Slovak University of Technology Bratislava

Faculty of Civil Engineering

Department of Concrete Structures and Bridges

Annual report

2011

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ANNUAL REPORT 2011

Department of Concrete Structures and Bridges
Faculty of Civil Engineering
Slovak University of Technology Bratislava
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CONTENT

PREFACE

Expert knowledge and skills always pass from one generation to the next. In every crisis, the greatest potential for firms is the staff: the technical knowledge and quality of character of employees. Nowadays, technical knowledge is primarily offered by universities. Higher education is for free in Slovakia. Nevertheless, somewhere it is paid. Although not for knowledge but for degrees themselves. Where a demand appears, also a supply does. Who has not got an experience with offers of foreign degrees to gain, without teaching, only for a "lifetime experience" and of course, for money? For similar issues it is not necessary to go abroad. In recent years it has become obvious that at a number of Slovak higher education institutions (universities) express degrees are easy to obtain. "*Scire volunt omnes, mercedem solvere nemo*" - "*Everyone wants to know; no one wants to pay what is required*" is a more or less known Latin proverb of the Roman satirist Juvenal.

The situation is not very jolly and the proverb could be paraphrased today: "Everyone wants a diploma and is willing to pay for it." Thus titled "experts" do not meet any of the criteria for greatest potential for firms. Therefore, also the only criterion in the tenders - price - seems to be an unacceptable simplification. "*As simple as possible, but not simpler*" says another proverb. Can the lowest price, poor quality and lack of awareness be an alternative to quality and knowledge? Everyone may take his own view…

You receive the new Annual Report (AR) of the Department of Concrete Structures and Bridges. We find the current crisis as a challenge and an opportunity to get better. We have significantly modified the layout of the AR and moderated its content. We have been inspired by the outstanding AR of the Department of Civil, Environmental, and Geomatic Engineering at ETH Zurich. Next time we will also try to get closer in the content quality of papers.

Juraj Bilčík

In every crisis
lies the seed
of opportunity

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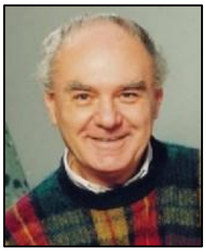

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I. NEW at D-CoSaB

I.1 Defenses of the Doctoral Theses

PAULÍK, P.: Contribution to an Evaluation of the Properties of Concrete Surface Layers
Supervisor: Hudoba, I.

VÖRÖS, Š.: Analyses of Biaxial Hollow Core Slabs
Supervisor: Fillo, Ľ.

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 fibre 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.) VEGA1/0458/11, Factors Affecting the Effectiveness of Utilization of High Strength Concrete in Load-Bearing Elements and Structures (2011-2013, HUDOBA, I.),

3.) VEGA 1/0306/09, Application of Probabilistic Methods to Improve the Reliability of Concrete Structures (2009-2011, BILČÍK, J.)

IV. EVENTS

Design of Concrete Structures and Bridges Using Eurocodes

Second International Workshop

On 12th and 13th September 2011 the second international workshop Design of Concrete Structures and Bridges Using Eurocodes was organized by a few employees from our department led by professor Fillo. The conference took place at Austria Trend Hotel Bratislava, Slovakia. The current European practice in structural engineering tends more to accepting the benefits of using the Eurocodes also for design of bridges and the other Civil Engineering Works.

The Second International Workshop focused on application of EN 1991-2, EN 1992-1-1 and EN 1992-2 since these standards have been introduced and accepted as the national standard in many European countries and there already are some practical experiences with its use. There was a creative discussion about the following topics:
- Background and Future of Eurocodes
- National Standards and National Annexes
- Design according to EN 1992-1-1
- Experiences with Bridges Design
- Proposal of Major Changes

The workshop was organized in cooperation with the Czech Technical University in Prague, Czech Republic, the Technical University in Vienna, Austria. It is anticipated that the next Workshops will be held in Austria.

Recognized experts and specialists in design of concrete structure from all over Europe participated, thus offering a unique opportunity to share their experience and acquired knowledge.

Major guests:

Jean-Armand Calgaro
Chairman of CEN/TC250 - Structural Eurocodes

György L. Balázs
fib President

Konrad Zilch
Eurocode 2 Project Team Member

Hugo Corres Peiretti
Member of fib Presidium

Bo Westerberg
Eurocode 2 Project Team Member

Jaroslav Procházka
Czech Technical University in Prague

The rich accompanying social program was also organized. The workshop started on 11th September (Sunday evening) in Theatre "Malá scéna" with a performance of classical music and folk dance of ensemble "TECHNIK".
Next day, the boat trip to ruins of "Devín" Castle with the presentation of history of Devín and Slavs and reception in "Hradná Brána" Hotel**** was organized.

All the 70 guests expressed their enjoinment with the whole technical program and we are looking forward to meeting again!


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Repair of Concrete Structures

7th Conference

On 7th and 8th December 2011 the seventh conference Repair of Concrete Structures was organized by the head of department professor Bilcik and other representatives of our department in cooperation with Association for repair of Concrete Structures beside the Slovak Chamber of Civil Engineers.

The intention of the conference was to allow for a meeting of a wide technical community focused on the field of repair of concrete and masonry structures. Recognized experts and specialists presented the reasons of failures of concrete and masonry structures, new technologies and materials, prepared and realized repairs.

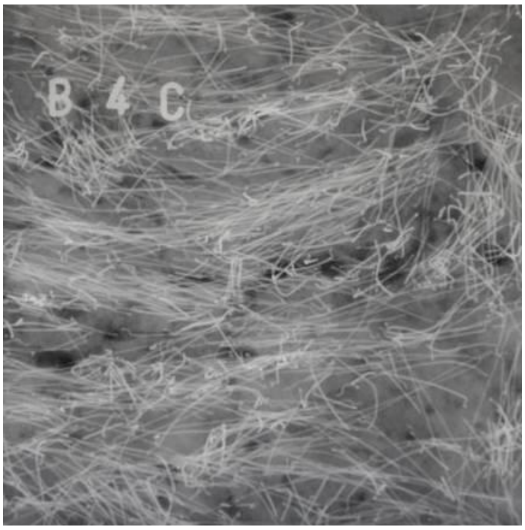
The rich discussion provided a big space for sharing of information and experiences. The personal interaction among guests during breaks and the evening event were an important detail of the conference.

The conference took place at Congress Centre Smolenice of Slovak Academy of Sciences – in Smolenice Castle.

At the end of the meeting, the opponency of a prepared directive for design of watertight structures passed.

Total of 60 active participants took their places in a technical discussion, learn new facts available for their work in the field of repair of concrete and masonry structures and established many new partnerships.

Conference is organized each two years and we are even now looking forward to meet again in December 2013.



prof. Hobst et al.: X-ray photography of fiber concrete

Eng. Roth: Concrete leek filled with crystal structure during impregnation

Meeting of Departments and Research Institutes

Meeting in Košice

The annual meeting of departments and research institutes from Slovak and Czech Republic, which operate in the field of concrete, was this year held on 3.10.2011 at the Faculty of Civil Engineering of the Technical University in Kosice. The participants were welcomed by assoc. prof. Ing. Sergej Priganc, PhD., who was also the main organizer of the event. After this welcome speech presentation of the delegates followed in which they informed their colleagues from other institutions about research and educational activities of the institution they represent.

These information were also available on the DVD, which all participants of the meeting were given. In the later afternoon participants were invited to take a short guided tour through Košice old town. The meeting then ended with social event in the Education and Training Establishment of TU Košice in Herľany, which is a small village known especially for the only geyser in Slovakia. Our department was represented on the meeting by assoc. prof. Ing. Július Šoltész, PhD and Ing. Peter Paulík, PhD.

Saying goodbye to year 2011

Christmas session

It's already a tradition at our department to meet all together around a small Christmas tree just few days before Christmas. Before each one goes home to have a rest and to spend time with his family, we spend one afternoon together. Talking about every event and memory we spent with colleagues. The head of the department, professor Bilcik, always addresses a speech on an interesting topic. This year, he was talking about a misunderstood researcher, who finally got a Nobel price. And we decided for a new resolution to year 2012: Be better and not to give up!

V. COOPERATION

V.1 International Cooperation

1.) Klokner Institute ČVUT Prague, Czech Republic
2.) Faculty of Civil Engineering, VUT Brno, Czech Republic
3.) Department of Civil, Environmental and Geomatic Engineering, Zurich, Switzerland
4.) Baustoffinstitut, TU Munich, Germany
5.) Institut für Massivbau und Baustofftechnologie, University of Leipzig, Germany
6.) Katedra Budowy Mostow Politechniki Slaskiej, Gliwice, Poland
7.) Department of Civil and Materials Engineering, University of Illinois at Chicago, USA
8.) RIB Bausoftware, Stuttgart, Germany
9.) Betosan, s.r.o., Prague, Czech Republic
10.) Seidl & Partners, G.m.b.H., Regensburg, Germany
11.) European Commission, DG Research, Brussels, Belgium
12.) Imperial College for Science, Technology and Medicine, London, U.K.
13.) St. Paul University, Brussels, Belgium
14.) Fachhochschule Braunschweig – Wolfenbütel, Germany
15.) Institut für Massivbau, TU Darmstadt, Germany
16.) Fachhochschule Coburg, Germany

V.2 Visitors to the Department

1.) prof.Ing.Leonard Hobst, CSc. – VUT Brno, Czech Republic
2.) Ing.Václav Pumpr, CSc. - Betosan, s.r.o., Prague, Czech Republic
3.) prof.Ing.Jaroslav Procházka, CSc. – ČVUT Prague, Czech Republic
4.) prof. Jean-Armand Calgaro - Ecole Nationale des Ponts et Chaussées, Paris, France
5.) prof. Hugo Corres Peiretti – FHECOR, Madrid, Spain
6.) prof. Gyorgy L. Balázs - Budapest University of Technology and Economics, Budapest
7.) prof. Konrad Zilch - Technische Universität München, München, Germany
8.) Dr. Bo Westerberg - KTH Architecture and Built Environment, Stockholm, Sweden
9.) Dr. Tony Jones - Ove Arup & Partners International Limited, London, England
10.) Dr.-Ing. Frank Fingerloos – DBV, Berlin, Germany
11.) Ing. Dr. Walter Potucek - fh-campus wien, Vienna, Austria

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

1.) BENKO, V.: Seminar on Lehrgang für Baudynamik und Erdbebeningenieurwesen für die Praxis, Vienna; Austria, 27.01.2011.
2.) BILČÍK, J.: Seminar on Execution and Control of Concrete Structures Repair, Prague, Czech Republic, 7.02.2011
3.) BENKO, V.: TU Vienna, Institut für Tragkonstruktionen – Betonbau, Vorlesung-Betonbau 3, Vienna, Austria, 10.3, 17.3, 24.3 31.3, 7.4, 14.4, 5.5, 12.5, 30.06 2011.
4.) BILČÍK, J.: XXI International Symposium on Concrete Structures Repair, Brno, Czech Republic, 18.-20.05.2011
5.) HALVONÍK, J.: 39th CEN TC250 Meeting, Ispra, Italy, 26.-27.5.2011.
6.) FILLO, Ľ. - HALVONÍK, J. - BELLOVÁ, M.: *fib* Symposium PRAGUE 2011: Concrete Engineering for Excellence and Efficiency, Prague, Czech Republic, 8. - 10.6.2011
7.) BENKO, V.: SCIA Engineer Seminar, Salzburg, Austria, 30.06 2011
8.) HALVONÍK, J.: 7th Central European Congress on Concrete Engineering, Hungary, 22.-23.9.2011 - Balatonfured
9.) BILČÍK, J. – BRONDOŠ, J.: Conference on Testing and Quality in the Building Industry, Brno, Czech Republic, 4.-5.10.2011
10.) BENKO, V.: Second Balkan Seminar on Earthquake Engineering, Sofia, Bulgaria, 6 .-7.10.2011
11.) HALVONÍK, J.: 40th CEN TC250 Meeting, Prague, Czech Republic, 14. -15.11.2011
12.) BENKO, V.: SCIA USER Meeting, Mondsee, Austria, 25.11. 2011
13.) BENKO, V.: 1st European Engineers' Day and GAM ECEC (European Council of European Chambers), Brussels, Belgium, 8. - 9.12.2011
14.) BENKO, V. - BORZOVIČ, V. - DOLNÁK, J. - FILLO, Ľ. - HALVONÍK, J. - PAULÍK, P. – PORUBSKÝ,T.: 18th Concrete Days 2011, Hradec Králové, Czech Republic, 23. -24.11.2011

V.4 Membership in International Professional Organizations,

1.) BILČÍK, J.: American Concrete Institute
2.) FILLO, Ľ.: Representative of the Slovak Republic in CEN TC 250 – SC2 Eurocodes – Design of Concrete Structures
3.) FILLO, Ľ.: Member of Task Group *fib* TG 1.1 Design Application
4.) FILLO, Ľ.: Honorary Member of Czech Concrete Society, Hradec Králové, Czech Republic, 25.11.2009
5.) HALVONÍK, J: Representative of the Slovak Republic on CEN TC 250 – SC1 Eurocodes – Actions on Structures
6.) HALVONÍK, J: Representative of the Slovak Republic on CEN TC 250 Eurocodes

VI. ACTIVITIES

Commercial Activities for Firms and Institutions

1.) BILČÍK, J.: Evaluation of the film thickness of a coating system Cooling Tower in Nuclear Power Plant Mochovce. Report for: Cooling Towers Praha, 2011

2.) BILČÍK, J.: Long-term measurement of cracks in the secondary lining of tunnel Branisko. Report for: National Highway Company, 2011

3.) FILLO, Ľ. BENKO, V. HALVONÍK, J.: Development of expertise reports for structural reliability of CBC 3, 4, 5 administrative building, Karadžičova St., Bratislava.

4.) FILLO, Ľ. BENKO, V. HALVONÍK, J.: Development of expertise reports for structural reliability of Multifunctional Houses – Mlynské Nivy building, Turčianska St., Bratislava – blocks C, F, G, H after the execution of construction works according to contract documents.

