp. 159-164 (in Slovak)

9.) BRONDOŠ, J. - BILČÍK, J.: The Strengthening of Columns under the Impact of a Load. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 361-364 (in Slovak)

10.) ČABRÁK, M.: A New Standard for the Assessment of Existing Structures. In: Proceedings of the 17th Conference on Statics of Buildings 2012, Piešťany, Slovak Republic, 14.-15.03.2012, ISBN 978-80-970037-3-9, pp. 259-266 (in Slovak)

11.) FILLO, Ľ.: Verification of the Strut-and-Tie Models of the D-Regions of Concrete Structures. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 237-242 (in Slovak)

12.) GAJDOŠOVÁ, K.: Effects of High Temperatures on Structures Strengthened by FRP Materials. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 365-370 (in Slovak)

13.) GRAMBLIČKA, Š. - LELKES, A.: Design of Composite Steel and Concrete Columns: Detailing, Joints and Examples of Application. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 139-144 (in Slovak)

14.) GRAMBLIČKA, Š.: Longitudinal Shear Resistance of Composite Slabs. In: Proceedings of the 17th Conference on Statics of Buildings 2012, Piešťany, Slovak Republic, 14.-15.03.2012, ISBN 978-80-970037-3-9, pp. 151-158 (in Slovak)

15.) GRAMBLIČKA, Š. - VERÓNY, P.: Reinforced Concrete Columns: Analysis of the Influence of Transverse Reinforcement. In: Proceedings of the 9th International Scientific Conference of the Faculty of Civil Engineering, Košice, Slovak Republic, May 22-25, 2012, Košice, University of Technology in Košice, ISBN 978-80-553-0905-7 (in English)

16.) HARVAN, I.: Some Requirements for Assessing Horizontal Loads in High-Rise Building Design. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 293-298 (in Slovak)

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19.) HUDOBA, I.: Method of Fixing Position of Steel Bar Reinforcement in Reinforced Concrete Elements and Its Consequences for the Reliability of Reinforced Concrete Structure. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 217-222 (in Slovak)

20.) LACO, J. - BORZOVIČ, V.: Bond Behaviour of Prestressing Units Loaded with Corrosion Protection Emulsions. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 213-216 (in Slovak)

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22.) PAULÍK, P.: Concrete Bridges on the Territory of Slovakia. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 77-84 (in Slovak)

23.) PAULÍK, P. – HUDOBA, I.: Degradation of High Performance Fibre-Reinforced Concrete Beams in an Aggressive Solution of Amonium Nitrate in a Loading Condition. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 381-386 (in Slovak)

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25.) ŠOLTÉSZ, J.: Design of Beam and Shell Concrete Structures in a CS STRAP at DeCoSaB. In: Proceedings of the 17th Conference on Statics of Buildings 2012, Piešťany, Slovak Republic, 14.-15.03.2012, ISBN 978-80-970037-3-9, pp. 195-203 (in Slovak)

26.) ŠOLTÉSZ, J. – IGNAČÁK, M.: Marginal Seismic Resistance of 3D Shell Concrete Structures. In: Proceedings of Concrete Days 2012 Conference, Bratislava, Slovak Republic, 25.-26.10.2012, Slovak University of Technology in Bratislava, ISBN 978-80-8076-104-2, pp. 243-248 (in Slovak)

27.) ŠOLTÉSZ, J. - IGNAČÁK, M.: Strength Assessment of the HVAC Pipeline Distribution at the EMO34 NPP. In: Proceedings of the 17th Conference on Statics of Buildings 2012, Piešťany, Slovak Republic, 14.-15.03.2012, ISBN 978-80-970037-3-9, pp. 213-220 (in Slovak)

prof. Eng. Igor Hudoba, PhD.

Eng. Ján Mikuš

Ultra High Performance Concrete and Its Application in Composite CC Columns

In the recent time a great attention is paid for research and practical utilization of Ultra High Performance Concrete (UHPC). The maim principles concerning its constituence composition, technological procedures of dosing, mixing and casting are already well known. Excellent mechanical and other physical properties of UHPC are used mostly for special purposes. 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 a core made of UHPC will be applied..

In the frame of research project VEGA No.01/0458/11 – Factors influencing the HSC application for load-bearing elements and structures the possibility of UHPC application is investigated. In collaboration with company STACHEMA (Eng.M.Doval',PhD) a mix composition of UHPC with the compressive strength level 140 MPa was developed. The mix constituence are Slovak origin except of the steel microfiber reinforcement (made by BEKAERT). Informative mix composition and other basic parameters of UHPC are presented in Tab.1.

The consistence of fresh concrete had the character of self-compacting concrete (Fig.1). There were not used any other way for compacting of fresh concrete during its casting into tube formwork. Fib.2 shows UHPC bars cast in two types of forms – plastic tubes and corrugated steel tubes of diameters of 72, resp.80 mm and length of 1500 mm. Combined reinforcement of composite column consisting of traditional steel bar reinforcement (4ø12mm) , steel stirrups ø6 mm and UHPC bar show (Fig.3). Side view on traditional and composite column reinforcement shows Fig.4. Test elements – columns with dimensions of 200x200x1500 mm will be cast by using of wooden formwork in horizontal position. Normal strength concrete will be used for composite columns concreting. The behavior of composite CC columns will be tested under centric compressive performance. The total number of 15 columns will be investigated.

Tab.1 Informative mix composition and basic parameters of UHPC

Mix constituance	
Cement CEM A	831 kg
Silica fume	191 kg
Silica powder 0,9 µm	205 kg
Fine aggreate 0,1 – 0,4mm	914 kg
Superplasicizer	28 kg
Water	180 kg
Steel microfibers 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 mm

Fig.1: Mixing of UHPC

Fig.2: UHPC reinforcing bars

Fig.3: Cross section of complete composite column reinforcement

Fig.4: Side view on traditional and composite column reinforcement

Eng. Mária Bellová, PhD.

Eurocodes Background and Applications
Structural Fire Design

1 INTRODUCTION

European Commission: DG Enterprise and Industry – Joint Research Centre together with European Committee for Standardization CEN/TC 250 – HG FIRE organized on 27-28 November 2012 in Brussels, Belgium **Workshop with worked examples: STRUCTURAL FIRE DESIGN.** 117 participants from 28 countries (especially from Europe, but also from Africa and Asia) took part in this enterprise.

2 OBJECTIVES

Workshop with emphasis on **worked examples** intended to contribute towards the transfer of background knowledge and expertise on Eurocodes-Structural Fire Design Parts from CEN/TC250 Horizontal Group-Fire to potential trainers at national level and Eurocode users. The principal objectives of the workshop were to:

o transfer knowledge and information to representatives of key organisations/institutions, industry and technical associations in Member States;

o provide state-of-the-art training material, background information and worked examples to Eurocodes trainers and users;

o facilitate exchange of views, networking and cooperation.

3 PROGRAMME

The workshop sessions presented the fire resistance assessment of structures according to the Eurocodes. At the beginning of the workshop sessions were presented definitions of actions in fire situations also with basic principles and examples. Each session focused on a specific structural material and addressed the principles and design methods followed by worked examples according to the following order:

- **Actions in fire situation**
- **Fire resistance assessment of steel structures**
- **Fire resistance assessment of steel and concrete composite structures**
- **Fire resistance assessment of concrete structures**
- **Fire resistance assessment of masonry structures**
- **Fire resistance assessment of timber structures**

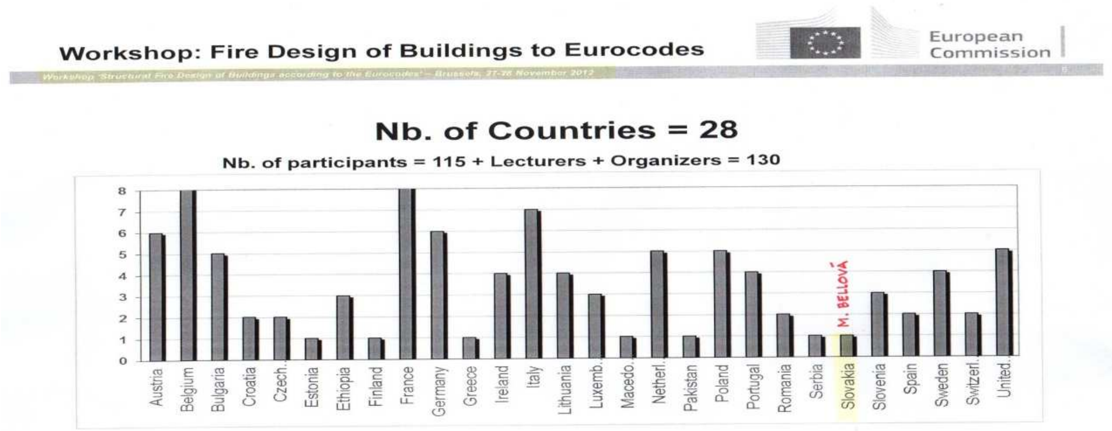


Fig.1: Number of participants at the workshop STRUCTURAL FIRE DESIGN

Fig.2: Announcement to the workshop

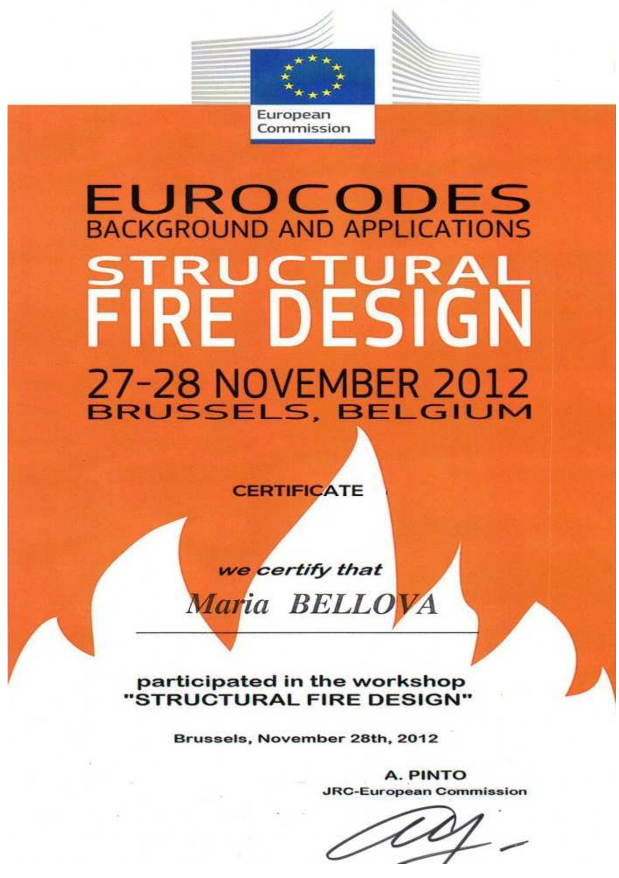


Fig.3: Certification of the participation in the workshop

Eng. Jakub Brondoš

Strengthening of Columns under Impact Load

1 INTRODUCTION

A lot of old parts of bridges, especially their columns aren´t designed under impact load. Impact load includes impacts of vehicles, trains, ships etc. The most frequent kind of impact is impact of vehicles. The most dangerous is impact of trucks for their big weight.

2 IMPACT LOAD ON COLUMN AND THEIR STRENGTHENING

There are two kinds of impact. The 1st is a hard impact, when energy of impact is dissipated on impact body and 2nd one is a soft impact, when energy is dissipated on the structure. The most important value for designing of strengthening of columns under impact load is equivalent static force (ESF) in the point of impact. During the impact dynamic force grows, their maximum is the peak dynamic force (PDF). We needs evaluate ESF from function of dynamic force. When we know ESF, we can take it on structure and do static calculation of structure. The results are the functions of bending moments and shear forces on the column. Then we can design strengthening. The most common method for strengthening is by CFRP (carbon fibre-reinforced polymers). There are two methods of strengthening : CFRP laminates, which increase bending resistance of columns and CFRP fabrics, which increase shear and the compressive resistance of columns. We can stick CFRP laminates longitudinally on the surface of the column or into cut wrinkles. We can stick CFRP fabrics transverse on the surface of the column. A good way of strengthening under impact load is by combination of CFRP fabrics and laminates, because during the impact arise bending moment and shear force together. Inter alia during the impact concrete of column can crush in the point of impact. CFRP fabrics can prevent this effect, concrete can´t crush easy, when it is wrapped.

3 CONCLUSIONS

When designing columns strengthening first we need to know potential ESF, which is the biggest problem at all. If we know ESF, we can design stengthening of column easily. If we don´t know that force, it´s necessary to model structure in some sophisticated FEM software, for example in LS-DYNA, which is oriented on dynamic problems. But this method isn´t simple and it isn´t common among structural engineers. Now we prepare experiment for verification response of concrete column (Fig.1).

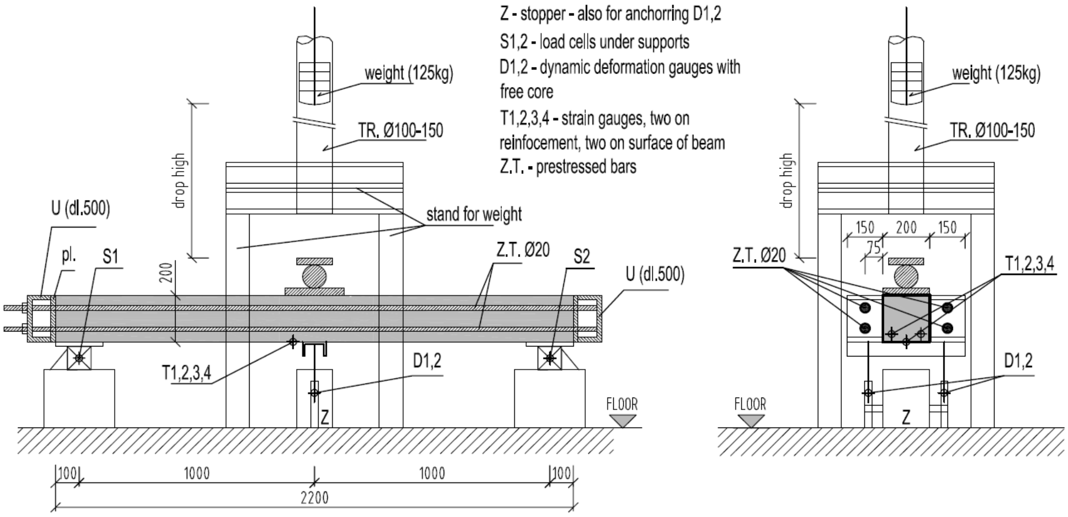


Fig.1: Planned experiment – impact load with free fall

Fig.2: Real impact accident

Fig.3: Experiment - crack of specimen under impact

Eng. Marek Čuhák

Assessment of Reinforced Concrete Column with Mathcad and Matlab

1 INTRODUCTION

By slender columns we have to consider, that the given element can be disrupted stably. When the slenderness of columns is greater than the slenderness limit, the impact of the second order moment has to be taken into account. The influence of the second order bending moment M_2 cannot be ignored. The second order moment can be calculated by application of general non-linear method.

2 ASSESSMENT OF REINFORCED CONCRETE COLUMN

Using computational processor, I analyzed the slender pillars. For the calculation I used three different procedures of general non-linear method. These methods are incremental, which means, that in every step of calculation I add ΔM, and as a result get increases in second order in both directions y and z. Simultaneously, I checked maximum deformations at four corners of the concrete cross section and randomly situated reinforcement. The data about concrete and reinforcement were determined according to EN 1992-1-1 norm. Method 1 – in the first step I gradually add ΔM to the final moment, which is equal to the vector sum of moments from both directions z and y. A normal force N_{Ed} will be then calculated from the final moment and the final eccentricity. Subsequently, I can calculate moments $M_z=N_{Ed}.e_z$ a $M_y=N_{Ed}.e_y$. These external forces N_{Ed}, M_z, M_y enter the equations for calculation of the cross-sectional area deformation. From the results of proportional transformation and after determination of the curvature in both directions z,y, I can calculate the eccentricity increases of the II. order e_{2z}, e_{2y}. With increased eccentricities e_z, e_y, I finish counting e_v and enter the next step in the cycle. The cycle will be interrupted after reaching maximum values of reinforcement and concrete. Method 2 – In this method I consider the impact of cracks on the calculation, the final forming and material nonlinearity. Method 3 – Method C is in principle similar to Method B as it takes into account the impact of cracks emergence along the height of the column, the final forming and material nonlinearity. It differs from the Method B in the fact, that I substitute the bending moments' rate and bending stiffness for curvature, which is changing along the height of the column.

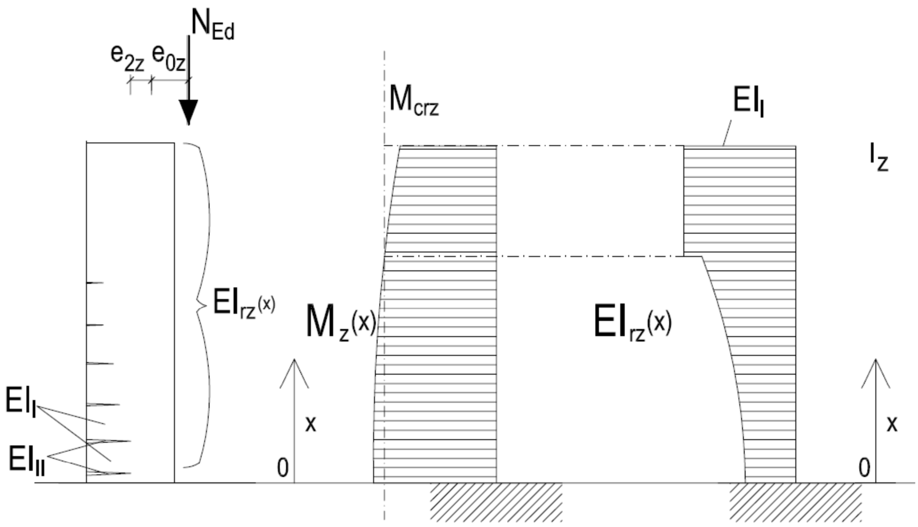


Fig.1: The principle of cracks' effect evaluation with the assistance of average flexural stiffness along the height of the column

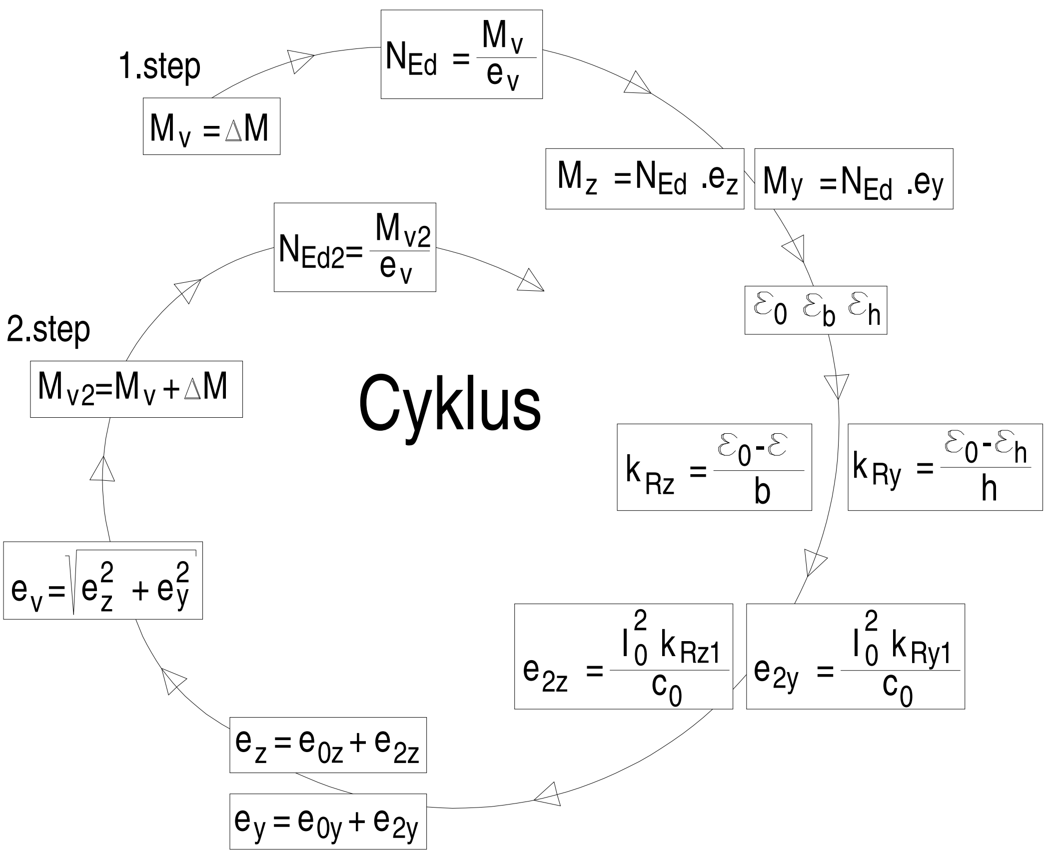


Fig.2: Schematic cycle of methods

Eng. Juraj Dolnák

Experimental Research of Concrete Rheology Properties

1 INTRODUCTION

In this contribution I will describe experimental part of my PhD thesis. The main task of this analysis is a comparison of rheology properties depending on two different classes of concrete. One concrete is with normal strength C40/50 and other is from high performance concrete C70/85.

2 EXPERIMENTAL ANALYSIS

This analysis consists of eight pretension girders with a length of 2,5 m (figure 1), twelve prisms with length of 0,45 m (figure 2) and all this samples are made from two different classes of concrete. A cross section of the girder is displayed on figure 2. On pretension girders I measure total deformation in the time. On the six prisms I measure deformation caused by shrinkage in the time. On the other six prisms in the special steel device I measure deformations caused by creep in the time (figure 3). There are also samples to determine actual strength and deformation properties of concretes (modulus of elasticity). All these experimental samples are placed out on the external environment. In this environment, they are exposed to all weather conditions as well as the real precast bridge girders in structure.

3 MEASUREMENT AND EQUIPMENT

I used two different methods to measure deformations. A part of samples were equipped by strain gauges type EDS-20V-E. The capacity of this gauge is 3000 μstrain and sensitivity is about 1 μstrain. This gauge allows to measure internal temperature in concrete in range -20°C to +80°C. On the all experimental girders were mounted measuring targets and for parallel control, for measuring deformations is also used deformeter. On each girder there are 3 to 4 target bases. From the beginning of experiment is continuously measured humidity and temperature by thermometer with hygroscope type TFA 30.3039.IT DTHL HydrologgPro with three external sensors. On the strands were installed elastomagnetic sensors for measuring prestressing force in the time. In pretension girders are placed four strands with diameter $\emptyset$12,5 mm and area 91,3 mm^2. Maximal stress which was introduced is 1416 MPa.

4 ANALYSIS TARGETS

I will compare deformations caused by shrinkage, deformations caused by creep and also recorded prestressing losts for two different types of concrete in time and this results will be confirmed by available models defined in Eurocode and other standards like Model Code.

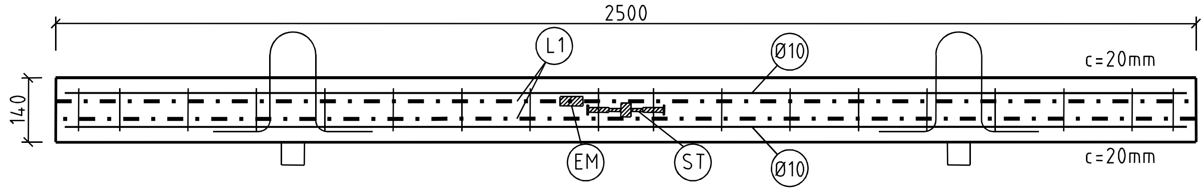


Fig.1: An experimental pretension girder, steel and strands placement

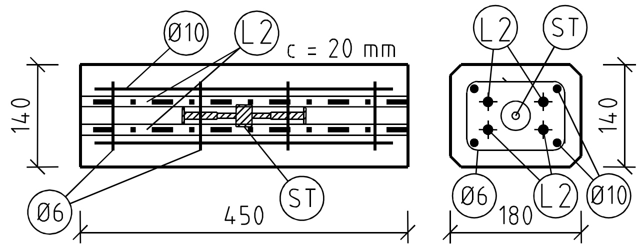


Fig.2: An experimental girder, cross section of girder and strain gauge installation

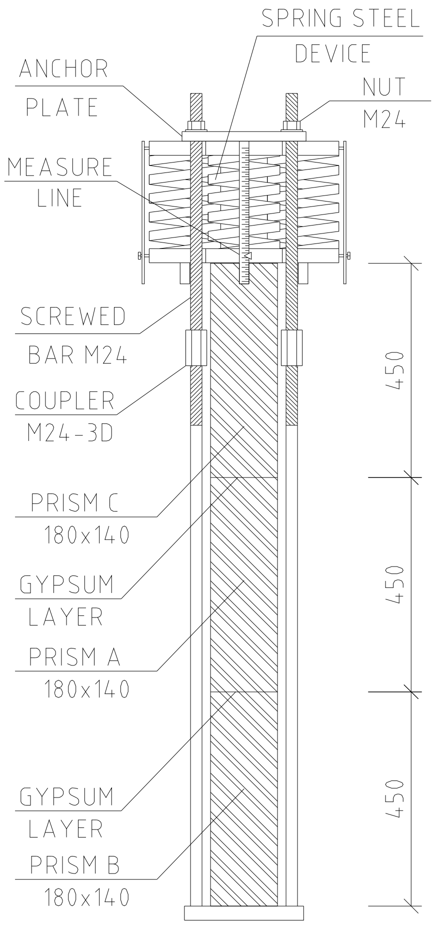


Fig.3: A steel device with system of four springs for modelling of creep

Eng. Marian Kišac

Interaction Diagram of Column with High Slenderness

The impact of slenderness on design resistance of concrete column can be expressed in the interaction diagram that shows the relationship between the bending moment M_{Ed} (horizontal axis) and axial force N_{Ed} (vertical axis).

The cross-section interaction diagram determines the resistance of the critical cross-section.

The most usual stress path of concrete columns is the loading by an axial compressive force at a given eccentricity, which we also call the basic eccentricity e_0. In our case, the column has slenderness λ=120. Calculations were made with tree different eccentricities e_0=0, 20, 30 mm. In the graph we have six lines, which are paired. There are three cases but each of them is calculated in 2 programs: Atena 2D and Atena 3D.

In the case e_0=0 at the increase in axial force on the eccentricity e_0 just a negligible deformation of compressed column occurs (Fig. 1 blue continues and dash line). In the coordinate system M_{Ed} – N_{Ed}, an increase in force is represented by a linear with the directive e_0. The maximum resistance of a section $N_{Rd(a)}$ can be obtained as the intersection of this line with the curve of resistance of the critical cross-section.

At the higher column slenderness with eccentricity e_0=20 (Fig. 1 green and purple dash line), the increase in force N_{Ed} causes column deformation. In the critical cross-section the critical normal force N_{cr} is reached. At this value of force the column loses its stability (Figure 1 dash green and purple line) before the strength of cross-section curs. At this point, without increasing the load, the strain increase and thus emphasizes the bending moment to the moment when the cross-section fails in strength.

At the eccentricity e_0=30, the deformation of columns increases. The column lost its stability at lower load, but the effect is the same, column loses its stability before the strength of cross-section occurs.

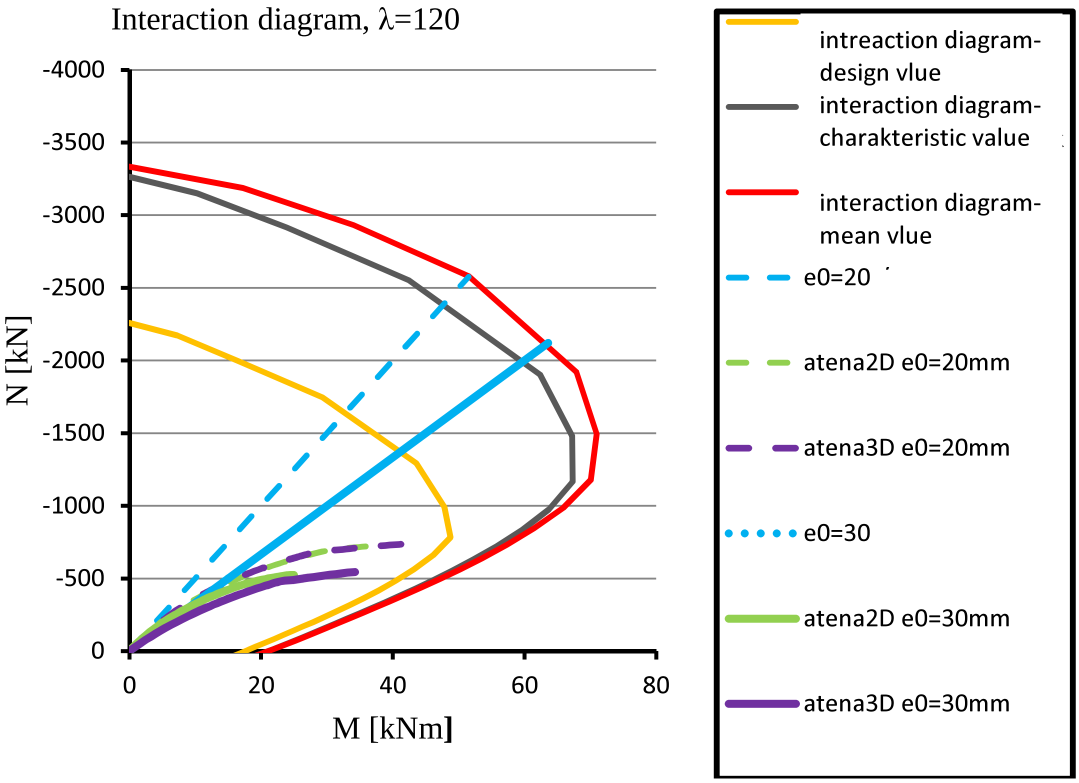


Fig.1: Interaction diagram of column with slenderness 120

Fig.2: Case of slender column failure

Eng. Tamás Porubský

Methods of Analysis of Reinforced Concrete Columns

1 INTRODUCTION

By the analysis of concrete members subjected to axial force and bending moments, such as columns, pillars of bridges etc. it is necessary to consider the Theory II. order, if the limit slenderness of members is exceeded.

2 METHODS OF ANALYSIS OF SLENDER COLUMNS

Design in accordance with Eurocode EN 1992-1-1 based on the Theory of limit states, compares the values of the resultant extreme external and internal forces in cross-section, which are calculated from the design values of strength of both materials, f_{cd} (concrete) and f_{yd} (concrete reinforcing bars).
STN EN 1992-1-1 provides three methods to consider second order effects of slender concrete columns. Method of the nominal rigidity (1), method of the nominal curvature (2), and general method (3).
(1) Method based on nominal stiffness, is allowed to use for isolated columns, also for whole structures, when the nominal values of stiffness are reasonably estimated. In the second order analysis nominal values of bending stiffness have to be used and the effects of cracks, material nonlinearities and creep have to be taken into account to overall behaviour.
(2) Method based on nominal curvature is particularly suitable for isolated columns with a constant normal force and defined effective length ℓ_0, but with realistic assumptions concerning the distribution of curvature, it can also be used for constructions. This method gives nominal second order moments, resulting from the deflection, which is calculated from the effective length and from the estimated maximum curvature.
(3) The general method is based on nonlinear analysis, which allows to use for ULS and SLS provided that the conditions of equilibrium and continuity are accomplished and assumed non-linear effect of materials. Acceptable is analysis of the first or second order. For structures exposed to predominantly static loading is allowed to neglect the previous load effects and assume monotonically increasing load intensity.

3 CONCLUSION

The method of nominal curvature is the most restrictive, comparing with the general method it is less economical.
It is clear that with more precise and more stringent method we should get more efficient results in accordance with safety criteria. Differences in the amount of reinforcement are not negligible, therefore it is advisable to design by using the general method and check the results with approximate methods.

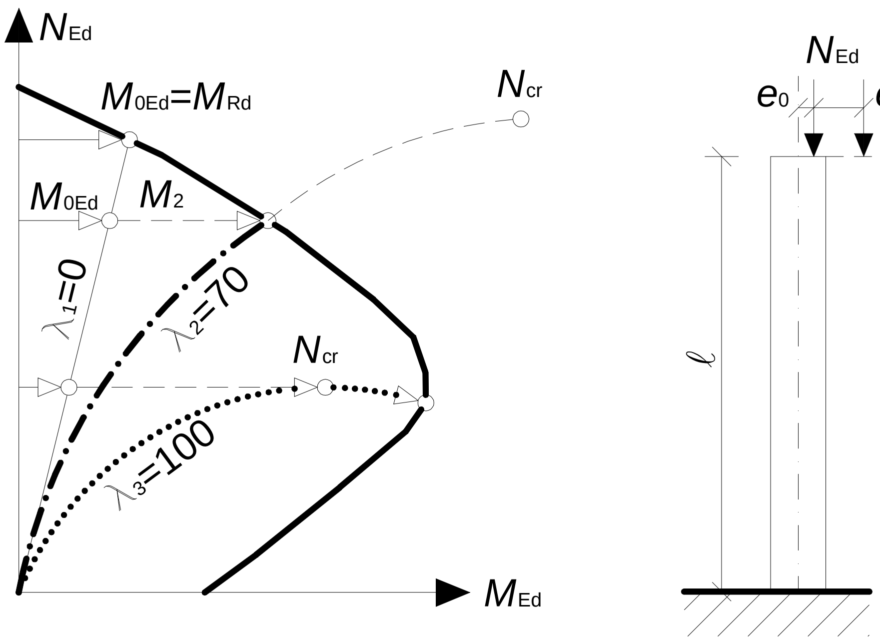


Fig.1: Behavior of slender elements

Eng. Peter Rozložník

Method of Redistribution of Bending Moments for ULS

1 INTRODUCTION

Redistribution is one of several means of the hyperstatic structures analysis using the possibilities of different ratio of the bending moments in the fields and supports. It was anchored in *fib* Bulletin 56*: Model Code 2010* in clause 7.2.2.4.2.

2 ENVIRONMENTAL ACTIONS

Bending moments at the ULS calculated using the linear elastic analysis may be redistributed, provided that the resulting distribution of moments remains in equilibrium with the applied loads. Redistribution of bending moments without explicit check on the rotation capacity is allowed for continuous beams or slabs which are predominantly subjected to flexure and have the ratio of lengths of adjacent spans in the range of 0,5 to 2. In this case the following relations should be applied:

$$\delta = 0{,}44+0{,}125\left(0{,}6+\frac{0{,}0014}{\varepsilon_{cu2}}\right)\frac{x}{d} \qquad \leq C50/60$$

$\varepsilon_{cu2} = 0{,}0035$

$$\delta = 0{,}54+0{,}125\left(0{,}6+\frac{0{,}0014}{\varepsilon_{cu2}}\right)\frac{x}{d} \qquad \geq C55/67$$

$\varepsilon_{cu2} = 0{,}0026+0{,}035[(90-f_{ck}/100]^4$

Analysis of the δ redistribution formulas and with a variable ratio of x/d comes from a concrete strength class and an area and a location of reinforcement over a cross-section. Fig. 1 obviously shows that by the concrete strength class C50/60 is the possible redistribution with $\delta=0{,}7$ under the reinforcement ratio $\rho=0{,}012$ (by taking into account only A_{s1}). By tension and compression reinforcement at the ratio of $A_{s2}=0{,}5A_{s1}$ the possible redistribution is with $\delta=0{,}7$ under the reinforcement ratio $\rho=0{,}031$. It is also necessary to take into account other conditions, especially the reinforcement class A – D. By the concrete strength class C80/95 the possible redistribution is with $\delta=0{,}7$ under the reinforcement ratio $\rho=0{,}009$ (by taking into account only A_{s1}). By tension and compression reinforcement at the ratio of $A_{s2}=0{,}5A_{s1}$ the possible redistribution is with $\delta=0{,}7$ under the reinforcement ratio $\rho=0{,}013$.