5.) ČABRÁK, M.: The revision of the standards STN ISO: 2010 Bases for design of structures - Assessment of existing structures. For the Slovak Standards Institute, 2011

VII. PUBLICATIONS

VII.1 Books and Textbooks

Books

1.) GAJDOŠOVÁ, K.: Strengthening Slender Reinforced Concrete Columns with the Use of Fibre-Reinforced Polymers, Slovak University of Technology in Bratislava, 2011, (Edition of Scientific Works; Workbook No. 109), ISBN 978-80-227-3562-9, 144 pp. (in Slovak)

2.) HARVAN, I.: Concrete Structures, Tall Buildings, Design According to Common European Standards, Slovak University of Technology in Bratislava, 2011, ISBN 978-80-227-3458-5, 206 pp. (in Slovak)

Textbooks

1.) ABRAHOIM,I. - BORZOVIČ, V.: Reinforced Concrete Supporting Structures, Instructions for Practice, Slovak University of Technology in Bratislava, 2011, ISBN 978-80-227-3598-8, 189 pp. (in Slovak)

VII.2 Journals

Scientific Papers Abroad

1.) BELLOVÁ, M.: Determining the Fire Resistance of the Design of Concrete Members According to Current Standards. In: Materiály pro stavbu, ISSN 1213-0311, Vol. 17, No. 2 (2011), pp. 42-43 (in Slovak)

2.) BELLOVÁ, M.: Determining the Fire Resistance of the Design of Masonry Members According to Current Standards. In: Materiály pro stavbu, ISSN 1213-0311, Vol. 17, No. 8 (2011), pp. 46-49 (in Slovak)

3.) BELLOVÁ, M.: Fire Resistance of Concrete Structures and Its Assessment According to Current Standards. In: Stavebnictví, ISSN 1802-2030, Vol. 5, Nos. 11-12 (2011), pp. 24-28 (in Slovak)

4.) BILČÍK, J.: Products and Systems for the Protection and Repair of Concrete Structures. In: Beton. Technologie - Konstrukce - Sanace, ISSN 1213-3116, Vol. 11, No. 3 (2011), pp. 52-55 (in Slovak)
5.) BORZOVIČ, V. - HÁJEK, F.: Analysis of Floating Floor Screed Slab Loaded with a Hand Pallet Truck. In: Beton. Technologie - Konstrukce - Sanace, ISSN 1213-3116, Vol. 11, No. 2 (2011), pp. 58-61 (in Slovak)
6.) GRAMBLIČKA, Š.: Reinforcing Cast-in-Situ Reinforced Concrete Bearing Structures. In: Stavebnictví, ISSN 1802-2030, Vol. 5, Nos. 6-7 (2011), pp. 4-9 (in Slovak)

Scientific papers in Slovak Journals

1.) BARTÓK, A.: Impact of an Earthquake on Building Structures. In: Stavebné materiály, ISSN 1336-7617, Vol. 7, No. 2 (2011), pp. 18-19 (in Slovak)
2.) BILČÍK, J. - HALAŠA, I.: Concrete for Watertight Concrete Structures (White Tanks). In: Stavebné hmoty, ISSN 1336-6041, Vol. 7, Nos. 1-2 (2011), pp. 15-16 (in Slovak)
3.) GAJDOŠOVÁ, K. - BILČÍK, J.: Slender Reinforced Concrete Columns Strengthened with Fibre-Reinforced Polymers. In: Slovak Journal of Civil Engineering, ISSN 1210-3896, Vol. 19, No. 2 (2011), pp. 27-31 (in English)
4.) HARVAN, I.: Assessment of the Concrete Structures of Existing Panel Buildings. In: Správa budov, ISSN 1337-6233, Vol. 5, No. 1 (2011), pp. 32-34 (in Slovak)
5.) HARVAN, I.: New Code for the Verification of the Concrete Structures of Existing Panel Buildings. In: Normalizácia, ISSN 1335-5511, Vol. 17, No. 1 (2011), pp. 3-7 (in Slovak)
6.) HUDOBA, I.: Problems of the Execution of Concrete Structures in Light of the New Standard STN EN 13670. In: Stavebné hmoty, ISSN 1336-6041, Vol. 7, Nos. 1-2 (2011), pp. 23-25 (in Slovak)
7.) HUDOBA, I. - ŠOLTÉSZ, J.: Repair of Reinforced Concrete Floor and Balcony Slabs Damaged by Cracks. In: Stavebné materiály, ISSN 1336-7617, Vol. 7, No. 7 (2011), pp. 38-41 (in Slovak)
8.) HUDOBA, I.: Standard STN EN 13670: Execution of Concrete Structures and Adopting Its Aspects in Concrete Practice. In: Stavebné hmoty, ISSN 1336-6041, Vol. 7, No. 6 (2011), pp. 45-49 (in Slovak)
9.) PRIECHODSKÝ, V. - NIČ, M. - CICANIČ, M.: Identify the Reasons for the Collapse of a School Building. In: Almanach znalca. - ISSN 1336-3174. - Vol. 10, Nos. 2-3 (2011), pp. 48-52 (in Slovak)

VII.3 Conferences

Contributions to Proceedings Abroad

1.) BELLOVÁ, M.: Detailing of Members According to the New Model Code. In: Concrete Engineering for Excellence and Efficiency, Vol. 1, *fib* Symposium Prague 2011, Czech Republic, 8.-10.6.2011, ISBN 978-80-87158-29-6, pp. 235-238 (in English)

2.) BILČÍK, J. - HOLLÝ, I.: Design and Verification of the Durability of a Concrete Infrastructure. In: Repair of Concrete Structures 2011, Proceedings of the 22th International Symposium, Brno, Czech Republic, 18.-20.5.2011, Brno, The Concrete Structure Repair Association, 2011, pp. 241-245 (in Slovak)

3.) BRONDOŠ, J. - BILČÍK, J.: Testing of Columns Strengthened with CFRP under an Impact Load. In: Investigation and Quality in Civil Engineering 2011, Reviewed Proceedings of Conference, Czech Republic, 4.-5.10.2011, Brno University of Technology, ISBN 978-80-214-3438-9, pp. 37-43 (in Slovak)

4.) FILLO, Ľ.: Structural Analysis of Beams by the New Model Code. In: Concrete Engineering for Excellence and Efficiency, Vol. 1, fib Symposium Prague 2011, Czech Republic, 8.-10.6.2011, ISBN 978-80-87158-29-6, pp. 103-107 (in English)

5.) GRAMBLIČKA, Š. - VALACH, P.: Composite Steel-Reinforced Concrete /SRC/ Columns: Theoretical Design Analysis. In: 11th International Scientific Conference VSU' 2011, Vol. 1, Sofia, Bulgaria, 2.-3.6.2011, L. Karavelov Civil Engineering High School Sofia, 2011, pp. 139-145 (in English)

Contributions to Proceedings in Slovak Republic

1.) BARTÓK, A.: Flat Slabs: Moments Distribution and Design of Reinforcement. In: Design of Concrete Structures and Bridges Using Eurocodes, Second International Workshop, Bratislava, Slovak Republic, 12.-13.9.2011, Slovak University of Technology in Bratislava, ISBN 978-80-8076-094-6, pp. 169-172 (in English)

2.) BELLOVÁ, M.: Contribution of Surface Reinforcement to the Resistance of Reinforced Concrete Members. In: 16th Conference on Statics of Buildings 2011, Piešťany, Slovak Republic, 17.-18.3.2011, ISBN 978-80-970037-6-0, pp. 47-52 (in Slovak)

3.) BELLOVÁ, M.: Fire Resistance of Concrete Structures in Accordance with the European Standards. In: 16th Conference on Statics of Buildings 2011, Piešťany, Slovak Republic, 17.-18.3.2011, ISBN 978-80-970037-6-0, pp. 23-32 (in Slovak)

4.) BENKO, V. - BORZOVIČ, V.: Tanks and Buildings. In: Proceedings of 4th Postcongress Colloquium on Concrete at the Third International fib Congress and Exhibition in Washington, D.C., Nitra, Slovak Republic, 6.-7.4.2011, Jaga Group 2011, ISBN 978-80-8076-091-5, pp. 159-175 (in Slovak)

5.) BENKO, V. - FILLO, Ľ.: Ultimate Resistance of an RC Cross-Section Subjected to Compressive Force. In: Design of Concrete Structures and Bridges Using Eurocodes, The Second International Workshop, Bratislava, Slovak Republic, 12.-13.9.2011, Slovak University of Technology in Bratislava, ISBN 978-80-8076-094-6, pp. 141-147 (in English)

6.) BILČÍK, J. - HOLLÝ, I.: Probability Models for the Deterioration of Concrete Structures. In: Preparation, Design and Execution of Civil Engineering Works, CONECO 2011, International Research Conference, 31.3.2011, SUT in Bratislava, ISBN 978-80-277-3469-1 (in Slovak)

7.) BILČÍK, J. – HALAŠA, I.: Measures for Extending the Service Life of Parking Structures. Proceedings of the 7th seminar Repair of Concrete Structures, Smolenice 7. – 8.12.2011, Slovakia (in Slovak)

8.) BORZOVIČ, V. - LACO, J.: Design of Bridge Approach Slabs for a Motorway and Highway in Slovakia According to EN 1992-2. In: Design of Concrete Structures and Bridges Using Eurocodes, Second International Workshop, Bratislava, Slovak Republic, 12.-13.9.2011, Slovak University of Technology in Bratislava, ISBN 978-80-8076-094-6, pp. 259-266 (in English)

9.) BORZOVIČ, V. - KÓŇA, D. - HALVONÍK, J.: Continuous Composite Concrete Girder. In: Design of Concrete Structures and Bridges Using Eurocodes, Second International Workshop, Bratislava, Slovak Republic, 12.-13.9.2011, Slovak University of Technology in Bratislava, ISBN 978-80-8076-094-6, pp. 197-205 (in English)

10.) BORZOVIČ, V.: Plain Concrete Loaded With an Eccentric Compressive Force. In: 16th Conference on Statics of Buildings 2011, Piešťany, Slovak Republic, 17.-18.3.2011, ISBN 978-80-970037-6-0, pp. 67-74 (in Slovak)

11.) BORZOVIČ, V. - HÁJEK, F.: Static Analysis of a Screed Floating Floor. In: 16th Conference on Statics of Buildings 2011, Piešťany, Slovak Republic, 17.-18.3.2011, ISBN 978-80-970037-6-0, pp. 99-106 (in Slovak)

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Eng. Peter Paulík, PhD.

The Use of Radioactive Reinforcement in the Structure of Concrete Bridges

1 INTRODUCTION

Nowadays the nuclear industry is facing new challenges related to 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. 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). 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.

Processes that precede the utilization of conditionally released very low level radioactive steel in bridge structure 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 bars or steel grids. Melting of metal scrap and fabricating of simple reinforcement bars will be carried out in specialized facility. It is clearly proven that the radiation impact of the proposed specific utilization on the workers and population will be negligible.

2 MODELING

Complex civil engineering structures and complicated construction processes were modeled in software, which was originally developed for calculations in the field of nuclear engineering. Real constructions were suitably simplified to enable the calculations along with keeping the results credible. Calculated models of the bridges also include trajectories that describe the procedures performed by the personnel on the construction site. The calculated results of these trajectories comprise the radiation doses absorbed by workers. These results can be confronted with the legislatively given limits and subsequently it is possible to define the limits and conditions of the utilization of very low level radioactive reinforcement steel in concrete bridge.

Research project is cooperated by the Institute of Nuclear and Physical Engineering at the Slovak University of Technology in Bratislava.

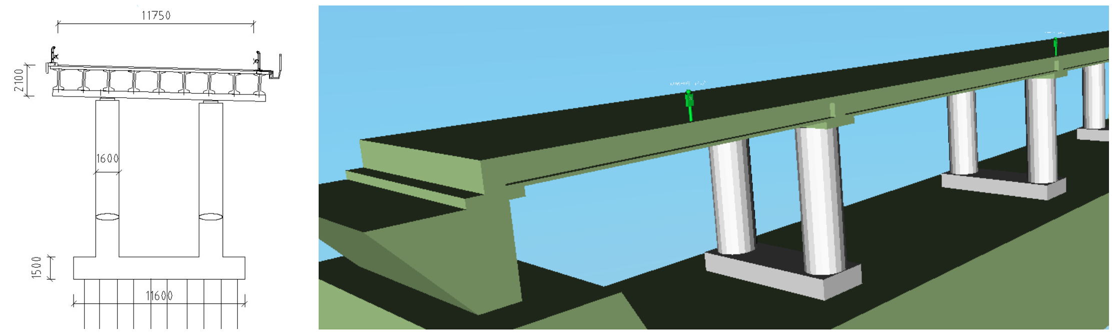

Fig.1: *Scheme of the bridge made of prefabricated components and its software model in VISIPLAN 3D ALARA*

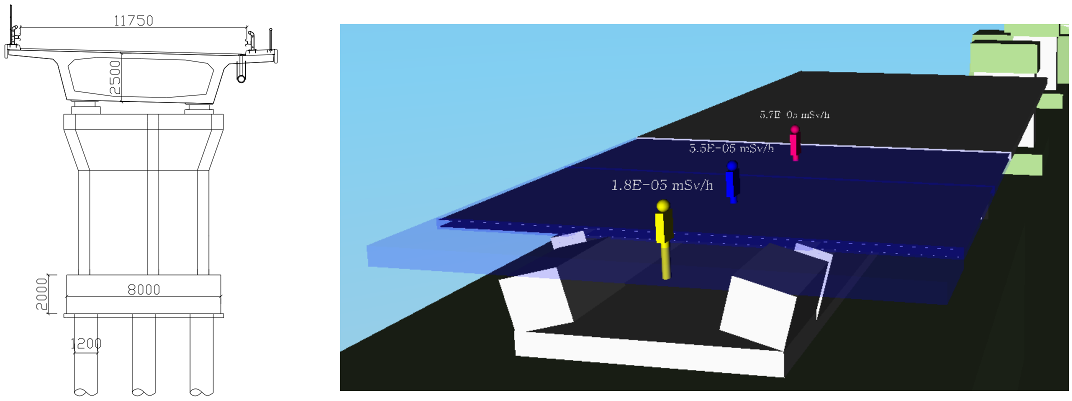

Fig.2: *Scheme of the bridge built using the launching technology and its software model in VISIPLAN 3D ALARA.*

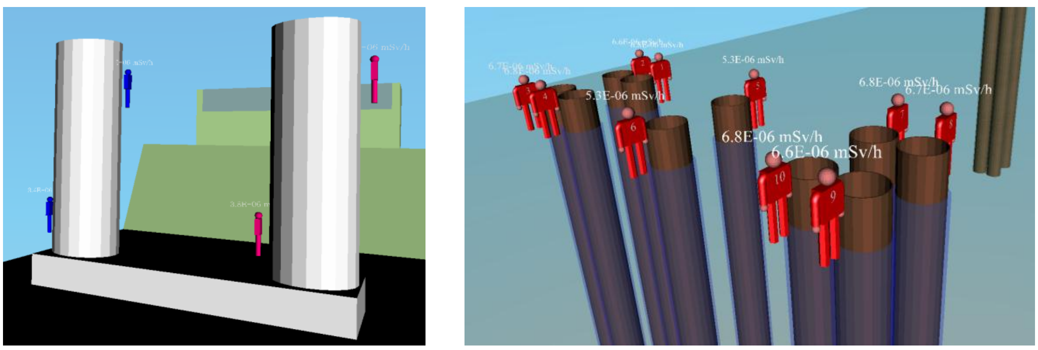

Fig.3: *Working procedures connected with building the foundations and the constructing the piers processed in VISIPLAN software*


Assoc. prof. Eng. Július Šoltész, Csc.

Eng. Miroslav Ignačák

Strength Assessment of the HVAC Pipeline Distribution at the EMO34 NPP

1 INTRODUCTION

The aim of this paper is the presentation and assessment of the solutions for the selected section of pipelines of the HVAC systems with respect to seismic and operational loads within the process of completion of the construction of EMO34. The paper is primarily focused on the assessment of thin-walled pipeline sections in relation to combination of internal forces and external pressure. Based on digital source materials in the DXF format and load specifications for respective structures (own weight, floor response spectra, temperature), an FEM calculation model was prepared in STRAP and used for the calculation of internal forces. The pipes are made of steel or stainless steel, mainly with circular cross-section and diameter of Φ1400, Φ1250, Φ1000, Φ400, Φ830, Φ630 and rectangular basic cross-section of 960x625, 800x1000, 960x 250, 960x 125, while certain cross-sections change linearly along. The pipeline wall thickness is 3-4 mm. The steel pipes are made of the material 11 373.1, while the stainless-steel ones of the material 17 248.4. *Figures 1 and 2* show examples of two rendered 3D models of the selected HVAC duct lines in *Revit* and the corresponding calculation models from *STRAP*.

2 ANALYSIS ASSUMPTIONS

All the pipeline elements have been assessed according to the ASME (American Society of Mechanical Engineers) based on the investor's request. In the given case, the US standards have been used. These standard serve as source materials for the design and assessment of safety of air or, as the case may be, gas distribution systems and structures of fittings of such systems in nuclear facilities. Since this equipment is used in a nuclear power plant, high demands with regard to safety and selection of responsible and reliable assessment of the aforementioned structures have to be applied. It is necessary to design the structure subject to evaluation and to prove by means of a reliable calculation that no damage to the structure will occur neither during operating nor emergency conditions and the structure will continue fulfilling its function.