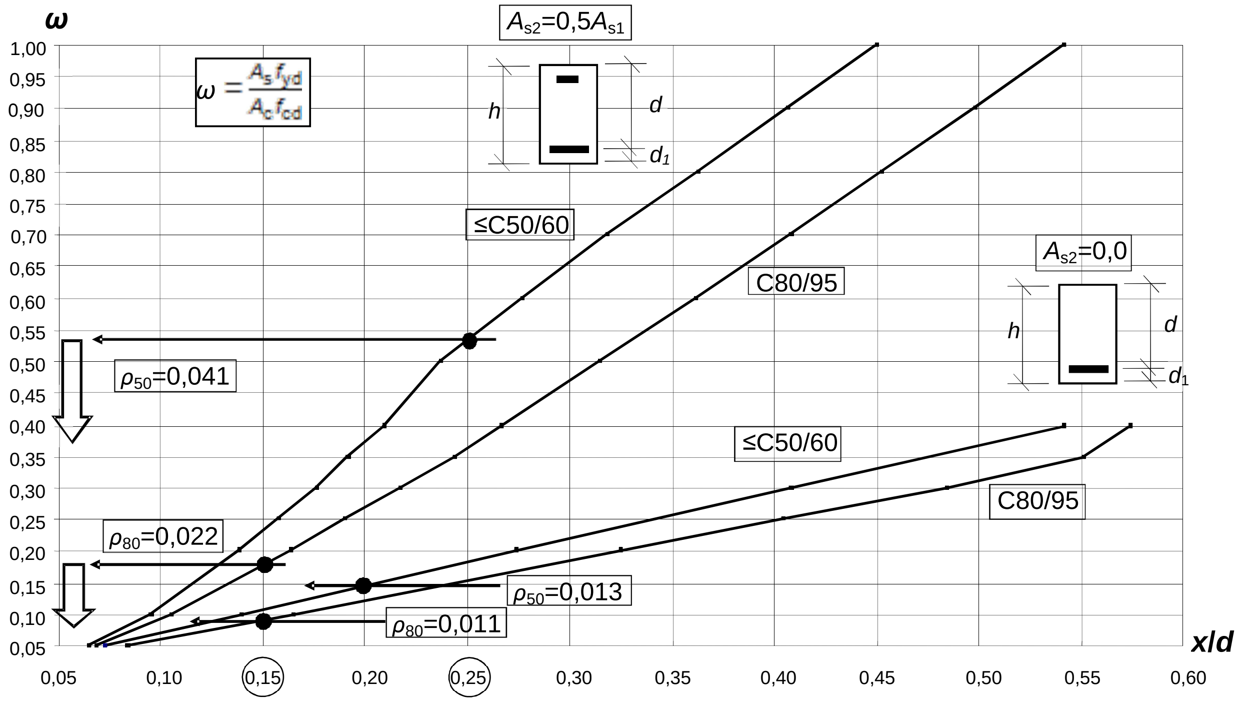
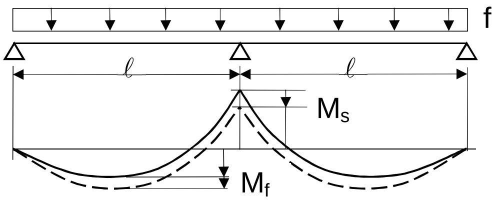
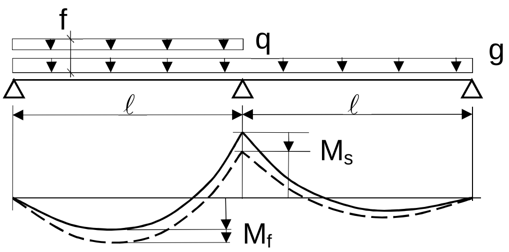


Fig.1: ω-x/d diagram for concrete up to C50/60 and concrete C80/95 with reinforcement steel B 500B



δ	M_f	M_s	
1,0	0,070	0,125	x $f\ell^2$
0,9	0,075	0,113	
0,8	0,080	0,100	
0,7	0,085	0,088	

Fig.2: Redistribution of bending moments



q/g	M_f	M_s	
0,0	0,070	0,125	x $f\ell^2$
0,25	0,075	0,112	
0,67	0,080	0,099	
0,73	0,085	0,088	

Fig.3: Bending moments of continuous beams by different ratios of q/g

Eng. Róbert Sonnenschein

Design of Reinforcement for Crack Width Limitation in Concrete Cross-Section

1 INTRODUCTION

Currently, there are a few wags for designing watertight structures-according to STN EN 1992-1-1, the German standard DIN 1045-1, the Austria guideline, the Model Code 2010 and this year, The Guideline for Watertight concrete structures "White tanks" was published in Slovakia. This paper deals with standards STN EN 1992-1-1 and DIN 1045-1.

2 DESIGN ACCORDING TO STN EN 1992-1-1

Relations for the direct calculation of the area of reinforcement required for selected maximum crack width has been derived based on the relationships described in Section 7.3 of EN 1992-1-1: Cracks control. After derivation we get a quadratic equation. Coefficients of quadratic equations have to be calculated by two manners a) and b). For each case, we get the required area of reinforcement. Higher values decide.

$$A_s = \frac{-b_k + \sqrt{b_k^2 - 4.a_k.c_k}}{2.a_k}$$

a)

$$a_k = E_s.w_k + 3{,}4.c.\alpha_e.k_t.f_{ct,eff}$$

$$b_k = -k_3.c.(F_s - k_t.F_{ct,eff}) + k_1.k_2.k_4.\alpha_e.k_t.F_{ct,eff}$$

$$c_k = -k_1.k_2.k_4.d_s.A_{c,eff}.(F_s - k_t.F_{ct,eff})$$

b)

$$a_k = E_s.w_k$$

$$b_k = -k_3.c.0{,}6.F_s$$

$$c_k = -k_1.k_2.k_4.d_s.A_{c,eff}.0{,}6.F_s$$

3 DESIGN ACCORDING TO DIN 1045-1

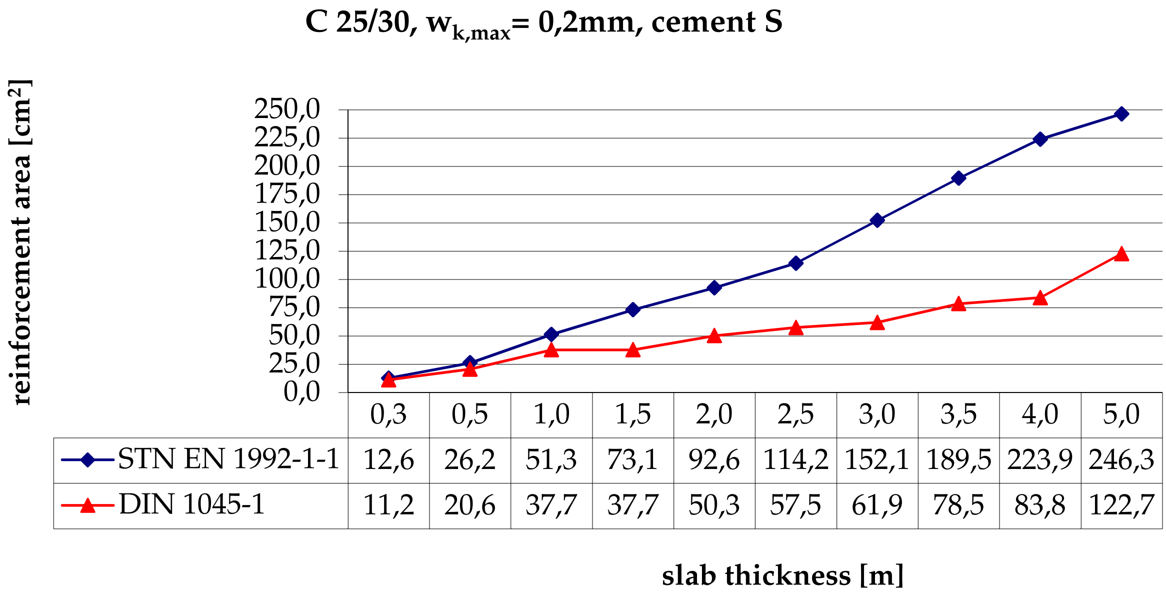
Relations for the direct calculation of the area of reinforcement required for selected maximum crack width has been derived based on the relationships described in Section 11.2 of DIN 1045-1: Crack control and check for decompression. After derivation we get four equations. Definitely, lower values of A_{S1} and A_{S2}, and higher values of A_{S3} and A_{S4} decide.

$$A_{s1} = \sqrt{\frac{d_s.A_{c,eff}.(F_s - 0{,}4.F_{ct,eff})}{3{,}6.w_k.E_s}}$$

$$A_{s2} = \sqrt{\frac{d_s.F_s.(F_s - 0{,}4.F_{ct,eff})}{3{,}6.w_k.E_s.f_{ct,eff}}}$$

$$A_{s3} = \sqrt{\frac{d_s.F_s.A_{c,eff}}{6.w_k.E_s}}$$

$$A_{s4} = \sqrt{\frac{d_s.F_s^2}{6.w_k.E_s.f_{ct,eff}}}$$



	0,3	0,5	1,0	1,5	2,0	2,5	3,0	3,5	4,0	5,0
STN EN 1992-1-1	12,6	26,2	51,3	73,1	92,6	114,2	152,1	189,5	223,9	246,3
DIN 1045-1	11,2	20,6	37,7	37,7	50,3	57,5	61,9	78,5	83,8	122,7

Fig.1: Diagram of relationship between slab thickness and need reinforcement area for standard EN 1992-1-1, DIN 1045-1

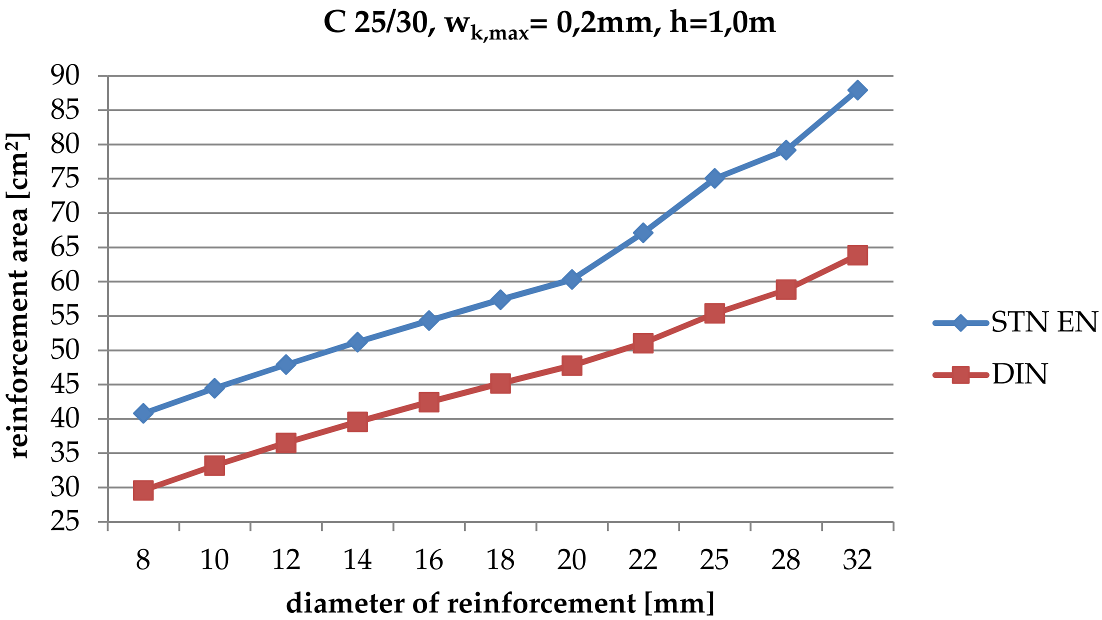


Fig.2: The dependence of the average concrete reinforcement and reinforcement areas required for each standardized procedure, slab thickness h = 1.0 m and limit crack width $w_{k,\,max}$ = 0.2 mm.

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Assessment of Shear Resistance of Reinforced Concrete Members According to EN 1992-1-1

1 INTRODUCTION

The goal of the article is to apply the European standard EN 1992-1-1 on the design of reinforced concrete structural elements, and its effect on shear forces together with normal forces. Critical sections for the shear forces expertise. Compression diagonal crush and shear crack failure expertise. Shear resistance of elements without shear reinforcement reinforced concrete slabs). Shear resistance of elements with shear reinforcement (beams, bearers, girders).

2 LOAD AND FORCES IN REINFORCED CONCRETE MEMBERS

Overall vertical load f carried by the slab or beam element consists of its own weight g_1, other continuous loads g_2 (partition walls, floors, enclosure covering etc.) and variable load (in typical levels it is represented by utility load q, on the top level it is the snow load s). Design values of the continuous load g_d we can obtain by multiplying the characteristic value gk by the load coefficient $\gamma_{G,sup} = 1{,}35$, and the design value of variable load q_d we can obtain in a similar process of multiplying the characteristic value q_k by load coefficient $\gamma_Q = 1{,}50$. The resistance of reinforced concrete elements and pre-stressed elements with unbonded tendons and their failure under the effect of shear and normal force we measure in the utility stage when the time $t = \infty$, when element is affected by the total design vertical load
$f_d = 1{,}35\,(g_{1k} + g_{2k}) + 1{,}50\, q_k$
Force and moment effects of pre-stressing unbonded reinforcement we can consider outer load which we can suitably superpose with the vertical load f_d. Design values of the forces which cause the failure under the effect of shear force V_{Ed} and normal force N_{Ed} in the critical section 1-1' of the load-bearing element we can obtain from the design load f_d overall effect and the design value of the pre-stressing force ${}^{(-)}P_d(t=\infty) = \gamma_p\,{}^{(-)}P_m(t=\infty)$, where $\gamma_p = 1{,}0$ is the coefficient of pre-stress reliability and ${}^{(-)}P_m(t=\infty)$ is the medium value of the pre-stressing force.

$$\begin{aligned} V_{Ed} &= {}^{(+)}V_{fd} + {}^{(-)}V_{pd} \quad (kN) \\ N_{Ed} &= {}^{(-)}N_{pd} \quad (kN) \end{aligned} \qquad (1)$$

Parameters in these formulas
V_{fd} … design lateral force in the critical section 1-1' caused by the design load f_d ,
V_{pd} ... shear force in the critical section 1-1' caused by the shear force of suitably shaped pre-stressing units defined by inclined design pre-stressing force ${}^{(-)}P_d(t=\infty)$,
N_{pd} … design normal force in the critical section 1-1' caused by the horizontal force of total design pre-stressing force ${}^{(-)}P_d(t=\infty)$.
In case of reinforced concrete elements without pre-stress we consider the forces V_{pd} and N_{pd} as having zero value. The

value of normal force N_{Ed} can have value not equal to zero also in elements without pre-stress, depending on the static scheme of the load-bearing construction. Locations of the critical sections 1-1' are illustrated on the fig. 1. In this article we will consider the compression normal stresses as having minus values and the lateral forces caused by vertical load we will consider as having plus values.

3 SHEAR RESISTANCE OF ELEMENTS WITHOUT SHEAR REINFORCEMENT

If inclined cracks appear near its abutments of a reinforced concrete element, we can verify the shear resistance of a structural element without shear reinforcement. Determining part of the shear resistance $V_{Rd,c}$ of an element without shear reinforcement is crumpling of the grains of gravel aggregate.

Effectiveness of the crumpling depends on the width of shear crack and tensile strength of concrete. Tensile strength of concrete depends on its quality which is expressed by the category of concrete. The width of shear crack depends on the area $A_{s\ell}$ of the reinforcement which crosses the inclined crack. Another important factor to consider is the distance between this reinforcement and the neutral axis, as the growing distance causes the impact of reinforcement on shear crack width reduction to diminish. Shear resistance is also affected by normal stresses caused by normal force N_{Ed} in the element. Compression stresses increase the shear resistance and tensile stresses decrease it. Shear resistance $V_{Rd,c}$ of the element without shear reinforcement meets the requirements if complying with the condition

$$V_{Rd,c} \geq V_{Ed} \qquad \text{(kN)} \qquad (2)$$

The shear force V_{Ed} caused by the outer load we can obtain from the formula (1) in the critical section 1-1', its distance from the front of elements abutment being equal to the effective height d of the element. If not compliant with the condition (2), it is necessary to design shear reinforcement for the element. Calculation of the element's shear resistance $V_{Rd,c}$ without shear reinforcement in the hear crack we can obtain from the following formulas

$$V_{Rd,c} = \begin{bmatrix} C_{Rd,c}\, k \left(100\, \rho_\ell \, \frac{f_{ck}}{MPa}\right)^{1/3} \\ -\, 0{,}15\, \sigma_{cp} \end{bmatrix} b_w\, d$$

$$V_{Rd,c} \geq V_{Rd,c,min} \qquad (3)$$

$$V_{Rd,c,min} = (v_{min} + 0{,}15\, \sigma_{cp})\, b_w\, d$$

Parameters in formulas (3)

$C_{Rd,c}$ ... empirical coefficient, its value being $C_{Rd,c} = 0{,}18$ MPa / γ_c , when the partial factor of concrete is $\gamma_c = 1{,}50$,

K ... parameter of the element's height impact $k = 1 + (200 \text{ mm} / d)^{1/2}$, Its maximum value being $k \leq 2$,

f_{ck} ... characteristic compressive strength (MPa) of concrete ,

b_w ... smallest width of tensile area of a concrete element in the section 1-1' without considering the weakening caused by the holes for unbonded cables,

ρ_ℓ ... degree of reinforcement by longitudinal bars behind the critical section 1-1' anchored at least $(d + \ell_{bd})$, where according to the fig. 1 $\rho_\ell = A_{s\ell} / (b_w\, d)$, its maximum value being $\rho_\ell \leq 0{,}02$,

d ... effective height of the element in the section 1-1' according to the fig. 1,

$A_{s\ell}$ ... cross-section area of the reinforcement according to the fig. 1,

ℓ_{bd} ... design anchorage length of the longitudinal reinforcement,

σ_{cp} ... normal stress (MPa) caused by the normal force $^{(-)}N_{Ed}$ (MN) according to (1), when compression is considered as having minus value $\sigma_{cp} = {}^{(-)}N_{Ed} / A_c$, its maximum value being $|\sigma_{cp}| \leq |0{,}2\, f_{cd}|$,

f_{cd} ... design compressive strength (MPa) of concrete $(-)f_{cd} = (-)f_{ck} / \gamma_c$,

v_{min} ... minimum value of the shear stress (MPa), which can be withstood by the concrete in the inclined crack $v_{min} = 0{,}035\, k^{3/2} f_{ck}^{1/2}$.

Another problem besides the element's failure in the inclined crack can happen with the crush of the compression diagonal, which forms among the inclined cracks and its angle is equal to the angle of the cracks. Shear resistance $V_{Rd,max}$ of an element without shear reinforcement meets the requirements concerning the crush of the diagonal if in compliance with

$$V_{Rd,max} \geq V_{Ed,max} \quad (kN) \qquad (4)$$

The shear force $V_{Ed,max}$ caused by the external load can be obtained in a process similar to (1), the only difference being that it is calculated in the face of the element. If the condition (4) is not met, it is necessary to increase the dimensions of the cross section or increase the quality of concrete. Calculation of the shear resistance $V_{Rd,max}$ of an element without shear reinforcement against the crush of the compression diagonal can be obtained from the following formula

$$V_{Rd,max} = \frac{1}{2} b_w \; d \; \nu \left| f_{cd} \right| \quad (MN)$$
$$\nu = 0{,}6 \left(1 - \frac{f_{ck}}{250 \text{ MPa}} \right) \qquad (5)$$

The reduction coefficient ν of the design compressive strength f_{cd} of concrete in single-axis compression incorporates the fact, that part of the shear stresses τ_c along the shear cracks is distributed into the compression diagonal. This phenomenon is the cause of disaccord between the stress in the compression diagonal and the main compressive stress $^{(-)}\sigma_{c2,d}$ and thus its angle α_2 is not totally accordant to the angle of the compression diagonal.

In the calculation of the shear resistance according to the formula (5) we marked the smallest width of the tensile area in the abutment face of the element b_w. If multiple parallel pre-stressing cables with the diameter ϕ_{duct} cross one spot in the tension area, the calculation by the formula (5) incorporates the least favorable nominal width of the tension area instead of b_w

$$b_{w,nom} = b_w - 1{,}2 \, \Sigma \, \phi_{duct} \qquad (6)$$

Reinforced concrete elements and partially pre-stressed elements we can design as having no shear reinforcement only in case these are slabs, hollow panels, and less important load-bearing members which do not have a significant impact on the overall resistance and stability of the construction (e.g. capping with span of less than 2 m). In the remaining cases, even if the conditions (2) and (4) are met, it is always necessary to design at least minimum shear reinforcement according to the construction principles for the elements made from reinforced concrete. The shear reinforcement is usually applied in the form of vertical stirrups or shear grates.

4 SHEAR RESISTANCE OF ELEMENTS WITH SHEAR REINFORCEMENT

If the requirements of the element without shear reinforcement are not met, it is necessary calculate shear reinforcement in the abutment area of the element. When designing the shear reinforcement, we use the method of variable angle of the compression diagonals according to the fig. 2. With this method, we model the element as single-apron truss beam with compressed concrete chord and steel chord being affected by tension. These chords are joined by a system of concrete compression diagonals and tensile steel verticals. Compressive stresses in the element are thus thought of as being withstood by concrete, whereas the tensile stresses remain with the longitudinal and shear reinforcement. When designing the shear reinforcement with using the truss analogy, we assume that the design shear force V_{Ed} distributes into the vertical force $(-)F_{cwd}$ in the compression diagonal and the horizontal force $(+)H_{Ed}$ in the tensile chord according to the fig. 2. Forces developed this way are borne by the concrete in the compression diagonal, longitudinal reinforcement bears the horizontal force and the vertical reinforcement bears the vertical force. The angle of the compression diagonals Θ is variable, its value sometimes being incompliant with the orientation of the main stresses in the abutment area of the element. Experiments have shown that the value of Θ ranges from $\Theta = 39°$ to $40°$ in reinforced concrete elements. In pre-stressed elements, the angle is usually flatter, ranging from $\Theta = 30°$ to $36°$, dependant on the pre-stressing force. We usually think of the diagonal as having the angle Θ and we try to design the reinforcement in accordance with the flow of forces. The smaller the angle Θ is, the less reinforcement will be needed to bear the shear force V_{Ed} in the shear crack. At the same time the horizontal force $^{(+)}H_{Ed}$ in the longitudinal reinforcement increases. And, in relation to these facts, the bigger the angle is, the more vertical reinforcement and less longitudinal reinforcement is needed to withstand the shear force V_{Ed}.

The angle of the compression diagonal Θ is limited by these conditions

$1{,}0 \le \cot g\,\Theta \le 2{,}5$ which complies with angles $45° \ge \Theta \ge 22°$ (7)

The value of the compression diagonal angle Θ and the angle of the inclined shear cracks depends on the values of proportional deformations in concrete and reinforcement caused by all forces in the abutment area according to the fig. 2. The proportional deformations of the concrete diagonal and compressed upper chord, as well as the deformations of the lower tensile chord and vertical shear reinforcement, are influenced by many parameters (stresses, geometry, shear and bending reinforcement, development of shear and bending cracks), which cannot be predicted in common civil engineering. The angle of the compression diagonal Θ can be then estimated according to the following formula

$$\cot g\ \Theta = 1{,}25 + \frac{3\ \sigma_{cp,eff}}{f_{cd}} \tag{8}$$

Parameters in these formulas

f_{cd} ... design compressive strength of concrete (MPa) $^{(-)}f_{cd} = {}^{(-)}f_{ck} / \gamma_c$,

$\sigma_{cp,eff}$ ... effective normal stress (MPa) caused by the normal force $^{(-)}N_{Ed}$ (MN) , compression is considered to have minus value $\sigma_{cp,eff} = [\ ^{(-)}N_{Ed} + f_{yd}\ A_{sc}\ ]\ /\ A_c$, maximum value being $|\sigma_{cp,eff}| \le |0{,}2\ f_{cd}|$,

γ_c ... partial factor for concrete $\gamma c = 1{,}50$,

f_{yd} ... design yield strength of the reinforcement (MPa) $f_{yd} = f_{yk} / \gamma s$, maximum value being $f_{yd} \le 400$ MPa ,

f_{yk} ... characteristic yield strength of reinforcement (MPa) ,

γ_s ... partial factor for reinforcement $\gamma_s = 1{,}15$,

A_{sc} ... area (m^2) of the bars of reinforcement located in the compression area of the concrete cross-section,

A_c ... overall area (m^2) of the concrete element's cross-section.

The highest angle of the compression diagonal $\Theta = 39°$ we obtain form the formula (8) for the elements without pre-stress and without pressed reinforcement bars, when the amount is $\sigma_{cp,eff} = 0$ ($\cot g\ \Theta = 1{,}25$). For the pre-stressed elements with the normal force $^{(-)}N_{Ed}$ the angle Θ decreases to the value from $\Theta = 32°$ to $36°$ ($\cot g\ \Theta > 1{,}25$). This case requires the least shear reinforcement.

Design shear resistance concerning the crush of the compression diagonal

In order to prevent the crush of the diagonal, the stress in it cannot be greater than the reduced design compressive strength of concrete $\alpha_{cw}\, \nu_1\, f_{cd}$. When using the truss model illustrated in the fig. 2, we can divide the shear force V_{Ed} into $^{(-)}F_{cwd}$ force in the compression diagonal and the horizontal force $^{(+)}H_{Ed}$ in the reinforcement. Following formula then takes effect

$$^{(-)}\sigma_{cwd} = \frac{^{(-)}F_{cwd}}{b_w\ a} = \frac{V_{Ed}}{z\ b_w}(\mathrm{tg}\ \Theta + \cot g\ \Theta) \ge \alpha_{cw}\, \nu_1\, {}^{(-)}f_{cd} \tag{9}$$

Reduction factor ν_1 of the design compressive strength of the concrete f_{cd} in single-axis compression incorporates the fact, that part of the shear stresses τc along the shear cracks is distributed into the compression diagonal. This phenomenon is the cause of disaccord between the stress $^{(-)}\sigma_{cwd}$ and the main compressive stress $^{(-)}\sigma_{c2,d}$ and thus its angle α_2 is not totally accordant to the angle Θ of the compression diagonal. Another reason for the reduction is the forming of laterally acting tensile stresses (approximately orthogonal in relation to the direction of the shear cracks) by the shear reinforcement into the compression diagonal, which makes its loading less favorable case of double-axis load than single-axis compression. The coefficient α_{cw} on the right side of the equation (9) incorporates parallel effect of the compressive stresses $^{(-)}\sigma_{cwd}$ in compression diagonals of the truss model according to the fig. 2, at the same time taking into account the effect of the longitudinal normal stress caused by the design normal force $^{(-)}N_{Ed}$ from the external load according to the formula (1). Design shear resistance $V_{Rd,max}$ concerning the crush of the compression diagonal can be obtained from the formula (9), if we switch the inequality sign with the equality sign and the design shear force V_{Ed} we replace with the design shear resistance $V_{Rd,max}$ when reaching the load-bearing capacity of the element. When using vertical shear reinforcement under the angle of $\alpha = 90°$ in relation to the longitudinal reinforcement, we can apply the following formula

$$V_{Rd,max} = \frac{\alpha_{cw}\ z\ b_w\ \nu_1\ f_{cd}}{\text{tg}\ \Theta + \text{cotg}\ \Theta} \quad \text{(MN)} \qquad (10)$$

In the formulas (9) and (10) we used the following parameters

z ... internal forces arm (m) , which we obtain during the design of longitudinal reinforcement in the face of the element. For the with simple abutment at the edge of the element we can calculate the value approximately as $z = 0{,}9d$,

d ... effective height of the cross-section according to the fig. 1 ,

f_{cd} ... design compressive strength of concrete (MPa) $f_{cd} = f_{ck} / \gamma_c$,

f_{ck} ... characteristic compressive strength of concrete (MPa),

γ_c ... partial factor for concrete $\gamma_c = 1{,}5$,

ν_1 ... reduction factor, which can be put equal to the factor ν in formula (5),

b_w ... smallest width (m) of the tension area in the element face,

σ_{cp} ... normal stress (MPa) caused by design normal force $^{(-)}N_{Ed}$ (MN) compression is considered to have minus value $\sigma_{cp} = {}^{(-)}N_{Ed} / A_c$,

α_{cw} ... coefficient used to incorporate parallel effect of the compressive stresses in compression diagonals and the effect of longitudinal horizontal strain caused by the normal force $^{(-)}N_{Ed}$ from the external load according to (1). It equals

$\alpha_{cw} = 1 - \sigma_{cp} / f_{cd}$ if $0 \le |\sigma_{cp}| \le 0{,}25 f_{cd}$,

$\alpha_{cw} = 1{,}25$ if $0{,}25 f_{cd} \le |\sigma_{cp}| \le 0{,}5 f_{cd}$,

$\alpha_{cw} = 2{,}5 (1 + \sigma_{cp} / f_{cd})$

If $0{,}50 f_{cd} \le |\sigma_{cp}| \le 1{,}0 f_{cd}$.

If multiple parallel pre-stressing cables with the diameter ϕ_{duct} cross one spot in the tension area, the calculation by the formula (10) incorporates the least favorable nominal width of the tension area $b_{w,nom}$ instead of b_w

$$b_{w,nom} = b_w - 1{,}2\ \Sigma\ \phi_{duct} \ , \qquad (11)$$

Shear resistance of an element with shear reinforcement meets the requirements concerning the crush of the diagonal if in compliance with

$$V_{Rd,max} \ge V_{Ed,max} \quad \text{(kN)} \qquad (12)$$

The shear force $V_{Ed,max}$ caused by the external load can be obtained in a process similar to (1), the only difference being that it is calculated in the face of the element. If the condition (12) is not met, it is necessary to increase the dimensions of the cross section or increase the quality of concrete.

Design shear resistance in terms of breach of shear reinforcement

Shear resistance in the inclined crack is determined by the capacity of the shear reinforcement which crosses the crack in the length of its horizontal projection from cotg Θ according to the fig. 2. The compression normal force ${}^{(-)}N_{Ed}$ calculated from the formula (1) does not directly participate in the resistance. Its effect is incorporated in the steeper angle Θ of the compression diagonals according to (7), which helps to reduce the amount of shear reinforcement in comparison with common reinforced concrete elements. When using vertical shear reinforcement under the angle of $\alpha = 90°$ to the longitudinal axis of the element, we can write equations for calculating the resistance of the element in the crack, which express the effect of shear forces in the shear reinforcement as illustrated in fig. 2

$$V_{Rd,s} = f_{ywd} \frac{A_{ss1}\, n_{st,s}}{s} z \cot g\Theta \quad (MN) \qquad (13)$$

$$A_{ss1} = \pi\, (\phi_{ss}/2)^2 \qquad (m^2)$$

Following parameters were used in these formulas

z ... internal forces arm (m) , which we obtain during the design of longitudinal reinforcement in the distance d from the face of the element in the section 1-1' For the with simple abutment at the edge of the element we can calculate the value approximately as $z = 0{,}9\, d$,

d ... effective height of the cross-section in critical section 1-1' ,

f_{ywd} ... design yield strength (MPa) of the shear reinforcement $f_{ywd} = f_{ywk} / \gamma_s$,

f_{ywk} ... design characteristic strength (MPa) of the shear reinforcement,

γ_s ... partial factor for reinforcement $\gamma_s = 1{,}15$,

A_{ss1} ... cross-section area of one stirrup of shear reinforcement with diameter ϕ_{ss} ,

$n_{st,s}$ ... number of shear cuts of one vertical row of shear reinforcement,

s ... distance (m) between the stirrups of shear reinforcement in longitudinal direction according to fig. 2 .

Shear resistance $V_{Rd,s}$ of an element with shear reinforcement meets the requirements if in compliance with

$$V_{Rd,s} \geq V_{Ed,s} \quad (kN) \qquad (14)$$

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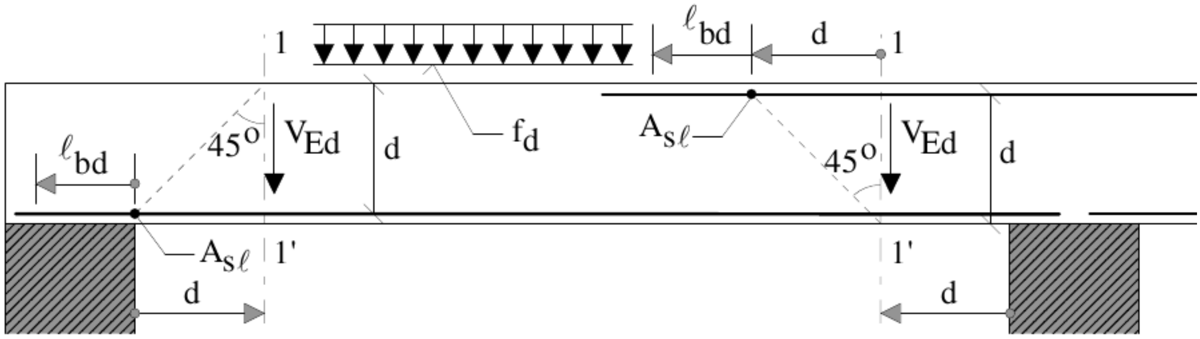

Fig.1: Definition of the longitudinal reinforcement $A_{s\ell}$ and critical sections 1-1′ for the expertise of shear resistance of an element without shear reinforcement

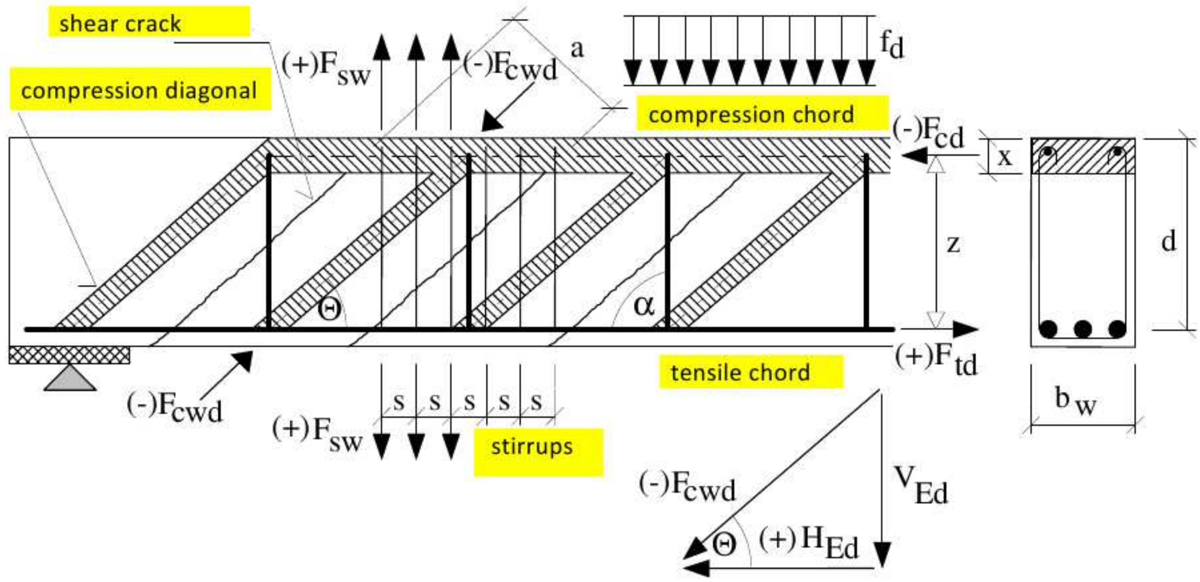

Fig.2: Truss model for the design of shear reinforcement. Distribution of the shear force V_{Ed} into $^{(-)}F_{cwd}$ in the compression diagonal and the horizontal force $^{(+)}H_{Ed}$ in the tensile chord

SLENDER RECTANGULAR CONCRETE COLUMNS STRENGTHENED WITH CFRP CONFINEMENT AND NSMR

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Abstract

Present trends demand the preservation of slenderness of strengthened structures, especially columns. Results of introduced investigation demonstrate a significant difference in slender and short column strengthening. This paper presents the effectiveness of the use of carbon fibre reinforced polymers for slender reinforced concrete columns strengthening. To verify the theoretical assumptions and the 3D nonlinear numerical analysis, full-scale specimens with rectangular cross-sections were tested under eccentric compressive loading to failure. Tested specimens were divided into four series: non-strengthened columns, columns confined with a single layer of CFRP sheet, columns strengthened by CFRP strips in NSMR form and columns strengthened by combination of mentioned methods. Failure mode, compression force of resistance, deflection, strains in the most stressed cross-section and utilization of CFRP in strengthened members were the most serious variables measured. The results indicated that effects of strips in NSMR method enhanced the behaviour of slender columns and CFRP sheets of short columns.