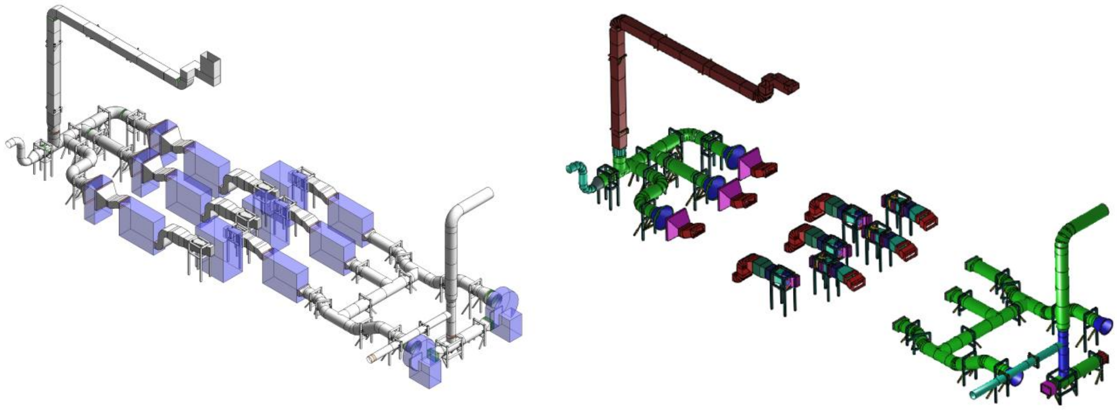

Fig.1: *The rendered model in Revit and the subsequent export of the geometry into VS STRAP*

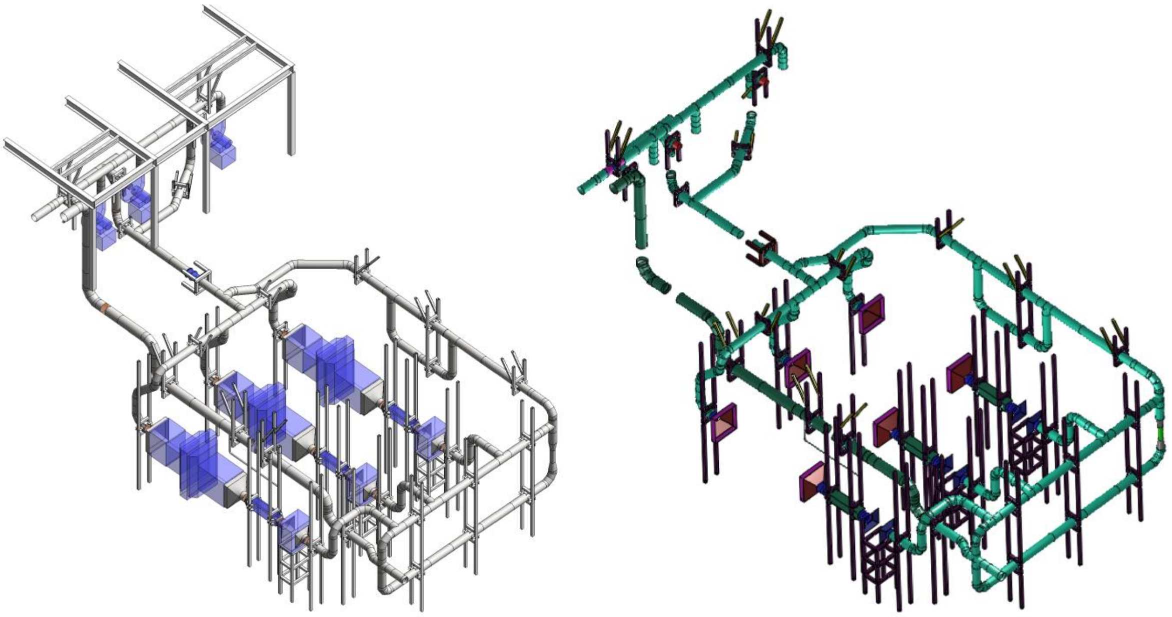

Fig.2: *The rendered model in Revit and the subsequent export of the geometry into VS STRAP*

Based on the calculated internal forces (AXIAL, M2, M3, V2, V3, MT) and the acting external pressure P from respective load combinations, each pipeline element has been assessed, both at its beginning and its end. It should be noted that these are thin-walled cross-sections with extremely high slenderness (200-500), and it is impossible to evaluate the individual elements based on standard relationships of the technical theory of elasticity. The pipeline elements' cross-sections themselves have been assessed pursuant to relevant articles from the set of the US ASME standards based on the cross-section shape (circle, rectangle). In this way, it has been possible to conduct a reliable assessment of cross-sections and to take account of such effects as local buckling, additional membrane stresses caused by external pressure, and also the shear relaxation. In the given case, these effects significantly contribute to the reduction of the load capacity of thin-walled pipeline cross-sections.

3 ASSESSMENT OF RECTANGULAR THIN-WALLED CROSS-SECTIONS

For the assessment of rectangular pipeline cross-sections see [13]. The assessment method deals with rectangular cross-sections subjected to stress resulting from the combination of axial forces, lateral forces, torques, and external or internal excess pressure, respectively.Based on the internal forces obtained from a global FEM analysis of the entire piping system and external pressure, the walls have been assessed as thin plates with high deflection, and thus with the impact of membrane stresses. The philosophy of assessment pursuant to the ASME lies in the calculation of the maximum permitted stresses and in the comparison with the values of actual acting stresses. The maximum permitted stresses take account of the influence of slenderness, material characteristics, local (wall buckling) and global (pipeline braces) loss of stability, shear relaxation, and also of the influence of additional stresses resulting from external pressure. The inclusion of the effects of local buckling of walls requires to use during the calculation of the maximum permitted stresses the cross-section characteristics of the effective cross-section, since the loss of stability of the wall occurs already with very low stresses. The ASME does not directly include a manual for the calculation of the effective cross-section of a rectangular pipeline; however, it offers several methods or specifies several publications which can be used for the determination thereof. Our calculation has been based on [15]. The buckling of slender walls is discussed in *Chapter 4.* Besides slenderness and material characteristics, the effective cross-section calculation is also a function of the pattern of normal stresses through the cross-section. The normal stresses in the cross-section are produced in result of the interaction between the normal forces and biaxial bending. Thus, based on Chapter 4, a universal macro has been designed for the calculation of cross-section characteristics of the effective cross-section, for a cross-section loaded by any combination of N_x (AXIAL), M_y (M2), M_z (M3), and also for any dimensions and thickness of the cross-section.

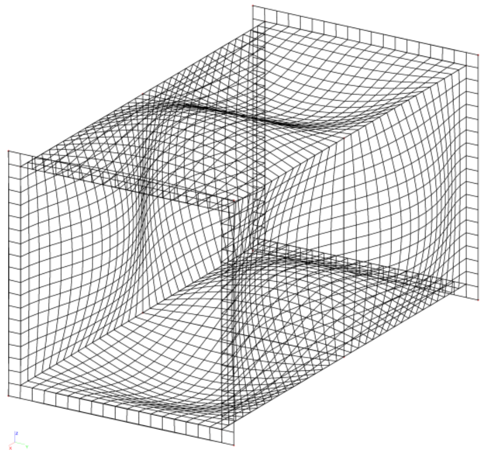

Fig.3: *Local buckling on a pipeline element*

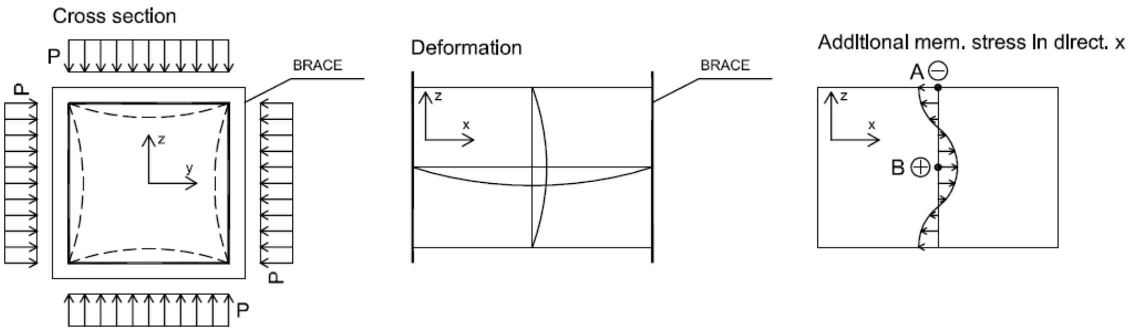

Fig.4: *Figure 6: a) cross-section + external excess pressure b) deformation c) additional membrane stresses*

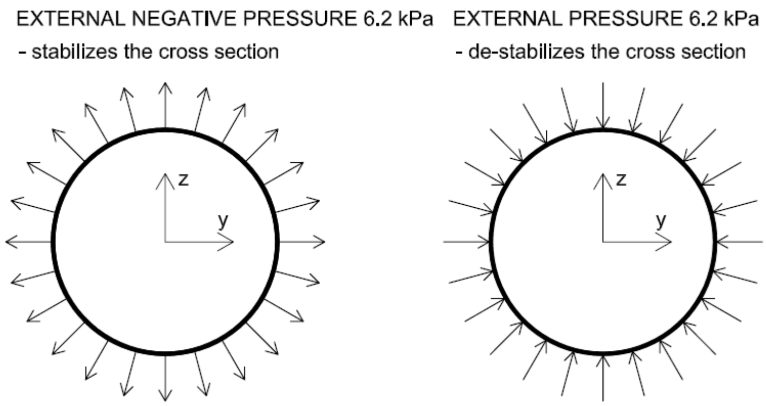

Fig.5: *Loading of a circular cross-section by external negative pressure or excess pressure, respectively*

In contrast to a circular cross-section, the excess pressure always has a negative impact on angular pipelines. Any pressure acting perpendicular to the pipeline wall causes a membrane tension inside the pipeline. Membrane stresses have been calculated in two ways, namely according to the standard, and for the verification purposes, with the use of the Finite Element Method in the *Scia Engineer* application, while it has been necessary to take account of the geometric non-linearity (Newton-Raphson). Calculation of membrane stresses on thin-walled plate elements is discussed in [14]. Based on the aforementioned standard, the values of membrane stresses for individual elements differing in particular in the geometry, however, also in material characteristics depending on temperature, have been calculated. The result of the verification calculation of additional membrane stresses in the *Scia Engineer* have very well (± 9%) matched the values calculated according to [14]. After the determination of cross-section characteristics of the effective cross-section, the acting stresses have been calculated. Subsequently, the respective cross-sections have been assessed for separate and combined acting force effects. In general, an element is in compliance if the acting stresses caused by the acting load are smaller than the permitted stresses.

4 ASSESSMENT OF CIRCULAR THIN-WALLED CROSS-SECTIONS

The assessment of the circular pipeline has been prepared according to [13]. The assessment method is focused on circular cross-sections subjected to stress resulting from the combination of axial forces, lateral forces, torques, and external or internal excess pressure, respectively. Again, during the assessment of cross-sections, it is necessary to take account in particular of local and global stability, and also of the effects of external pressure. Compared to the rectangular cross-sections, the consideration of local buckling of walls is substantially simpler, since this effect is considered in relatively simple relationships for the calculation of the maximum permitted stresses, and the calculation of the effective cross-section is not required. Also, the excess pressure assessment is substantially simpler than in the case of rectangular cross-sections. The acting of external negative pressure stabilizes the slender walls and increases their resistance. Therefore, for the benefit of safety, the impact of the external negative pressure has been neglected. The acting of the external excess pressure de-stabilizes the cross-section, and this effect must be considered by means of the calculation of the critical external excess pressure P_{cr} under which the cross-section collapses. The cross-section is in compliance with regard to the external excess pressure if the excess pressure is lower than the critical excess pressure.

5 IMPLEMENTATION OF THE ASSESSMENT IN PRACTICE

In the first phase, verification assessments have been prepared in the Mathcad application. However, the assessment of one duct line provided

	A	B	C	D	E	F	G	H	I
1	MAXIMUM BEAM RESULTS (Units: kN, kN*meter)								
2	Beam			Axial	V2	V3	MT	M2	M3
3	1	1	1	0.128	0	0	0	0	0
4			2	-0.106	0	0	0	0	0
5									
6	2	1	2	0.106	0	0	0	0	0
7			3	0.106	0	0	0	0	0
8									
9	3	1	3	-0.106	0	0	0	0	0
10			4	0.149	0	0	0	0	0

Fig.6: *Example of export of internal forces from Strap to MS Excel*

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Vstupné hodnoty										Výsledky / Results			
2	Prvok / Member		Medza klzu / Material yield stress	Modul pružnosti / Modulus of elasticity	Vonk. Rozmer / Outside dimension	Vonk. Rozmer / Outside dimension	Hrúbka prierezu / Duct thickness	Nevystuž. Dl. prvku / Unbraced lenght of member	Efektívny faktor dĺžky / Effective lenght factor	Aplikované memb. napatia / Applied membrane stress	Ax. tlak a ohyb / Axial compression and bending	Torzia a priečna sila / Tors. and transverse shear	Tl. a ext. tlak / Compress. And ext. Pressure	Maximálne využitie / Maximal exploitation
3	Beam	Prop.	σ_y	E	B	H	t	L	K	f_p	N+M+P	T+V	N+M+T+V	MAX
4	[-]	[-]	[MPa]	[GPa]	[m]	[m]	[m]	[m]	[-]	[MPa]	[-]	[-]	[-]	[-]
5	1	5	229	210	0.2	0.2	0.002	5	1	0	0.016	0.016	0.001	0.016
6		5	229	210	0.2	0.2	0.002	5	1	0	0.004	0.024	0.001	0.024
7														
8	2	5	229	210	0.2	0.2	0.002	5	1	0	0.026	0.015	0.001	0.026
9		5	229	210	0.2	0.2	0.002	5	1	0	0.016	0.016	0.001	0.016
10														
11	3	5	229	210	0.2	0.2	0.002	5	1	0	0.027	0.013	0.001	0.027
12		5	229	210	0.2	0.2	0.002	5	1	0	0.028	0.013	0.001	0.028

Fig.7: *Example of assessment of rectangular cross-sections in MS Excel*

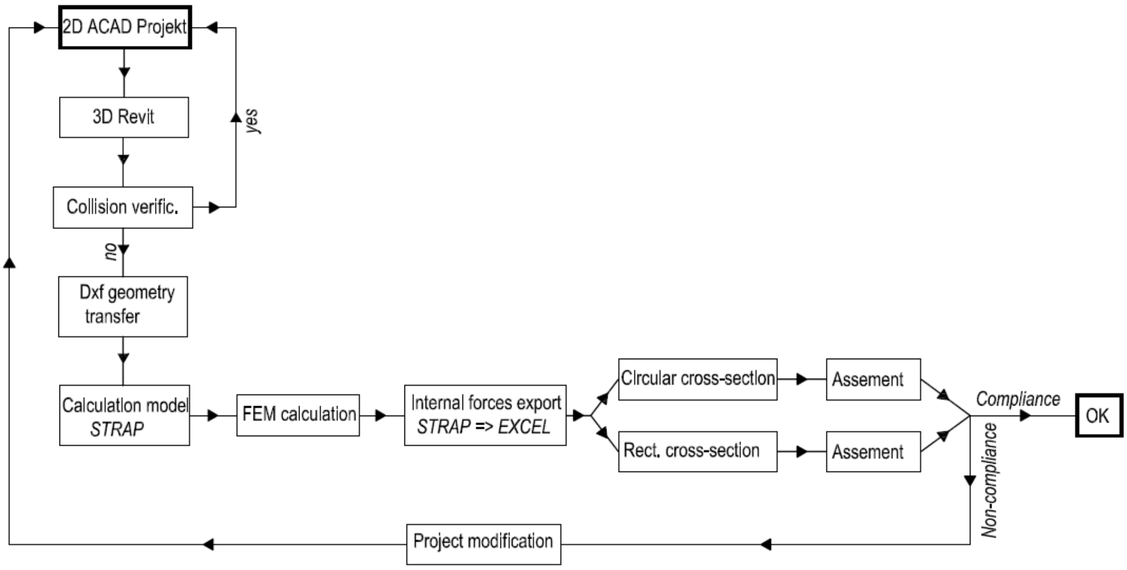

Fig.8: *Calculation chart*

here as an example requires to evaluate, for 3 combinations, approx. 80,000 cross-sections. It would be technically impossible to evaluate such an amount of cross-sections in Mathcad; therefore the entire algorithm has been transcribed into MS Excel. Programming of relatively complex and extensive relationships with the elements of propositional logic in MS Excel is cumbersome and poorly arranged, however, inevitable in the given case. The assessments themselves are relatively extensive, in particular the assessment of rectangular cross-section. It fills 1 row in MS Excel. For the purpose of the assessment itself, the internal forces for the respective load combinations have been divided separately for rectangular and circular cross-sections. Subsequent assessment goes very quickly and effectively. Import of the data (internal forces) in a delimited format into MS Excel (Figure 6), allows us to virtually immediately assess the required amount of cross-sections (Figure 7).

6 PARAMETER HCLPF

For the purpose of assessment of seismic resistance of buildings, equipment, and systems at EMO34, a value defining the reached limit seismic resistance is determined, i.e. the so-called HCLPF. During the calculation of the HCLPF parameters, the FS safety factor is defined, while this factor corresponds to the multiple of the final seismic response determined for PGA at which exhaustion of the crucial acceptance criterion used for the assessment of the given component occurs. In order to ensure the required level of seismic resistance, the final value of the parameter HCLPF (determined for crucial way of breaking) has to be at least equal to the value of PGA. If this condition is complied with, the structure subject to assessment is regarded as being in compliance with respect to strength requirements. Thus, it is clear that the HCLPF parameter characterizes the structure subject to assessment as a whole from the perspective of resistance of its weakest part, and expresses the maximum possible PGA value at identical permanent and operating loads. Should we wish to place a similar technology structure to a different location (geographically or within the same facility) with a different design PGA value, it shall be possible, even without a further static analysis, to observe based on the basic HCLPF parameter that the structure complies/fails to comply with strength requirements.

7 CONCLUSION

Designing of pipelines is performed with the use of iteration procedure. This includes, in particular the correct positioning of compensators and support frame structures laid or hung-up on the concrete structure of EMO. In certain cases, it has been necessary to locally increase the pipeline wall thickness from the initial 3 mm up to 6 mm. It is necessary that all pipeline distribution systems are in compliance with respect to strength requirements. If the structure fails to comply after the first design, appropriate measures are taken and the structure is subjected to assessment using the procedure described above. In that way we continue until all the parts of the pipeline comply with respect to strength requirements with the prescribed load combinations (Figure 8).

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STN EN 1993-1-5 Plated structural elements

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prof. Eng. Igor Hudoba, PhD.

Eng. Ján Mikuš

Factors Influencing the Use of High-Performance and Ultra High Performance Concrete in Practical Applications

1 INTRODUCTION

The last twenty years could be defined as a period of significant progress in research and development of the new generation of concrete. The results of extensive research and existing knowledge in the area of high performance concrete (hereinafter as HPC) and ultra high performance concrete (hereinafter as UHPC) have been utilizing in many successful projects of various kinds of concrete structures all around the world. Codification of high strength concrete classes (HPC) in the current legal standards created legal space and tools for its application in practical design and construction execution. Despite certain resistance from the side of contractors and partly of ready mix concrete producers, HSC, respectively HPC is slowly finding more and more applications in concrete practice. Currently an increasing interest in research area of UHPC in many countries is visible as well. UHPC is gradually moving from the laboratory into the concrete practice. As the matter of fact the design of elements, resp. constructions of UHPC are not yet reflected in valid standards. Only some regulations were published in the recent time. In spite of this fact a number information about successful UHPC applications have been presented and published.

2 FACTORS INFLUENCING THE USE OF HPC AND UHPC IN PRACTICAL APPLICATIONS

There are many factors which influence the practical application of HPC a UHPC. They depend especially on material basis, technology of fresh concrete production and many other processes of concreting in the local, resp. regional frame. The most important fact and condition in decision making process whether use HPC (resp. UHPC) or not must be the higher added value for the element, resp. concrete structures made of HPC, resp. UHPC. The present research project is focused on the utilization possibility of UHPC in composite elements exposed compressive load. UHPC of high compressive strength on the level more 150 MPa by using mostly local and regional materials is under development. UHPC steel bars as the reinforcement of different types (in stead of classical steel reinforcement) will be used for composite CC columns (Concrete-Concrete). At present a detailed stress analysis is under investigation. Theoretical results will be compared with experimental laboratory investigation. (Reseach Project VEGA No.01/ /0458/11 - Factors influencing the HSC application for load-bearing elements and structures).

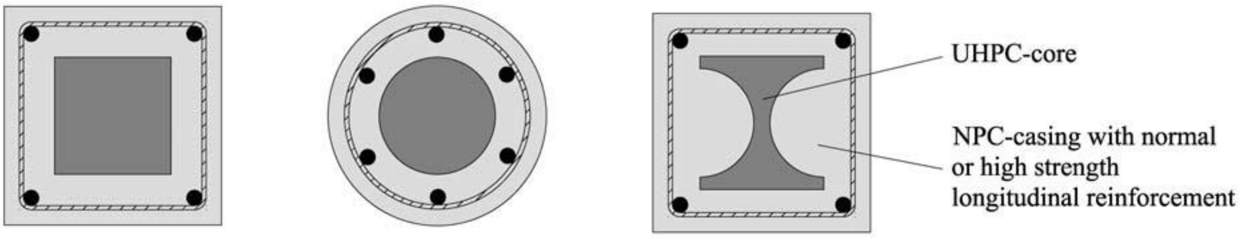


Fig. 1: “CC-Column“– possible design of the UHPC-core

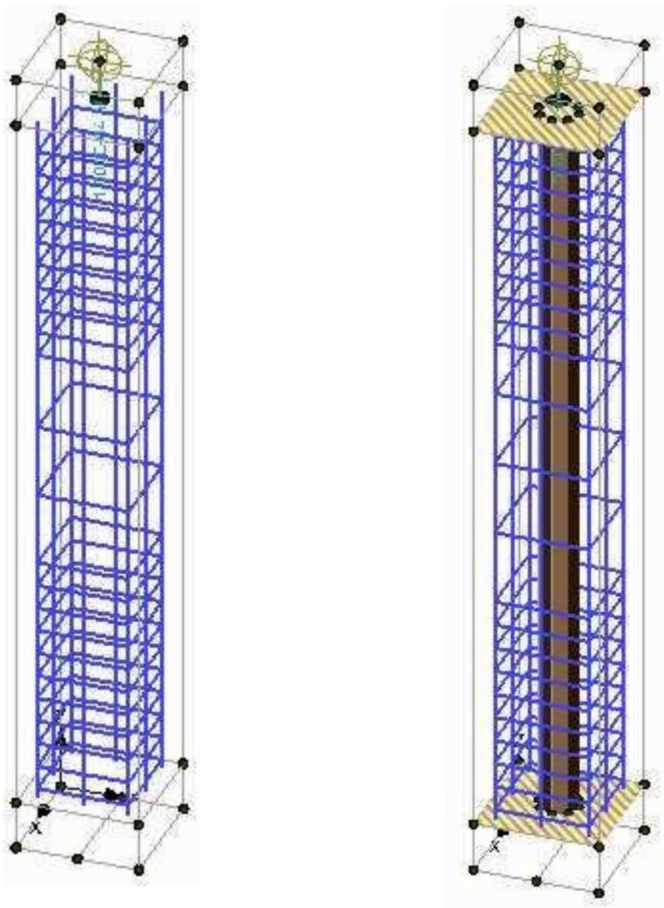


Fig. 2: FEM Model of NPC and “CC-Column“ for theoretical analysis

Eng. András Lelkes

Theoretical Analysis of Composite Steel-Concrete Columns

1 INTRODUCTION

The paper presents some results of the analysis of the second order theory effects. According to the results of the experiments, which were carried out by Department of Concrete Structures and Bridges in Bratislava we analyzed the effects of the second order theory. The results of the experiments by S. Matiaško were compared with theoretical results from the calculation based on non-linear software Athena.

2 THEORETICAL AND EXPERIMENTAL STUDY

European standards for composite structures can be applied to columns and compression members with steel grades S235 to S460 and normal weight concrete of strength classes C20/25 to C40/50, which is the limit between high strength and normal strength concrete. If high-strength concrete (HSC) is used in composite column, the resistance will be greater, respectively, we will achieve smaller area of the cross section. In slender composite columns it is necessary to take into account an increase in bending moments according to the second-order theory.

According to the strength of the concrete the second order-effects could be different and it would be useful to modify the factor *k* (which is given by the European standard, it can be applied only for normal strength concretes). Second-order effects may be calculated by multiplying the greatest first-order design bending moment M_{Ed} by a factor *k* given by:

$$k = \frac{b}{1-\frac{N_{Ed}}{N_{cr,eff}}} \geq 1{,}0$$

where:

$N_{cr,eff}$ is the critical normal force for the relevant axis

b is an equivalent moment factor in Table 6.4(STN EN 1994-1-1)

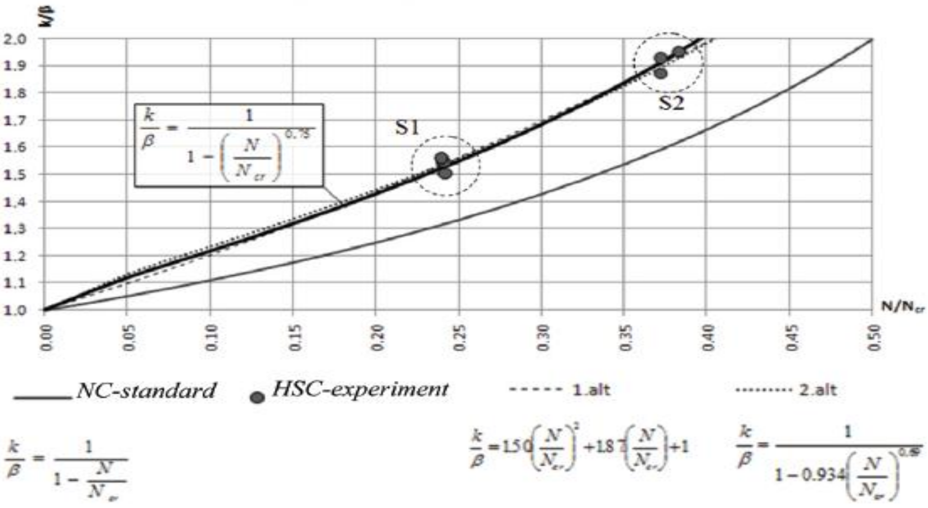


Fig.1: Possible solutions for the modification of the "k" factor relationship for HSC composite columns with the use of HSC

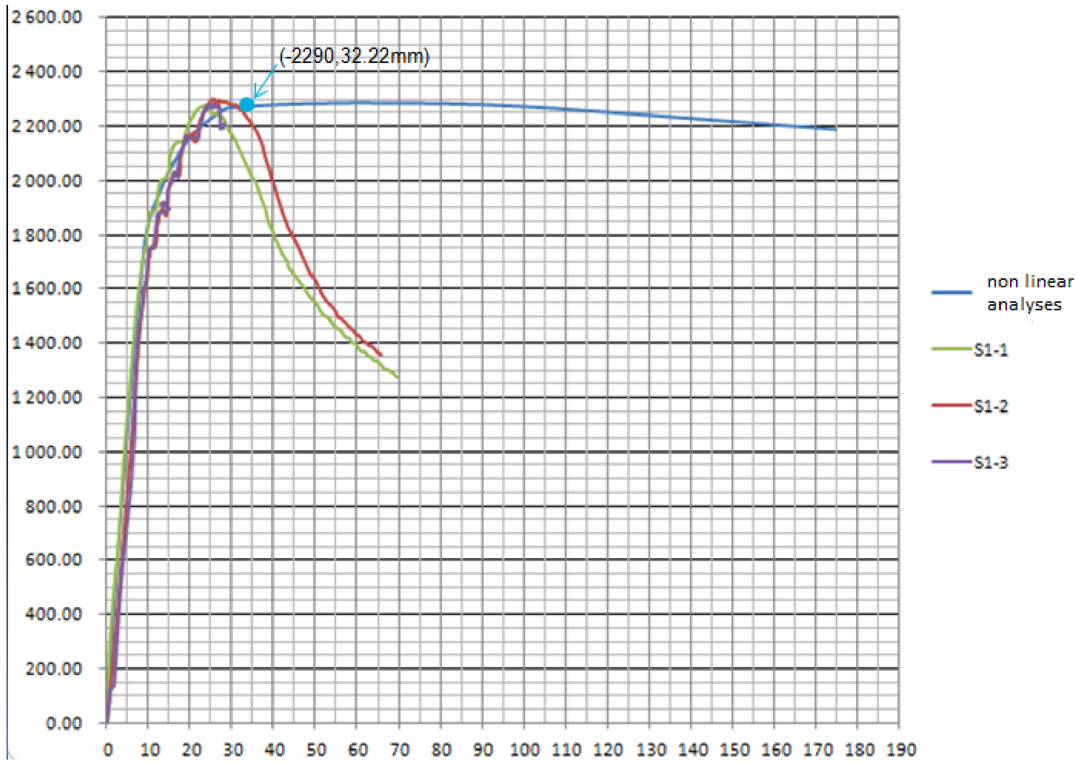


Fig.2: Comparison Force – Deflection relationship of the experiment series S1 using HSC and non-linear calculation in software Athena

	Resistance of the column					Horizontal deflection		
	Experiment		Non-linear analyses.		Gap	Experiment	Non-linear analyses	Gap
	N [kN]	M [kNm]	N [kN]	M [kNm]	[%]	[mm]	[mm]	[%]
Series S1	-2288	157	-2290	165	2.6	25.97	32.22	19.4

Fig.3: Comparison of non-linear analyses and experimental measurements

Eng. Jakub Brondoš

Strengthening of Columns under Impact Load

1 INTRODUCTION

A lot of old parts of bridges, especially their columns were not designed under impact load. Impact load includes impacts of vehicles, trains, ships etc. The most frequent kind of impact is the vehicle impact. The most dangerous one is the impact of a truck due to its big weight.

2 IMPACT LOAD ON COLUMN AND ITS STRENGTHENING

There are two kinds of impact. The first one is a hard impact, where the impact energy is dissipated in the impact body and second one is a soft, where energy is dissipated into the structure. The most important value for design of strengthening of columns under impact load is equivalent static force (ESF) in the impact point. During the impact dynamic force grows, its maximum is the peak dynamic force (PDF). We need to evaluate ESF from function of the dynamic force. Once we know ESF, we are able to apply it on the structure and carry out its static calculation. The results are the bending moment functions and shear forces in the column. Then we are able to design the strengthening. The most common method for strengthening is using of 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 mean of strengthening under impact load is by a combination of CFRP fabrics and laminates, because during the impact the bending moment arises together with the shear force. Inter alia during the impact concrete of column can crush in the point of impact. CFRP fabrics could prevent from this effect, concrete can´t crush easy if it is wrapped.

3 CONCLUSIONS

When designing column 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 do not, it is necessary to model structure in a sophisticated FEM software, for example in LS-DYNA, which aims at dynamic problems. Anyway, this method is not simple and is not commonly used among structural engineers. Now we are preparing an experiment for verification of concrete column response (fig.1).