Keywords: CFRP, column, slenderness, strengthening.

1. Introduction

Fibre reinforced polymers are used for strengthening in a civil engineering practise mostly for their biggest advantage – the possibility of resistance enhancement without significant cross-sectional dimensions increase. This is especially useful for columns strengthening with the demands for the preservation of their slenderness. The effect of strengthening techniques on short columns has been demonstrated in numerous tests. The investigation of eccentrically loaded slender columns is insufficient, and so this method of application is limited.

The enhancement of the column load-carrying capacity by CFRP transverse confinement decreases with an increase of load eccentricity and column slenderness [1]. For slender columns with slenderness of about 70, the ultimate strength for the columns strengthened by unidirectional transverse CFRP is quite close to that of unstrengthened ones [2]. Transverse confinement with a bidirectional CFRP jacket or the confinement with unidirectional CFRP jacket in longitudinal direction improves the behaviour of slender concrete columns [3]. The longitudinal fibres become more effective when bending becomes predominant.

This paper says about strengthening slender reinforced concrete columns with CFRP strips in near surface mounted reinforcement (NSMR) form, in the form of confinement with CFRP sheet and their combination.

2. Experimental investigation

2.1 Specimens

The experimental investigation was performed on full-scale slender rectangular reinforced concrete columns. Columns were divided into four series: non-strengthened columns, columns strengthened by longitudinal NSM CFRP strips, transverse CFRP sheet wrapping and a combination of these two methods. All columns were 4.1m long with cross-section of 210 x 150 mm. Reinforcement of a column consisted of 8 bars of diameter 10 mm in longitudinal direction and stirrups of diameter 6 mm in distances 150 mm, concerned by up to 30 mm in the heads of a column.

Columns were hinge supported at both ends – the initial end eccentricity (e_0) 40 mm was put into by these supports. At this eccentricity, columns were loaded by a compression force, value of which was growing continuously step by step to the failure.

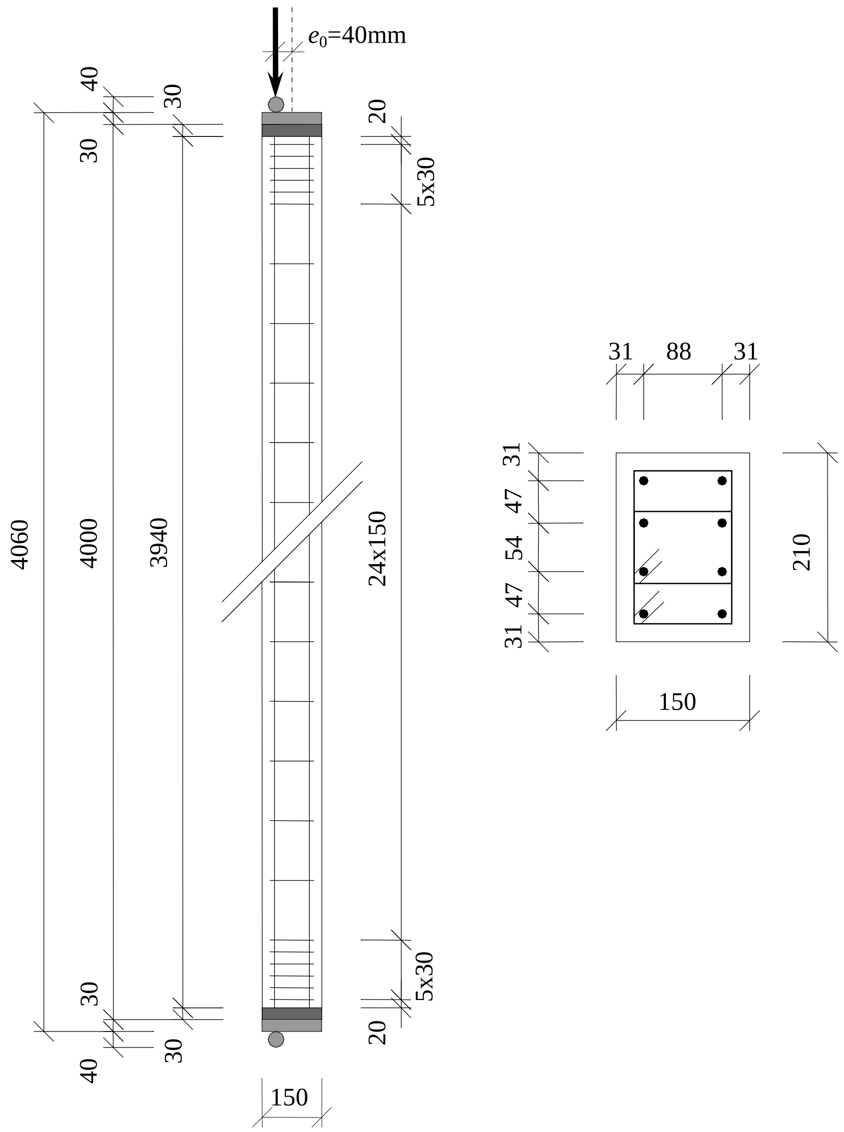

Figure 1. Geometry and loading of columns.

2.2 Strengthening preparation

Selected columns were strengthened before experimental investigation. At first, columns for NSMR method were prepared – three grooves (3x15mm) on each longer side of cross-section were cut over the whole length of a column. After cleaning, the grooves were filled with epoxy adhesive and CFRP strips (1,4x10mm) were pushed into them.

For columns later strengthened by confining, all edges were chamfered to radius 20mm. Unidirectional CFRP sheet with fibres in transverse direction (width of 300mm) was wrapped in “stirrup form” with distances 50mm. The sheet was anchored by an overlap of 170mm on a shorter side of the cross-section.

a) NSM CFRP strips

b) CFRP sheet wraping

Figure 2. Columns prepared for testing.

2.3 Measurements and results

The main objective of this experimental investigation was the observation of the relationship between the deflection at the mid height and the eccentrically acting compression force for determination of the most effective strengthening method for this type of columns and loading. The most detailed plotting of the whole loading behaviour was allowed by linear variable displacement transducers (LVDTs) measurement. To check the accuracy of the main measurement other direct and indirect measurement methods were also used.

For direct checking, the horizontal displacement in five geodetic points was measured by theodolite. Indirect methods for checking the mid height deflection were based on deflection calculations from the curvature which was determined from strains measured on compression and tensile part of the cross-section. Strains were measured on concrete faces – by deformeters and in steel and concrete – by tensometer gauges. Comparison of results of all measurements methods for a chosen column is shown in Figure 3.

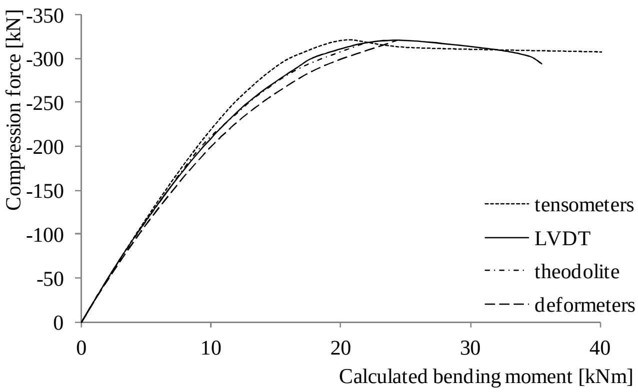

Figure 3. Measurements comparison for column strengthened with NSM CFRP strips.

The most accurate measurement was enabled by LVDTs. The increase in the resistance of the compression force beside the non-strengthened reinforced concrete column can be estimated as follows: 12.9% for the columns strengthened by adding the CFRP strips into the grooves in the concrete cover layer; 2.4% for the columns strengthened by confinement with one layer of the CFRP sheet; 15.4% for the columns strengthened by a combination of CFRP strips and a CFRP sheet.

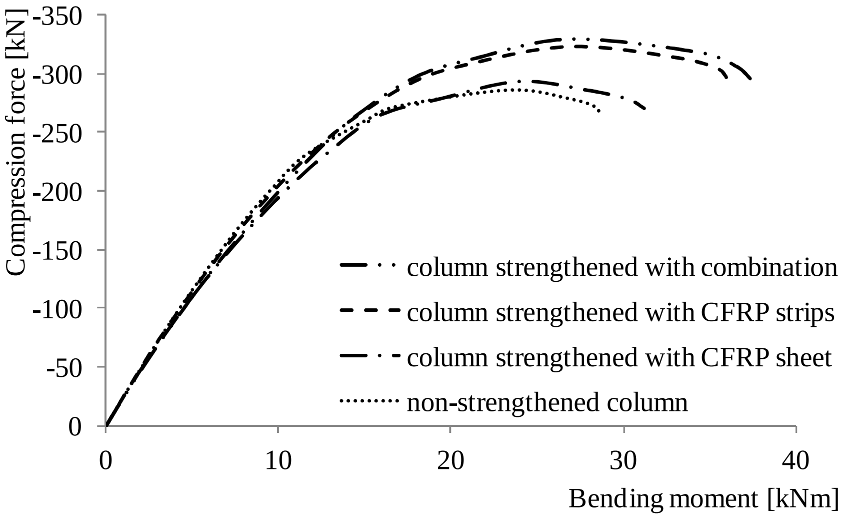

Figure 4. Load-moment relationship measured with LVDT during experimental testing.

From Figure 4 it is evident, that NSM CFRP strips take more apparent effect on the strength enhancement of slender reinforced concrete columns than confinement with CFRP sheet.

3. Theoretical and numerical procedure, comparison with experimental results

With the material properties observed during the experimental testing, a numerical model in ATENA 3D software and theoretical calculations were provided.

In theoretical investigation, at first, the interaction (load-moment) diagrams for reinforced concrete column cross-section were made out (= short columns interaction diagrams). With the making a provision for second order effect bending moments, the interaction diagrams were modified to the slender column interaction diagrams with a given slenderness.

The second step of the theoretical calculation was to include the strengthening effects in interaction diagrams and into the determination of column resistance. Near surface mounted CFRP strips were assumed as additional reinforcement with relevant characteristics. The strengthened structure is unloaded during application of CFRP strips, even though there are some initiate strains concerning the self weight of a structure and a provision has to be made for these strains during design.

Confining with CFRP sheet is considered as an increased concrete strength in a triaxial state of stress. There are many models available to calculate the lateral confining pressure and to enhance the confined concrete compressive strength. The model that mostly approximates the experimental results was the Lam and Teng's stress-strain model (2003) [4]. Confined concrete compressive strength f_{cc} and ultimate strain ε_{cc} are calculated by the initial strength f_{co} and strain ε_{co} of unconfined concrete and lateral pressure f_l as follows:

$$f_{cc} = f_{co} \cdot \left(1 + 3{,}5 \cdot \frac{f_l}{f_{co}}\right) \tag{1}$$

$$\varepsilon_{cc} = \varepsilon_{co} \cdot \left(1 + 17{,}5 \cdot \frac{f_l}{f_{co}}\right) \quad \text{when} \quad f_l \geq 0{,}07 \cdot f_{co} \tag{2}$$

$$\varepsilon_{cc} = \varepsilon_{co} \cdot \left(1 + 17{,}5 \cdot \left(\frac{f_l}{f_{co}}\right)^{1,2}\right) \quad \text{when} \quad f_l < 0{,}07 \cdot f_{co} \tag{3}$$

The lateral confining pressure f_l is a reduced value by the effect of not fully confined area of the cross-section (cross-section shape, radius of rounded edges) and by the effect of stirrup form confinement over the column height (not continuous confinement).

Results of these proposed calculation approaches are shown in Figure 5. Comparison of all three investigation methods is shown in Figure 6.

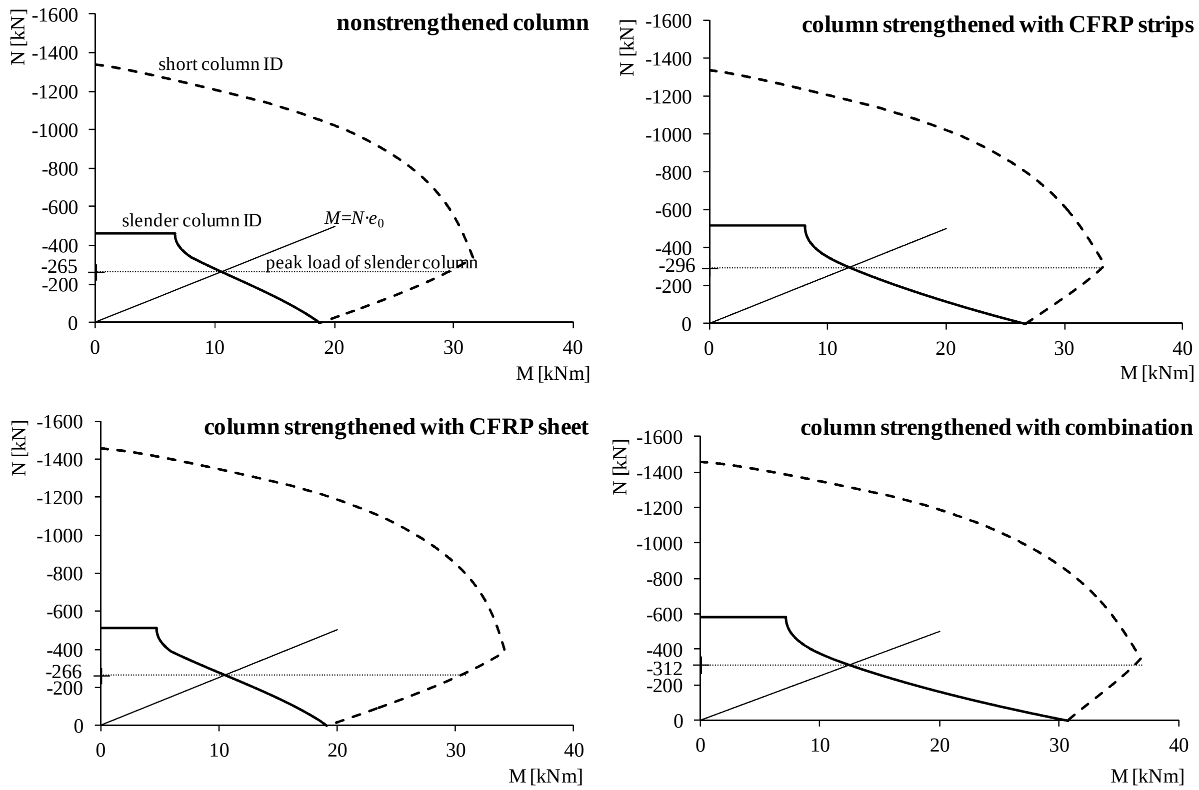

Figure 5. Theoretical analysis results.

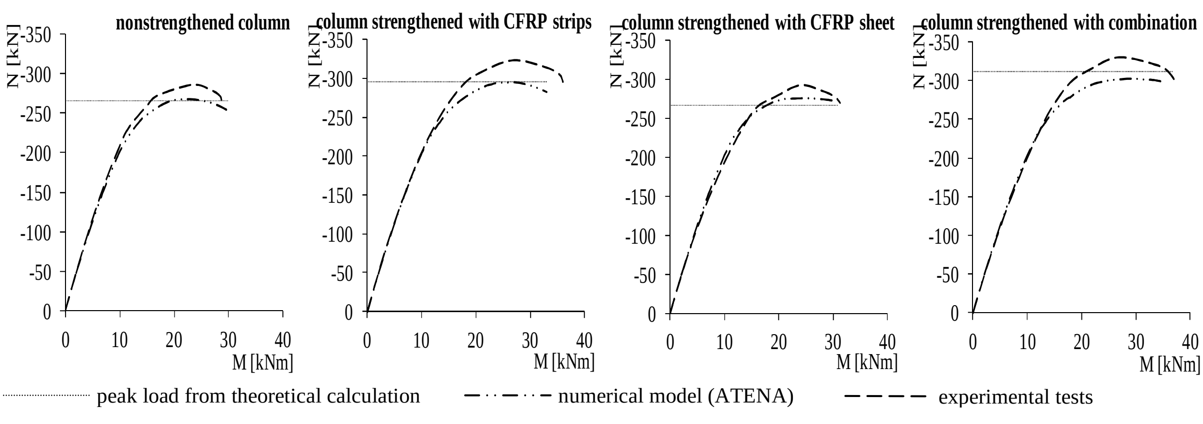

Figure 6. Comparison of all three investigation methods.

4. Conclusions

The most effective strengthening technique for the slender columns strength enhancement results from all three investigation methods equally – it is a near surface mounted CFRP strips form. CFRP strips as additional reinforcement increase the interaction diagram in the predominant bending part (low value of compression force, bigger bending moments) and especially slender columns fail in this part of their loading. For design of NSM CFRP strips strengthening it is recommended to calculate with additional reinforcement with right material properties and take care for initial strains in unloaded structure. The material properties, especially the tensile strength, should not be the same like from the tensile test. CFRP strips are not stressed in pure tension, because there is a big curvature in the most stressed cross-section and ultimate strain of CFRP strip has to be reduced.

The use of CFRP sheet for concrete columns confinement is effective for short concrete columns – it is a result of type of stress. CFRP sheet confinement increases the concrete strength in the triaxial state of stress and the interaction diagram increases in predominant compression region (high value of compression force, low bending moment). Just short columns can achieve this part if interaction diagram during failure.

Good correlation between the results of theoretical assumptions, computer modelling and experimental testing confirms the possibility of short and slender columns strengthening design according to the aforementioned assumptions.

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MARGINAL SEISMIC RESISTANCE OF REINFORCED SHELL CONSTRUCTIONS


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Abstract

The objective of this article is to present a practical method for calculation of the parameter of marginal seismic resistance, which can be used to assess the slab-wall reinforced concrete constructions during the process of finishing the 3rd and 4th block of the nuclear power plant Mochovce. This parameter is used for the assessment of seismic resistance of buildings, equipment and systems in nuclear power plants. This article is a theoretical description of the assessment and a subsequent application of the theoretical solution to practical calculations and assessments of seismic resistance of reinforced concrete shell constructions.

Key words: seismic design, shell, concrete structure, European standards, Finite Element Method (FEM), HCLPF parameter

1 Introduction

Currently accepted and used practice for the design of nuclear power plants can be characterised as a deterministic approach which uses deterministic computational models.

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(The following applies for deterministic models: elements and relations among them are firmly determined, the behaviour of models under certain conditions is controlled by these terms and conditions.) Conservatism of the plan is guaranteed by controlling every input parameter as well as the individual steps in calculations of seismic analysis, the following design and construction or element details.
The following segments of the design process are an example of the typical conservatism:

- specification of design earthquake,
- modelling of interactions and parametric solutions to the interaction of the geo-body and the construction,
- defined characteristics of damping,
- balanced and widened floor spectra,
- method of combination of seismic impacts,
- design strength of materials.

The real marginal resistance of the construction element is not determined by the actual process of planning and it is obvious that this process does not provide sufficient information for an estimate of seismic risks.
Alongside the deterministic approach, a probabilistic assessment of potential risks of a seismic event is also frequently used. One of the methods, called SPRA (SPRA – seismic probabilistic risk assessment), integrates insecurity and randomness of the seismic risks, dynamic reaction of the construction, material coefficients and determines the probabilistic risks estimate. The other method, named SMA (SMA – seismic margin assessment), determines the margins of seismic load beyond the margin of the design seismic load and in this way will quantify seismic safety. Statistic description of seismic risks for construction elements is problematic, therefore the recommendation to use the CDFM method (CDFM – conservative deterministic failure method) was accepted. This method is understandable from an engineering point of view and has its theoretical background in the framework of the current deterministic approach to the design of building constructions.
Determination of the value specifying the achieved marginal seismic resistance, the so-called HCLPF parameter, is required for the assessment of the seismic resistance of buildings, equipment and systems in nuclear power plants, e.g., in completion of the 3rd and 4th block of Mochovce (hereinafter MO34). For the completion of Mochovce we denote this parameter $HCLPF_{MO34}$. For the purpose of building constructions it is mostly determined by "a simpler" deterministic calculation. The final value of the parameter $HCLPF_{MO34}$ (given for the ultimate way of failure) has to be at least equal to the value of PGA (peak ground acceleration) to guarantee the seismic resistance at the required level. If this condition is implemented, the assessed component or construction element will be satisfactory from the point of view of strength. In this contribution we submit a solution which was developed for slab-wall reinforced concrete systems consisting of slabs, walls, central cores where we model the reinforced concrete construction elements as 3D Shell elements in computational MKP models. The theoretical solution has been developed on the basis of theoretical recommendations [3] and [4]. The solution has been programmed in an EXCEL spreadsheet. Quantities illustrated in figures 3, 4 have been exported from the dynamic calculation, in our case from VS STRAP [2], in the format of a "delimited" file.

2 Definition of parameter $HCLPF_{MO34}$ (High confidence of low probability of failure)

The first step to calculation of $HCLPF_{MO34}$ is the calculation of safety coefficient F_s, which corresponds to the multiple of the resultant seismic reaction determined for a value of PGA (peak ground acceleration), in which the decisive acceptance criterion used for the assessment of the given component or resistance of the construction element will be depleted.

For F_S it holds that:

$$F_S = \frac{C - R_{NS}}{R_S} \tag{1}$$

Where

C capacity of the component of construction element (e.g., resistance, permissible tension, permissible reshaping, etc.)

R_{NS} resultant reaction to the non-seismic load (shown in the same format as C – capacity of the component, element)

R_S resultant reaction to seismic load defined by the value of PGA (peak ground acceleration, shown in g (ms^{-2})).

F_S safety coefficient

For the calculation of parameters of marginal seismic resistance *HCLPF* the following relation holds:

$$HCLPF_{MO34} = F_S \,.\, PGA\ (g) \tag{2}$$

The resultant value of parameter $HCLPF_{MO34}$ (determined for the decisive process of damaging) must be at least equal to the value of PGA to guarantee the seismic resistance at the required level. If this condition is satisfied, the assessed construction will be acceptable from the point of view of strength.

Thus, it is apparent that the parameter $HCLPF_{MO34}$ characterises the assessed construction as a whole from the point of view of its weakest component and represents the maximum possible value of PGA at the same level of dead and live load. In case we wanted to set the given construction in some other place (geographically or within the site), for which the design value PGA was different, from the parameter $HCLPF_{MO34}$ it is possible to state whether the construction is acceptable from the point of view of strength even without further static analysis.

3 Solutions for concrete shell components in compliance with STN EN 1992-2

The attachment *MM* deals with the theoretical solution of laminated concrete sandwich components. Its recommended use is for the revision of the walls of box bridge girders in

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the parts where the simplified beam analogues cannot be used. Shell elements generally consist of 8 components of inner forces (fig. 1)

- 3 elements in the level n_{Edx}, n_{Edy}, $n_{Edxy} = n_{Edyx}$
- 3 slab elements m_{Edx}, m_{Edy}, $m_{\mathrm{Edxy}} = m_{\mathrm{Edyx}}$
- 2 cross shear forces v_{Edx}, v_{Edy}

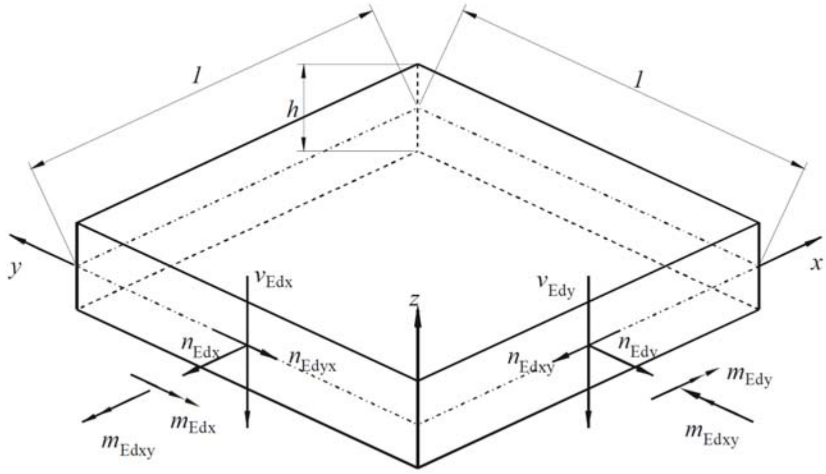

Fig. 1 Shell component

Three layers are defined in the sandwich model (fig.2): two external layers transfer membrane effects formed by n_{Edx}, n_{Edy}, n_{Edxy}, m_{Edx}, m_{Edy}, m_{Edxy}; and the inner layer transfers shear forces v_{Edx}, v_{Edy}. The thickness of individual layers can be determined by iterative method.

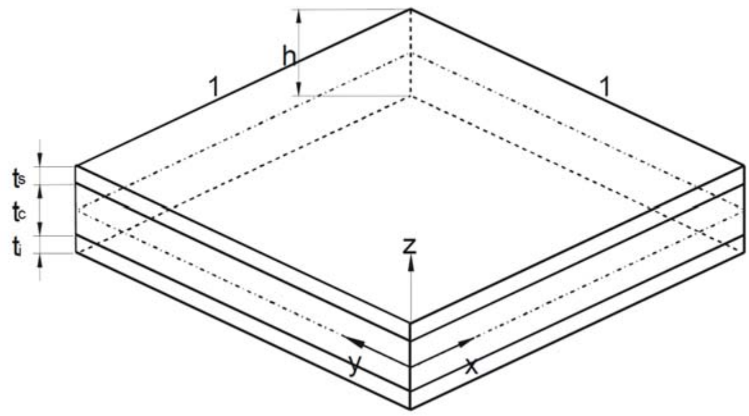

Fig. 2 Sandwich model

6 elements of inner forces n_{Edx}, n_{Edy}, n_{Edxy}, m_{Edx}, m_{Edy}, m_{Edxy} are further transferred to membrane stresses in individual external layers (fig. 3) by means of particular relations of the technical theory of elasticity.

Furthermore it is possible to dimension the individual layers of shell model as separate two-dimensional membrane elements by combining inner forces determined by linear analysis of the finite element method. The membrane elements are stressed only in the level of forces namely σ_{Edx}, σ_{Edy}, τ_{Edxy} (fig.4). Actual assessment of membrane elements has been considered in Article 6.109 [4].

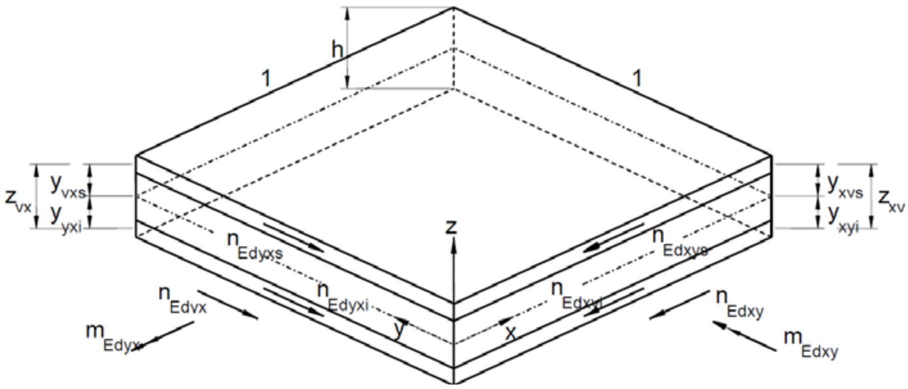

Fig. 3 The effects of membrane shear forces and moment of torsion in the external layer

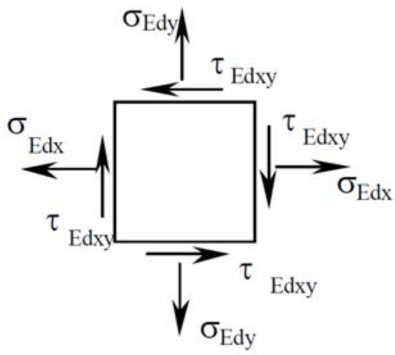

Fig. 4. Membrane element

4 Practical calculation of marginal seismic resistance of reinforced concrete shell components

The objective of this task is to find the marginal seismic resistance of reinforced concrete structures on the boundary of failure of their weakest member. Thus we are searching for a multiplier (safety level F_S) of linear seismic combination, the application of which with constant/dead load will cause the failure of construction in at least one finite element. The weakest member of the construction is considered to be the finite element with the lowest safety level F_S, to which the value $HCLPF_{MO34}$, characterizing the whole construction from the point of view of seismic resistance, corresponds when the element is multiplied by basic acceleration. Considering the non-linear character of the task and complexity of the whole algorithm for the assessment of one shell component (in compliance with STN EN 1992-2), the analytical solution of the task would be very complicated and hard to monitor, which is apparent from individual functionalities in fig. 5 and 6.

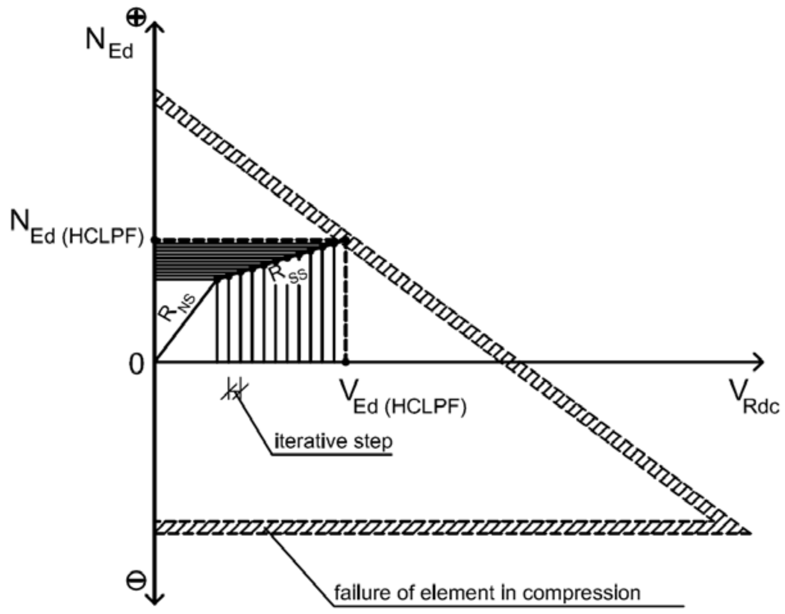

Fig. 5 Dependence of shear resistance of a member on normal force magnitude

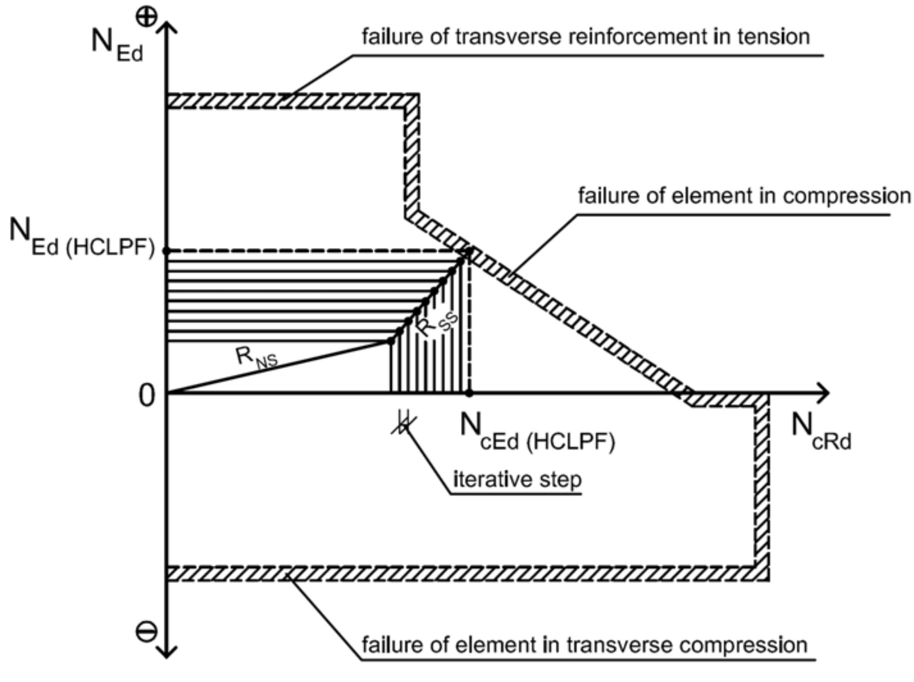

Fig. 6 Dependence of compression resistance of a member on normal force magnitude in cross section

The task can also be solved iteratively and considering the fact that finite element load is imported from MKP VS, the solution in Excel seems to be the most practical and fastest. The calculation proceeds automatically in individual stages, assessing elements up to the moment when the safety level of all finite elements has been found and the lowest one constitutes the seismic resistance. The accuracy of the solution is determined by the area of marginal resistance (hatched areas in fig. 5 and 6) and the magnitude of the iterative step. The area of marginal resistance means the interval of utilization of the construction (e.g., 95 – 100%), when the iterative calculation for the given element is completed. Higher accuracy of the calculation is naturally more time-consuming. For several thousand finite elements the calculation can take a few minutes.

Using the described method, the assessment proceeds fast and efficiently. After exporting a set of inner forces as shown in the example of fig. 7 (especially R_{NS} – constantly and separately R_S – 24 seismic combinations) and assigning input parameters (thickness of a member, reinforcement and material characteristics) the calculation resulting with the value $HCLPF_{MO34}$ (fig..8). can start.