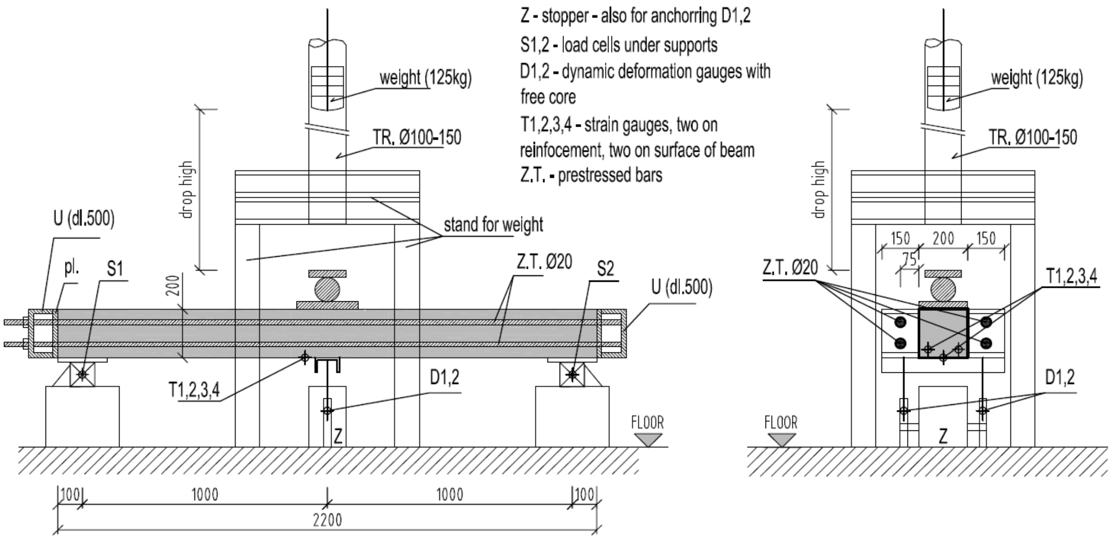


Fig.1: planned experiment – impact load with free fall

Fig.2: real impact accident

Eng. Ivan Hollý

Degradation of Concrete Structures Due the External Environments

1 INTRODUCTION

Concrete is considered a durable mate-rial, which in most cases does not need additional protection from the effects of the environment. The most common causes of failure of concrete structures is corrosion of reinforcement and cor-rosion of concrete. Corrosion of reinfor-cement can be induced by carbonation or chloride penetration into concrete. Time-devepoment of reinforcement corrosion is shown on (fig. 1)

2 DURABILITY OF CONCRETE STRUCTURES

The rate of reliability of the structures in not constant value in time. Reducing the reliability (probability of failures in-crease) is due to an increase in the load effects *E(t)* and decrese in resistance of the structure *R(t)* during its lifetime. (Fig.2) Calculation of reliability is based on mathematical statistics and proba-bility theory. Reliability function can mathematically write in the form:

$$Z = g(R - E) \begin{cases} \geq 0 & \text{reserve of reliability} \\ < 0 & \text{failure} \end{cases} \qquad (1)$$

In equation (1) *R* is the resistance of the structure and *E* is the effect of the load.

Problems of service life of concrete structures is engaged in the fib document "Model Code for service Life Design". In the document are full-probabilistic models for carbonation initiated corrosion, corrosion due to chloride penetration, degradation of concrete due freezing cycles.

The processes of material degradation effects of surrounding environment are important time-dependent phenomena that can significantly affect the relia-bility of the building structure. In this case, it is appropriate to calculate the required level of reliability to the proce-dures based on probabilistic approach. Application of the degradation model assumes considerable knowledge of concrete technology, knowledge in the use of appropriate software and proba-bilistic characteristics of the input para-eters.

Benefits of using a probabilistic ap-roach to structural design is reflected mainly in construction works with long life, such as the construction of trans-port infrastructure.

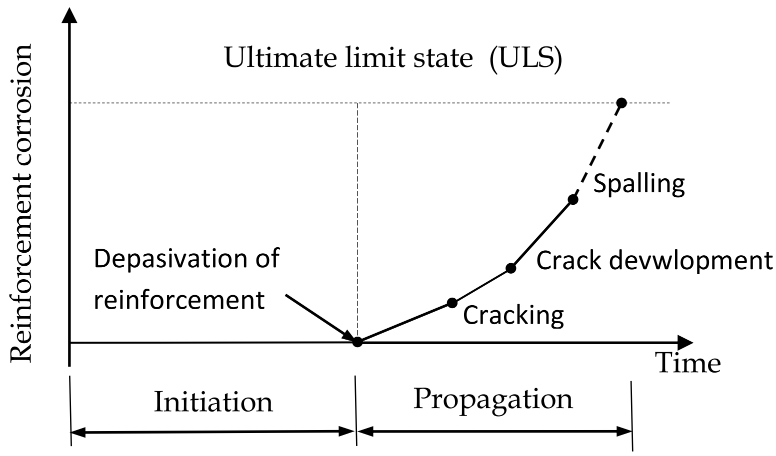


Fig.1: Time-dependent development of reinforcement corrosion

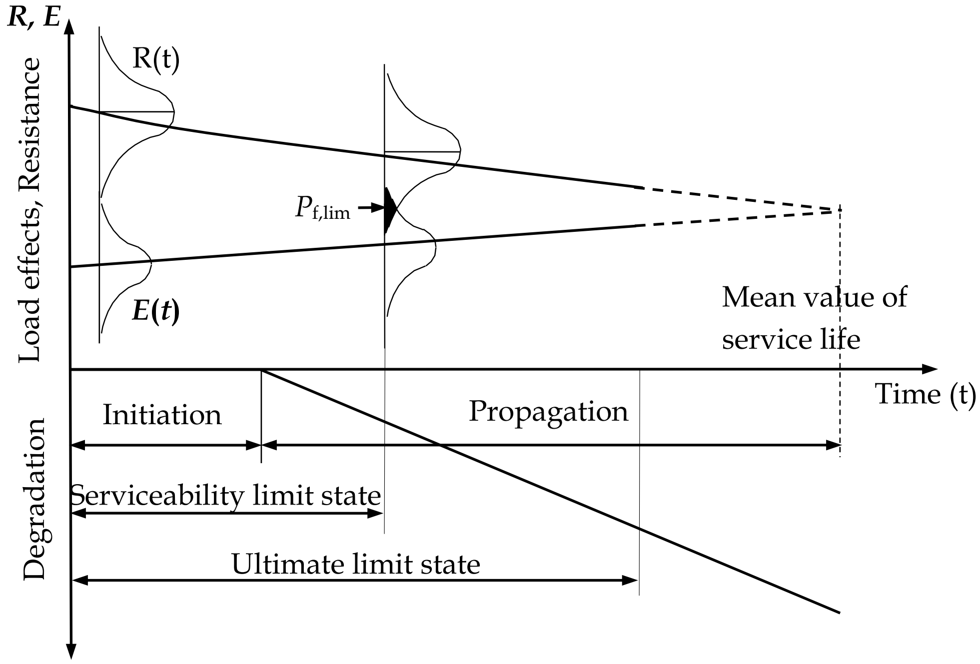


Fig.2: Development of reliability as a result of the effects of change in load and resistance of concrete at the time

Eng. Peter Rozložník

Redistribution of Bending Moment

1 INTRODUCTION

Design of concrete structures provides several means of analysis. The possibility to use redistribution by design of hyperstatic concrete structures 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:

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

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

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

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

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 the redistribution at δ=0,7 is possible under the reinforcement ratio of ρ=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 of δ=0,7 under the reinforcement ratio ρ=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 at δ=0,7 under the reinforcement ratio of ρ=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 at δ=0,7 under the reinforcement ratio of ρ=0,013.

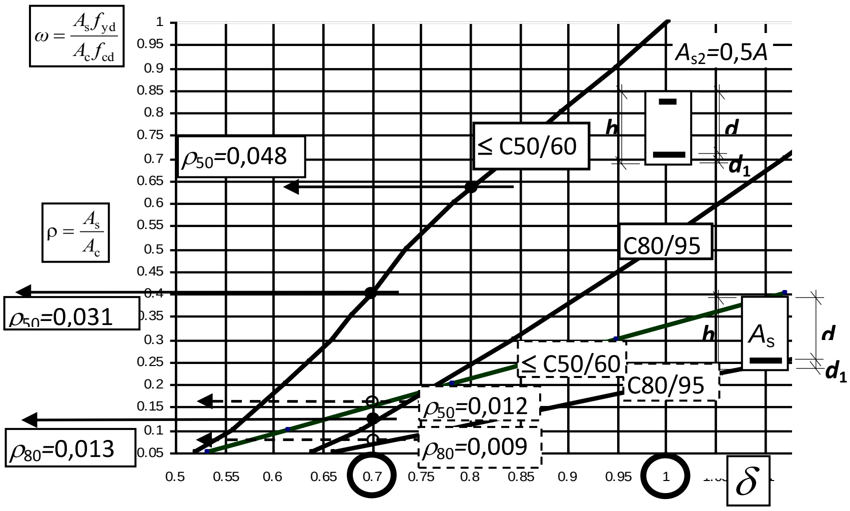
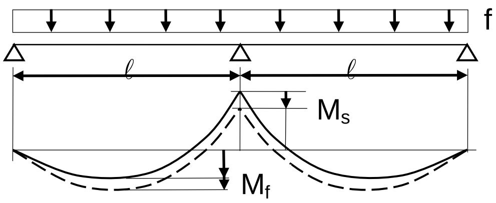
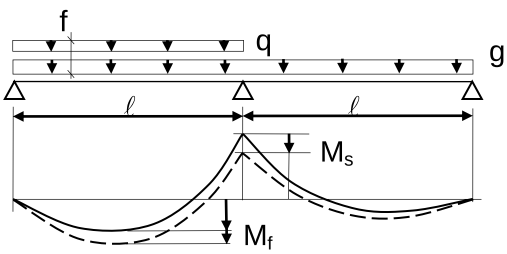


Fig. 1: δ - ω ratio



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 moment



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. Marian Kišac

Interaction Diagram of Slender Columns

The effect 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).

Fig. 1 shows the interaction diagram, which determines the resistance of the critical cross-section.

In the case according to Fig. 1 it is a cross-section at the fixing of the column. 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 the case a) of a massive column – at the increase in axial force on the eccentricity e_0 just a negligible deformation of compressed column occurs (Fig. 1 black continues 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 achieved as the intersection of this line with the curve of resistance of the critical cross-section.

At the higher column slenderness, the case b) (Fig. 1 dash-dot line), the increase in force N_{Ed} causes column deformation, this leads to an increase in the eccentricity of normal force in the critical cross section. That is reflect in the additional axial force eccentricity e_2. In Fig. 1 the increase of eccentricity is shown as the dash-dot line. The maximal normal force $N_{Rd(b)}$ is the intersection of the dash-dot curve with the interaction diagram.

In very slender columns, case c), a bigger columns deformation occurs. 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 dot 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.

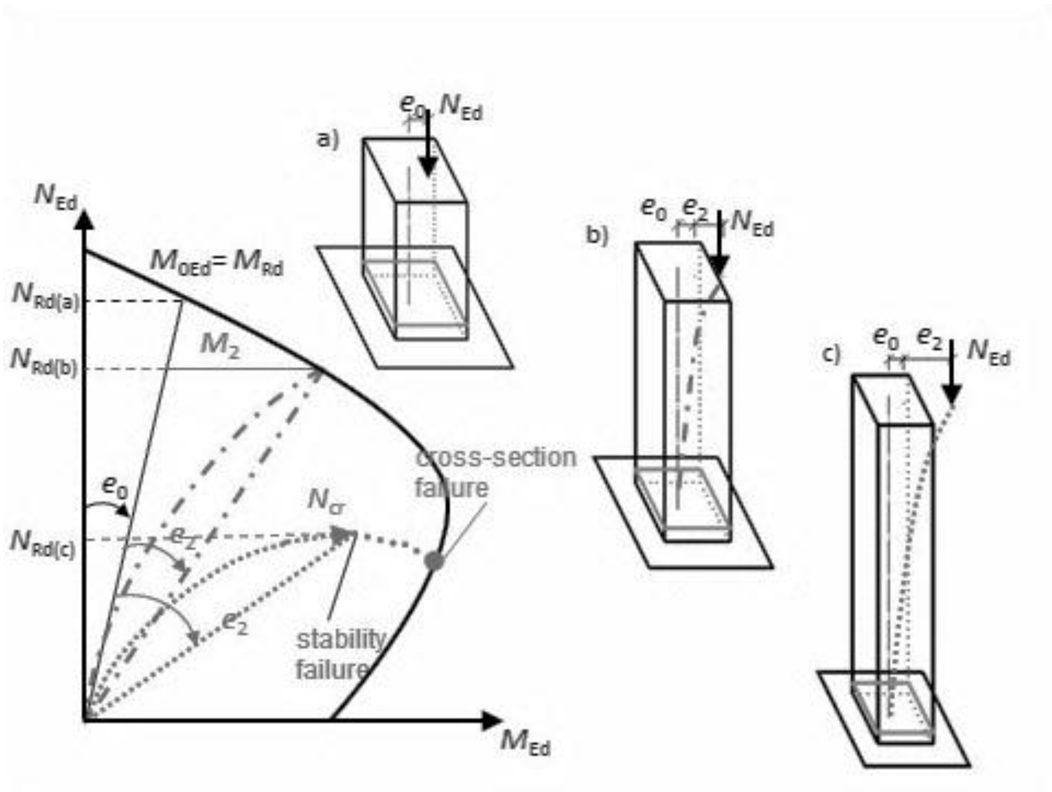


Fig.1: Interaction diagram of slender columns

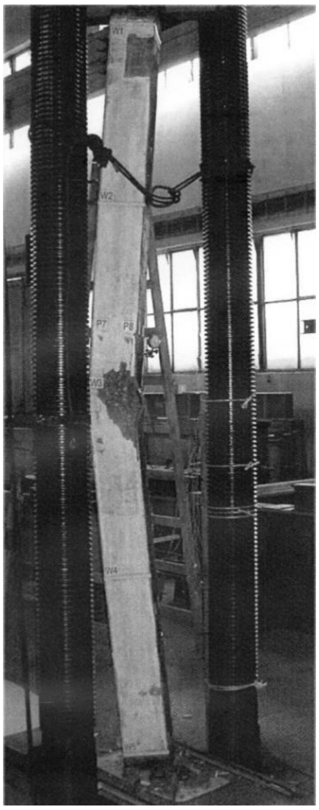


Fig.2: Case of failure slender column failure by stability failure

Eng. Peter Veróny

Reinforced concrete columns - Analysis of the influence of transverse reinforcement

1 INTRODUCTION

The paper is dealing with the influence of transverse and longitudinal reinforcement over the resistance of a column cross-section and also with the effectivity of design guides of column reinforcement. We are verifying correctness of these design guides of column reinforcing for transverse and longitudinal reinforcement. We are also taking into account the diameter of stirrups and its influence over transverse column deformation.

2 ANALYSIS OF COLUMNS

To determine the size of the influence of the transverse and longitudinal reinforcement to column cross-section resistance, to the effectiveness of the design principles of column-reinforcing and to determine the size of transverse column strain we used finite-element calculation program ATENA. In ATENA we modeled many columns and gradually we edited them, while four series of nine columns (36 columns) incurred. We analyzed these columns, compared with one another, and we finally draw conclusions from.

Outputs we processed in tables. We found out that the transverse displacements of stirrups with smaller diameter are larger than the transverse displacement of stirrups with larger diameter. Furthermore, we confirmed, that the additional stirrups reduce transverse displacement of the column.

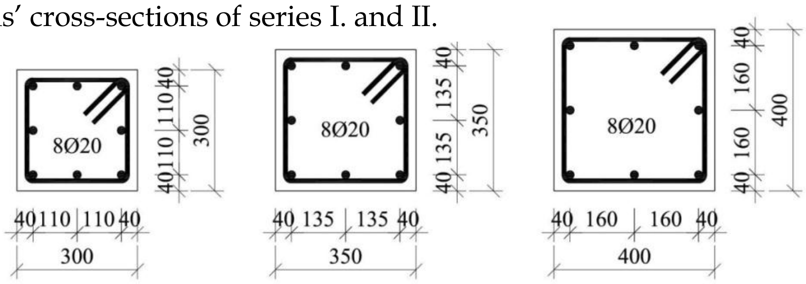
3 CONCLUSIONS

From the results it is obvious, that the design of auxiliary stirrups also depends on other parameters:

- stirrup diameter,
- longitudinal reinforcement diameter,
- quality of concrete, etc.