Rns							
n_{Edx} [kN/m]	n_{edy} [kN/m]	n_{edxy} [kNm/m]	m_{edx} [kNm/m]	m_{edy} [kNm/m]	m_{edxy} [kNM/m]	v_{Edx} [kN/m]	v_{edy} [kN/m]
3.78	4.31	2.46	0.83	0.77	0.87	-0.53	-0.69
5.38	2.05	-1.39	-0.82	1.18	0.27	0.55	-7.32
7.86	5	-6.88	-0.01	1.04	-0.83	-1.48	-5.08
10.3	10.75	-11.28	0.76	0.62	-0.71	0.79	2.33

Fig. 7 An example of exporting inner forces on individual finite elements (STRAP=>Excel)

Globálne vstupné veličiny / Global inputs								Nastav. Riešiča / Solver sett.							Vyhod. / Evaluation		
hrúbka prvku /element thickness		char. pev. betónu v tl / concr. strenght in pressure	char. medza klzu výstuže char. yeld stress	krytie betónu-povrch 1/cover-surface 1	krytie betónu-povrch 2/cover-surface 2	parc. súč. spolah. pre betón / p. coef. of rel. for concr.	parc. súč. spolah. pre ocel / p. coef. of rel.. for steel	dolná hranica intervalu / low border of solving	horná hranica intervalu / high border of solving	max. hodn. stupňa bezpeč. / max. value of safety factor	iteračný krok / iteration step	pomocná hodnota / advisory value	maximálne využitie elementu / maximal utilization of element	kompletn. riešenia / inclus. Of compute	peak ground acceler.	stupeň bezpečnosti / safety factor	hight confidence in low probability of failure
h [mm]		f_{ck} [MPa]	f_{yk} [MPa]	c_1 [mm]	c_2 [mm]	γ_c [-]	γ_s [-]	I_1 [%]	I_2 [%]	F_{smax} [mm]	k [mm]	H_o [-]	μ_{s_max} [%]	U [n]	PGA [g]	F_{smin} [-]	HCLPF [g]
200		35.0	500.0	25	25	1.5	1.15	99	100	100	0.01	0	99.093	281	0.15	1.3205	0.1981

Označenie / Appelation		Lokálne vstupné veličiny / Locall inputs								Výsledky / Results									
		povrch 1 / surface 1				povrch 2 / surface 2				povrch 1 / surface 1				povrch 2 / surface 2				p1,2/ s1,2	
č. ploš. prvku / number of planar member	číslo koneč. prvku / n. of fin. element	priemer výstuže /perimeter of armature	osová vzd. prútov / axial distance betw. bars	priemer výstuže /perimeter of armature	osová vzd. prútov / axial distance betw. bars	priemer výstuže /perimeter of armature	osová vzd. prútov / axial distance betw. bars	priemer výstuže /perimeter of armature	osová vzd. prútov / axial distance betw. bars	využ. ocele v smere x / capacity utilitation of steel in dir.x	využ. ocele v smere y / capacity utilitation of steel in dir.y	využ. bet. v tlaku / cap.utiliz. of concrete in pressure	využ. z.b. prvku v tl. / cap.utiliz. of r.c. elem. in pressure	využ. ocele v smere x / capacity utilitation of steel in dir.x	využ. ocele v smere y / capacity utilitation of steel in dir.y	využ. bet. v tlaku / cap.utiliz. of concrete in pressure	využ. z.b. prvku v tl. / cap.utiliz. of r.c. elem. in pressure	využ. z.b. prvku v šmyku / cap.utiliz. of r.c. el. in shear	bezpečnostný koeficient / safety factor
Č.p.p [-]	Č.k.p [-]	ϕ_{x1} [mm]	s_{x1} [mm]	ϕ_{y1} [mm]	s_{y1} [mm]	ϕ_{x2} [mm]	s_{x2} [mm]	ϕ_{y2} [mm]	s_{y2} [mm]	μ_{s_x1} [%]	μ_{s_y1} [%]	μ_{c1} [%]	μ_{1_comp} [%]	μ_{s_x2} [%]	μ_{s_y2} [%]	μ_{c2} [%]	μ_{2_comp} [%]	μ_{shear} [%]	F_s [-]
11963	1	12	200	12	200	14	200	14	200	99.03	97.3	0	0	56.2	70	2.8717	0	10.20	4.9134
11964	1	12	200	12	200	14	200	14	200	90.71	99.0	0	0	77.9	71	0	0	14.08	4.4234
11965	1	12	200	12	200	14	200	14	200	99.03	91.9	0	0	70.9	54	0	0	11.44	4.3444
11966	1	12	200	12	200	14	200	14	200	99.04	54.7	0	0	42.3	41	0	0	27.19	3.3184

Fig. 8 Calculation of parameter $HCLPF_{MO34}$ in Excel program

5 Conclusion

The calculation of marginal seismic resistance of reinforced concrete shell constructions is a difficult task, especially in comparison to the calculation of parameter $HCLPF_{MO34\ for}$ steel rod constructions with which we have extensive experience. As this parameter has been used for the assessment of seismic safety of nuclear plants and its structural members a responsible approach is imperative. The use of the system of calculation is universal; import of data is possible from any MKP VS which supports the export of the results in the format of "delimited files".

6 Acknowledgement

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Use of Radioactive Reinforcement in the Construction of Concrete Bridges

Peter Paulík, Michal Pánik, Vladimír Nečas

ABSTRACT:

Nowadays, many nuclear power plants are approaching 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, contains radionuclides with relatively short half-life. Disposal of all this kind of steel in specialized repositories would require considerable financial investments. Therefore, re-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 and the risks related to this issue.

Keywords: radioactive reinforcement; concrete bridges, decommissioning, very low level radioactive waste,

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INTRODUCTION

Nuclear industry is nowadays facing new challenges connected with decommissioning and dismantling of shut-down facilities and power plants. Management of various kinds of waste produced during these activities becomes an issue that needs effective solution. Substantial part of this waste comprises solid metallic materials with different levels of radioactivity [1]. Radioactivity of these metals originates in contamination or activation process. Contamination is caused by the contact of metallic materials with active media (fluid that contains radionuclides). Activation can be described as the direct impact of radiation after which metals become sources of radiation.

Considerable amount of the radioactive metals consists of steel with very low level of radioactivity that just slightly exceeds the legislation limit for unconditional release into the environment (an example is the limit of specific activity 300 Bq/kg for ^{60}Co, valid in the Slovak legislation [2]). Standard practice in this case is to classify the material as radioactive waste that has to be treated, conditioned and disposed in the respective type of radioactive waste repository. This process requires significant, not only financial resources.

Another option that combines saving of financial funds and valuable raw material is to use the concept of conditional release of materials and their utilization for a specific industrial purpose. Concentration of radionuclides in considered very low level radioactive steel exceed the legislative limits [2] but it is clearly proven that the radiation impact of the proposed specific utilization on the workers and population will be negligible.

The general basic principles of conditional release are as follows:

a) Conditionally released material comprises solid radioactive materials (metals, concrete…).
b) Solely materials that contain radionuclides with a relatively short half-life are considered.
c) Long-term fixation of the released materials at one place is considered. This allows the sufficient decrease of radioactivity due to the natural radioactive decay.
d) International recommendations that define the limits of the annual individual effective dose are applied.
e) Requirements of the Slovak Statutory Order 345/2006 Coll. (based on EU Directive 96/29/EURATOM) are obeyed [3].

Processes that precede the utilization of conditionally released very low level radioactive steel in bridge construction comprise melting of metal scrap originating mostly in the decommissioning of nuclear power plants and production of ingots. Steel ingots can be processed in various reinforcement elements including reinforcement rods or steel grids. Melting of metal scrap and fabricating of simple reinforcement rods will be carried out in specialized facility [4].

RESEARCH SIGNIFICANCE

Conditional release of very low level radioactive material following with its recycling and utilization for special purpose is becoming an important topic which is receiving an increased attention from major world and international organizations working in the field of nuclear energy (IAEA, USNRC) [5-7]. Especially in the light of recent news announcing untimely shut-down of many nuclear power plants that have to be decommissioned in short time period, it will be necessary to deal with large waste material flow. Recycling of valuable material, particularly metals could save considerable financial funds, capacity of disposal facilities and last but not least the mineral resources.

RADIOACTIVE STEEL AND BRIDGE CONSTRUCTION

Reason for use of radioactive steel in bridge construction could be summarized in these four points:

1.) geographical location of bridges – many bridges are built in non-occupied territory
2.) reinforcement cover is large enough to provide adequate cover against radiation
3.) high quality concrete with low permeability provides a very good protection against release of radioactive particles into the environment before their activity falls below legislation limits
4.) long required service life of bridges significantly exceeds the requirements for natural decline of radioactivity of steel below legislation limits

Irradiation of workers

Workers building bridge with very low level radioactive reinforcement steel can receive radiation dose from gamma rays radiated by radionuclides present in the steel. Identification parameter of radioactive material is its specific mass activity in Bq/kg (Becquerel per kilogram). In a simplified way, Becquerel unit can be understood as intensity of radioactive decay or number of nuclei decayed during period of time. Irradiation or received dose in μSv (microSievert) represents amount of energy received by human body and its influence on the human body.

Expected radiation dose absorbed by workers during bridge building process does not exceed 5% of the average radiation dose absorbed from the typical natural radiation background [8]. This radiation dose is also many times lower than that absorbed by the staff of nuclear power plants.

Majority of the radiation dose is obtained by workers during reinforcement works, during manipulation with reinforcement or reinforcement cages and during moulding and concreting. After concrete covers the reinforcement it provides a partial shielding against radiation. Typical reinforcement cover of 4 cm made of ordinary concrete could decrease gamma radiation by 25% and 10 cm cover could decrease radiation by nearly 50%.

In some cases also the use of so-called mixed reinforcement cages could be chosen. These cages are made of ordinary reinforcement mixed with radioactive reinforcement. In this solution radioactive rods could be placed just before concreting and this way the duration of irradiation of workers could be significantly reduced. Prestressing tendons will be made of non-radioactive steel. Mixing of radioactive and non-radioactive metal scrap before melting is not an option because legislation recognizes this process as `dilution with aim at unconditional release` and does not allow it.

In our study, as a first step, we divided the bridge into three basic parts – superstructure, piers and foundations. These parts were at first examined separately and then these results were incorporated to a complex bridge construction.

The use of radioactive reinforcement in bridge construction could take place in all of bridge main parts with these recommendations:

Foundations

Deep foundations made of large-diameter piles seems to be very suitable for this purpose since their construction is very fast and a large amount of radioactive reinforcement could be incorporated in their design. Reinforcement cages of these piles are very simple with possibility of incorporating the radioactive rods just before concreting.

Piers

Simple shaped piers could be built of radioactive reinforcement very effectively. The thickness of concrete cover is still determined by the requirements of durability and must not be determined by the radioactive shielding requirements. Of course more attention should be made on quality of works to avoid the transport of radioactive particles into the environment before their activity falls below a legislation limit.

Superstructure

Most of requirements specified for piers are also valid for the superstructure but in the case of superstructure also the chosen building technology is very important.
It is expected that this kind of reinforcement would be incorporated only in long enough bridges, where a largest possible amount of radioactive reinforcement could be used. Therefore only technologies with relatively fast construction speed come into consideration. After preliminary study of construction speeds the launching technology and the technology using prefabricated I-shaped girders were chosen for detailed study [9].
Foundations and piers were then adapted to the chosen technology.
The amount of reinforcement in the bridge parts (foundations, piers and superstructure) was estimated by preliminary design and on the basis of several previous projects.
Both modeled bridges were 1650 m (5413ft.) long. Once the dimensions of bridges were designed in preliminary process, the durations of working procedures were estimated in close cooperation with construction practices. Also the number of needed workers was estimated and fictional working groups were created. It was assumed that these working groups leave the construction site after they finished their works. Subsequently, for each group the radiation dose was evaluated and the critical group has been selected. Radiation doses obtained by this critical group were then compared with the legislation limits and these results were considered as crucial in making conclusions. Description of the bridges and modeling is described in the respective chapters.

DESCRIPTION OF MODELED BRIDGES

Dimensions of foundations, piers and the superstructure are based on experience and they are resulting from the preliminary structural design.

<u>Prefabricated alternative with I-shaped girders</u>

This first alternative is a typical bridge made of longitudinal prestressed prefabricated I-shaped girders of 1.4 m (4.6 ft.) height with a typical field span of 31 meters (102 ft.). These prefabricated girders are placed during construction process on prefabricated crossbeam which is placed on a pair of circle-shaped piers of 1.6 m (5.2 ft.) diameter. Once the girders are placed into the designed position the completion of the upper deck takes place.
It has been assumed that the production of prefabricated girders is located in a separate production hall next to which an interim storage of these completed girders is located. Also the interim storage of radioactive steel near the hall has been considered.

Deep foundations for this alternative are consisting of large diameter piles of an average length of 14 meters (45.9 ft.) and with a diameter of 1.2 m (3.9 ft.). Foundation slab is 1.5 meters(4.9 ft.) thick. Scheme of the bridge piers with superstructure and its model in VISIPLAN 3D ALARA is shown on **Fig. 1.**

<u>Incremental launching alternative</u>

Next to be evaluated for the bridge building technology was the incremental launching technology. The superstructure of the bridge was assumed to be constructed at so-called stationary formwork situated behind the abutment from where the segments were gradually moved over the valley. This technology also required modeling the bearings replacement (from sliding bearings to permanent bearings) after the launching process has been finished.

Piers and the foundations were adapted to this technology. Deep foundations remained the same as in the first alternative. Foundation slab is 2.0 meters (6.6 ft.) thick as well as the pier which has nearly a rectangular shape. After completion the bridge the finishing works on bridge superstructure, as well as on the first alternative, were assessed. Scheme of the bridge piers with superstructure and its model in VISIPLAN 3D ALARA is shown on **Fig. 2.**

APPLIED COMPUTATIONAL TOOLS

The exposure of workers that work with very low level radioactive steel can be calculated using appropriate computational tool, e.g., VISIPLAN 3D ALARA [10], or MICROSHIELD[11] both simulating the external gamma irradiation and GOLDSIM [12] calculating the internal exposure. The VISIPLAN software has been chosen for the calculations connected with respective working procedures during the construction of the bridge because this tool enables simulation of complex working environments. VISIPLAN 3D ALARA software allows the calculation of the absorbed individual effective dose in complex environments. It enables a creation of the simplified geometry model of working environment based on the technical documentation. Radiation sources, identified by the measurement, are placed in this geometry. Dimensions, material and radionuclide composition of radiation sources are also taken into account. After creating geometry with radiation sources it is possible to create a grid of points in which the program calculates dose parameters. The output in this case is a dose map that can be displayed as a colored field or contours connecting the places with the same dose. A useful option of the program is a creation of “trajectory”. Trajectories describe the movement of the person in an environment with radiation sources. They consist of several points. Each point contains the information about the movement of the person in the environment, the duration and the type of its activity. The calculated trajectory contains a record of the dose absorbed by the person summarily or individually at each trajectory point and also allows the recognition of the contribution of individual radiation sources to the resulting dose [3].
Results from simulations implemented in GOLDSIM software comprise irradiation of humans caused by intake of radionuclides from the environment (inhalation, ingestion ...). This tool is therefore helps to estimate long-term effects of contaminants released in the environment.
Outputs of the computational tool VISIPLAN are important for the assessment of the doses received by personnel during the construction of the bridge. Thus attention will be paid on the results of calculations executed in this program. The VISIPLAN software has been verified and validated, so its results can be considered accurate and reliable.

DESCRIPTION OF BRIDGE MODELS

Two typical bridge constructions were selected as the alternatives for software modeling. Both bridge types fulfill all above stated requirements for utilization of very low level radioactive steel and they are described in Sections 3.1 and 3.2 in detail.

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

- Bridge foundation
- Piers and abutments
- Bridge superstructure

Separation of the bridge into these parts is useful for the assessment of radiation impact on workers that can be implemented for each bridge part separately. Simplified radiation source containing only ^{60}Co radionuclide was chosen for the software models. This radionuclide remains fixed in the steel during the melting process, and furthermore it is one of the most common contaminants in nuclear power plants.
Working procedures of building of both bridge model alternatives were transformed into trajectories in VISIPLAN software after the cooperation with the construction practice. Radiation exposure of workers was summarized in results and confronted with the legislation limits. Limits and conditions along with the possibilities of utilization of very low level radioactive steel were stated in the **Table 1** and **Table 2** in respective section. **Fig. 3** shows visualization of chosen working procedures simulated in VISIPLAN software.

RESULTS OF CALCULATIONS

Prefabricated alternative with I-Shaped girders

In general, all the three main parts of the prefabricated bridge are suitable for incorporation of very low level radioactive reinforcement. However, some components have to be produced with few restrictions, e.g. slightly modified design of reinforcement or change of working procedures. These changes ensure decrease of radiation dose of workers to the lowest achievable level which never exceeds the legislation exposure limits [2]. Slovak legislation specifies the exposure limit at the level of 50 µSv/year. Here it is suitable to remind that natural background radiation typically reaches a level of about 3 mSv/year or even more (to say this in another words, the radiation dose received from the natural background is about 60 times greater than that received from working with this radioactive steel).

Results of calculations, amount of very low level radioactive reinforcement that could be used for building of respective bridge part along with the approximate calculation of saved financial resources are shown in **Table 1**. In the first column of **Table 1**, calculated radiation dose received by workers is compared with the legislation dose limit. Ratio between calculated dose and the dose limit is stated for each part of the bridge. The percentage that is lower than 100% means that theoretical calculated dose received by workers is lower than legislation limit. There is direct relationship between radioactivity (in Bq/kg) and received dose rate (in μSv/year). Having this in mind it is possible to say that if the percentage stated in first column is lower than 100% (expected dose of workers is lower than limit), complete reinforcement of respective part of the bridge can be radioactive.
Last column of **Table 1** shows saved financial resources in case of maximum possible utilization of very low level radioactive reinforcement. Savings reach 90% of standard price of reinforcement because it is assumed that price of very low level radioactive reinforcement will not exceed 10% of price of regular steel reinforcement. This assumption was made based on calculation executed in computational tool OMEGA [13] (developed by Slovak company Decom, Inc.) which calculates price of melting of radioactive metal scrap and production of reinforcing components – these processes represent costs of nuclear operator. On the other side, operator`s savings is the cost of treatment and disposal of radioactive metal as radioactive waste. Amounts of nuclear operator`s costs and savings are approximately the same and the final price of very low level radioactive reinforcement steel on the level of 10% of price of regular steel reinforcement can be considered as profit or uncertainties in calculation.
The results of calculations show that very low level radioactive reinforcement steel containing ^{60}Co radiation source with specific activity 300 Bq/kg could be utilized for the complete construction of the prefabricated bridge with I-Shaped girders. Level of specific activity of the radiation source could even be increased along with keeping the legislation limits. This fact is especially welcome in nuclear industry.

Incremental launching alternative

Calculations connected with the bridge built by using incremental launching technology were made applying the same logic as in case of prefabricated bridge alternative. Results of calculations, amount of very low level radioactive reinforcement that could be used for building of respective bridge part along with the approximate calculation of saved financial resources are stated in **Table 2**. If the value in the first column of the table is 100% then the calculated dose is equal or higher than the dose limit. Using the same logic as in the case of prefabricated alternative, portion value equal to 100% prevents building of whole bridge utilizing very low level radioactive reinforcement. Applicable amount of very low level radioactive reinforcement is decreased proportionally to the calculated dose. In this case the reinforcement of bridge has to be made of combination of common and very low level radioactive steel, thus the share of this kind of steel must be decreases. Decreased amount of very low level radioactive steel based on the calculation results is stated in third column of the **Table 2.**

According to data in the **Table 2** it is possible to say that the part of this bridge alternative that is most suitable for the utilization of very low level radioactive reinforcement steel is the foundation of the bridge, especially piles. For other bridge parts very low level radioactive reinforcement can be used only in limited quantities.

DISCUSSION AND CONCLUSION

The object of this paper was to present the possibilities of the utilization of very low level radioactive reinforcement steel in the bridge building process. Two models of common widespread bridges 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. Long-term impact of utilizing very low level radioactive reinforcement of bridges on the environment and people will be the subject of further study. Based on the preliminary outputs of long-term impact simulations is possible to consider this effect as insignificant.
Results of calculations stated in the paper show that recycling and reuse of very low level radioactive steel in structural engineering is possible and it could be the way how to work more efficiently through the cooperation between nuclear and civil engineering industry. An example of benefit resulting from this issue is saving the

investments. It is expected that the price of very low level radioactive steel will not exceed 10% of the price of common reinforcing steel what is clearly profitable for construction industry.
Very low level radioactive steel could be used in a number of building applications. The alternative of utilizing this material as part of reinforcement of concrete bridges was described and presented in the paper.

ACKNOWLEDGEMENTS

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TABLES AND FIGURES

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Table 1 Portion of calculated radiation doses of legislation limit, amount of very low level radioactive reinforcement and resulting approximate saved financial resources for respective bridge parts – technology with prefabricated I-shaped girders

Bridge part	Portion of calculated dose received by workers of legislation dose limit* [%]	Amount of reinforcement in the bridge structure [ton]	Amount of very low level radioactive reinforcement that could be used [ton]	Saved financial resources** [€]
Piles	17 %	500	500	360 000
Piers	77 %	620	620	446 400
Superstructure	67 %	2450	2450	1 764 000

Table 2 Portion of calculated radiation doses of legislation limit, amount of very low level radioactive reinforcement and resulting approximate saved financial resources for respective bridge parts – incremental launching technology

Bridge part	Portion of calculated dose received by workers of legislation dose limit* [%]	Amount of reinforcement in the bridge structure [ton]	Amount of very low level radioactive reinforcement that could be used [ton]	Saved financial resources** [€]
Piles	17 %	500	500	360 000
Piers	100 %	1500	200	144 000
Superstructure	100 %	2250	730	525 600

* value is valid for the ^{60}Co radiation source with specific activity 300 Bq/kg (legislation limit for the unconditional release into the environment)
** calculation was based on the assumption that price of very low level radioactive reinforcement will not exceed 10% of the price of common reinforcement steel (ton of common reinforcement steel=800€)

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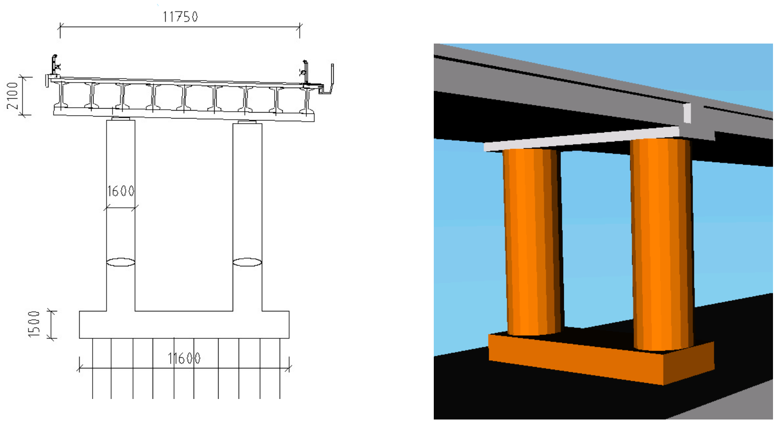

Fig. 1 - Scheme of the bridge piers and its software model in VISIPLAN 3D – alt. 1

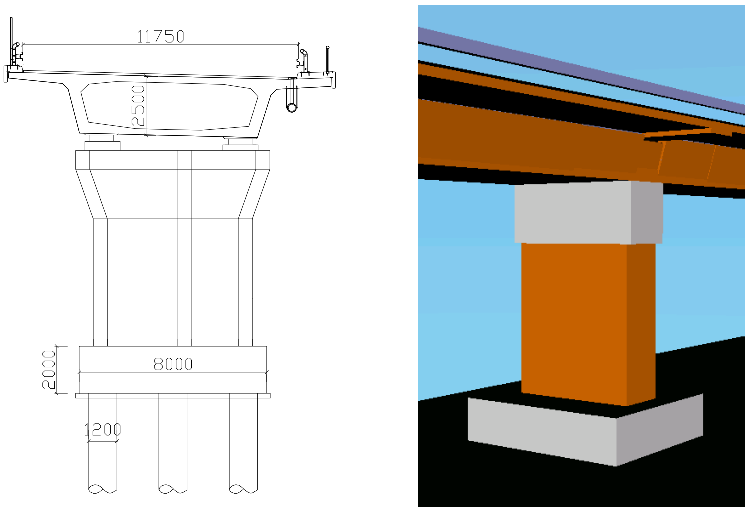

Fig. 2 - Scheme of the bridge pier and its software model in VISIPLAN 3D – alt. 2

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Analysis of composite steel-concrete columns

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Abstract

This paper presents some results of the analysis of the effects of the second order theory. According to the results of the experiments, which were made on the Department of Concrete Structures and Bridges in Bratislava we analyzed the effects of the second order theory. For the theoretical analysis of composite steel concrete columns was made a computational program. It was supplemented by a method for the calculation of the second order moment according to the model of Hanswille and Bergmann. The results of the experiments made by S. Matiaško were compared with theoretical results from the calculation based on non-linear software Atena.

Keywords: composite steel-concrete, second order theory, non-linear analysis

1. 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 standard for the design of composite columns gives us a simplified relationship according to what it can be calculated. This relationship can be applied only for columns made from concrete strength grade up to C50/60. This is the boarderline between high-strength concrete (HSC) and normal concrete. 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 Hanswille and Bergmann (general method). The main advantages of this method is that it can be applied in case if HSC is used. In this chapter we focused on the comparison of the results calculated according to STN EN 1994-1-1 and according to the general method. The general method according to Hanswille and Bergmann is described in reference [4].

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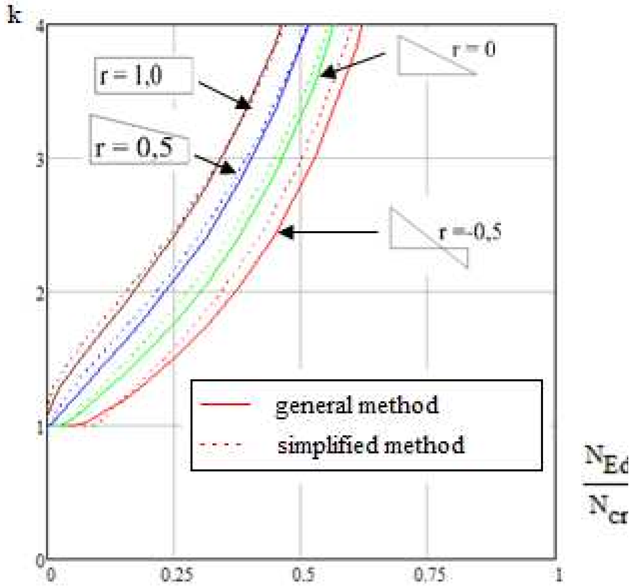
The general method and the simplified method according to the European standard for the design of composite columns was compared by the help of factor "k". In case of slender columns the factor "k" is used to calculate the second order moment by multiplying the first order moment with the factor "k".

$$k = M_{max,II} / M_{max,I} \quad (1)$$

The second order moments where calculated for fully encased composite cross-section (Fig. 1b). Ratio N_{Ed}/N_{cr} depends on the columns buckling length. Results of the comparison are presented in Fig.1a. The results of the functions according to the general method show a good agreement with the results calculated according to the simplified method. We can allege that the simplified method for the second order effects according to the European standards for the design of the composite columns shows a result on the side of safety in comparison with the general method according to Hanswille and Bergmann.



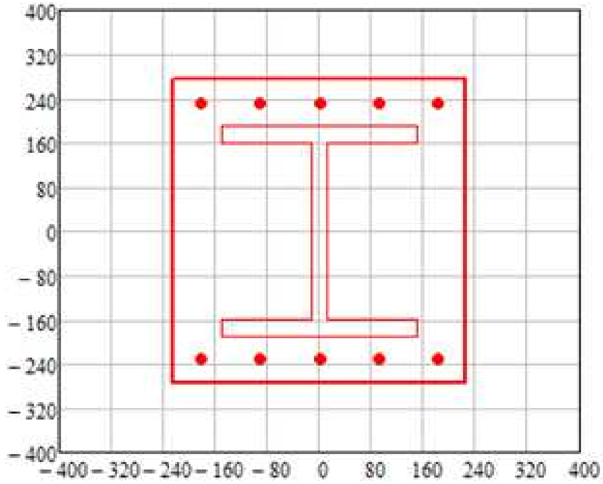


Fig. 1. (a) Comparison of the factor "k" ; (b) Composite cross-section (Concrete C20/25, Steel: S235, Reinforcement B500 B – 10Φ20. Normal force N_{Ed}=5MN

2. Experimental analysis of composite steel-concrete columns

On the Department of Concrete Structures and Bridges in Bratislava in the year 2006 was made a dissertation work with name of Navrhovanie spriahnutých oceľobetónových stĺpov (Design of composite steel-concrete columns), the author was Ing. Pavol Valach. The main objectives of the work were: analysis of simplified method, general method and comparison and the approval of the method of determination of the resistance of the cross-section, also analysis the effects of the second order theory.

Experimentally where tested partially encases composite steel-concrete columns from steel cross-section HEA 280, reinforced by longitudinal reinforcement 4ΦR16. Globally where tested 12 columns in two series by the length of 3m and 4 meters.
From the experiments author ensured the following conclusions:

- Measured initial imperfection of the columns were smaller than advised in the European standards
- The general method provides a smaller value of resistance than the simplified method for the measured material properties in centric compression, particularly for the parabolic-rectangle stress and strain diagram, because the structural steel and reinforcing steel is not fully exploited (the strain of the concrete in compression is limited).
- Measured value of resistance are corresponding with the calculated value of resistance according to the simplified method, using measured material characteristics
- Experiments confirmed the accuracy of the relationship for the factor "k" (in case we observance the rules according to European standards, concrete from strength grade from C20/25 to C50/60, steel strength grade from S235 to S460).

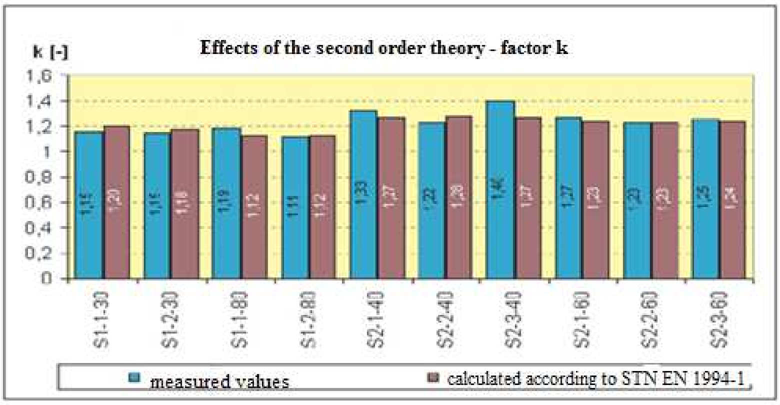


Fig. 2.Graphical comparison of the measured and the calculated values of the factor "k"

The next dissertation work about the design of composite columns made on the Department of Concrete Structures and Bridges in Bratislava was made by Ing. Slavomír Matiaško in 2010 with the name of Navrhovanie spriahnutých oceľobetónových stĺpov s použitím vysokopevnostného betónu (Design of the composite steel-concrete columns with the use of high strength concrete). The main objective of the experiments was to analyze the effects of the second order theory on slender columns using HSC. The tested columns were partially encased composite concrete columns from steel cross section HEA 200. They were reinforced by longitudinal reinforcement 4ΦR14 and from high strength concrete strength grade C60/70. Columns had length 3 and 4 meters.

From the experiments author ensured the following conclusions:

- The value of the second order bending moment was higher that it was calculated according to European standards
- Calculated value of the factor "k" according to design material characteristics where in the average 15% smaller than exactly measured values from the experiment. According to the European standard STN EN 1994-1, we can use simplified method to calculate the second order moments only in cases if normal concrete is used. In the experiment it was used HSC. The author advised to modify the factor "k" in cases if HSC is used.

$$\frac{k}{\beta} = \frac{1}{1-\left(\frac{N}{N_{cr}}\right)^{0,75}} \qquad (2)$$

Concrete used in the experiment was classed into strength grade C60/75, what means that was used high strength concrete. The conclusion of the author where based only on a small number of experiments (globally

6), and only in one type of HSC (C60/75) so his relationship have to be tested in several other examples. Experimental test of such many specimens would be a big challenge also financially such as time-consumingly. That is the reason we decided to do it in a software Atena 3D.

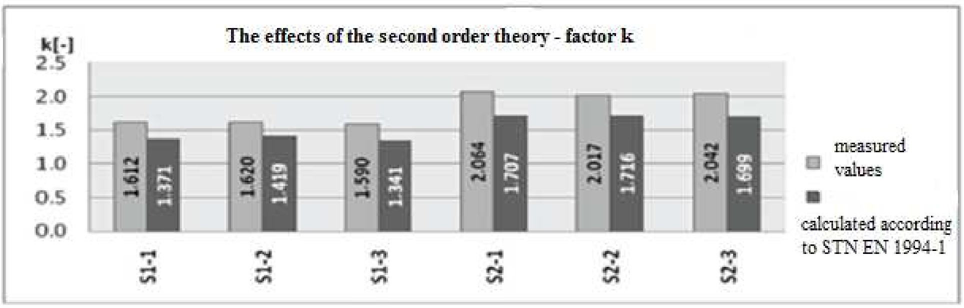
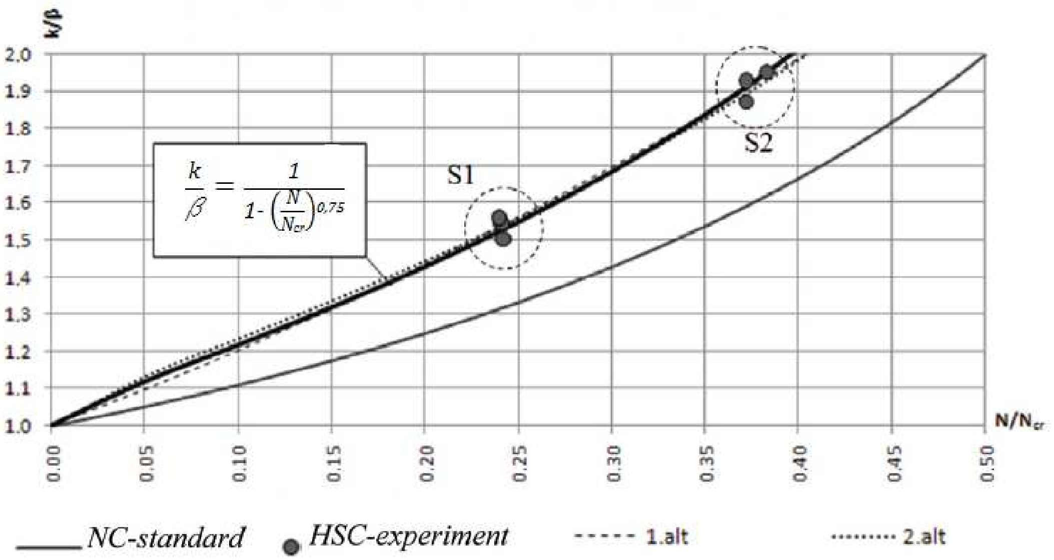


Fig.3. Graphical comparison of the measured and the calculated values of the factor “k”



$$\frac{k}{\beta} = \frac{1}{1-\left(\frac{N}{N_{cr}}\right)^{0,75}}$$

$$\frac{k}{\beta} = 1,5\left(\frac{N}{N_{cr}}\right)^2 + 1,87\left(\frac{N}{N_{cr}}\right) + 1$$

$$\frac{k}{\beta} = \frac{1}{1-0.934\left(\frac{N}{N_{cr}}\right)^{0.69}}$$

Fig.4. Available solution to modify the factor “k” if HSC is used

3. Analysis of composite steel-concrete columns in software Athena 3D

In computational program Athena 3D we compared the non-linear calculation with the experiment of Ing. S. Matiaško, which was characterized in the previous chapter. The main objective of the comparison is the matter that the author advised to modify the relationship for the factor “k” in cases if HSC is used based only in a small number of experiments. We want to do a bigger number of experiments in program Athena 3D so we could graphically compare the results and than to allege or disconfirm the advised relationship. The first step is to make the same model in Atena 3D and to compare it with the experiment to check if it gives us the same

results at all.
The steel cross section is from HEA 200 with dimensions 200mm x 190mm. The length of the columns is 3 meters, the column was modeled with hinged endings in both sides just as it was made in the experiment. The load of the construction was modeled with the help of load steps just as it was made in the experiment. The exact value of the resistance of the cross section was determinated from the diagram force/deformation, the maximal resistance is reached, when the deformation grows even if there is no load added.