At the same load of individual cross-sections by using stirrups of diameter Ø6 were observed larger transverse deflections 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 auxiliary stirrup.

Columns' cross-sections of series I. and II.



Columns' cross-sections of series III. and IV.

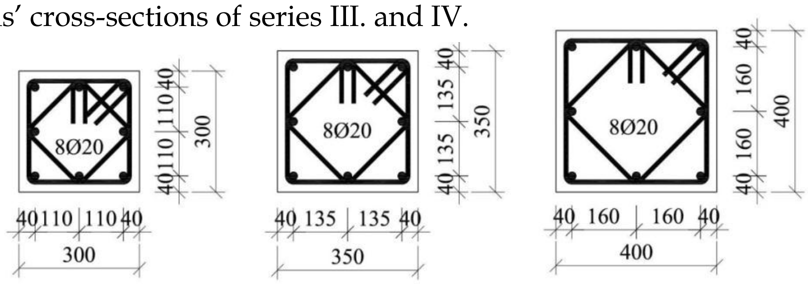
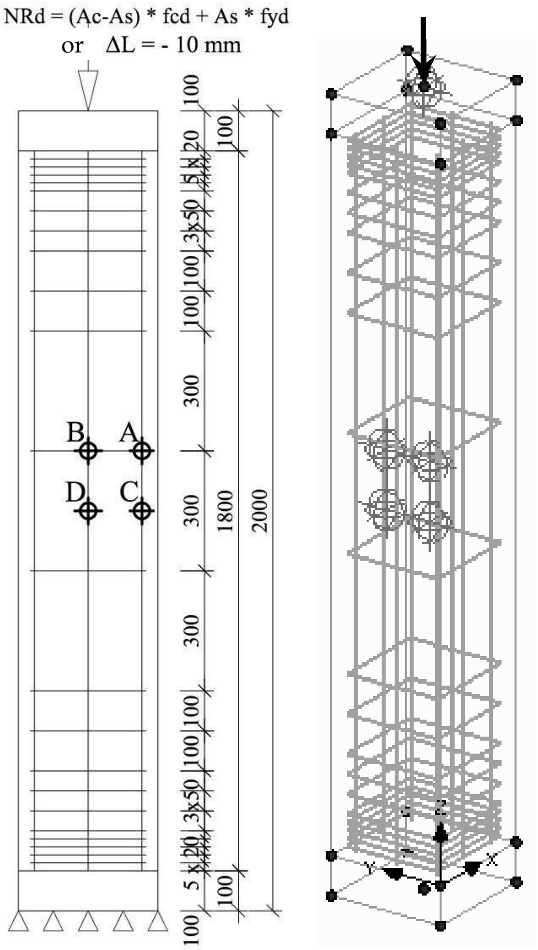
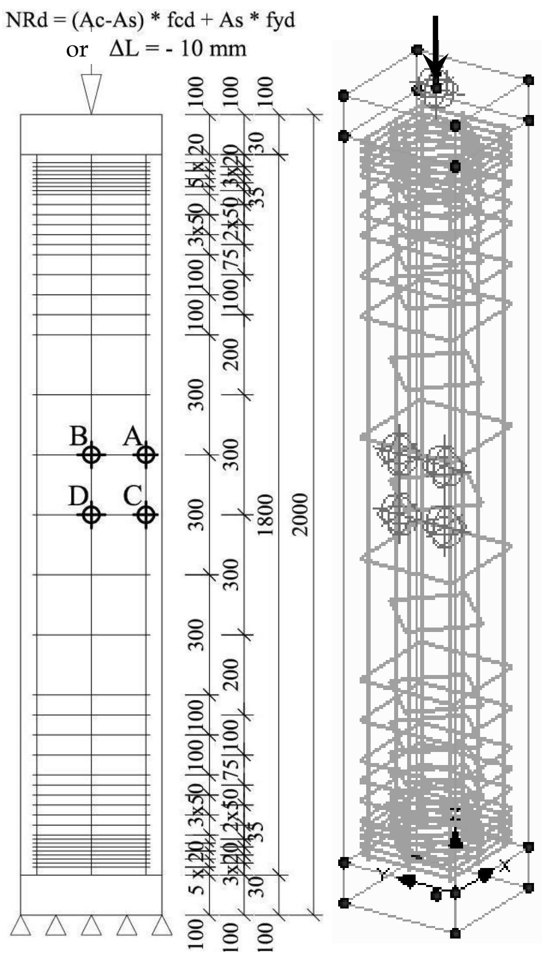


Fig.1: Geometry and reinforcing of columns cross-sections



Columns' geometry of series I. and II.



Columns' geometry of series III. and IV.

Fig. 2: Geometry, load and reinforcement of columns /location of column monitors/

Eng. Tamás Porubský

Methods of Slender Columns Analysis

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 II. Order theory, whether 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 behavior.

(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 non-linear 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.

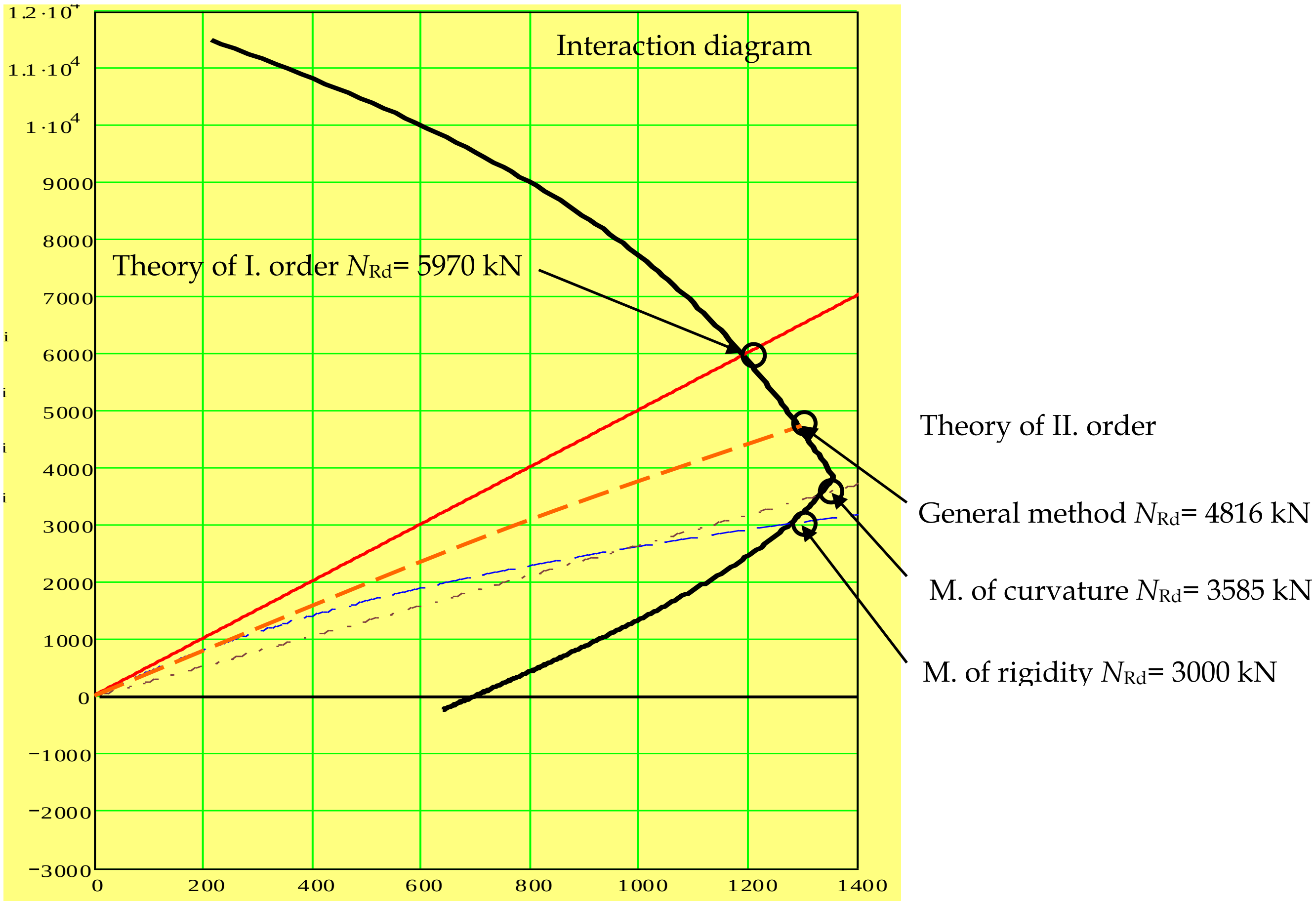


Fig.1: Interaction diagram


Assoc. prof. Eng. Ivan Harvan PhD.,

Eng. Iyad Abrahoim PhD.

Verification of Concrete Structures of Existing Panel Buildings

1 INTRODUCTION

New standard STN 73 1211:2011 "Assessment of concrete structures of existing panel buildings" is applicable since February 2011. It was completely revised to fulfil the newest requirements of actual European standards. Standard is intended for verification of prefabricated concrete structures of existing panel buildings made of dense plain concrete, under reinforced, reinforced and prestressed concrete. This standard will be applied in additional reconstructions, superstructures and interventions to load-bearing structure. It's not assumed to use this code for design of new panel buildings concrete structures.

2 SCOPE OF STN 73 1211: 2011

The assessment of concrete structures of existing panel buildings is a significant technical challenge these days. Engineers can use the specific evaluation methods, which allow extending the life of the constructions of concrete panel buildings, restoring or enhancing the use of residential buildings and reducing costs. The ultimate aim is to limit additional building reconstructions to the bare minimum, and this goal is fully consistent with the principles of sustainable development.

Some details of the extensive computational procedures required by the standard STN 73 1211: 2011 "Assessment of concrete structures of existing panel buildings" [2] for reconstructions of panel buildings and other details related to the static analysis of panel buildings are found in the publication [1]. The publication also contains a clear description of the structure and static action of different types of panel systems applied in Slovakia, the development history of their construction in the past and follow-up problems with using and reconstructions of the residential buildings at present.

3 MATERIALS AND SUBSOIL

The strength class of concrete/contact mortar is determined during the assessment of existing panel structures according to the provisions of STN ISO 13822 [4] mentioned in NC.2. It is based on a documentation of actual construction of wall panels, ceiling panels, load-bearing joints of panel structure and/or evaluation tests of concrete/contact mortar of load-bearing structure in accordance with STN EN 1990 and STN ISO 13822 Art. NA.2.6. Table NC.1 STN ISO 13822 is used for the conversion of older marks and classes of concrete/mortar to the strength class´s according to Table. 3.1 of STN EN 1992-1-1 or STN EN 206-1 .

The type of reinforcement is determined during the assessment of the existing panel buildings in accordance with the provisions of STN ISO 13822 listed in NC.3. It is based on the documentation of the actual construction of wall panels, ceiling panels, loadbearing joints of panel structure and/or empirically. Types of reinforcement may also be identified by surface treatment of reinforcement, which is shown in table NC.8 STN ISO 13822. Properties of reinforcement used for reinforced concrete load-bearing members of panel buildings, which were designed and made according to the applicable standards in the past, are taken into account by STN ISO 13822, tables NC.2 , NC.3 and NC.4 . If the type and the characteristics of reinforcement are difficult to determine, it is possible to take specimens from the reinforcement for tests. Taking of specimens must not compromise the structural function of panel building.

Load-bearing capacity and deformation characteristics of the subsoil (modulus of deformation) are determined according to the provisions of the standard STN EN 1997-1. The characteristics which correspond to the final settlement of foundations after the consolidation of subsoil shall be used in assessment of existing panel structures.

4 ACTIONS AND THEIR COMBINATIONS

Characteristic values of imposed load applied in ordinary floors of panel buildings shall be determined by taking into account the provisions of STN EN 1991-1-1. Imposed load is considered as a uniformly distributed over the surface of ceiling plate in most cases, it also can act as a linear or a concentrated load, or as their combination. Imposed areas of panel buildings are classified into the basic category A. It is allowed to take into account following characteristic values when uniform imposed area load is used: residential areas qk = 1,5 kN/m^2, stairways qk = 2,0 kN/m^2, balconies and loggias qk = 2,5 kN/m^2.

Characteristic values of horizontal wind loads applied to the panel building are determined with regard to the provisions of STN EN 1991-1-4. Usually it is sufficient to consider only two directions of bilateral action of the wind. In tower buildings with a roughly square floor plan it is recommended to take the diagonal direction of mutual action of wind in to account.

The values of horizontal seismic loads FA applied on panel buildings are determined by taking into account the provisions of STN EN 1998-1.

Settlement of the foundation gap is determined as an interaction of solid box load-bearing structure of the panel house, foundations and flexibility of the ground. Flexibility of the subsoil is expressed through the final (stabilized) settlement in the level of the foundation gap with regard to the provisions of STN EN1997-1. The solid load-bearing structure of the panel house and the foundations is simulated taking into account the final creep of concrete and flexibility of vertical joints in load-bearing walls. The values of the settlement of a foundation gap, which is caused by the possible changes in soil conditions (undermining, changing soil moisture and etc.), are referred in documents provided by the administrator of the panel house.

Design load combinations for the assessment of the panel building shall be designed to reflect the real possibility of simultaneous occurrence of loads acting from various sources and also probability of simultaneous occurrence of variable loads in their maximum values (snow, imposed loads, wind, etc.). Design load combinations for ultimate limit states to assess the mechanical resistance of structural elements, their joints and structure of panel building are created by combining schemes defined in STN EN 1990 as "Group B" labelled (STR / GEO).

5 STRUCTURAL ANALYSIS OF THE PANEL BUILDING

General requirements for the analysis of the structure have an objective to determine the course of internal forces, stresses, strains and displacements of the whole structure of a panel building, or just the parts of it. General description of the analysis of the structure is specified in 5.1 of STN EN 1992-1-1. Verification of the existing panel building needs to be done in the critical cross sections of structural elements and their joints in achieving ultimate limit state, as well as under serviceability limit state (occurrence or width of cracks, deformations) at the appropriate load combinations.

Assumptions of linear elastic action of the effects of load are allowed to be used to calculate the internal forces (M, N, V, T) and/or stresses (σ, τ) when the provision is made for specific characteristics of relations between the elements and joints of the panel building. Linear elastic analysis considers: the effect of cross-sections without cracking, linear relationship between stresses and strains, the mean value of modulus of elasticity E_{cm}.

Analysis of ceiling panels or the span of a slab to vertical loads effect can be realized by the plane 2D model with finite element method taking into account the action of longitudinal joints of ceiling panels on lateral distribution of vertical load and particular supports of the span of a slab. If the allowance of the positive effect of torsion moments m_{xy} on the magnitude of bending moments m_x, m_y was assumed, it is necessary to assess the reinforcement of ceiling panels for interaction effects of bending and torsion moments in this calculation.

Analysis of wall panels is included in the calculation of the effects of combinations of vertical and horizontal loads in the assessment of load-bearing and stiffening walls. Load-bearing structure of the existing building is created of a solid box system consisting of plane load-bearing elements of horizontal ceiling panels and an assembly of vertical load-bearing and stiffening walls. Analysis of the structural system to the effects of vertical and horizontal loads can be made by plane 3D model with finite element method taking into account

– weakening of wall panels with door and window openings,
– increased malleability of vertical joints between the wall panels and horizontal joints in floor level between the wall panels and the ceiling panels,
– interaction with the foundations and consolidated bedrock.

Shear malleability of vertical joint between the wall panels **is considered to be effectively enhanced** if the following requirements are met simultaneously:

– Vertical joints of the wall panels and longitudinal joints of ceiling panels are shifted from each other in ground plan by a distance Δ_{st} so it is valid $\Delta_{st} \geq 4$ h , where h is the thickness of the wall panel (Fig. 2).

– Wall panels are connected by concrete anchors and horizontal connecting elements in vertical joints. Horizontal connecting elements in the levels of the ceiling consist of reinforcement of band, welded scabs or short gaff spikes. There is no connecting element in the height of the vertical joint in some systems.
– Vertical joint shows no significant defects. In the case of obvious faults (overdimensional cracks, missing parts of the contact concrete or mortar, bare or corroded reinforcement in the joints and connections) it is necessary to decide on a possible method of restoration of damaged joints. If the joint of external wall panels is adapted that the crack in it cannot be the cause of failures in the use of the building (the rainwater penetrate into the crack), and if the reinforcement is protected against the corrosion in the joint, the width of the crack may be greater than it is permitted according to 7.3 STN EN 1992-1-1, but less than 1 mm.

If the shear malleability of vertical joint is effectively increased, it is allowed to do **the analysis of wall panels without modeling of joints** of structural elements (such as the monolithic plate-wall structure) and malleability of joints is accounted by using the effective modulus of elasticity of concrete $E_{c,eff}$ and the effective shear modulus of elasticity of concrete $G_{c,eff}$ as follows:

– for concrete with dense aggregate

$E_{c,eff} = 0{,}80\ E_{cm}$, $G_{c,eff} = 0{,}80\ G_{cm}$, $G_{cm} = 0{,}417\ E_{cm}$,

– for lightweight concrete with lightweight aggregate

$E_{c,eff} = 0{,}60\ E_{cm}$, $G_{c,eff} = 0{,}60\ G_{cm}$.

Assessment of resistance of load-bearing wall taking into account the interaction of stresses may be simplified in appropriate cases by using the following procedure:

– shear malleability of vertical joint between the wall panels can be regarded as effectively increased,
– critical horizontal cross sections of load-bearing wall are assessed for effects caused by vertical compression and bending perpendicular to the plane of the wall,
– the effects of horizontal normal stress and shear stress are negligible,
– if the wall is weakened by openings in an investigated area, the vertical load of the upper beam is moved into parts of the wall around the opening.

The classification of joints in the prefabricated structure of the panel building affects the way of static calculation of the load-bearing structure under the action of the considered load combinations. The mode of possible failure of the joint with another load combination needs to be considered in the classification of joints. It is necessary to pay attention in research of construction of existing panel building which is done according to STN ISO 13822. The condition of horizontal and vertical joints in gaps of bearing walls must be checked carefully.

If there are defects in some sections of the horizontal joints of bearing walls, the vertical tensile stress cannot be taken to the analysis of structure in these sections. If there are defects in the vertical joints, it is necessary to consider reduction of the vertical shear stress between wall panels.

5.1 THE INSIDE WALLS OF PANEL BUILDING

Technical parameters of the wall panels depend on the type of panel system. Thickness of the bearing walls is usually 140 mm or 150 mm. Thickness of the panels of bracing walls may be less. Wall panels were produced largely as concrete panels (Fig. 3a, without vertical load-bearing reinforcement) or exceptionally as reinforced concrete panels (Fig. 3b, with vertical load-bearing reinforcement). Reinforced concrete panels were used in the lower (maximum three) floors of certain types of panel high-rise buildings.

Vertical wall panel faces usually have vertical groove constructed of various cross- sections. Surface of vertical groove is normally indented, exceptionally smooth. Concrete anchors are created through the joint concrete/mortar in indented groove of a vertical joint of the bearing walls. These concrete anchors are significantly involved to shear resistance of a joint. Shear function of concrete anchors is provided by avoiding mutual separation of neighbouring wall panels using their connections by horizontal reinforcement applied in a wall plane. This reinforcement significantly in-creases the shear resistance of vertical joints of bearing walls and also ensures the spatial rigidity of panel building.

Connection of neighbouring panels in some types of panel systems is only provided in the top and the bottom of the wall. Connection is provided by welding of short scabs. Vertical cracking was observed in these simplified and adapted vertical joints. The vertical cracks were opening during a long time after the construction of the building and their growth rate was diminished.

5.2 EXTERNAL WALLS OF PANEL BUILDING

External walls have to satisfy static functions and also other requirements for thermal resistance, air resistance and protection against ingress of rainwater. The construction of external walls and bearing joints is addressed differently in different types of panel systems. External wall panels are divided according to the number of layers of different materials into:

- Single–Layer – from light structural concrete with surface finishes. External panels were designed as overall or as folded from four or five horizontal chord elements of light-weight concrete, which are switched vertically by two steel bars. These panels are usually on the face of the panel building. Failures are occurred on majority of switched external panels currently. Failures are occurred by increasing of horizontal cracks between the chord elements and transgression of cracks in the chord elements.
- Double-layer – stacked from the structural concrete panel and the external panel from lightweight insulating concrete, these solutions are usually used in the shield. They are also used on the face of the building in some types of the panel building.
- Three-layer – consisting of the inner load-bearing layer of concrete, from the middle layer of high efficient insulating material and from an external concrete layer (membrane). The external layer with thickness of 50 - 70 mm was made from a very dense concrete reinforced with welded mesh $\phi 4/150$-$\phi 4/150$, the cover of which was at least 25 mm from the outside of the building. The external layer is clipped

to the inner layer of yielding stirrups or diagonal spatial coupling anchors from stain-less steel ϕAK 8 mm.

Static action of the external wall panels depends on whether the components form a load-bearing wall, self-supporting or carried according to Fig.1. The multilayer external components are considered as a selfsupporing structure without retracted ceiling elements in some types of panel systems. These components are connected to load-bearing structure of inner walls only with the structural horizontal connection.

Serviceability limit states of the panel building

Assessment of crack width of reinforced structural elements is made by one of methods specified in 7.3 STN EN 1992-1-1. Width of cracks in the wall panels of plain concrete (as shown in fig. 3a) is not demonstrated by calculation. If the angular slope γ is $\leq 1/2000$, it is allowed to assume, that cracks in these panels do not rise because of load effects. It is allowed to substitute this condition of limitation of angular slope with the assessment of horizontal displacement f_W on the top of highest ceiling of the panel building. Displacement f_W is calculated due to the characteristic load of wind on the solid load-bearing structure, which is modeled by assuming zero rotation of foundation gap. The condition of the size of the angular slope γ is assumed satisfied if the high H_Z of the upper ceiling above the top level of foundation is

$$f_W \leq H_Z / 2000 . \qquad (1)$$

If the joint of the external wall panels is adapted that the crack in it cannot be the cause of failures in the use of the building (e.g. the rain water would penetrates the crack etc.), and if the reinforcement is protected against corrosion in the joint, crack width can be greater than it is allowed according to STN EN 1992-1-1 , but less than 1 mm.

Assessment of the lateral displacement of the building f_W due to the wind can be calculated according to equation (1) following the assumptions given by the assessment of crack widths.

6 RECONSTRUCTION AND ITERVENTIONS IN THE LOAD-BEARING STRUCTURE OF EXISTING PANEL BUILDINGS

It is necessary to realize a professional diagnosis before any intervention in the load-bearing structure of the panel building (to change the use of residential areas for commercial, before setting up one or more openings in load-bearing wall or ceiling, the super-structure of the building, the building insulation). Extent of necessary diagnostic tests of load-bearing structure of the panel building should be done individually based on the state of the load-bearing structure and based on type and extent of the anticipated intervention in accordance with the requirements of STN ISO 13822. The visual inspection of the state of the external surface sheet, load-bearing walls, ceilings and their joints is initial foundation for diagnosis.

Documentation of all additional interventions made into the initial load-bearing structure and their position sketch in the areas of panel building (description, drawings, structural analysis, and the agreement to intervene in the structure) must be registered by administrator of the building.

The scope of the static calculation for interventions into the structure of the

panel system results from the requirements for conservation of the static security of the building. Additional interventions should not cause excessive shear stress concentration around openings in possible places of new localized supporting of bearing walls, vertical compressive stress between the openings in the walls and horizontal tensile stresses in the head of the openings. In the case of need, it is necessary to design and realize a strengthening of load-bearing walls or ceilings not to threat the static function of load-bearing system.

7 CONCLUSIONS

Mass developing of homebuilding based on panel houses began about sixty years ago and its use lasted about forty years in Slovakia. At present, this means that the age of our panel buildings is rising and concerns directly to their intended life. Determination of technical real-life of panel buildings is a professionally challenging process and it requires the knowledge of the principles of design and static features of individual panel systems. But it is also necessary to professionally respond to the great interest of the public for changes and adjustments of accommodation in panel houses. The creation of new openings in load-bearing wall panels, super-structure of panel buildings and in some cases also additional insulation of an external cladding belong among the major interventions in the statics of panel buildings.

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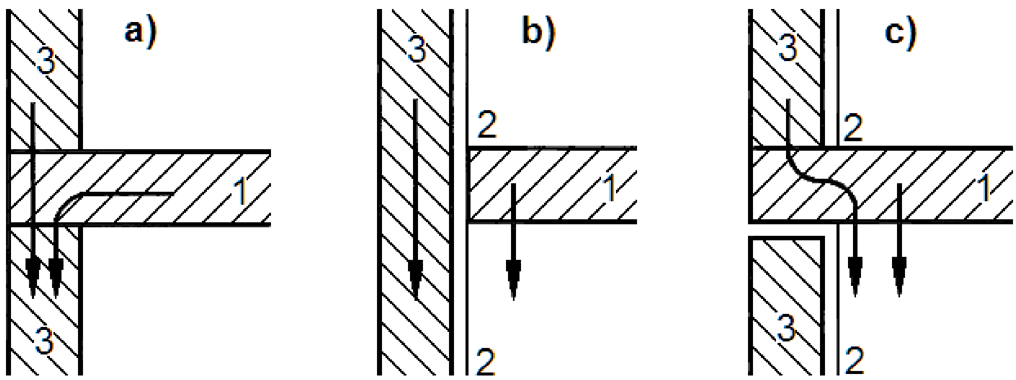

Fig. 1: Schemes of static action of the external wall, a) supporting wall b) self-supporting wall, c) carried wall, 1 – ceiling panels, 2 – transverse load-bearing wall in a direction perpendicular to the plane of the external cladding,
3 – external wall panels

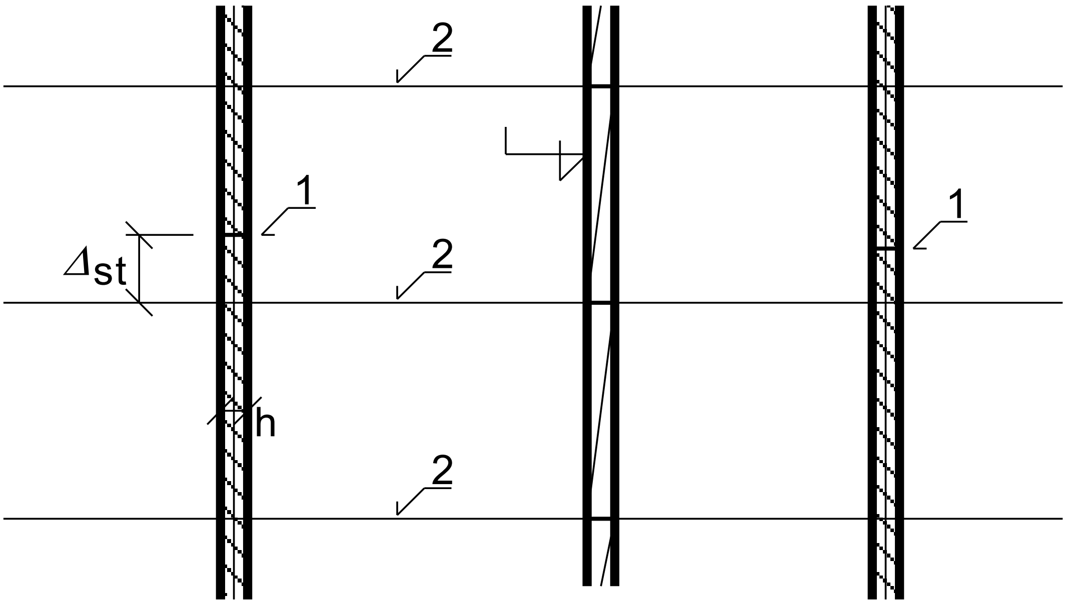

Fig. 2: Ground plan of joints of the wall panels and the ceiling panels, 1 – vertical joint of wall panels, 2 – joint of ceiling panels

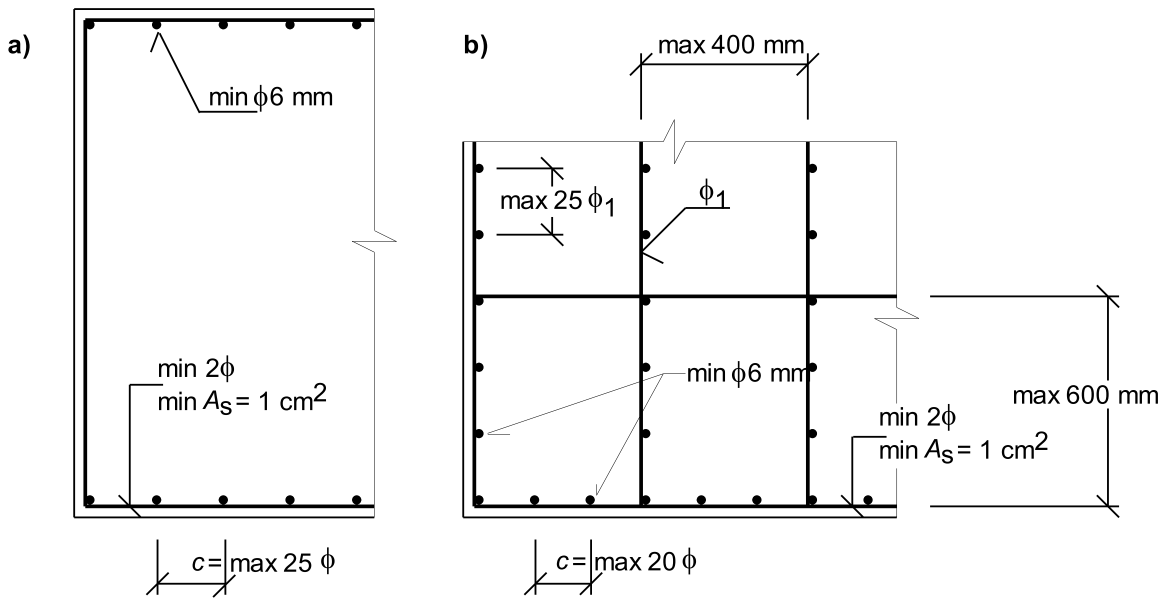

Fig. 3: The detailing of wall panel reinforcement, a) concrete wall panel, b) reinforced concrete wall panel

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SLENDER REINFORCED CONCRETE COLUMNS STRENGTHENED WITH FIBRE REINFORCED POLYMERS

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ABSTRACT

The requirement for a long life with relatively low maintenance costs relates to the use of building structures. Even though the structure is correctly designed, constructed and maintained, the need for extensions of its lifetime can appear. The preservation of the original structure with a higher level of resistance or reliability is enabled by strengthening. Conventional materials are replaced by progressive composites – mainly carbon fibre reinforced polymers (CFRP). They are used for strengthening reinforced concrete columns in two ways: added reinforcement in the form of CFRP strips in grooves or CFRP sheet confinement and eventually their combination. This paper presents the effect of the mentioned strengthening methods on slender reinforced concrete columns.