In software Atena 3D can be each macro-element meshed separately. Concrete macro-element is meshed with mesh type – brick, other macro-element are meshed by mesh type – tetrahedral. The accuracy of the calculation depends on the global element size of the meshing. Reinforcement is meshed by the program automatically so there is no need to mesh it separately.

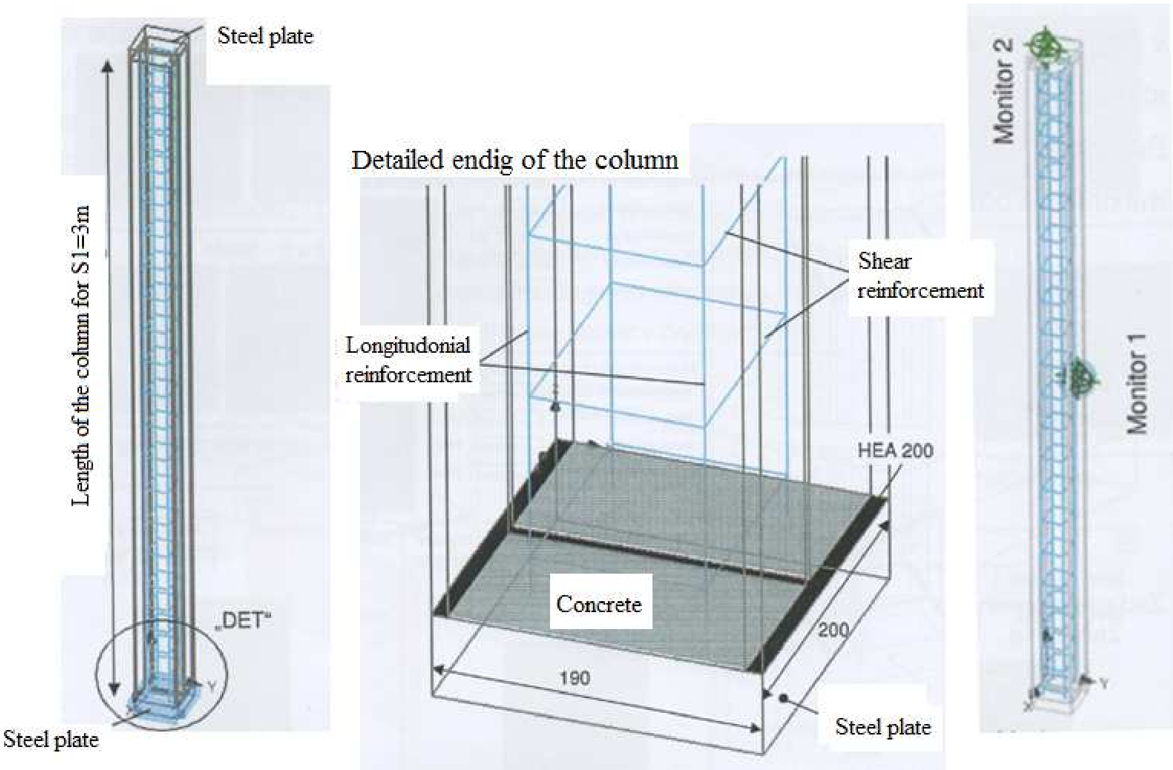


Fig. 5. Model of the columns in Athena 3D with monitor points and the detail of the ending

During the non-linear analysis it is advised to follow the change of force, deformations or stresses in each points of the columns. To follow these parameters we can use monitoring points. We used two monitoring points, the first is situated at the middle of the column to check the deformations (monitor 1), and the second is situated at the bottom of the column to check the forces (monitor 2). From these two monitoring points we generated the graph force/deformation. The results of the non-linear analysis are presented in Figure.6. The maximal resistance of the column is 2,29MN. The resistance was specified from the calculation, when the deformation started to grow without any added load. This value shows a good agreement with the results of the experiment, where it was specified average value of resistance as 2,288MN. In figure 6, we compared the graphs of the results made according to the non-linear calculation with the results from the experiments.

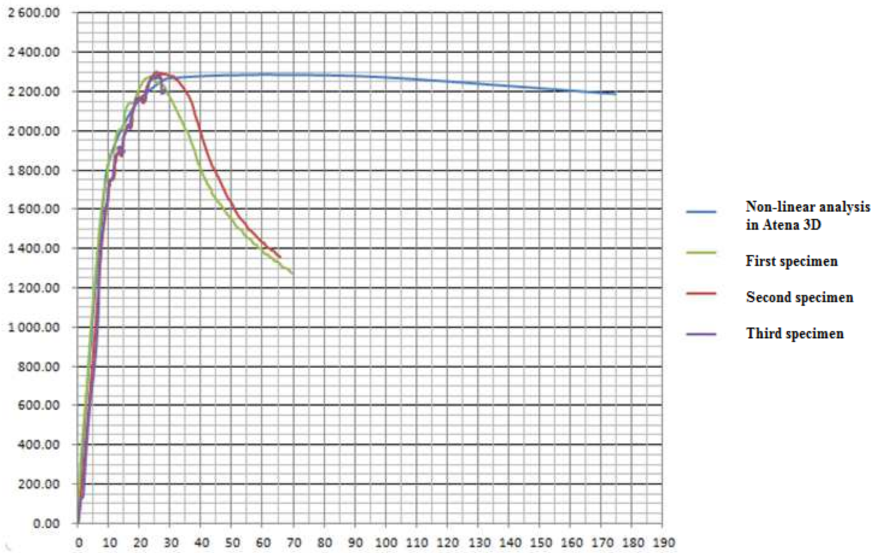


Fig.6.Comparison of the graphs force/deformation

4. Conclusions

From the experiments we can ensure the following conclusions:

- The values of resistance calculated in non-linear program Atena 3D shows a good agreement with the experimentally tested columns.
- The force/deformation graph made in non-linear program Atena 3D shows a good agreement with the experiments

The next step of the research is to make a bigger amount of calculation in program Atena 3D. We would like to make a complete parametric study of the effects of the second order theory in case if high strength concrete is used. After this study we would like to check the accuracy of the relationship of factor "k" and make conclusion for the experience in cases HSC is used in composite steel-concrete columns.

Acknowledgements

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BOND BEHAVIOUR OF PRESTRESSING UNITS COATED WITH CORROSION PROTECTION EMULSIONS


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Abstract

The use of corrosion emulsions at post-tensioning is currently very resonant issue in Slovakia and neighbouring countries. This protection measures affect many parameters that are inputs to the design of prestressed concrete bridges. In this article authors are presenting a part of results from the experimental program.

Keywords: post-tensioning, bond, emulsifiable oil

1 Introduction

Time between prestressing tendons and injection of the ducts at concrete post tensioned structures can be several weeks. In this time it is necessary to protect tendons against weathering and atmospheric humidity. Tendons are usually protected with various oil based agents, which can have influence on their bond with construction and also on friction losses during prestressing. This fact is well aware for designers and contractors and therefore experimental research on prestressing units coated with corrosion protection emulsions has been preformed with support of company Doprastav a.s..

2 Bond influence factories

Bond between prestressing unit and concrete or injection grout is realized with three primary mechanisms. There are adhesion, friction and wedging. Contribution of every one

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of these mechanisms depends from surface texture of the strand and therefore their proportion is different by different prestressing units.

Adhesion – interaction between cement stone and microscopic structure of steel surface. It contains from physical and chemical adhesion. Previous research shown better adhesion between strand and injection grout than strand and structural concrete. That means smaller aggregate better adhesion. Bond failure in adhesion comes in vary small strains. Therefore adhesion represent just a small part of total bond between strand and surrounding material. After adhesion exceed the force is transmitted by friction.

Friction – coefficient of friction based on previous research is between 0,3 and 0,6. Exact value of friction coefficient and also influence is still object of scientific works. Another significant friction factor is so called Hoyer effect. During prestressing strand diameter is getting smaller. After releasing strand from jack the strand end is getting to his original diameter. That causes wedge where the stress is acting perpendicular to the wedge surface.

Another interesting phenomenon by friction is that the strand wires have tendency to straighten themselves. After strand built-in to the concrete structure is this phenomenon prevented. That initiates torsion moment in the strand which is in equilibrium with contact stress on his surface. But this stress contribution is small because total torsion stiffness of the strand is small.

Wedging – aggregate grains are wedging on the strand spiral surface. This effect is not so significant as in case of classic reinforcement but it is not possible to ignore it. Figure 1 represents influence of strand spiral finishing on the bond stress. From the figure is evident that after breakthrough the adhesion and friction wire bond is getting rapidly down while strand bond is growing.

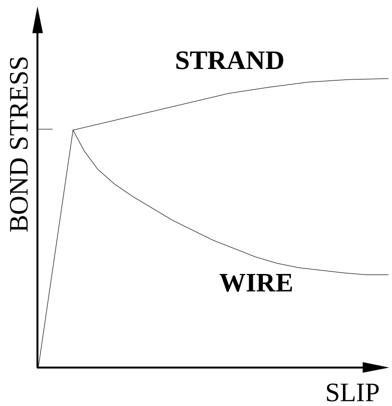

Fig. 1 Bond behaviour for 7-wire strand and plain wire

3 Bond experimental measuring

3.1 Experiment description

One of the methods of bond measuring is pullout test, where the unstressed strand is pulling out from the concrete specimen. In the first phase of experimental program was performed measuring of the bond on 21 specimens. Specimens were cylindrical shape with length 600 mm and diameter 165 mm made from concrete and injection grout placed to the plastic tubes. In their centre were placed strand ϕ15,7 (0,62“) with characteristic strength 1860 MPa. Strand was in some cases coated with anticorrosion emulsifiable oil Rostschutz 310® applied in different time. After reaching 28 day strength of the grout that means approximately 50 MPa were specimens tested with device constructed for this purpose. The test arrangement is shown on figure 3. A part of the test arrangement was calibrated hollow jack. Cylindrical specimen was fixed and the strand was pulling out with the jack.

On the specimen were measured displacements on both ends and also corresponding pulling force.

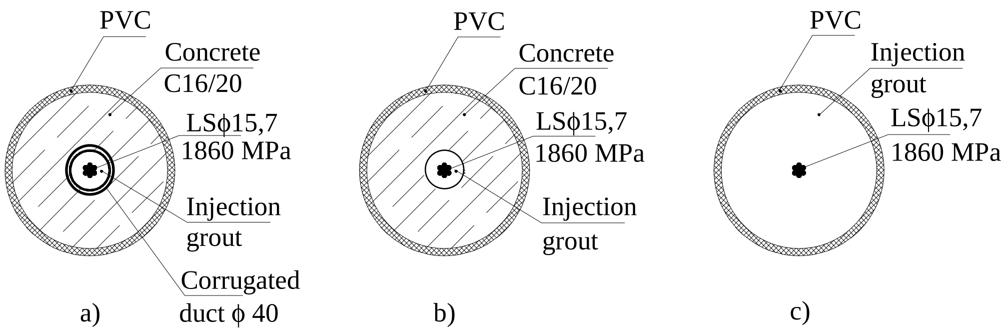

Fig. 2 Specimens types and their cross sections

Primary goal was to compare bond stress behaviour on different types of specimens with strand coated or not coated with emulsifiable oil. The first type of specimens were specimens made from concrete C16/20 where was in the centre placed corrugated steel duct. In that duct was injected strand (Fig 2a). Specimen labelling was “D” as duct or “DC” as duct with coated strand. Second type of specimens was made from concrete C16/20 with hollow in the centre of specimen. In the hollow was placed strand (Fig 2b). Specimen labelling was “WD” as without duct and “WDC” as without duct with coated strands. Third type of specimens was specimens made only from injection grout placed to the plastic tube and the strand was placed in the centre of the specimen (Fig. 2c). In this type of specimens concrete and ducts were not used. Specimens labelling was “G” as grout, “GCW” as grout with strand coated with emulsifiable oil one week before grouting and “GC” as grout with coated strands that means the strand was coated immediately before grouting. There were made 3 specimens from each type.

Fig. 3 Hardening of specimens and test arangement

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Fig. 4 Failure mode of some types of specimens

3.2 Experiment results

Typical mode of failure was represented by longitudinal splitting cracks accompanied with crushed grout wedge around strand on the active pulling side as is shown on Figure 4. Strands losing their adhesion if were coated with anticorrosion emulsifiable oil. Adhesion losing were obviously due the fact that the strand was possible to screw out from specimen. Therefore was prevented against twisting on both end of the strand. Results and comparing maximal pulling force is shown in chart on Figure 5.

Maximum bond between strand and surrounding grout was reached in specimens with corrugated duct type “D”. The strand was pulling out by 157 kN which is corresponding with bond stress 6,038 MPa. Failures of these specimens were accompanied by wide splitting cracks of the weak concrete body of the specimen. Specimens with duct coated with emulsifiable oil reached bond stress in average 2,115 MPa what is approximately 35% of unoiled ones.

Specimen type “WD” reached much lower values of bond stress then type “D” specimens. Specimens coated with emulsifiable oil marked as “WDC” were not even measurable. The strand was pulled out almost without any resistance.

In case of last type of specimens made only from injection grout there was not significant difference in pulling force between coated and not coated strands. Also age depend relationship of emulsion coat in strands coated one week before grouting and strands immediately grouted was not clarified. Average value of bond stress for freshly coated strands type “GC” was 3,077 MPa and for type “GCW” was 3,192 MPa. Specimens without emulsifiable oil marked as “G” had average bond stress 3,423 MPa.

In figure 6 are shown displacements of the strand in specimen types “D” on its active and passive side. From displacement behaviour is obviously that in the beginning are bigger displacement on active side to the adhesion failure between strand and grout. After that there was higher development of displacements on passive side. Curves gradient shows that the displacements on active and passive side are after adhesion breaking very similar. In other cases is obviously smaller adhesion caused strand coating with emulsifiable oil or missing corrugated duct. Measured displacement on their active and passive side did not have characteristic behaviour.

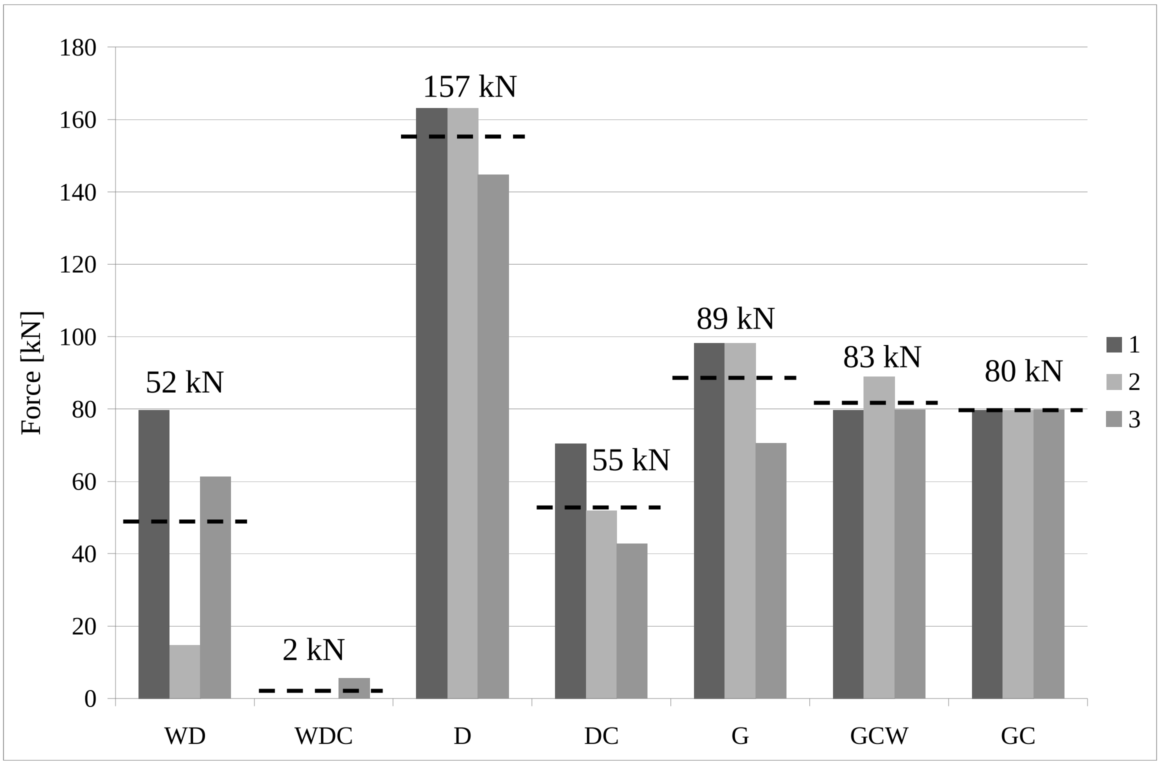

Fig. 5 Resulting of maximal pulling force by specimen failure with average values

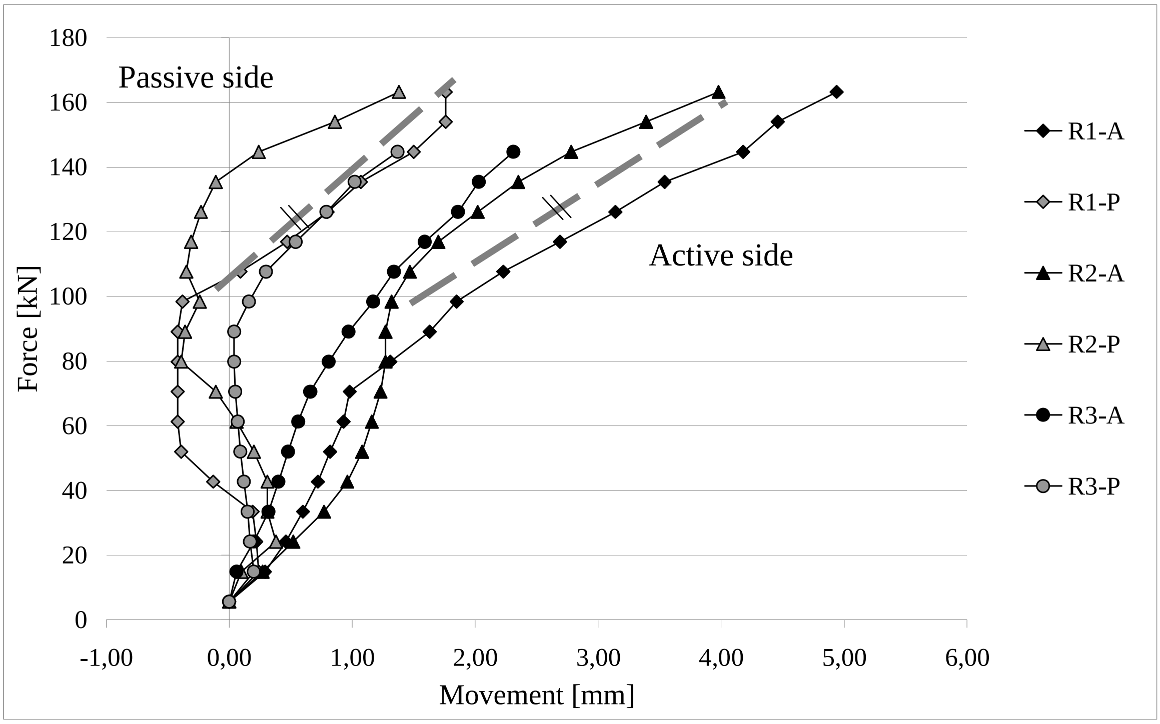

Fig. 6 Displacement behaviour of a "D" type specimens on active and passive side

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4 Conclusions

The use of corrosion emulsions decrease bond between prestressing units and grout. Experimental test also show that an important part for bond behaviour is corrugated duct.
Bonded strands and tendons interact with concrete cross-section, while on the other hand unbonded strands and tendons do not. That means change of strains in prestressing are not the same as in surrounding material on the interface of these two materials. For this reason it is not possible to include that type of strands to the resistance of cross-section directly. In this case prestressing act as external load. If bond failure of tendon occurs, then tendon or its part behave as unbounded. This may cause higher deflections, larger crack width and also lower bending capacity or durability of the construction. Therefore tendons with lower bond capacity should be checked the bond stress from external load in sections where tensile stress or even crack can occur.

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Design of foundation slabs and footings for shear

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Abstract. The paper is focused on the design of foundation slabs and footings for shear. Foundation slabs and footings are structural members where control perimeters within 2*d* should be also checked for punching. High load usually requires additional shear reinforcement to be provided in this area. The paper introduces proposal how to design this reinforcement in control perimeters within 2*d* and how to verified shear resistance of the member.

1 Introduction

Foundation slabs and footings are structural members with thickness which frequently exceeds one meter, but for larger buildings or bridges it can be over three meters. Structural respond to action is ground pressure or reaction in piles with very high magnitude. Therefore the foundation slabs and footings are subjected to the shear forces which are changing significantly with distance from the column face. In order to take into account this effect EN1992-1-1 requires verification of punching resistance also inside the basic control perimeter with minimum distance $a = 0{,}5d$ from the column face. Symbol d is an effective depth of the member.

2 Verification for punching in foundation slabs and footing

Verification for punching includes calculation of shear stresses $v_{\mathrm{Ed}}(a)$ at assumed control perimeter located at distance a from the column face and calculation of shear resistance without shear reinforcement $v_{\mathrm{Rd,c}}$ or $v_{\mathrm{Rd,c}}(a)$ if $a < 2d$. Condition $v_{\mathrm{Ed}}(a) \leq v_{\mathrm{Rd,c}}(a)$ should be fulfilled otherwise shear reinforcement is necessary to provide in order to meet requirement $v_{\mathrm{Ed}}(a) \leq v_{\mathrm{Rd,cs}}(a)$.

2.1 Punching resistance without shear reinforcement

Punching resistance without shear reinforcement $v_{\mathrm{Rd,c}}$ can be determined by formula (1) for control perimeters with distance $a \geq 2d$ from the face of column. Punching resistance inside basic control perimeter $a < 2d$ can be increased by ratio $2d/a$, see formula (2).

$$v_{\mathrm{Rd,c}} = C_{\mathrm{Rd,c}}\, k \left(100.\rho_\ell\, f_{\mathrm{ck}}\right)^{1/3} \tag{1}$$

$$v_{\mathrm{Rd,c}}(a) = \frac{2d}{a} C_{\mathrm{Rd,c}}\, k \left(100.\rho_\ell\, f_{\mathrm{ck}}\right)^{1/3} \tag{2}$$

Where $C_{\mathrm{Rd,c}}$ is an empirical coefficient $C_{\mathrm{Rd,c}} = 0{,}18/\gamma_{\mathrm{C}}$

γ_C partial safety factor for concrete, γ_C = 1,5 (1,2)
k factor $k = 1+(200/d)^{0,5} \leq 2$ (d in [mm])
ρ_ℓ reinforcement ratio based on main reinforcement in two orthogonal direction.

2.2 Shear stresses for punching verification

Shear stresses v_{Ed}(a) are related to the assumed control perimeter and depend on the value of axial force N_{Ed} in the column at the junction with column base and on so called unbalanced bending moments. Unbalanced bending moments are bending moments $M_{Ed,x}$ and $M_{Ed,y}$ at the foot of the column in a case of foundation slab or footing. Shear stresses may increase e.g. openings and recesses in the slab close to the column. Shear stresses result from the ground pressure which is behind the assumed control perimeter. Therefore, shear forces can by reduced by upward force due to the ground pressure $\sigma_{gd,avrg}$ inside the control perimeter, see equation (3).

$$V_{Ed,red}(a) = N_{Ed} - \sigma_{gd,avrg} A(a) \tag{3}$$

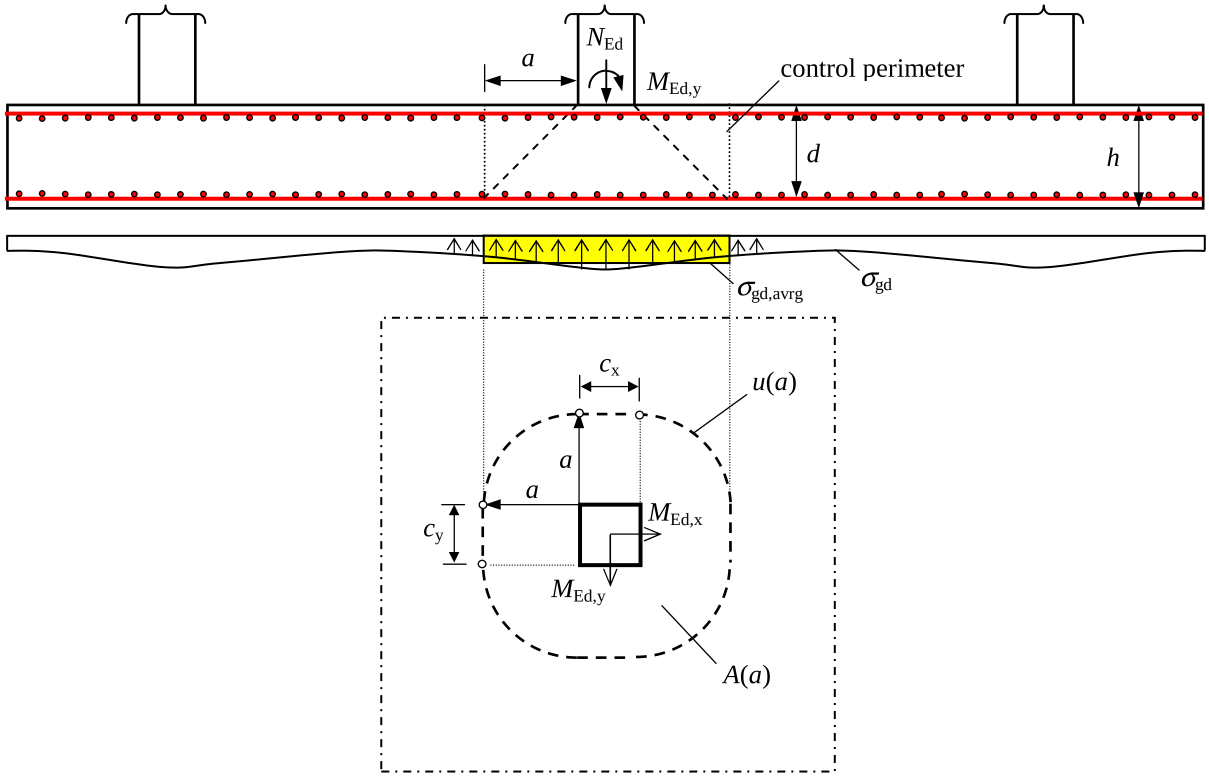

Fig. 1. Foundation slab – model for calculation of shear stresses

An effect of unbalanced bending moments can be taken into account by factor β. Opposite to the floor flat slabs the factor should be calculated for each control perimeter $\beta = \beta$(a), see equations (4) and (6). Equation (4) is valid for rectangular column and equation (6) for circular column with diameter D.

$$\beta(a) = 1 + k_x \frac{M_{Ed,x}}{V_{Ed,red}(a)} \frac{u(a)}{W_x(a)} + k_y \frac{M_{Ed,y}}{V_{Ed,red}(a)} \frac{u(a)}{W_y(a)} \tag{4}$$

$$W_x(a) = \frac{c_x^2}{2} + c_x c_y + 2c_x a + 2a^2 + \frac{\pi}{2} c_x a \tag{5}$$

$$\beta(a) = 1 + 0{,}6.\pi \frac{M_{\mathrm{Ed}}}{V_{\mathrm{Ed,red}}(a)} \cdot \frac{1}{(D + 2a)} \tag{6}$$

Factors k_x and k_y in equation (4) are defined in table 6.1 of [1]. Factor k_x depends on the ratio of column dimensions c_y/c_x while factor k_y is based on the ratio c_x/c_y. Geometrical parameter $W_x(a)$ can be determined by equation (5) as a function of variable a and $W_y(a)$ by the same equation only column dimensions is necessary exchanged here. Shear stresses for punching verification can be calculated by equation (7).

$$v_{\mathrm{Ed}}(a) = \beta(a) \frac{V_{\mathrm{Ed,red}}(a)}{u(a).d} \tag{7}$$

2.3 Verification for punching

Verification for punching should be carried out at the set of control perimeters with regard to the minimum value of $[v_{\mathrm{Rd,c}}(a)\text{-}v_{\mathrm{Ed}}(a)]$. The first control perimeter should be assumed at distance $d/2$ from the face of the column. If value $[v_{\mathrm{Rd,c}}(a) - v_{\mathrm{Ed}}(a)] < 0$ shear reinforcement for punching is necessary to provide within the control perimeter a or change parameters which influence shear capacity $v_{\mathrm{Rd,c}}(a)$, e.g. increase slab thickness, quality of concrete or increase amount of bending reinforcement. The cheapest solution is an application of shear reinforcement at the vicinity of the column. However EC2 does not provide suitable formula for calculation of shear resistance with shear reinforcement for control perimeters at $a < 2d$.

2.4 Maximum shear capacity for punching

The maximum shear capacity for punching is determined by crushing of compression diagonal at the column perimeter $a = 0$. Reliability condition is following.

$$v_{\mathrm{Ed,max}} = v_{\mathrm{Ed}}(0) \le v_{\mathrm{Rd,max}} = k.0{,}6.\left(1 - \frac{f_{\mathrm{ck}}}{250}\right) f_{\mathrm{cd}} \tag{8}$$

Originally factor k was proposed by value 0,5 in [1]. Amendment of the code has changed value to 0,4 , because factor β underestimates an effect of unbalanced bending moments at the column perimeter in the floor flat slabs, where β is calculated base on the geometrical parameters associated with basic control perimeter at $a = 2d$. However, because factor β should be determined for each assumed control perimeter in foundation slabs and footings there is no reason to take this lower value k and 0,5 should be acceptable.

2.5 Punching resistance with shear reinforcement

Punching resistance with shear reinforcement $v_{\mathrm{Rd,cs}}$ can be determined by equation (9) for control perimeters with distance $a \ge 2d$.

$$v_{\mathrm{Rd,cs}} = 0{,}75.v_{\mathrm{Rd,c}} + \left(\frac{1{,}5.d}{s_{\mathrm{r}}}\right) \frac{A_{\mathrm{sw}} \cdot f_{\mathrm{ywd,ef}}}{u(\ge 2.d).d} \sin\alpha \tag{9}$$

Where A_{sw} is the area of one perimeter of shear reinforcement around the column
s_{r} – radial spacing of perimeters of shear reinforcement
$f_{\mathrm{ywd,ef}}$ – effective design strength of shear reinforcement $f_{\mathrm{ywd,ef}}$ = 250 MPa + 0,25d [mm]

For punching resistance with shear reinforcement inside 2d Eurocode 2 does not offer suitable model. My proposal is expressed by equation (10):

$$v_{\mathrm{Rd,cs}}(a)=\frac{2d}{a}\left[0{,}75v_{\mathrm{Rd,c}}+\left(\frac{0{,}75a}{s_{\mathrm{r}}}\right)\frac{A_{\mathrm{sw}}f_{\mathrm{ywd,ef}}}{u(a)d}\sin\alpha\right] \tag{10}$$

or more conservative model by equation (11):

$$v_{\mathrm{Rd,cs}}(a)=\frac{2d}{a}0{,}75v_{\mathrm{Rd,c}}+\left(\frac{0{,}75a}{s_{\mathrm{r}}}\right)\frac{A_{\mathrm{sw}}f_{\mathrm{ywd,ef}}}{u(a)d}\sin\alpha \tag{11}$$

Formula (10) is based on the assumption that part of load (ground pressure) within 0,5$d \le a \le 2d$ flows to the support (column) through the compressive diagonals, see fig.2. This effect can be expressed by reduction of shear force with factor $a/2d$, see 6.2.2 (6), 6.2.3 (8) of EN1992-1-1, or by increasing of shear resistance with factor $2d/a$, see 6.4.4 (2).

3 Footings

Generally, design of footings for punching is the same as in foundation slabs. However the effect of unbalanced bending moments should be taken into account rather by an increment of ground pressure σ_{gd} than application of the factor β. It results from an arrangement of the action under the footing and flow of internal forces in the member, see fig.2.

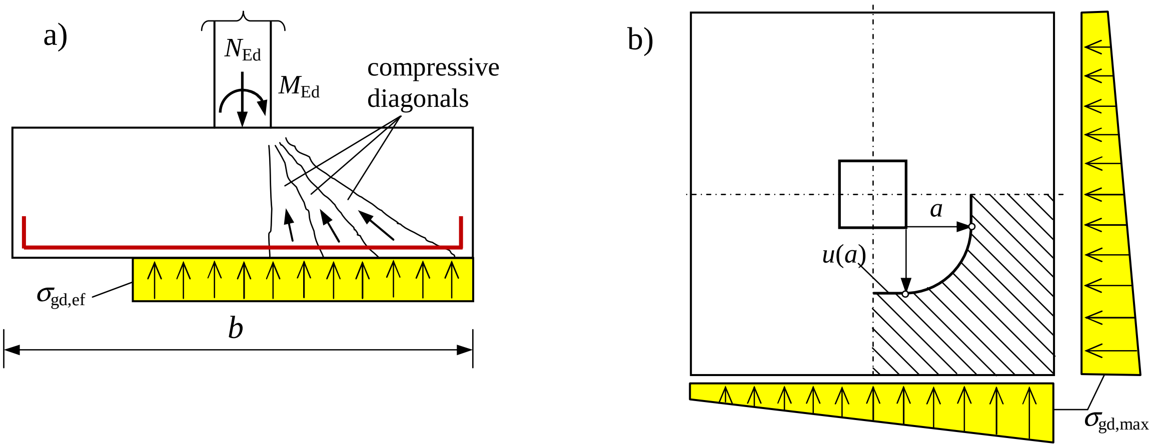

Fig. 2 Footings - a) constant ground pressure, flow of internal forces , b) linear ground pressure

3.1 Footings without piles

The procedure for shear stress calculation depends on assumed distribution of ground pressure. Very simple solution brings assumption of constant ground pressure see fig.2. Increased value of σ_{gd} can be obtained from formula (11) and reduced shear force per full length of control perimeter $u(a)$ by equation (13).

$$\sigma_{\mathrm{gd,ef}}=\frac{N_{\mathrm{Ed}}}{(b-2.e).l} \quad \text{where } e=\frac{M_{\mathrm{Ed}}+h.V_{\mathrm{Ed}}}{N_{\mathrm{Ed}}} \tag{11),(12}$$

$$V_{\mathrm{Ed,red}}(a)=\sigma_{\mathrm{gd,ef}}\left[b.l-A(a)\right] \tag{13}$$

Due to the higher level o reliability for geotechnical design compare to design for mechanical resistance linear distribution of ground pressure is recommended, see fig.3. In order to take into account higher shear stresses $v_{Ed}(a)$ in a part of control perimeter due to the different ground pressure under the footings it is necessary to divide member e.g. into quadrants. Then for the quadrant with maximum ground stress $\sigma_{gd,max}$ calculate shear force $V_{Ed,red}(a)$ by integration of ground pressure under the area of footing behind assumed control perimeter, cross-hatched area in fig.3.
Shear stresses for punching verification can be then simply calculated by formula (7) with β(a) = 1,0 and corresponding length of control perimeter $u(a)$ which depends on the way of $V_{Ed,red}(a)$ calculation.

3.2 Footings with piles

Similar procedure which is introduced in chapter 3.1 can be used also for punching verification of footing with piles. Basic difference is in distribution of shear forces $V_{Ed,red}(a)$. Change of shear forces is not continuous but staggered and depends the on the arrangement of piles and value of reactions in the piles R_{Edi} with regard to assumed control perimeter $u(a)$. In fig.3 is shown example how to determine length of control perimeter $u(a)$ in order to maximize shear stresses and how to calculate shear forces $V_{Ed,red}(a)$ for punching verification.

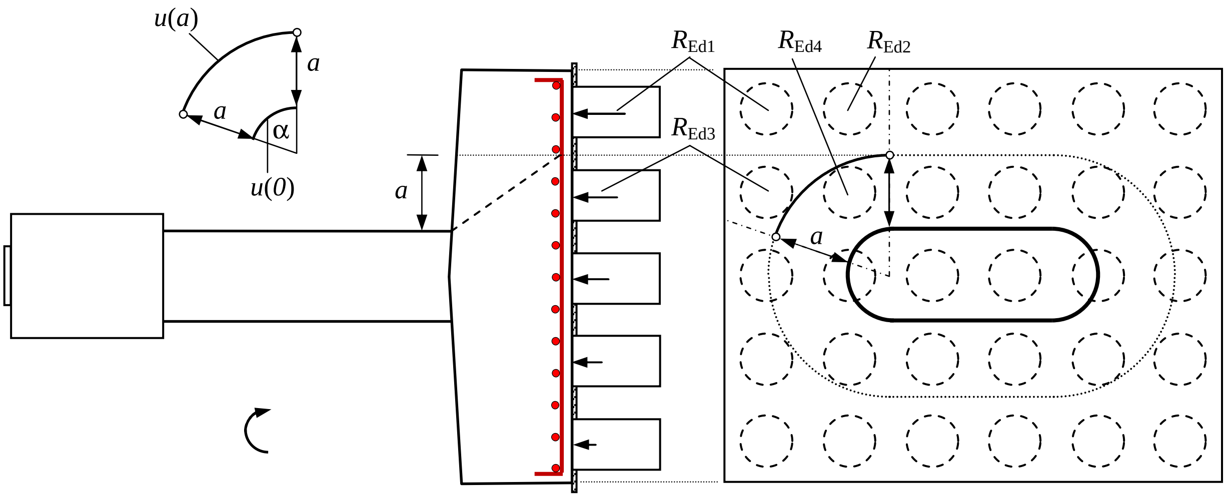

Fig. 3 Footing with piles - model for calculation of maximum shear stresses $v_{Ed}(a)$

$$V_{Ed,red}(a) = R_{Ed1} + R_{Ed2} + R_{Ed3} \quad \rightarrow \quad v_{Ed}(a) = \frac{V_{Ed,red}(a)}{u(a)d} \leq v_{Rd,c}(a) \tag{14}$$

$$V_{Ed,max} = R_{Ed1} + R_{Ed2} + R_{Ed3} + R_{Ed4} \quad \rightarrow \quad v_{Ed,max} = \frac{V_{Ed,max}}{u(0)d} \leq v_{Rd,max} \tag{15}$$

4 Example

Bridge over the river Nitra has 5 spans with maximum span length of 120 m. Bridge is constructed using cantilever balanced method. Substructure consists of pier footings with dimensions 15×12 m and one column with diameter D = 4 m, see fig.4. Thickness of the footings is variable and changes from 3 m to 2,47 m with pitch of 7%. Bending reinforcement B500B consists of 2 layers ϕ32 bars spaced at 125 mm in longitudinal directions and 2 layers ϕ25 bars spaced at 125 mm in transverse directions. Effective depth is d = 2,705 m. Concrete C25/30.

4.1 Internal forces

Axial force: N_{Ed} = 74,1 MN

Shear forces: $V_{\mathrm{Ed,x}}$ = 2,770 MN ; $V_{\mathrm{Ed,y}}$ = 1,178 MN

Bending moments: $M_{\mathrm{Ed,x}}$ = 38,0 MN.m ; $M_{\mathrm{Ed,y}}$ = 22,0 MN.m

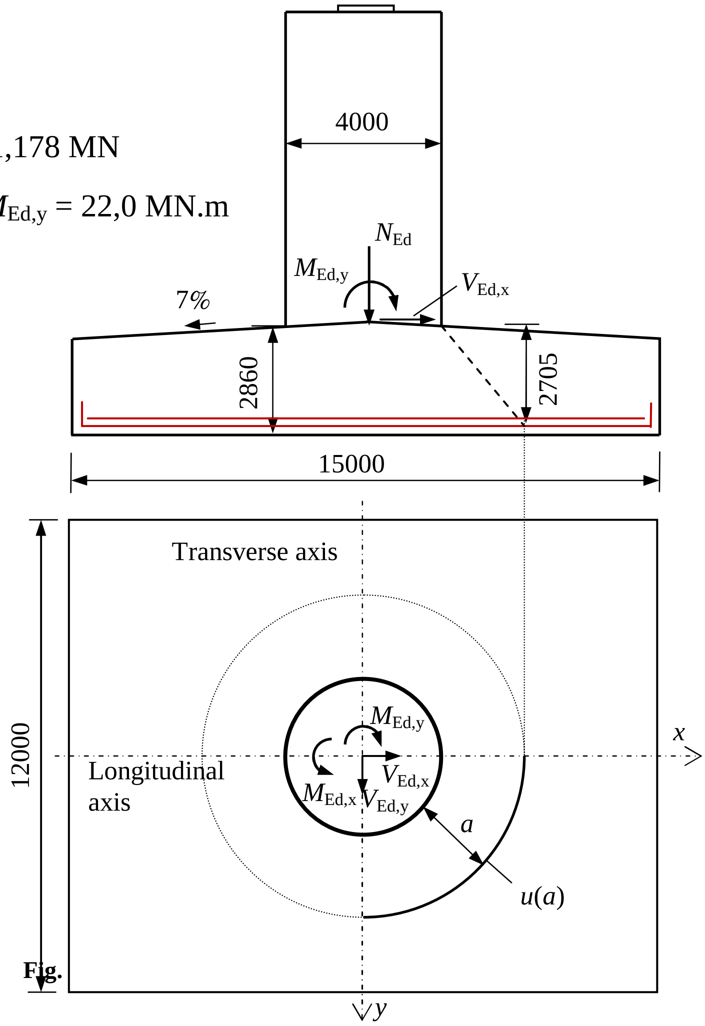

Fig.

4.2 Punching resistance

k = 1+(200/2705)^{0,5} = 1,272

A_{sx} = 16.8,04 = 128,64 cm^2

A_{sy} = 16.4,91 = 78,54 cm^2

ρ_{sx} = 0,012864/(1,0.2,72) = 0,004733

ρ_{sy} = 0,007854/(1,0.2,69) = 0,002919

ρ_{sl} = (0,00473.0,00292)^{0,5} = 0,003717

$$v_{\mathrm{Rd,c}} = C_{\mathrm{Rd,c}}\, k\left(100.\rho_{\ell}\ f_{\mathrm{ck}}\right)^{1/3}$$

$$v_{\mathrm{Rd,c}} = \frac{0,18}{1,5}1,272\left(100.0,00372.25\right)^{1/3} =$$
$$= 0,320 \text{ MPa}$$

$$v_{\min} = 0,0035.(1,272)^{3/2}(25)^{1/2} = 0,251 \text{ MPa}$$

$$v_{\mathrm{Rd,c}}(a) = 0,320.2d/a = 0,320.5,41/a$$

4.3 Shear stresses

Length of control perimeter: $r(a) = 0,5D + a \rightarrow u(a) = 0,5\pi r(a)$ (one quadrant)

$$e_{\mathrm{y}} = \frac{M_{\mathrm{Ed,x}} + h.V_{\mathrm{Ed,y}}}{N_{\mathrm{Ed}}} = \frac{38 + 2,86.1,178}{74,1} = 0,558 \text{ m}$$

$$e_{\mathrm{x}} = \frac{M_{\mathrm{Ed,y}} + h.V_{\mathrm{Ed,x}}}{N_{\mathrm{Ed}}} = \frac{22 + 2,86.2,77}{74,1} = 0,404 \text{ m}$$

$$\sigma_{\mathrm{gd,ef}} \approx \frac{N_{\mathrm{Ed}}}{(b_{\mathrm{x}} - 2e_{\mathrm{x}}).(b_{\mathrm{y}} - 2e_{\mathrm{y}})} = \frac{74,1}{(15 - 2.0,404).(12 - 2.0,558)} = 0,48 \text{ MN/m}^2$$

$F_{\mathrm{Ed,max}} = 0,5b_{\mathrm{x}}0,5b_{\mathrm{y}}\sigma_{\mathrm{gd,eff}}$ = 0,25.15.12.0,48 =21,6 MN

$V_{\mathrm{Ed,max}} = F_{\mathrm{Ed,max}} - 0,25\pi(D/2)^2\sigma_{\mathrm{gd,eff}}$ = 21,6 – 0,25.3,14.(4/2)^2.0,48 =20,1 MN

$v_{\mathrm{Ed,max}} = V_{\mathrm{Ed,max}}/(u_0 d)$ = 20,1/(0,25πDd) = 20,1/(0,25.3,14.4.2,705) = 2,365 MPa

$v_{\mathrm{Rd,max}}$ = 0,5.0,6.(1- f_{ck}/250).f_{cd} = 0,5.0,6.(1- 25/250).14,17 = 3,825 MPa

$v_{Rd,max} = 3{,}825$ MPa $\geq v_{Ed,max} = 2{,}365$ MPa

$V_{Ed,red}(a) = F_{Ed,max} - \sigma_{gd,eff}\, 0{,}25\pi r(a)^2 \rightarrow\; v_{Ed}(a) = V_{Ed,red}(a)/(u(a)d)$

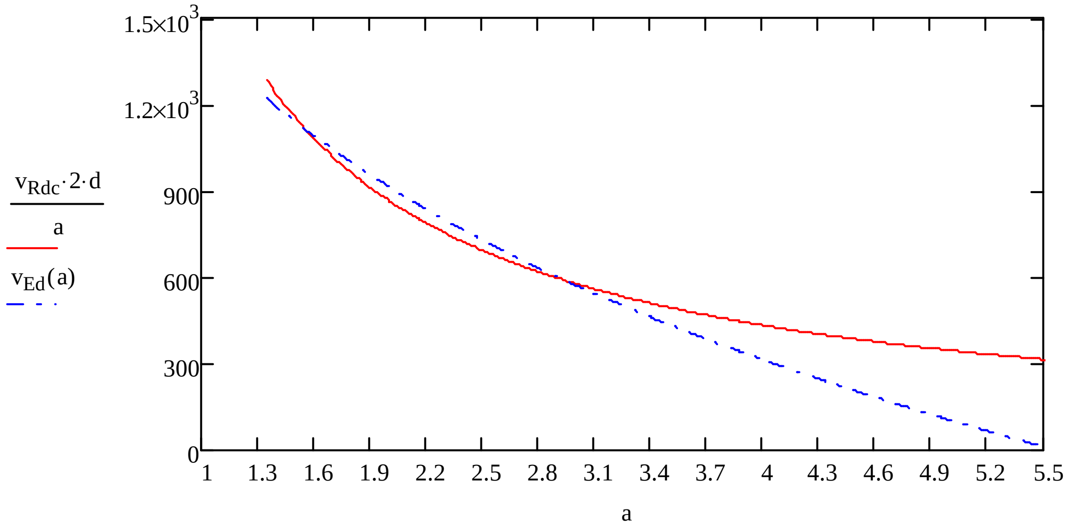

Fig.5 Footing without shear reinforcement - punching verification where *a* ranges from 0,5.*d* to 2,0.*d*

Verification for punching resistance is shown in fig.5. Red line indicates shear resistance, dashed blue line shear stresses. For control perimeters with distance *a* varying from 0,56.*d* = 1,52 m to 1,09.*d* = 2,95 m is not fulfilled reliability condition and shear reinforcement is required. Proposed shear reinforcement ϕ12 mm consists of 64 links in perimeter around the column or 16 links per quadrant. Spacing of shear reinforcement perimeters in radial direction s_r = 210 mm was determined from $v_{Rd,cs}(a) \geq v_{Ed}(a)$.

Note: Proposed arrangement of shear reinforcement was chosen in order to simplify calculation. Orthogonal arrangement of links or bent up bars is typical for bridge footings.

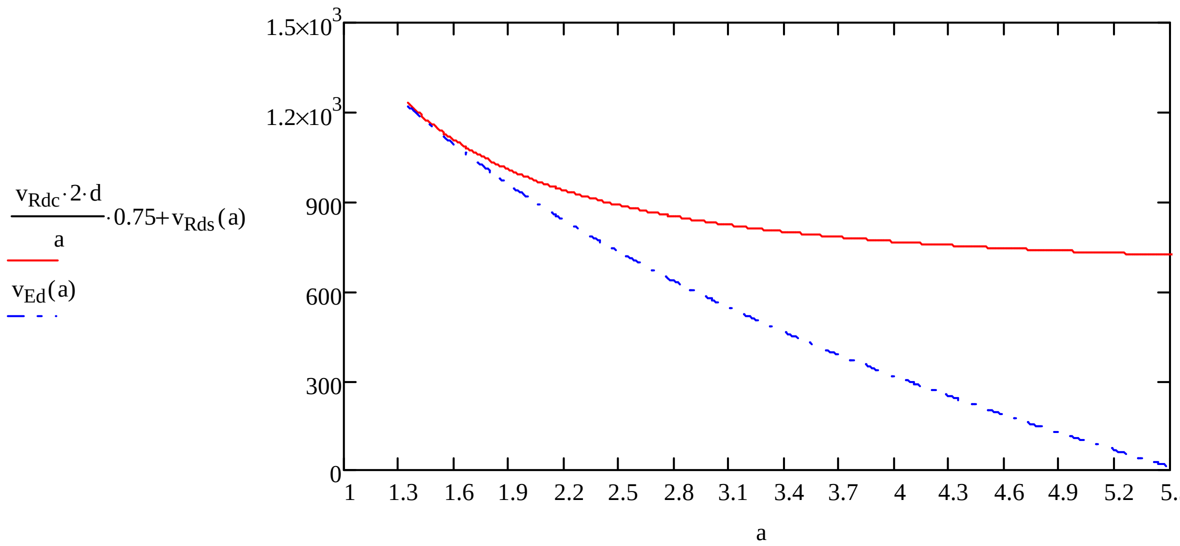

Fig.6 Footing with shear reinforcement - punching verification where $v_{Rd,cs}(a)$ is calculated by equation (11)

Detailing rules require to check ratio of shear reinforcement $\rho_w = A_{sw1}/(s_t\, s_r) \geq \rho_{w,min}$
$\rho_{w,min} = 0{,}08.(f_{ck})^{0,5}/f_{ywk} = 0{,}08.(25)^{0,5}/500 = 0{,}0008 \rightarrow s_{t,max} = \min\,[A_{sw1}/(\rho_{w,min} s_r)$, 600 mm] = min[0,000113/(0,0008.0,21); 0,6 m] = min[0, 672 m, 0,600] = 0,6 m.

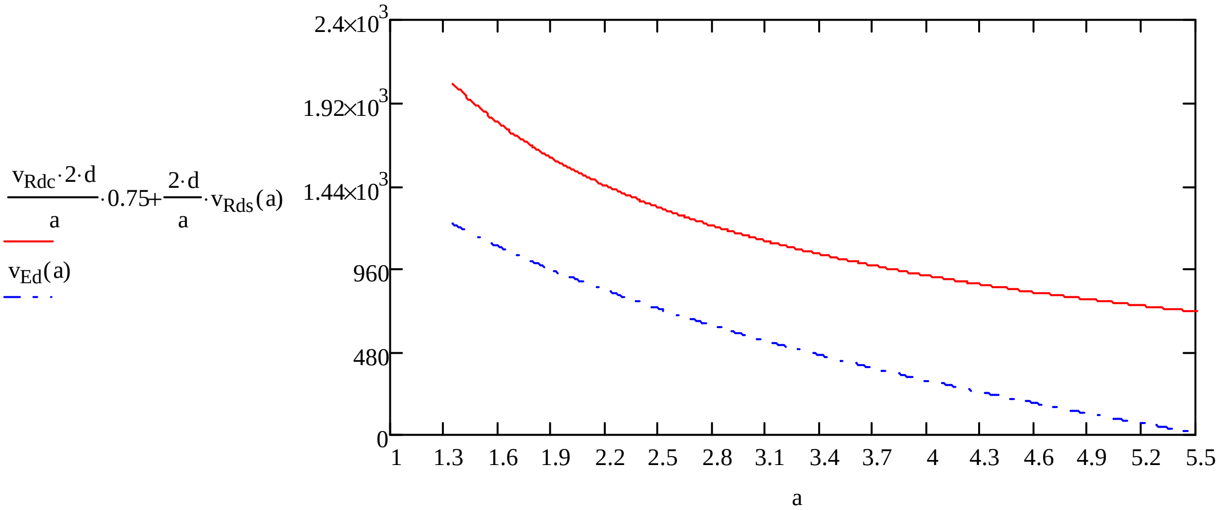

Fig.7 Footing with shear reinforcement - punching verification where $v_{Rdcs}(a)$ is calculated by equation (10)

In fig.7 is shown punching verification with $v_{Rd,cs}(a)$ calculated with less conservative equation (10) and the same reinforcement arrangement as in fig.6. Verification indicates possibility to save some amount of shear reinforcement. However this saving is limited by the detailing rules.

Conclusions

Design of foundation slabs and footings for shear is more complex than in a case of floor flat slabs because verification brings requirement to check shear stresses also inside the basic control perimeter $a < 2d$ and to take into account an interaction with ground pressure in calculation of shear forces. Current EC2 does not offer suitable model for punching verification in control perimeters inside $2d$ if shear reinforcement is required here. The paper offers two possible ways how to design shear reinforcement and how to determine shear resistance for punching in this area. The first one, equation (10), is compatible with model for calculation of $v_{Rd,c}$ inside basic control perimeter where shear resistance grows with decreasing of a/d. The second model, equation (11) is more conservative and better reflects knowledge obtained from extensive research in foundation performed by RWTH Aachen University. In [3] is stated that shear reinforcement substantially increase punching capacity of footings but less effectively with decreasing of a/d.

Acknowledgment

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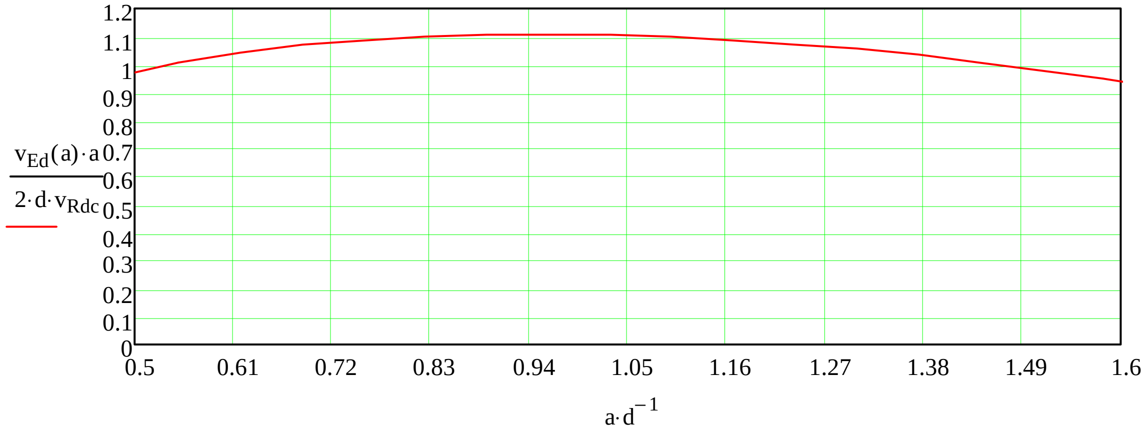
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
$\frac{v_{Ed}(a)\cdot a}{2\cdot d\cdot v_{Rdc}}$
0.5
0.61
0.72
0.83
0.94
1.05
1.16
1.27
1.38
1.49
1.6
$a\cdot d^{-1}$

Reinforced concrete columns – analysis of the influence of transverse reinforcement

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1 Introduction

In the article we are dealing with the influence of transverse and longitudinal reinforcement to the resistance of a cross-section of a column and also with the effective detailing of the column reinforcement. We are verifying the correctness of design guides for detailing of transverse and longitudinal reinforcement. We are also taking into account the diameter of stirrups and its influence over transverse deformation of column.

2 Detailing of reinforced columns

For the design of elements and structures it is not enough to verify performance of the ultimate limit states and serviceability, but it is necessary to perform the detailing of reinforcement as well. It should be noted that the performance of detailing ensures reliability in those areas, where the calculation is not verified directly. This is mostly in those cases of loads, where the calculation would be disproportioned difficult, due to the achieved results, respectively cases, where it is not yet enough theoretical input dates to calculate (e.g. for durability). In most cases detailing gives results on the conservative side and it is because these requirements must cover the whole range of different input dates. The use of these dates with the next precision would be too difficult [1].

2.1 Detailing of longitudinal reinforcement

The detailing of column longitudinal reinforcement according to STN EN 1992-1-1, also taking into consideration STN EN 1992-1-1 / NA is: min. diameter of reinforcement: $\phi_s = 10$ mm; min. quantity of reinforcement: $A_{s,min} = 0,10.N_{Ed}/f_{yd}$, As,min $\geq$ 0,002 . A_c; max.

quantity of reinforcement: $A_{s,max} \leq 0{,}04.A_c$ (lapped bars area $A_{s,max} \leq 0{,}08.A_c$), where: N_{Ed} is design value of compressive axial force and A_c is area of concrete cross-section. The min. number of reinforcing bars: at least one bar in each corner and in a circular cross-section 4 (better 6). Spacing of reinforcement bars must enable correct location and correct compaction of concrete, thus ensuring adequate bond of reinforcement and concrete. Horizontal and vertical spacing between parallel reinforcement bars shall be not smaller than the max. of values: 1,5 $Ø_{max}$; 20 mm; d_g + 5 mm, where d_g is max. diameter of aggregate and $Ø_{max}$ is max. diameter of reinforcement bar. In determining the min. spacing between reinforcing bars are therefore also take into account the size of the max. diameter of aggregate, which is necessary to indicate in the proposed concrete on design documentation. Because lapped joints of longitudinal reinforcement it is recommended minimum space between bars of longitudinal reinforcement in columns 40mm. The values of the minimum distance between the longitudinal reinforcement of monolithic reinforced concrete columns should be unconditionally secure. By precast concrete elements and specific methods of compacting, such centrifugation, is assumption that we have not fully fulfill with those recommended values of min. spacing between the reinforcement. [2].

2.2 Detailing of transverse reinforcement

Reinforced concrete columns are mainly subjected to relatively great axial forces and bending moments. The stress because of transverse force is usually very small. The main role of transverse reinforcement in reinforced concrete columns is not a transfer of shear stress as it is mainly in the beams, but it is prevention buckling of longitudinal reinforcement and transfer of tensile stresses, that arise in the orthogonal direction to the direction of compressive force action in the column [1].Transverse reinforcement needs to be anchored adequately if it is to function effectively. By failure of columns comes to buckling of longitudinal reinforcement (Fig. 2.1). Detailing of column stirrups according to [4], also taking into consideration NA :

diameter of stirrups: $\phi_{ss} \geq \max [0{,}25\ \phi_{sl,max}\ ;\ 6\text{ mm}]$

distance of stirrups: $s_{cl,t} \leq 300$ mm $\quad s_{cl,t} \leq \min [b;\ h] \quad s_{cl,t} \leq 15\ \phi_{sl,min}$

where: $\phi_{sl,max}$ - max. diameter of longitudinal column reinforcement

$\phi_{sl,min}$ - min. diameter of longitudinal column reinforcement

$s_{cl,t}$ - distance of transverse reinforcement

Figure 2.1: Buckling of longitudinal reinforcement by failure of column [2]

The distance of stirrups in the place of: 1.) lapped joints of longitudinal reinforcement, 2.) within a distance equal to the larger dimension of the column cross section above or below a beam or slab, reduces to the value as follows: $s'_{cl,t} \leq 0{,}6.s_{cl,t}$
Every longitudinal bar in a corner should be held by transverse reinforcement. No bar within a compression zone should be further than 150 mm from a restrained bar. If there is a longitudinal bar in the greater distance, so it is should be held with an another stirrup (Fig.2.2). In the case of spiral stirrups is valid: $\varnothing_{ss} \geq 6$ mm; $\varnothing_{ss} \geq \frac{1}{4} \varnothing_{sl,max}$

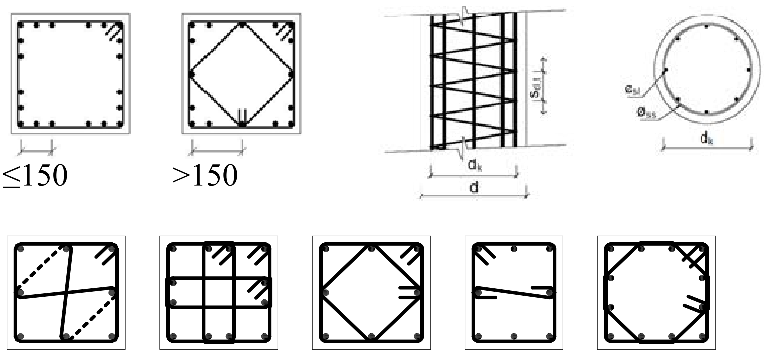


Figure 2.2: Transverse reinforcement of columns

The spacing between spirals should be determined from these conditions:

$$s_{cl,t} \leq 0{,}2\ d_k\ ;\ s_{cl,t} \leq 100\ \text{mm}\ ;\ s_{cl,t} > 20\ \text{mm} + \varnothing_{ss}$$

d_k - diameter of the spiral, $\varnothing_{ss}$ - diameter of transverse reinforcement,
$\varnothing_{sl}$ - diameter of longitudinal reinforcement, $s_{cl,t}$ - spacing between spirals

3 The influence of transverse reinforcement to the resistance of a column

Transverse reinforcement with small diameter works as a tie member between opposite corners of the rectangular stirrup, because the bending stiffness of stirrup is very small and stirrups deviate to the outside before, as they should act against the transverse strain of concrete between corners. In this case is protected against transverse strain of concrete only at the corners of the stirrup (Fig. 3.1 a, b, c). It is supposed that the area of effective influence of column stirrups create a kind of arches between the points where the stirrups cause pressure on the concrete. Improvement of the effect of transverse reinforcement to the resistance of column can be achieved by placing the transverse reinforcement in smaller distances. When the arch arises smaller (the distances between stirrups are smaller), then it is more effective protect against the transverse strain of concrete. The effectiveness of protection of concrete transverse strain of the rectangular shape stirrups can be rapidly increased by using additional stirrups (Fig.3.1a). The suitable spaced longitudinal reinforcement in the perimeter of cross-section also helps to increase the column's resistance. The longitudinal reinforcement resists of the strained concrete and the reactions from this reinforcement are transferred to the transverse reinforcement. So the effectiveness of transverse reinforcement to the resistance of the column depends on the distances of arches, which arise between the longitudinal reinforcement and also between transverse reinforcement (Fig. 3.1 b, c) [3].

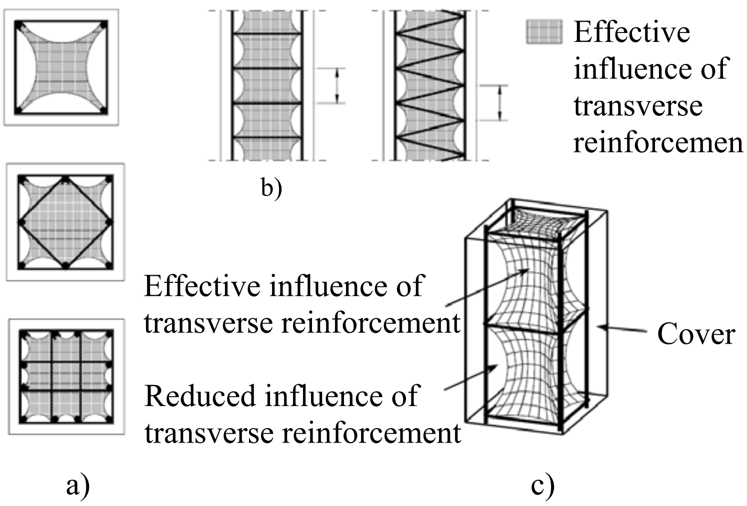


Figure 3.1: Effective influence of transverse reinforcement of reinforced columns; a) squared and b) circular cross-section; c) 3D view on a column of squared cross-section

The longitudinal reinforcement has usually sufficient bending stiffness between stirrups. However it resists transverse pressure, and if the reinforcement is in larger distances from each other, bending stiffness is no sufficient. To effectively protected transverse strain it is also necessary to thicken sufficiently the distances of the longitudinal reinforcement.

4 Analyzed model

To determine the size of the influence of the transverse and longitudinal reinforcement to resistance of column cross-section, to the effectiveness of the detailing of column reinforcement and to determine the size of transverse strain of column we used finite-element calculation in program ATENA.

4.1 Material characteristics, geometry and reinforcement of columns

We used in the models of columns concrete class C20/25 due to the relatively small modulus of elasticity and thus the better readability of the results – strain in the transverse direction. For the longitudinal and transverse reinforcement was used reinforcing steel Bst500. All columns had length of 1800mm and the cross-sections are on the Fig. 4.1. These types of cross-sections were chosen in the order to observe the behavior of the stirrups depending on the distance of longitudinal reinforcement from the corner of stirrup. In the case of column cross-section 300x300mm was the distance of longitudinal reinforcement from the corner of stirrup shorter than 150mm. In the case of column cross-section 350x350mm was this distance approximately 150mm and in the case of column cross-section 400x400mm was this distance longer than 150mm (Fig. 4.1). Transverse reinforcement was modeled in all cases with equal distances „s“, so the differences in the results should be cause only with the diameter of the stirrup.

Column’s cross-sections of series I. and II. Column’s cross-sections of series III. and IV.

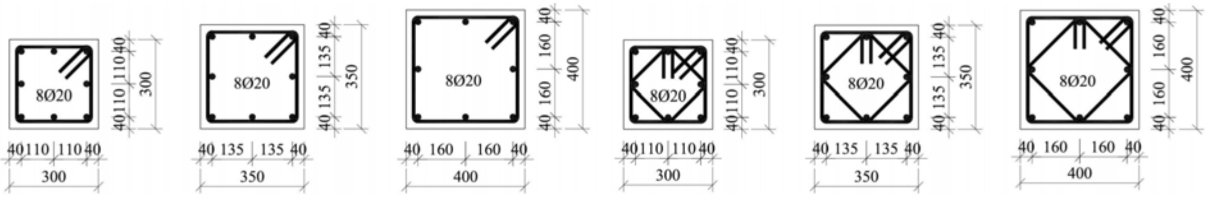
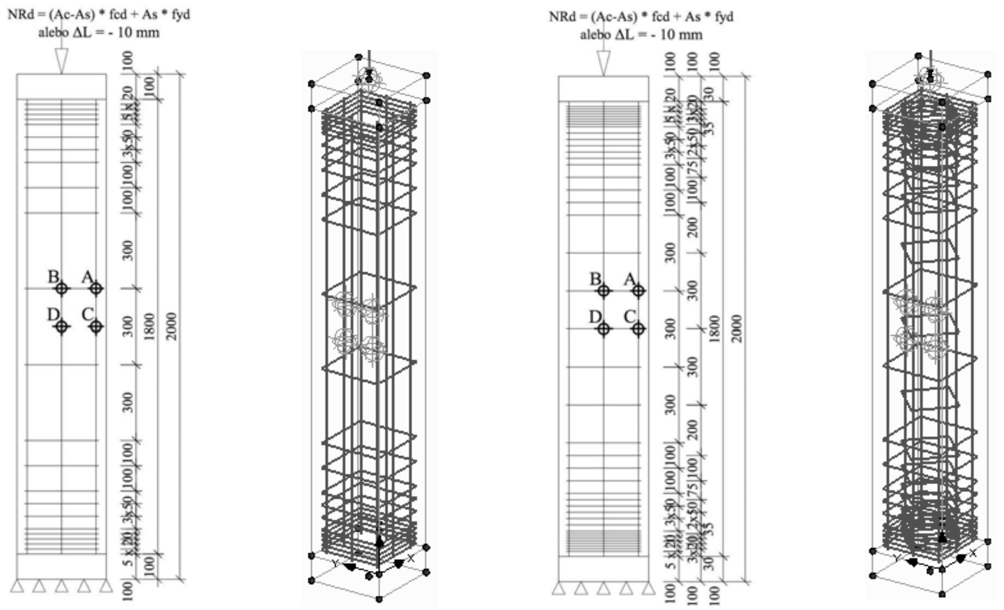


Figure 4.1: Cross-sections of analyzed columns

The distances of the transverse reinforcement in the head and in the foot of the column were smaller due to stress concentration in the head and in the foot of the column. The smaller distances were according to the results of models, those we analyzed before. In the head and in the foot of the column were modeled steel plates of thickness 100mm (Fig. 4.2).

4.2 Monitoring of results and loading of a column

Results were monitored by five monitors. One monitor was placed in the head of the column on the spread plate. This monitor monitored column load and its progress at each step. The other monitors were marked with letters „A“, „B“, „C“, „D“. These monitors monitored the progress of transverse strain in each step. Monitor „A“ monitored transverse strain in the corner of the stirrup, monitor „B“ in the middle of the stirrup, monitor „C“ and „D“ on the longitudinal reinforcement (Fig. 4.2).

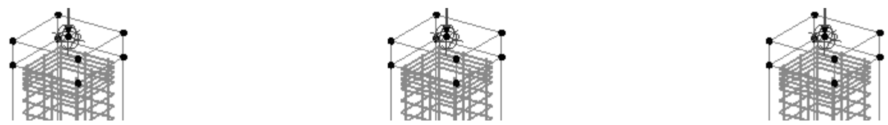


Column’s geometry of series I. and II. Column’s geometry of series III. and IV.

Figure 4.2: Geometry, load and reinforcement of columns /location of monitors on the column

All the columns were loaded by concentric compression, respectively by concentric force in order to eliminate transverse movements due to column buckling. These transverse movements due to column buckling would be made impossible reading and comparison of the results. Load was transmitted into the steel plate that ensures the spreading of the load into the column cross-section. Two of the four series were loaded by centric force that has risen during the calculation. Load-strain curves were generated. The final size of the force was calculated from the equation $N_{Rd} = (A_c - A_s) \cdot f_{cd} + A_s \cdot f_{yd}$. It results that each cross-section was loaded by another force. (Fig.4.3):

$N_{Rd} = 2259$ kN $\quad N_{Rd} = 2692$ kN $\quad N_{Rd} = 3192$ kN



300x300mm $\quad$ 350x350mm $\quad$ 400x400mm

Figure 4.3: The maximum load of columns for each cross-section

Such a load of columns allowed us to compare the columns of the same cross-section with different diameter of stirrup and thus to determine the influence of the stirrup to buckling of longitudinal reinforcement bar. However we also want to compare the efficiency of stirrup diameter at different distances of longitudinal reinforcement from the corner of the stirrup, we had loaded the column so that the longitudinal and transverse stress in cross-section was the same in all three types of cross-sections. The other two series of columns were loaded by vertical strain of the final value Δl = -10mm. This strain increased in the process of calculation, what also ensured results in the form of load-strain curves. (Fig.4.6). In both cases of the loading was the calculation in progress in 100 following steps. The only variable was the size of the diameter of stirrup (Ø6, Ø8 a Ø10) in the compared columns and using of additional stirrup. Overview of modeled columns is in the table (Table 4.1):

Table 4.1: List of analyzed columns (N-loading by increasing force, L- loading by increasing support displacement)

series	Marking	cross-section	diameter of stirrup
I.	S 1.1 N	300x300	Ø6
	S 1.2 N		Ø8
	S 1.3 N		Ø10
	S 1.4 N	350x350	Ø6
	S 1.5 N		Ø8
	S 1.6 N		Ø10
	S 1.7 N	400x400	Ø6
	S 1.8 N		Ø8
	S 1.9 N		Ø10
II.	S 1.1 L	300x300	Ø6
	S 1.2 L		Ø8
	S 1.3 L		Ø10
	S 1.4 L	350x350	Ø6
	S 1.5 L		Ø8
	S 1.6 L		Ø10
	S 1.7 L	400x400	Ø6
	S 1.8 L		Ø8
	S 1.9 L		Ø10

series	Marking	cross-section	diameter of stirrup	diameter of add. stirrup
III.	S 2.1 N	300x300	Ø6	Ø6
	S 2.2 N		Ø8	Ø8
	S 2.3 N		Ø10	Ø10
	S 2.4 N	350x350	Ø6	Ø6
	S 2.5 N		Ø8	Ø8
	S 2.6 N		Ø10	Ø10
	S 2.7 N	400x400	Ø6	Ø6
	S 2.8 N		Ø8	Ø8
	S 2.9 N		Ø10	Ø10
IV.	S 2.1 L	300x300	Ø6	Ø6
	S 2.2 L		Ø8	Ø8
	S 2.3 L		Ø10	Ø10
	S 2.4 L	350x350	Ø6	Ø6
	S 2.5 L		Ø8	Ø8
	S 2.6 L		Ø10	Ø10
	S 2.7 L	400x400	Ø6	Ø6
	S 2.8 L		Ø8	Ø8
	S 2.9 L		Ø10	Ø10

5 Results from the analysis

Results are in tables (Table 5.1 - Table 5.12). From the tables placed side by side it is shown that the additional stirrups reduce transverse strain of column.

Table 5.1: Transverse strain of columns of series I. for cross-section 300x300mm [mm]

cross-section	300 x 300 mm		
marking	Ø6	Ø8	Ø10
Monitor A	6,462E-05	6,237E-05	5,973E-05
Monitor B	6,700E-05	6,642E-05	6,580E-05
Monitor C	6,735E-05	6,700E-05	6,661E-05
Monitor D	6,743E-05	6,713E-05	6,681E-05

Table 5.2: Transverse strain of columns of series III. for cross-section 300x300mm [mm]

cross-section	300 x 300 mm		
marking	Ø6	Ø8	Ø10
Monitor A	6,447E-05	6,220E-05	5,955E-05
Monitor B	6,700E-05	6,648E-05	6,594E-05
Monitor C	6,676E-05	6,614E-05	6,551E-05
Monitor D	6,327E-05	6,043E-05	5,735E-05

Table 5.3: Transverse strain of columns of series I. for cross-section 350x350mm [mm]

cross-section	350 x 350 mm		
marking	Ø6	Ø8	Ø10
Monitor A	6,523E-05	6,367E-05	6,180E-05
Monitor B	6,696E-05	6,660E-05	6,617E-05
Monitor C	6,719E-05	6,682E-05	6,639E-05
Monitor D	6,727E-05	6,694E-05	6,656E-05

Table 5.4: Transverse strain of columns of series III. for cross-section 350x350mm [mm]

cross-section	350 x 350 mm		
marking	Ø6	Ø8	Ø10
Monitor A	6,503E-05	6,336E-05	6,138E-05
Monitor B	6,682E-05	6,638E-05	6,587E-05
Monitor C	6,673E-05	6,610E-05	6,539E-05
Monitor D	6,465E-05	6,258E-05	6,020E-05

Table 5.5: Transverse strain of columns of series I. for cross-section 400x400mm [mm]

cross-section	400 x 400 mm		
marking	Ø6	Ø8	Ø10
Monitor A	6,562E-05	6,438E-05	6,258E-05
Monitor B	6,702E-05	6,687E-05	6,664E-05
Monitor C	6,785E-05	6,756E-05	6,719E-05
Monitor D	6,800E-05	6,775E-05	6,745E-05

Table 5.6: Transverse strain of columns of series III. for cross-section 400x400mm [mm]

cross-section	400 x 400 mm		
marking	Ø6	Ø8	Ø10
Monitor A	6,548E-05	6,411E-05	6,241E-05
Monitor B	6,694E-05	6,667E-05	6,628E-05
Monitor C	6,748E-05	6,694E-05	6,628E-05
Monitor D	6,625E-05	6,478E-05	6,303E-05

Table 5.7: Transverse strain of columns of series II. for diameter of stirrup Ø6 [mm]

marking	Ø6		
cross-section	300x300	350x350	400x400
Monitor A	4,244E-04	4,959E-04	5,629E-04
Monitor B	4,826E-04	5,435E-04	6,071E-04
Monitor C	4,857E-04	5,475E-04	5,935E-04
Monitor D	4,894E-04	5,476E-04	5,910E-04

Table 5.8: Transverse strain of columns of series IV. for diameter of stirrup Ø6 [mm]

marking	Ø6		
cross-section	300x300	350x350	400x400
Monitor A	4,985E-04	5,874E-04	6,636E-04
Monitor B	5,798E-04	6,561E-04	7,344E-04
Monitor C	5,726E-04	6,602E-04	7,504E-04
Monitor D	4,466E-04	5,861E-04	7,033E-04

Table 5.9: Transverse strain of columns of series II. for diameter of stirrup Ø8 [mm]

marking	Ø8		
cross-section	300x300	350x350	400x400
Monitor A	4,146E-04	5,102E-04	5,794E-04
Monitor B	5,295E-04	6,044E-04	6,702E-04
Monitor C	5,310E-04	6,156E-04	6,826E-04
Monitor D	5,404E-04	6,223E-04	6,904E-04

Table 5.10: Transverse strain of columns of series IV. for diameter of stirrup Ø8 [mm]

marking	Ø8		
cross-section	300x300	350x350	400x400
Monitor A	4,821E-04	6,001E-04	6,863E-04
Monitor B	6,495E-04	7,292E-04	8,193E-04
Monitor C	6,446E-04	7,282E-04	8,367E-04
Monitor D	3,707E-04	5,842E-04	7,484E-04

Table 5.11: Transverse strain of columns of series II. for diameter of stirrup Ø10 [mm]

marking	Ø10		
cross-section	300x300	350x350	400x400
Monitor A	3,739E-04	4,946E-04	5,703E-04
Monitor B	5,721E-04	6,489E-04	7,211E-04
Monitor C	5,691E-04	6,613E-04	7,418E-04
Monitor D	5,846E-04	6,756E-04	7,606E-04

Table 5.12: Transverse strain of columns of series IV. for diameter of stirrup Ø10 [mm]

marking	Ø10		
cross-section	300x300	350x350	400x400
Monitor A	4,143E-04	5,747E-04	6,720E-04
Monitor B	7,375E-04	7,840E-04	8,849E-04
Monitor C	7,371E-04	7,685E-04	8,918E-04
Monitor D	3,015E-04	5,402E-04	7,479E-04

6 Conclusions

The resistance of column was bigger by using a stirrup of diameter Ø10, than the resistance by using a stirrup of diameter Ø8, resp. Ø6 and from what results the bigger stress in the core of column cross-section and with that bigger transverse strain by maximum column resistance. At the same load of individual cross-sections by using stirrups of diameter Ø6 were observed larger transverse strains than by using stirrups of diameter Ø8 or Ø10.
Currently, no bar within a compression zone should be further than 150 mm from a restrained bar. From the results it is evident, that the design of additional stirrups also depends on other parameters, as for example: diameter of stirrup, diameter of longitudinal reinforcement, - quality of concrete, etc. The results confirm expected behavior of stirrups in the column. They confirmed that the bending stiffness of transverse reinforcement influences on the transverse column strains and also buckling of the stirrup and the longitudinal reinforcement too. At the same load of individual cross-sections by using stirrups of diameter Ø6 were observed larger transverse strains than by using stirrups of diameter Ø8 or Ø10. Even a stirrup diameter is supposed to determine both, in the case of a structural design of stirrups and about the use of an additional stirrup. It is necessary to model more columns, to take into account the influence of stirrup diameter by designing of stirrups and consequently these designs to verify by experiment.

Acknowledgements

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Stresses and Crack Width Control According to EN 1992

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Abstract. In calculation of crack width it is very important to look after time dependant volume changes. EN 1992 model for direct crack width calculation makes provision for creep effect by a factor for long term loading. Calculation of stress in steel reinforcement without creep effect can therefore lead to slight of this stress. When including creep by effective modular ratio from the beginning of cracks calculation, the results are incorrect and unsafe.

1 Introduction

Serviceability limit states are applied to ensure the functionality of structural members under service loading conditions. Three parts of serviceability limit states control are interconnected by stresses calculation and consideration of time dependent volume changes. For limitation of stresses in structural materials, there is no recommendation for creep effect consideration in EN 1992. The model for direct crack width calculation (for crack width check) makes provision for creep effect by a factor for long term loading. For deflection check, the effective modular ratio can be used to include creep effect in bending stiffness calculation.

2 Consideration of Time Dependent Volume Changes

In following paragraphs the consideration of time dependent volume changes in three serviceability limit states checks is analyzed. It is very important to look after the parameters for creep consideration in particular calculations, because in some cases, these steps can lead to incorrect and unsafe results.

2.1 Deflection control

Structural mechanics principles for deflection calculation are based on elastic deformation of a cross-section. The behavior of reinforced concrete cross-section, when cracks occur and long-term loading is applied, is different. Heterogeneous composition of the reinforced concrete cross-section is reflected by an idealized cross-section, in which the amount of steel reinforcement is replaced with concrete and different material properties are calculated with the use of the effective modular ratio:

$$\alpha_{eff} = \frac{E_s}{E_{c,eff}} \tag{1}$$

where
E_s is the modulus of elasticity of steel reinforcement

E_{ceff} is the effective modulus of elasticity of concrete:

$$E_{c,eff} = \frac{E_{cm}}{1+\varphi(\infty,t_0)} \tag{2}$$

where
E_{cm} is the secant modulus of elasticity of concrete
$\varphi(\infty,t_0)$ is the creep coefficient relevant for the load and time interval

With the effect of creep during a long-term loading, the final deflections increase. After the cracks occur and long-term loading is applied, the deflection increases several times compared to the initial elastic deformation. For example for a one way slab, the final deflection is about five or six times bigger.

2.2 Crack width control

A crack width according to direct calculation depends on the maximum crack spacing ($s_{\text{r,max}}$) and the difference between mean strain in the reinforcement and the mean strain in concrete between the cracks (ε_{sm} - ε_{cm}). The theoretical model for direct crack width calculation is derived for the immediate crack position and width and the long-term loading is reflected by the factor dependent on duration of the load (k_{t}).

For calculation of geometrical properties of a solved cross-section and a tensile stress in steel reinforcement, the basic modular ratio α_{e} should be used:

$$\alpha_e = \frac{E_s}{E_{cm}} \tag{2}$$

To use an effective modular ratio here and calculate with creep effect in the modification of the modulus of elasticity of concrete can lead to incorrect and unsafe results.

This is an example of a one way slab, 200mm in thickness, reinforced with 5 bars of diameter 14mm. In model A – the crack width was calculated using the basic modular ratio (α_{e}=6.4) for determination of stresses in steel reinforcement, as it is correct according to assumptions in a code. In model B – the modified effective modular ratio (α_{eff}=25.7) was used for stresses calculation. The results are in Table 1.

Table 1. Results of an example of crack width calculation

Model	Shrinkage included	Crack width	Difference
A	no	0.112mm	
B	no	0.098mm	B:A= -11.9%
A	yes	0.210mm	
B	yes	0.180mm	B:A= -14.4%

From Table 1 it can be seen, that the consideration of creep effect in direct crack width calculation can lead to results markedly more than 10% on the unsafe side.

2.3 Stresses limitation check

Values of stresses in materials in a reinforced concrete cross-section are controlled for following reasons:

- Compressive stress in concrete under characteristic combination is limited in order to avoid longitudinal cracks and micro-cracks, such cracking may lead to a reduction of durability. The limit value is $k_1 \cdot f_{ck}$ (recommended value of k_1=0.6).
- Compressive stress in concrete under quasi-permanent combination of loads is limited in order to avoid high levels of creep. The limit value is $k_2 \cdot f_{ck}$ (recommended value of k_2=0.45).
- Tensile stress in reinforcement under the characteristic combination of loads is limited in order to avoid inelastic strain, unacceptable cracking or deformation. The limit value is $k_3 \cdot f_{ck}$ (recommended value of k_3=0.8). Values for imposed deformation and prestressing tendons are limited to 1 and $0.75 \cdot f_{ck}$.

In EN 1992, there is no recommendation for creep effect consideration when checking stress limitations, despite this fact mostly effects on results of stress values. Four one way slabs were chosen for comparison of stresses calculation with or without creep effects. The results are shown in Table 2.

Table 2. Comparison of stresses in materials

Slab	Compressive stress in **concrete** **without** creep effect [MPa]	Compressive stress in **concrete** **with** creep effect [MPa]	Tensile stress in **reinforcement** **without** creep effect [MPa]	Tensile stress in **reinforcement** **with** creep effect [MPa]
A	11.29	6.69	287.77	304.14
B	10.33	6.14	236.83	251.77
C	7.69	4.94	291.63	304.14
D	9.32	5.34	260.18	275.60

From Table 2 it can be seen, that the consideration of creep effect in calculation of stresses in concrete leads to underestimation of these values, whereas the stresses in reinforcement are higher of more than 5% when considering creep effect.

The most precise calculation has to take into account the creep effect only for the long term part of loads; effects of remaining loads should be calculated without consideration of creep effect (Table 3).

Table 3. Stresses in materials including creep effects for long term loads

Slab	Ratio of long term part of loads	Compressive stress in **concrete** [MPa]	Tensile stress in **reinforcement** [MPa]
A	73.6%	7.9	299.8
B	74.6%	7.2	248.0
C	79.1%	5.0	301.5
D	83.8%	6.0	273.1

The values of stresses from Table 3 are between the values calculated with and without the consideration of creep effect for both concrete and reinforcement. Dividing the effects of load to two parts (long term and short term) and then calculating the stresses separately for both parts is a complicated process. The best way is to find a modular ratio that mostly approximates the results of detailed calculation. Found modular ratios are summarized in Table 4.

Table 4. Modular ratio α_{approx}

Slab	Ratio of long term part of loads	Approximate modular ratio α_{approx}	Effective modular ratio α_{eff}	α_{approx} / α_{eff}
A	73.6%	**17.7**	23.8	74.3%
B	74.6%	**18.5**	24.7	74.9%
C	79.1%	**19.2**	24.1	79.5%
D	83.8%	**22.4**	26.9	83.3%

From the results in Table 4 it can be seen, that the approximate modular ratio α_{approx} is almost the same proportion of effective modular ratio α_{eff} as the ratio of long term part of loads from total loads. The difference is not more than 1%. So the most accurate calculation of stresses in materials can be performed by the consideration of approximate modular ratio.

2.4 Conclusion

The consideration of time dependent volume changes in verification of serviceability limit states plays a very important role. Final deflection increases about five times when creep effect is taken into account. For crack width control, the calculation model is derived for the immediate crack position and width and the long term loading is reflected by the factor dependent on duration of the load. Here, the consideration of creep effect for calculation of stresses in tensile reinforcement can lead to unsafe design.

The most complicated for creep effect consideration are the stresses checks. For compressive stresses in concrete, the biggest values are achieved at the beginning of loading - calculated without creep effect and these values should be checked with the limit ones. For steel reinforcement, the tensile stresses calculated with basic modular ratio (without creep effect) are on the unsafe side. Gradually with time the tensile stresses in reinforcement increase. Their values calculated with the effective modular ratio are a bit oversized. The most accurate values are achieved by dividing the calculation to two parts – for long term loads assumed with effective modular ratio and for short term loads with basic modular ratio. The simplification can be performed by the consideration of approximate modular ratio, which can be calculated like the same proportion of effective modular ratio as the rate of long term part of load from the total load.

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Design of concrete structures for durability

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ABSTRACT: To make the design adequate for concrete structures against environmental effects, many factors should be considered, mainly the material parameters and environmental actions that cause concrete and reinforcement deterioration. In the paper the service life is modelled as the initiation and propagation of reinforcement corrosion, distinguished serviceability and structural failures. The equations of failure probability for carbonation and crack opening due to reinforcement corrosion are presented. For different crack width levels there are different consequences possible as regards the reinforcement area reduction, spalling and changes in bond strength.

1 INTRODUCTION

Design of concrete structures is currently focused especially on the effects of direct actions. Increasingly the consequences of indirect, accidental and environmental actions are manifesting. Long term exposure to environmental actions (chemical, biological and physical effects of the environment), causes deterioration of concrete and reinforcement. When considering the reliability of structures all types of actions should be taken into account. This holistic approach to the design and verification of structures shall be applied to all constructions, especially civil engineering works, because of their large ratio between the area exposed to the surrounding environment and cross-section dimensions as well as longer design life.

One of the most predominant factors responsible for the structural deterioration in concrete structures is identified as a corrosion of reinforcement, which may result in damage of the structures in the form of expansion, cracking and eventually spalling of the cover concrete. In addition to this, the structural damage may be caused by loss of bond between reinforcement and concrete and loss of reinforcement sectional area; sometimes to the extent that the structural failure becomes inevitable (Mehta 1997). While much research effort has been devoted to researching the causes and mechanisms of reinforcement corrosion, relatively little attention has been devoted to the problem of assessing the residual reliability with respect to cracking and spalling of concrete cover associated with reduction in bond strength.

2 ENVIRONMENTAL ACTIONS

In contrast to presently used design rules, the design of structures for durability should be based upon realistic and sufficiently accurate environmental actions, material parameters and degradation models. An environmental action, such as corrosion, decay or shrinkage, is a chemical, electrochemical, biological, physical or mechanical action causing material deterioration or deformation which is not considered as loads in structural design. Except for mechanical action, an environmental action is the consequence of the expected environmental agents, such as moisture, oxygen and temperature, the chemical, electrochemical and physical properties of the materials of the components, and the interaction of the different components, including electrochemical and physical interactions. Environmental actions, such as corrosion of steel, shrinkage or freeze-thaw of cement-based materials such as masonry or concrete, can result in loss of performance (ISO 13823 2008).

Environmental action is caused by micro-environmental agents, acting alone or in combination, over time, resulting in deterioration or deformation of a given material. These agents generally occur as a result of influences in the structure environment outside or within the structure being transferred on, into or through an assembly of components. Although little can be done to modify the structure environment (except indoors), the agents causing degradation can be controlled. Their control can be achieved from a fundamental understanding of the transfer mechanisms. The mechanisms of most concern are those that govern the flow of water, air, and heat.

In designing for durability, one can either select a material to resist degradation caused by the expected environmental action, or the environmental action must be altered to suit the material.

3 REINFORCEMENT CORROSION

3.1 *Criteria for reinforcement corrosion*

The corrosion of the reinforcement is caused by the carbonation of concrete and/or the chlorides penetration into the concrete. The end of technical service life in a structure subject to deterioration caused by reinforcement corrosion might potentially be defined by one, or possibly more, of the following limit states, related to the Serviceability Limit States (SLS) or Ultimate Limit States (ULS):
SLS 1: depassivation of reinforcement
SLS 2: appearance of cracking
SLS 3: reinforcement area reduction
ULS 1: excessive reinforcement and bond reduction
ULS 2: spalling with risk of falling pieces
ULS 3: the probability of failure under the design exceeding a predetermined value.

The service life is modelled as the initiation and the propagation of reinforcement corrosion, consequences of reinforcement corrosion and finally serviceability and structural failures (Figure 1). Corrosion may be initiated if the concrete around the reinforcement is carbonated and/or if chlorides reach the reinforcement.

Figure 1 shows in principle the performance of a concrete structure with respect to reinforcement corrosion and related events. Depassivation, small reinforcement cross section reduction and cracking represent events related to the serviceability, while excessive loss of steel cross section, spalling and bond reduction represent events related to the structural failures.

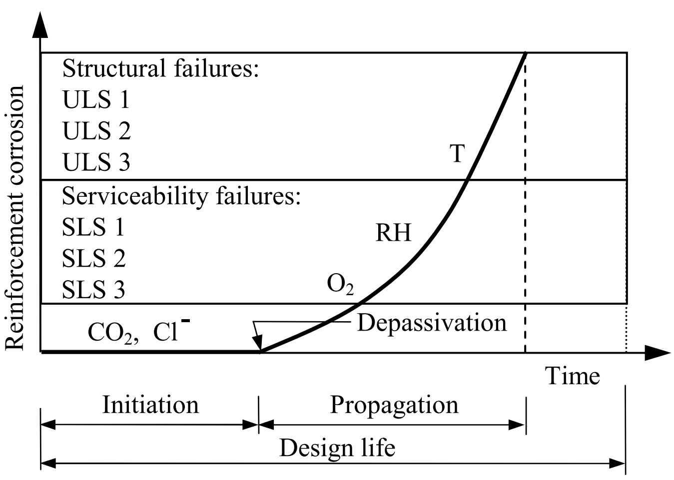

Figure 1. Service life of concrete structures due to the reinforcement corrosion.

A preliminary review of the service life of concrete structures suffering from corrosion leads to the general conclusion that a service life is considered finished when the depassivation of the bar is reached. This statement sometimes seems conservative, and in consequence, other authors prefer to identify the service life with the time for cover cracking to appear (Alonso1998). It is obvious that in any case, there is a clear need to distinguish between Ultimate and Serviceability Limit States when assessing the service life of a corroded structure.

3.2 *Probabilistic models*

A probabilistic approach is very appropriate to deal with problems with uncertainty and randomness which is the nature of the corrosion process and its effect on structural response (Li 2004).

With the formulation in Figure 1, the time period for each phase of service life of RC structures can be determined once a performance-based assessment criterion is established. This criterion can be the probability of failure P_f or alternatively reliability index β. The probability of failure describes the case when a variable resistance R is lower than a variable effect of load E. Generally, both R and E (and hence P_f and β) are time dependent. The failure probability P_f is required to be lower than the target probability of failure P_{target} expressed in Equation 1 as follows

$$P_f(t) = P\ \{R(t) - E(t) < 0\} < P_{target} \qquad (1)$$

where P denotes the probability of an event; t = time; P_{target} = target failure probability.

3.3 *Deterioration model - carbonation*

Carbonation of concrete is a chemical process causing a decrease of the pH value of the pore solution. Corrosion is initiated when the carbonation front reaches the reinforcement. By considering the depassivation of the reinforcement due to carbonation, the concrete cover is defined as the resistance and the carbonation depth as the load. As the carbonation depth increases with time, this load variable has to be defined as time dependent. The probability of corrosion initiation at a given time follows t, $P_{f,i}(t)$ due to carbonation may be written as

$$P_{f,i}(t) = P\{a - x_c(t) \le 0\} \le P_{target} \qquad (2)$$

where a = concrete cover; x_c = depth of carbonation at time, see Figure 2.

Full probabilistic models for carbonation induced reinforcement corrosion are available (fib 2006), where the concrete cover is compared to the carbonation depth at a certain time. In a probabilistic approach the variables a and $x_c(t)$ need to be quantified.

When depassivation of the reinforcement due to the carbonation is the relevant limit state (LS), it does not lead to severe consequences and the LS can be allocated with a corresponding relaxed target reliability level for failing, often in the order of 10^{-1} to 10^{-2} (fib 2010b).

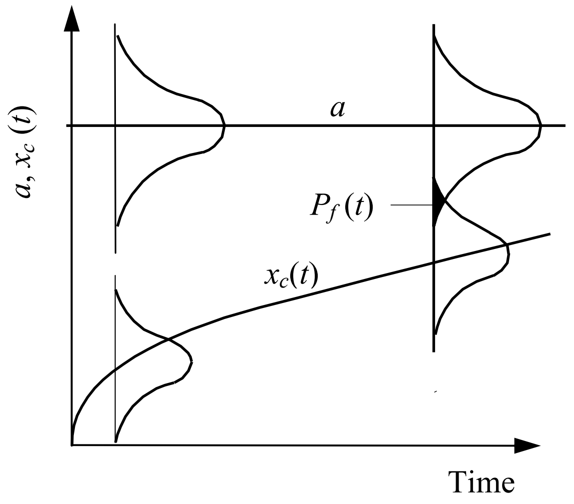

Figure 2. Relationship of carbonation $x_c(t)$ and cover depth a.

3.4 *Deterioration model - cracking*

The durability of concrete structures may be adversely affected by excessive cracking. Besides this, cracking shall be limited to a level that will not impair the proper operation and durability of the structure or cause its appearance to be unacceptable (Litzner 1999).

The corrosion of reinforcement results in the formation of various corrosion products due to the process of oxidation and causing an increase in the volume. Depending on the level of oxidation this volume increase would be up to about 6.5 times the original iron volume which gets consumed by the corrosion process. This volume increase generates radial stress (pressure) on the surrounding concrete at the interface between the reinforcement and concrete. The radial stress is the principal cause of the concrete expansion and ultimately the cover cracking along the line of bar when the maximal tensile stress σ_t exceeds the tensile strength of the concrete cover f_{ct}. Based on this process, the probability of the serviceability failure due to corrosion induced concrete cracking, $P_{f,c}(t)$,can be determined from Equation 3 as follows

$$P_{f,c}(t) = P\{\sigma_t(t) \geq f_{ct}\} \leq P_{target} \tag{3}$$

The reinforced concrete modelled as a thick wall cylinder under radial compressive stress σ_c, which generates the circumferential tensile stress σ_t, is shown in Figure 3. The dimensions of the concrete cylinder are determined by the least value c_x and c_y.

Another model for the reinforcement corrosion induced cracking and spalling of cover concrete is presented in (fib 2010b):

$$P_{f,c}(t) = P\{\Delta r_{(R)} - \Delta r_{(E)}\,(t_{SL}) < 0\} \leq P_{target} \tag{4}$$

where $\Delta r_{(R)}$ = maximal corrosion induced increase of the rebar radius which can be accommodated by the concrete without formation of cracks; $\Delta r_{(E)}$ (t_{SL}) = increase of the rebar radius due to reinforcement corrosion; t_{SL} = design service life.

A corrosion penetration of 0.05 mm for general corrosion, which is the average value found in literature to induce corrosion, represents a reduction of less than 2.5% of cross sectional area for an 8 mm diameter bar, and less for larger bars. Clearly, cracking will develop well before loss of bar section becomes significant. The relationship between surface crack width and corrosion penetration shows a mean surface crack width of 1 mm to be associated with an attack penetration of around 0.2 mm (Rodriguez at al. 1994).

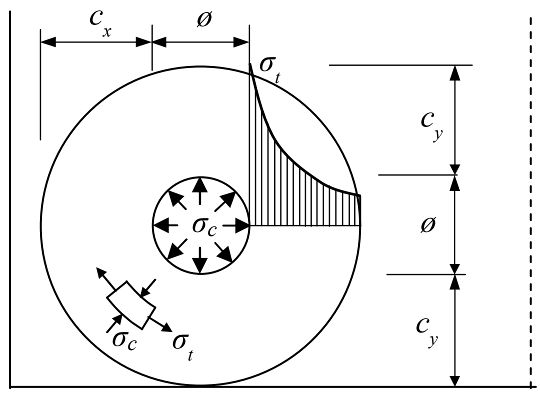

Figure 3. Stresses induced by expansive corrosion products: σ_c – radial compressive stress, σ_t – circumferential tensile stress.

3.5 *Deterioration model - spalling*

Another response related to structural failures is concrete spalling. The occurrence of spalling depends mainly on the following factors (Hunkeler 2006):

- reinforcement: cover thickness, diameter, spacing, position of the rebar and type of rebar
- corrosion: cause, corrosion attack (loss of cross section), corrosion rate, corroding area or length of the rebar, rust modification
- exposition: moisture content of the concrete , temperature, wetting and drying cycles
- concrete: concrete quality, mechanical properties, existing cracks.

For the assessment of the risk of spalling not only the general and corrosion related influences mentioned above have to be considered but also specific factors depending on the structure as a whole and its components.

In Table 1 there are the estimates for the necessary removal of spalling with those for the cracking compared. Compared to cracking in concrete cover, spalling needs approximately 15-times higher corrosion rate.

Table 1. Comparison of the minimum required corrosion rate for cracking and spalling (Hunkeler 2006).

Cover thickness/	Required corrosion rate in mm for	
Bar diameter	Cracking	Spalling
0.5	0.006	0.08
1.0	0.011	0.17
2.0	0.022	0.33
3.0	0.033	0.50

In various studies an attempt has been made to find the minimum distance between the reinforcing bars, which results that between the corroding rebars no interaction occurrs. (Capozucca 1995), for example, assumes in his calculations that this the case is, at least when the mutual distance between the rebars *s* 6 times the rebar diameter *d* is. If $s > 6\ d$ occurred over the rebar conical spalling with an angle of about 45 °. When $s < 6\ d$ preferred separations occur in the plane of the reinforcement.

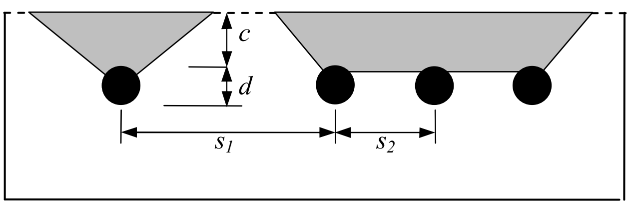

Figure 4. The geometric parameters affecting the spalling (Hunkeler 2006).

3.6 *Deterioration model- reinforcement reduction*

No models with broad international consensus are available for predicting the length of the corrosion period till cracking, spalling, loss of bond or collapse of the structure occurs.

Dissolution of iron from steel reinforcement results in a loss of bar cross section which may either be predominantly uniformly distributed over the length and circumference of the bar (general corrosion) or show concentration at localized sites (pitting corrosion).

Uniform corrosion is generally caused by carbonation of concrete. The residual cross sectional area A_{res} may be evaluated by:

$$A_{res} = A_0 - A_{corr} = \pi.(d_b - 2p(t))^2\ /4 \qquad (5)$$

where A_0 = original cross section area; A_{corr} = loss in cross section area; d_b = original bar diameter; $p(t)$ = corrosion penetration depth (bar radius reduction).

Local or pitting corrosion is invariably associated with chloride contamination and not with carbonation. In local corrosion, the area of the anode may be relatively small, and hence rate of loss of metal will be relatively high. Because the corrosion rate is rapid and the supply of oxygen restricted, the products of the corrosion reactions exhibit a lower degree of volumetric expansion, and the tendency to split the concrete cover is consequently less. Extreme loss of bar section may occur without external visual signs of cracking, although surface staining will usually be noticeable.

Corrosion effectively reduces not only the bar section area, but also the strength of the steel. At the present time, models for loss of strength and ductility are confined to empirical correlations with section loss, expressed as a percentage of original cross section:

$$f_y = (1{,}0 - \alpha_y.A_{corr}).f_{y0} \qquad (6)$$

$$f_u = (1{,}0 - \alpha_u.A_{corr}).f_{u0} \qquad (7)$$

where f_y, f_u = yield strength, ultimate tensile strength in corroded state; f_{y0}, f_{u0} = yield strength, ultimate tensile strength; α_y, α_u = regression coefficients (reported between 0.05 to 0.16) (LIFECON 2003). Force in a bar can increased until fracture occurs at the ULS at the reduced section. The effect of reinforcement section loss on residual strength may be estimated using conventional calculation procedures, but allowing for the reduction in cross sectional area of the bar and of the reduction in 'apparent' yield strength.

3.7 *Deterioration model - bond reduction*

Bond is necessary to anchor reinforcement and to ensure composite interaction between reinforcement and concrete. The reduction in confinement on cracking of the cover will lead to a progressive reduction in bond strength. Magnitudes of the bond strengths reported and the effect of corrosion on those bond strengths differ widely. Despite wide variations in test specimens and in conditioning techniques, the general reported trends are the same in almost all studies, as illustrated in Figure 5.

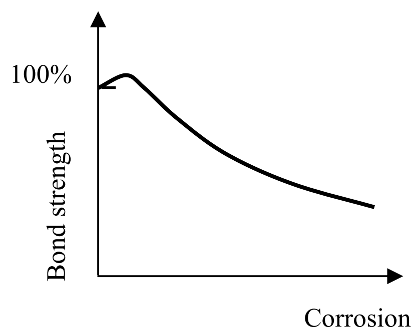

Figure 5. Variation in bond strength with corrosion (LIFECON 2003).

The same pattern is observed in tests on plain and ribbed surface bars. Initially, bond strength is increased by small amounts of corrosion, but with further increases, bond starts to reduce. It appears, however, that bond strength does not reduce below the 'as new' value prior to development of externally visible longitudinal cover cracks. For purposes of assessment of residual strength of concrete structures suffering from general corrosion, bond can be assumed to be sound in the absence of visible corrosion induced cracking. Once cracking develops, appreciable loss of bond strength may develop, particularly if no confining reinforcement is present (Lay & Schiessl 2003).

Studies conducted by (Auyeeun 2001) have confirmed that the loss of bond strength for unconfined reinforcement is much more critical than bar section area loss; that is, a low diameter loss could lead to 80% bond reduction. Study also showed that confinement provides excellent means to counteract the bond loss.

If the ratio ρ_{tr} of the transverse reinforcement area at anchorage length (considering the reduction due to corrosion) versus the area of the main bars is higher than 0.25 (minimum value established in EC2), the bond strength f_b (in N/mm²) can be predicted as follows (fib 2000):

$$f_b = 4.75 - 4.64.p(t) \tag{8}$$

where $p(t)$ = corrosion penetration depth.

This proposal gives bond strength values for each attack penetration, taking account the actual residual stirrup section at the anchorage length.

3.8 *Model - structural deterioration*

The last phase of service life is the period of time from loss of serviceability to final collapse of the structure. Few models exist for the structural strength deterioration. Also, failure can occur in many modes, such as loss of both flexural strength and shear strength. The recommended target reliability indices β for ULS verification, related to specific reference periods and consequences of failures given in (fib 2010b). The target reliability level for the existing structures may be chosen lower than for new structures.

4 CONCLUSIONS

The following conclusions can be drawn:

1. Significance of the environmental actions. The exposure environment has been found to have a large influence on the reliability of concrete structures, especially for civil infrastructure, because of their extended service life and great area exposed to the surrounding environment.

2. Deterioration models. Models for carbonation and chloride induced reinforcement depassivation are available (e.g. in fib 2006), in which the concrete cover is compared to the carbonation or critical chloride concentration at the rebar surface.

3. Structural and serviceability failures. For given degrees of corrosion rate the risk for cracking, spalling and bond strength decrease depends mainly on the geometry of the cross section and the confinement (transverse reinforcement area).

4. Significance of probability methods. The scatter of properties of structural materials and their response to the environment are so large, that they cannot be neglected. In partial factors method they are not taken into account to the necessary extent, but are subjected to explicit observation in performance based methods. More research needs to be devoted to modelling of the effect of reinforcement corrosion for structural response of concrete structures.

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Resistance of Columns Subjected to Axial Load

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Abstract. The paper deals with columns predominantly subjected to axial loads. Ultimate resistance of columns in critical cross-section is determined and verified according to EN 1992-1-1 and national codes ON B4700:2001; BS 8110-1; STN 731201/86, ON B 4200 and DIN 1045:1988. Recommended is the modification of Clause 6.1(4) EN 1992-1-1 after this study to erase the possible mistakes and presented ambiguities.

1 Introduction

One of the conditions for the ultimate limit state design of cross-sections subjected to bending moments with or without axial force is the condition in section 6.1 [1]:

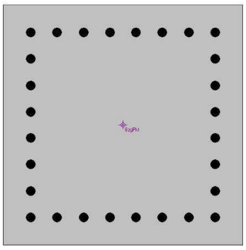
(4) For cross-sections loaded by the compression force it is necessary to assume the minimum eccentricity $e_0 = h/30$ but not less than 20 mm where h is the depth of the section.

The aim of this paper is to analyze the condition for the design of reinforced and prestressed concrete cross-sections.

2 Reliability of Centric Loaded Reinforced Concrete Cross-Sections

Effect of the condition for reduction of max resistance, represented by axial force and defined in [1] Chapter 6.1 (4) is graphically presented on reinforced concrete cross-sections in Fig.1 and Fig.2. This condition is suitable for symmetrical cross-sections, which are reinforced with symmetrical reinforcement. In a case of unsymmetrical cross-section or section reinforced with asymmetrical reinforcement, this requirement is not correct. In some cases the application of this condition leads to a decrease in reliability of design. Interaction-diagrams in Fig.1 and Fig.2 illustrate the possible ambiguity by definition of max resistance by min eccentricity. There are other examples of max resistance definition by different codes, previous valid in Germany, Austria, Great Britain and Czechoslovakia.

Cross Section 1: Square cross-section – with symmetrical reinforcement (see Fig. 1):



Dimensions a = 600 mm
Concrete C50/60 f_{cd} = 33 [MPa]
Steel B 460B f_{yd} = 400 [MPa]
Reinforcement 28 ϕ 25
Reinforcement ratio 3,8 %

$N_{Rd,max}$ = 17500 kN is the axial resistance force without eccentricity
$N_{Rd,EN}$ = 16000 kN is the resistance force with eccentricity e_0 [1]
$N_{Rd,ON\ B4700}$ = 13650 kN is the resistance force with eccentricity h/10 by [2]
$N_{Rd,STN}$ = 14381 kN is the max resistance force by STN 73 1201 [3]
$N_{Rd,BS}$ = 11323 kN is the max resistance force by BS 8110 [4]
$N_{Rd,DIN\ 1045:88}$ = 11666 kN is the max resistance force by DIN 1045 [5]
$N_{Rd,ON\ B4200}$ = 9831 kN is the max resistance force by ON B 4200 [6]

The Germany code DIN 1045:88 (valid untill 2001) and the Austrian code ON B 4200 (valid until 2001) belong to the group of codes with philosophy of design by global safety factor.

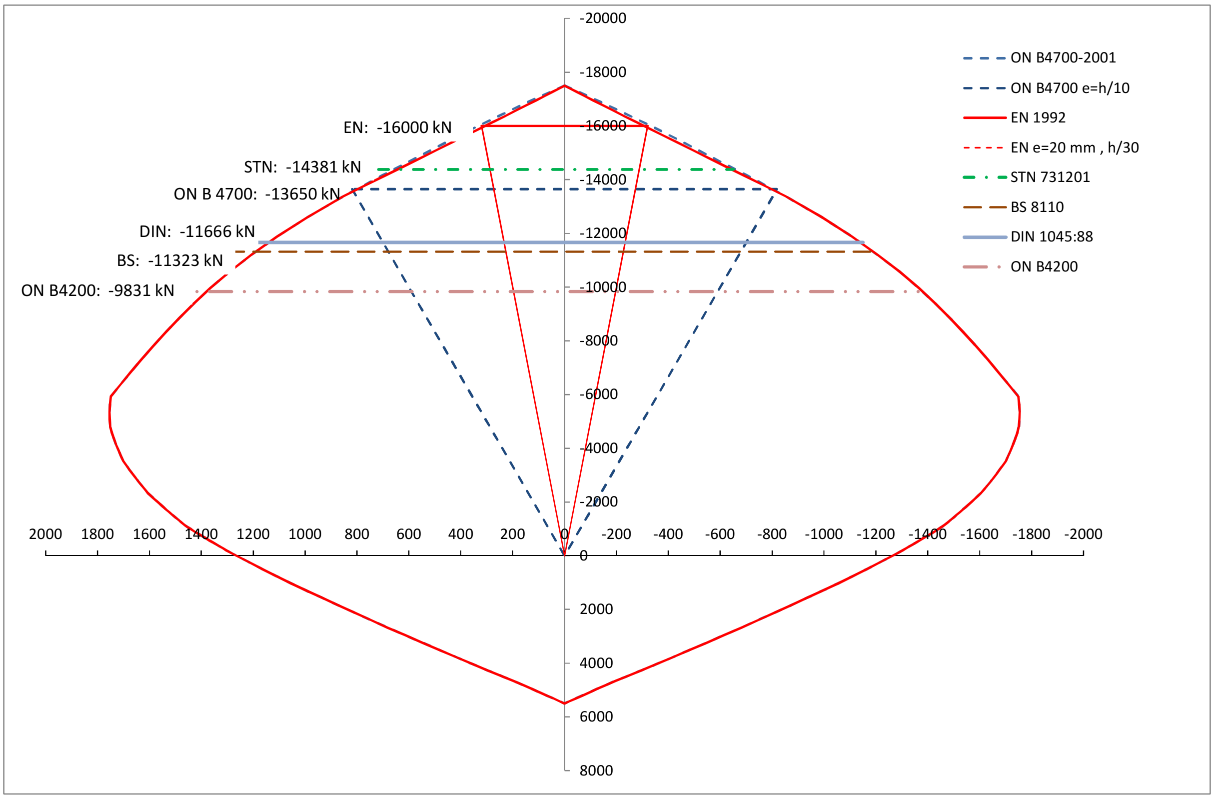
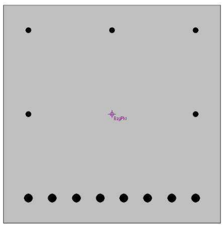


Fig. 1 Interaction-diagram - nonsymmetrical reinforcement 28ϕ25; B 460B; C50/60
a) reduction of axial force by the minimal eccentricity e_0 [1]
b) reduction of axial force by the eccentricity *h/10* [2]
c) reduction of axial force by the STN 72 1201-CL.5.2.7.3[3]
d) reduction of axial force by the BS 8110-1-Cl.3.8.4.3 [4]
e) reduction of axial force by the DIN 1045:88 [5]
f) reduction of axial force by the ON B 4200 [6]

For a possibility to compare these codes based on the safety factor with codes based on the semiprobalistic philosophy the authors put a part of the global probabilistic coefficient (value 1,4) on the loads side of verification what correspond dead versus live load ratio about 70:30.

Differences by design of columns predominantly subjected to axial force are evident from the diagram N-M on Fig. 1. The considerable differences in the reliability of design are in codes based on the method of the global reliability. By a cross-section defined in paper the lowest reliability represent EN 1992-1-1[1]. It is possible to increase reliability by reduction of concrete resistance in NAD by coefficient α_{cc}= 0,85 but by this step also the resistance of columns with prevailing bending moment would be reduced..

Cross Section 2: Square cross-section – with non-symmetrical reinforcement (see Fig 2):



Dimensions a = 600 mm
Concrete C50/60 f_{cd} = 33 [MPa]
Steel B 460B f_{yd} = 400 [MPa]
Reinforcement 8 ϕ 25 + 5 ϕ 16
Reinforcement ratio 1,4 %

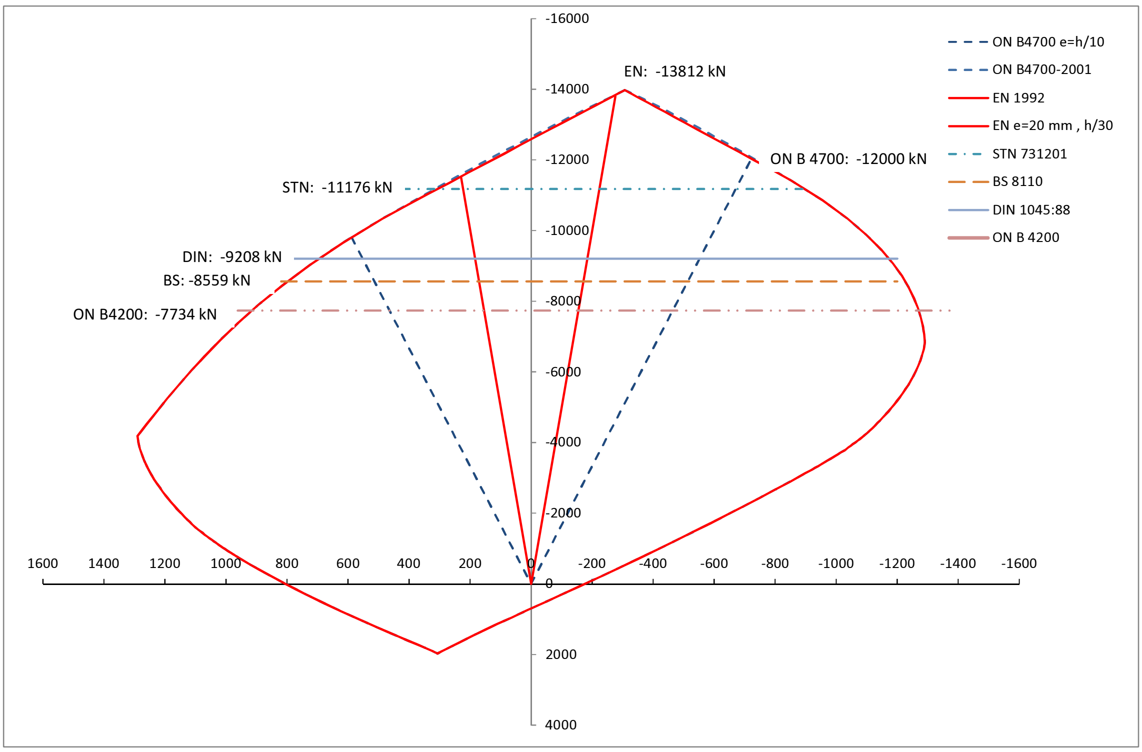


Fig. 2 Interaction-diagram - nonsymmetrical reinforcement 8ϕ25 + 5ϕ16; B 460B; C50/60

a) reduction of axial force by the minimal eccentricity e_0 [1]
b) reduction of axial force by the eccentricity *h/10* [2]
c) reduction of axial force by the STN 73 1201:1985 [3]
d) reduction of axial force by the BS 8110-1:1997 [4]
e) reduction of axial force by the DIN 1045:88 [5]
f) reduction of axial force by the ON B 4200 [6]

$N_{\mathrm{Rd,max}}$ = 13812 kN is the axial resistance force without eccentricity
$N_{\mathrm{Rd,EN}}$ = 13800 (11500) kN is the resistance force with eccentricity e_0 [1]
$N_{\mathrm{Rd,ON\ B\ 4700}}$ = 12000 (9800) kN is the resistance force with eccentricity h/10 by [2]
$N_{\mathrm{Rd,STN}}$ = 11176 kN is the max resistance force by STN 73 1201 [3]
$N_{\mathrm{Rd,BS}}$ = 8559 kN is the max resistance force by BS 8110 [4]
$N_{\mathrm{Rd,DIN\ 1045:88}}$ = 9208 kN is the max resistance force by DIN 1045 [5]
$N_{\mathrm{Rd,ON\ B4200}}$ = 7734 kN is the max resistance force by ON B 4200 [6]

The considerable differences of the global reliability of unsymmetrical reinforced cross-section defined in example 2 are evident for the presented codes from the Fig.2.

The definition of the minimal eccentricity e_0 according to [1] for the symmetrical cross-section and unsymmetrical reinforcement does not give a true picture about reliability design of the cross-section. For some cases of the cross-section design, the requested reliability is even decreasing.

Conclusion

The paper deals the reliability of columns predominantly subjected to axial loads. The ultimate resistance of critical cross-sections are determined and demonstrate on Fig. 1 and 2 (two examples of cross-sections symmetrically and non-symmetrically reinforced). Interaction diagrams presented on Fig.1 and 2 show a verification of max axial resistance by EN 1992-1-1 and by the other national codes (ON B4700:2001; BS 8110-1; STN 731201/86, ON B 4200 and DIN 1045:1988).

Recommended is the modification of Clause 6.1(4) EN 1992-1-1 after this study to erase the possible mistakes and presented ambiguities. A proposal can be applied with definition of max level of axial force resistance and can be valid for cross-sections for general design, even for the biaxial bending with axial force. This proposal takes into account the increased level of reliability in cases the materials are utilized to the full plastic value. The application of such proposal is even more important for reinforcement steel with yield strength less than 460 MPa.

Apart this proposal, defined above, authors of the contribution recommend also substituting the definition of the minimum eccentricity e_0, with final eccentricity, which takes into account also the influence of second order effects.

Acknowledgements

The paper has been partially supported by Slovak grant agency VEGA No. 1/0857/11.

References

[1] EN 1992-1-1: 2006 *Design of concrete structures.* CEN Brussels 2004. 225pp.

[2] ON B4700-2001 *Stahlbetontregwerke EUROCODE-nahe Berechnung, Bemessung und konstruktive Durchbildung.* ON Wien 2011 .

[3] STN 73 1201:1986 *Design of concrete structures.* Praha 1986. 282 pp.

[4] BS 8110-1:1997 *Structural use of concrete*. BSI 30 NOV 2005. 159 pp.

[5] DIN 1045:1988 *Stahlbeton- und Spannbetonbau*.DIB Berlin 1988. 96 pp.

[6] ON B 4200 Teil 9: Stahlbetontragwerke, Berechnung und Ausführung.

VIII. TEACHING

VIII.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 (in Slovak)	4.	2 – 2	J. Bilčík
Design of Concrete Members (in English)	4.	3 – 2	J. Bilčík
Reinforced and Prestressed Members	5.	2 – 2	Ľ. Fillo
Reinforced Concrete Structural Members	5.	3 – 2	J. Halvoník, Š. Gramblička
Reinforced Concrete Structural Members	5.	3 – 2	V. Benko
Design of Concrete Structures I	6.	2 – 2	I. Hudoba
Reinforced Concrete Structural Systems	6.	2 – 2	I. Harvan, V. Borzovič
Reinforced Concrete Structural Systems	6.	1 – 1	V. Benko

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 Concrete Structures	2.	2 – 2	J. Bilčík
Design of Concrete Bridges I	2.	2 – 2	J. Halvoník
Special Problems of Concrete Structures	2.	2 – 2	I. Harvan
High – Rise and – Span Structures	2.	2 – 2	I. Harvan
High – Rise and – Span Structures	2.	2 – 2	Š. Gramblička
Design of Composite Structures	3.	2 – 2	Š. Gramblička
Experimental Testing of Concrete Structures	3.	0 - 3	V. Priechodský

Optional Subjects

Bachelor's degree study	Semester	Hours per Week Lectures Seminars	Lecturer
Reinforced Concrete Structural Systems 2	6.	2 – 1	I. Harvan
Design of Concrete Structures	8.	2 – 2	I. Harvan

Master's degree study	Semester	Hours per Week Lectures Seminars	Lecturer
High – Rise and – Span Structures	2.	2 – 2	Š. Gramblička
High – Rise and – Span Structures	2.	2 – 1	I. Harvan
Design of Composite Structures	3.	2 – 2	Š. Gramblička
Execution of Concrete Structures	3.	2 – 2	I. Hudoba
Design of Concrete Bridges II	3.	2 – 2	J. Halvoník
Masonry Bearing Structures	4.	2 – 1	M. Čabrák
Execution of Concrete Structures	4.	2 – 1	I. Hudoba

Recommended Subjects

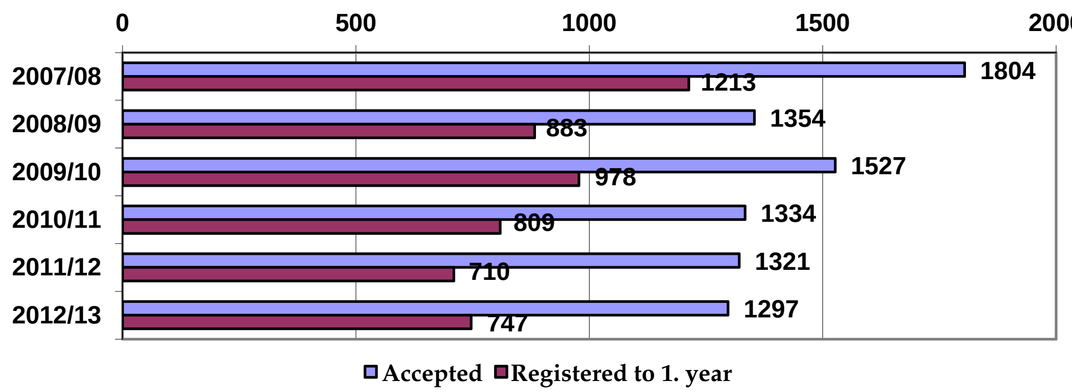
Master's degree study	Semester	Hours per Week Lectures Seminars	Lecturer
Modelling of RC Structural Systems I	1.	2 – 2	I. Harvan
Modelling of RC Structural Systems II	1.	2 – 2	I. Harvan
Modelling of RC 2D Structures	2.	2 – 2	J. Šoltész
Modelling of RC 3D Structures	2.	2 – 2	J. Šoltész

Postgraduate courses

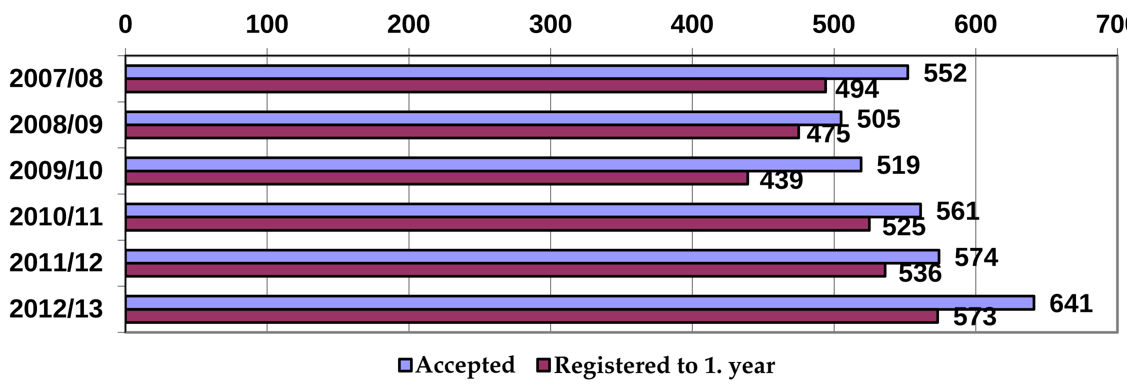
Advanced Concrete and Masonry Structures
Advanced Concrete Bridges
Reliability and Strengthening of Concrete Structures
Modelling of Concrete Structures
Structural Materials and Systems
Experimental Testing of Concrete Structures
Advanced Reinforced Concrete Structures

VIII.2 Acceptance test for the whole Faculty

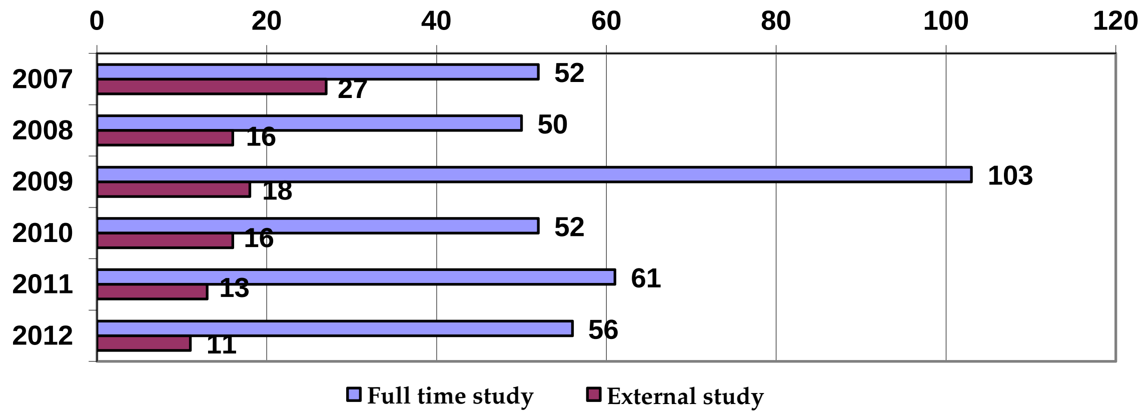
Graph VIII.1 – Number of accepted students and students registered to 1. year of Bachelor study at the whole Faculty



Graph VIII.2 – Number of accepted students and students registered to 1. year of Engineer study at the whole Faculty

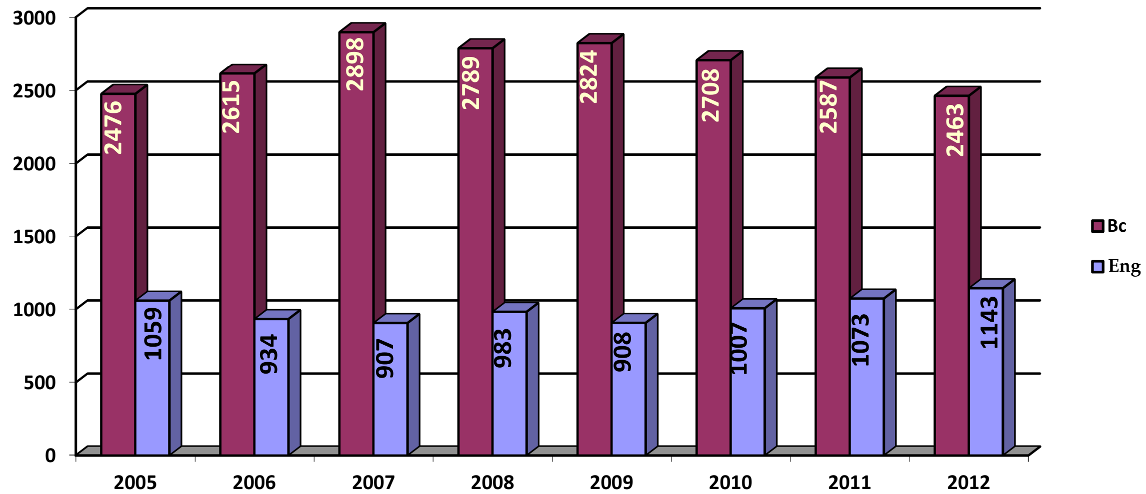


Graph VIII.3 – Number of accepted students for PhD study at the whole Faculty

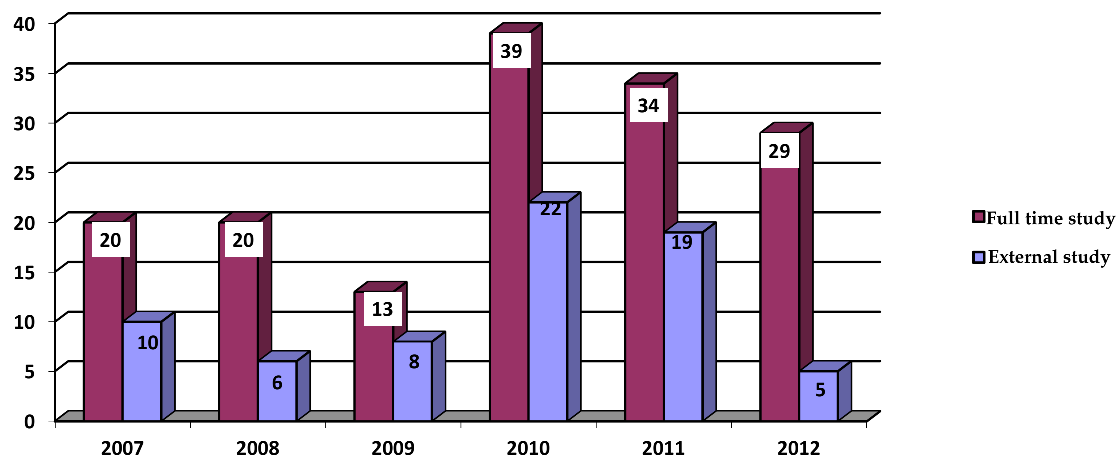


VIII.3 Results of studies for the whole Faculty

Graph VIII.4 Number of students of Bachelor and Engineer study at the whole Faculty



Graph VIII.5 Number of students successfuly finished on PhD study at the whole Faculty



IX. THESES

IX.1 Bachelor Theses

DOVIČOVIČ, J.: Cast-in-Situ Reinforced Concrete Floor Structure of an Office Building
Supervisor: Gramblička, Š.

JÁVOR, P.: Cast-in-Situ Reinforced Concrete Slab Structure of Residential House
Supervisor: Borzovič, V.

PETKOVÁ, V.: Reinforced Concrete Slab Structure of a Residential Building
Supervisor: Borzovič, V.

GURA, L.: Cast-in-Situ Reinforced Concrete Slab of Apartment Building
Supervisor: Borzovič, V.

KUBIŠ, J.: Design of Floor Structure as Cast-in-Situ Structure of Two-Way Floor Slab
Supervisor: Halvoník, J.

KUČERA, M.: Operational Floor Structure Solution Consisting of a One-Way Cast-in-Situ RC Slab
Supervisor: Halvoník, J.

VIDA, R.: Design of Floor Structure Using Lattice Girder Slab Panels
Supervisor: Halvoník, J.

MIŠKOVČÍK, A.: Supporting Structure for the Manufactural Technological Facility
Supervisor: Gramblička, Š.

PTÁK, M.: Engineering Solutions of Cast-in-Situ Floor Slab for Administrative Building
Supervisor: Bilčík, J.

ŠTEFULA, M.: Engineering Solution of Cast-in-Situ Reinforced Concrete Floor Slab for Water Tank of Rectangular Shape
Supervisor: Bilčík, J.

TUŽINSKÝ, M.: Engineering Solution of a Cast-in-Situ Floor Slab of Multi-storey Garage
Supervisor: Bilčík, J.

IX.2 Graduate Theses

BEDNÁR, M.: "Generali" Office Building: Cast-in-Situ RC Combined Bearing Structure with Flat Slabs and Stiffening Cores
Supervisor: Abrahoim, I.

BEUTELOVÁ, D.: Zvolen Shopping Centre: Large-Span Cast-in-Situ Concrete Structure
Supervisor: Gajdošová, K.

BRIŽEK, L.: Hotel: Circular Concrete Building
Supervisor: Benko, V.

BURANSKÝ, S.: Office Building: Hanging Structure
Supervisor: Benko, V.

DALKOVIČ, I.: "Uzbecká-Dolné Hony-Bratislava" Multifunctional Apartment Building: RC Cast-in-Situ Structure with Stiffening Cores
Supervisor: Abrahoim, I.

DÖMÖTÖROVÁ, A.: Multifunctional Building: 21-Storey RC Slab-Wall Combined Structure
Supervisor: Harvan, I.

DRHA, P.: Design of the Bearing System of a Production Factory: Precast Concrete Hall Structure
Supervisor: Šoltész, J.

ĎURICOVÁ, J.: Office Building of an Automobile Centre-Bratislava, Rožňavská: Cast-in-Situ RC Combined Bearing Structure with a Stiffening Core
Supervisor: Abrahoim, I.

FABIAN, P.: Staré Grunty-Bratislava Hotel: Combined Cast-in-Situ RC Structure with a Stiffening Core
Supervisor: Abrahoim, I.

FARAGOVÁ, J.: "Železná Studnička" Apartment House: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Gajdošová, K.

FRÓLO, J.: Load-Bearing Structure of a High-Rise Building Office Center
Supervisor: Gramblička, Š.

HALABRÍNOVÁ, A.: Supporting Structure of a High-Rise Building of a Residential Complex
Supervisor: Gramblička, Š.

CHLEBÍK, M.: Static and Dynamic Analysis of a High-Rise Building
Supervisor: Šoltész, J.

JALŠOVSKÝ, J.: Design of the Bearing System of a Production Factory: Cast-in-Situ RC Structure
Supervisor: Šoltész, J.

JOZEKOVÁ, A.: "GREEN PARK – VTÁČNIK – PART A, Podkolibská, Bratislava" Apartment House: Cast-in-Situ RC Structure with Stiffening Cores
Supervisor: Abrahoim, I.

KLABNÍK, M.: Bus Station with Hotel: Concrete Structure
Supervisor: Benko, V.

KOLLÁR, T.: Design of Underground Parking Space of "Vtáčnik-Part C" Residential Apartment Building Made as a Watertight Concrete Tank
Supervisor: Šoltész, J.

KURINEC, M.: High-Rise Building of an Office Centre
Supervisor: Benko, V.

LADIVER, M.: Hotel "TULA" - Banská Bystrica: Cast-in-Situ RC Slab-Wall Structure on a Skeleton Pedestal with a Stiffening Core
Supervisor: Harvan, I.

MACOSZKOVÁ, P.: Central Police Office in Bratislava. Multi-Storey RC Skeleton Structure with a Stiffening Core
Supervisor: Harvan, I.

MÁČIK, J.: Social and Service Building of Jablonecké sklárne: Cast-in-Situ RC Structure with a Stiffening Core
Supervisor: Harvan, I.

MALLO, M.: Office Building
Supervisor: Benko, V.

MATOVČÍK, Š.: Office Building in Banská Bystrica: Cast-in-Situ RC Structure
Supervisor: Borzovič, V.

MATUŠOV, M.: Apartment House in Piešťany: Cast-in-Situ RC Structure
Supervisor: Borzovič, V.

STICZA, G.: "Centrál" Multifunctional Complex: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Gajdošová, K.

BOZSÍK, J.: Road Bridge over Trnávka Stream on Northern Bypass of Town of Trnava
Supervisor: Borzovič, V.

FABÓ, J.: Bridge over the River Váh on the Motorway D1 Built by Cantilever Balanced Method
Supervisor: Halvoník, J.

CHOVANEC, D.: Load-Bearing Structure of a Post-Tensioned Bridge
Supervisor: Borzovič, V.

JAVUREK, M.: Cast-in-Situ Bridge on the R2 Expressway
Supervisor: Halvoník, J.

JUHÁSZ, G.: Repair of Reinforced Ramp and Footpath at the Faculty of Civil Engi-

neering of Slovak Technical University Bratislava
Supervisor: Bilčík, J.

NOVÁK, M.: Load-Bearing Structure of Post-Tensioned Bridge
Supervisor: Borzovič, V.

PAŽMA, P.: Prestressed Bridge over the River Poprad, at Road I/68 Plavnica
Supervisor: Halvoník, J.

SCHWARZ, Ľ.: Incremental Bridge in Poland-Structural Analysis of the Superstructure
Supervisor: Paulík, P.

SKAČÁNYIOVÁ, S.: Road Bridge over the River at Rabča
Supervisor: Fillo, Ľ.

SVĚTLÍKOVÁ, E.: Bridge Built by the Incremental Launching Method in Poland: Structural Analysis of the Substructure
Supervisor: Paulík, P.

IX.3 Stundent's Scientific Conference Theses

MICHLÍK, M.: Evaluation of Efficiency of Various Floor Structure Types in an Apartment Building

MATEJKA, J.: The Effect of Nonlinearities on the Deflection of Reinforced Concrete Slab

BEUTELOVÁ, D.: The Geometry of Strands in an Unbounded Post-tensioning System of a Flat Slab

LADIVER, M.: Solution of a Pair of Walls of Multi-storey Building Realized on the Skeleton Pedestal

DÖMÖTÖROVÁ, A.: Calculation of Crack Width on Reinforced Concrete Wall on the Skeleton Pedestal

MÁČIK, J.: Design Principles for Reinforcement of Concrete Structures in Seismic Areas

DRINKA, J., VAŠEK, T.: Ultimate Resistance of Concrete Columns

FRÓLO, J.: Design of Slender Columns of High-Rise Building

HALABRÍNOVÁ, A.: Reinforced Concrete and Composite Steel-Concrete Circular Columns of High-Rise Building

KUBIŠ, J.: Comparison of Results of Various Methods for Solution of Two-Way Reinforced Concrete Floor Slabs

MATOVČÍK, Š.: Effect of Construction Progress on Behavior of Tall Buildings Columns

PAVLAČKA, P., KUBALOVÁ, Z., MAJTÁNOVÁ, L., STRUHÁR, M., PAVLÍKOVÁ, V.: Complex Examination of a Structural Element Load-Bearing in One Direction

SVĚTLÍKOVÁ, E.: Analysis of Reinforced Concrete Bridge Piers Built in Poland

SCHWARZ, Ľ.: Analysis of Deformations of the Bridge Built by the Incremental Launching Technology and Their Compa rison with Experimental Data

DRHA, P.: The Innovative Design Principle of a Prestressed Beam

Aerodynamic Wind Tunnel

On the 5th October 2012 the representatives of faculty opened the trial run of the aerodynamic wind tunnel in Central Laboratories of Faculty of Civil Engineering, Slovak University of Technology in Bratislava. The tunnel provides a cost-effective solution for a wide range of applications from wind engineering, structures and other sectors. Tunnel is 26.3 m long, measuring 2.6 x 1.6 m with working parts and it is destined mainly for research on non-stationary loads of buildings and structures due to wind. A large working width of the tunnel is advantageous for modeling the atmospheric flow over realistic topography (scale 1:1000 to 1:1500) to detect exposure to local wind field and dispersion in the fields. Front model test area provides a steady flow of air in the speed range up to 32 m/s.