KEY WORDS

- *strengthening,*
- *slender concrete columns,*
- *fibre reinforced polymers,*
- *CFRP sheet,*
- *CFRP strips.*

INTRODUCTION

The design of slender load-bearing members is the most asserted trend today. The same requirement is then imposed on strengthened structures and members. That is the main reason for replacing strengthening techniques using conventional materials (concrete, steel) with new ones. Progressive composites – fibre reinforced polymers (FRP) – have many advantages such as high strength-to-weight and stiffness-to-weight ratios, corrosion resistance, ease of installation, etc.; their disadvantage is their relatively high cost. For static strengthening purposes, unidirectional composites with carbon fibres and epoxy resin are the most widely used.

There are two forms of carbon fibre reinforced polymers used for strengthening. Polymers in strip form are bonded to the structural member's surface or into the pre-cut grooves in the concrete cover – the well-known near surface mounted reinforcement method (NSMR). They transfer tensile forces and perform as added reinforcement with related characteristics. Polymer sheets in a confinement form can be shaped like stirrups or continuously like a spiral. The confinement effect is based on the well-known fact that the containment of the lateral deformation of concrete increases its strength. The confinement effects can be considered by the increase in the concrete's strength and the modification of the stress-strain model.

The confinement of concrete has a major effect on columns – its effect on axially loaded short squared and circular concrete columns has been demonstrated in numerous tests (also Olivová (2007)). The research on eccentrically loaded slender concrete columns is still quite limited, and there are few publications on this topic, which is why this application is not advanced.

Mirmiran, et al. (2001) started the research in this field with concrete-filled fibre-reinforced polymer tubes (CFFT), which

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showed that as the slenderness ratio is increased, the columns' strength rapidly drops [1]. Pan, et al. (2007) and Tao and Han (2007) investigated slender reinforced concrete columns wrapped with FRP; the behaviour of these columns differs from that of CFFTs, even though the strengthening effect decreases with an increase in the slenderness ratio and the initial end eccentricity [2], [3].
Confinement with a unidirectional and also a bidirectional CFRP jacket was the subject of Tao's and Yu's (2008) investigation. The ultimate strength measured for the columns (slenderness of about 70) strengthened by unidirectional CFRP is quite close to that of unstrengthened ones. The longitudinal fibres become more effective when bending becomes predominant [4]. A small-scale test of Fitzwilliam and Bisby (2010) contained columns wrapped with CFRP in a hoop and also in a longitudinal direction. For slender columns, wrapping in a hoop direction resulted in only a modest increase in capacity. Longitudinal CFRP wraps improve the behaviour of slender concrete columns and allow for the achievement of higher strengths and capacities [5].
To enable a wider application of strengthening techniques for slender concrete columns, it is necessary to engage in more research in this field and try to derive design methods for the strengthening effects of CFRPs.

EXPERIMENTAL INVESTIGATION

In order to assess the effectiveness of the CFRP laminate strips in the grooves method and wrapping with a CFRP sheet for strengthening slender concrete columns submitted to an eccentric compression load, four series of specimens were tested. The specimens were compounded of 4m long columns, with a cross section of 150 x 210 mm and reinforced with a 2 x 4 ϕ 10 hinge supported at both ends (Fig. 1). The first series consisted of non-strengthened columns; the second series was composed of concrete columns strengthened with CFRP laminate strips in the grooves; the third series was composed of columns confined with a CFRP sheet; and the last series was composed of columns strengthened with a combination of these methods.
All the columns were loaded in the same way – the compression force acted on the initial eccentricity of 40mm, and its value grew in ten loading steps to the resistance force expected from the theoretical analysis. The experimental test set-up is shown in Fig. 2. Strains of the concrete, steel and composite reinforcements and deflection from the increasing bending moment at the mid height were monitored during the increase in the compression force.
The observation of the relationship between the deflection at the mid height and the eccentrically acting compression force was the prime objective of the experimental investigation. The most accurate measurement allowing for descending branch plotting was enabled by linear variable displacement transducers (LVDTs) – Fig. 3. 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. These main measurements show good agreement with the check scaling (direct: theodolite and indirect: tensometers, deformeters). The indirect measurements allow for calculating the final deflection of the most stressed cross-section from the strains measured. Strains of the concrete, steel reinforcement and composite materials were observed.
From a comparison of the steel reinforcement and CFRP laminate strips and the tension and compression strains in the column strengthened by adding CFRP strips into the grooves in the concrete cover, it can be seen that the CFRP strip strains always have higher

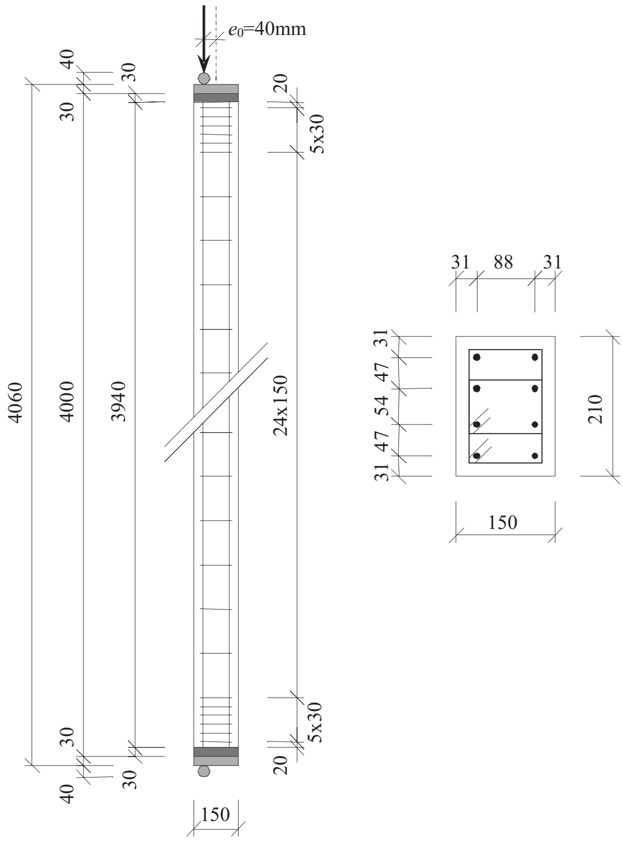

Fig. 1 *Geometry, reinforcement and fitting of a column*

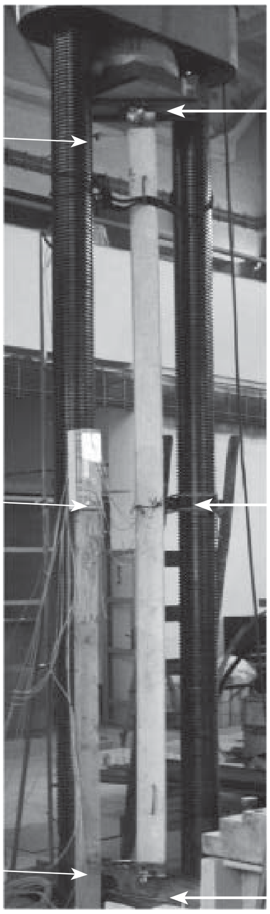

Fig. 2 *Set-up of the experimental test*

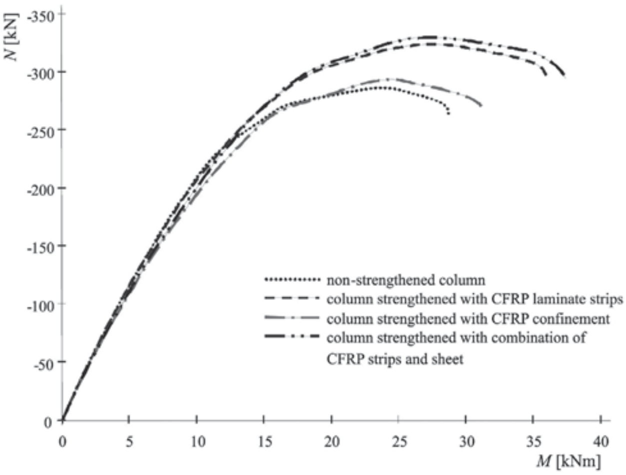

Fig. 3 *Relation between the axial force and bending moment – results from the LVDT measurements*

values, which result from their greater distance from the neutral axis. The rupture of the CFRP strips was observed at the ultimate strain of 2.5 – 2.8‰. Similar values were measured by Olivová (2007) [6], which was five times less than the values from the tension test, which can be caused by another type of stress: there is a large curvature in the most stressed cross-section of the column, and the strip is not stressed in pure tension as during the tension test but in something like direct and bending stress. That is why the utilization of the strain capacity of CFRP strips is limited.

The measured values of the CFRP sheet strains in the columns strengthened with the CFRP sheet confinement were very low – almost zero on the compressed face and a maximal 0.1‰ on the tensile face of the column's cross-section. The same values are listed in Tao and Yu (2008) [4]. These low strain values in the CFRP sheet led to a limited confinement effect – the necessary increase in the concrete' strength was not achieved.

THEORETICAL AND NUMERICAL ANALYSIS

Along with the experimental investigation, numerical modelling was executed; the ATENA 3D program was used. The material properties and static actions were adapted from full-scale tests. The numerical and experimental procedures give equivalent results. It was demonstrated that strengthening with CFRP strips in grooves is a more effective method for slender reinforced concrete columns. A theoretical analysis was carried out before testing the columns with the material properties measured according to a number of analytical models from sources abroad and then modified following the results of the experimental investigation. The models that mostly approximate the experimental results are further recommended for ordinary use.

The theoretical analysis consists of two parts. At first the resistance of a short column represented by a cross-section interaction diagram (ID) is constructed. In the second part, this diagram can be turned into a column interaction diagram to determine the resistance of a slender column without the additional second order effects. For the dimensions of the known column's cross-section, the material properties, slenderness and end eccentricity, the resistance axial force N_R can be taken from the column interaction diagram from the linear dependence of the axial force N and the first order bending moment $M_{R,0}$. In a critical cross-section, the axial force N_R and total bending moment M_R (denoting the sum of the first order moment $M_{R,0}$ and the second order moment $M_{R,II}$) act (see Fig. 4).

For the ordinary use of the mentioned strengthening techniques, it is necessary to derive the method of the CFRP materials' inclusive effect on the section and column capacity. When calculating the strengthening effect, the CFRP strips in grooves can be included

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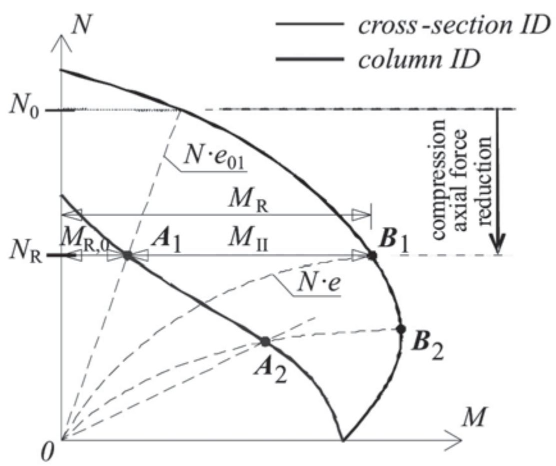

Fig. 4 *Interaction diagram of slender column*

as an additional reinforcement near the cross-section's surface; the values of their strains are determined based upon the concrete strains, which depend on their distance from the neutral axis. Following the strain values of the CFRP strips, stress is determined from the linear stress-strain relation. It is very important to make provision for the initial strains in the reinforced concrete section when the CFRP strips are applied; during strengthening the structure is maximally relieved even though there are some low strains in the concrete and steel reinforcement, and it is necessary to calculate with them.

CFRP sheet confinement can be included as an increasing concrete strength and modified stress-strain relation. Lam and Teng's stress-strain model (2003) [7] mostly approximates the experimental results. This diagram consists of a parabolic part followed by a linear part that ends at a point defined by the confined concrete's strength and ultimate strain. These values are calculated according to Teng (2007) [7]. The confined concrete's final stress and strain result from the confining pressure provided by the CFRP jacket, which depends on many factors that provision has to be made for. The most important factors are: the material properties of the CFRP sheet, the shape of the cross-section, the diameter of the rounded edges, and the continuous or stirrup-form confinement. Because of the linear σ-ε diagram of the CFRP materials, the confining pressure continuously increases with the increasing strains of the CFRP sheet. The confinement's effectiveness is highly dependent on these strains.

A comparison of the short (cross-section ID) and slender column interaction diagrams of the four series of specimens is shown in Fig. 5: As a result of the interaction diagrams, the effect of the strengthening techniques is different for short and slender columns.

CONCLUSIONS

The results of all three investigations presented indicate that strengthening columns with the use of fibre reinforced polymers allows for an increase in load-carrying capacity without a significant increase in a cross-section.

The evident difference in the effect of strengthening techniques on the short and slender reinforced concrete columns is especially conclusive from the theoretical analysis. As a result of the short

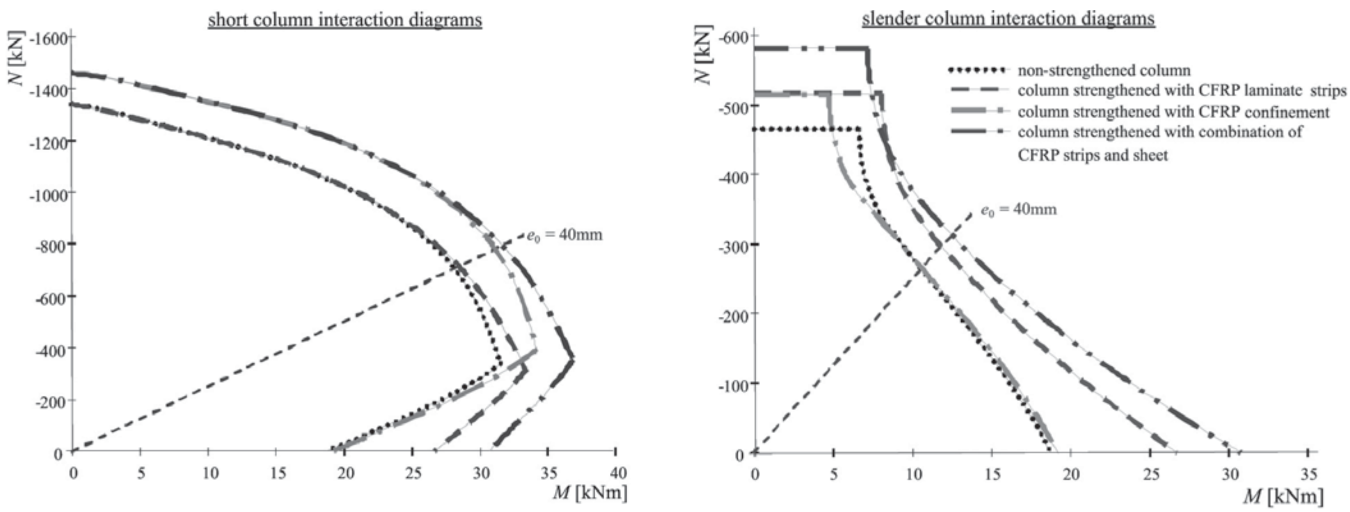

Fig. 5 *Comparison of short and slender column interaction diagrams*

columns' interaction diagrams (cross-section ID), the increase in load-carrying capacity is 1.3% for columns strengthened by near surface mounted CFRP strips, 10.8% for columns strengthened by confinement with a CFRP sheet, and 12.6% for a combination of these two methods. For slender columns, where provision is made for slenderness, the average increase in the resistance compression force beside a non-strengthened reinforced concrete column can be estimated as follows: 11.4% for columns strengthened by adding CFRP strips into the grooves in the concrete cover layer; 1.8% for columns strengthened by confinement with one layer of the CFRP sheet; 15.3% for columns strengthened by a combination of CFRP strips in the grooves and confinement with one layer of the CFRP sheet.

Generally it can be said that the most effective strengthening method depends on the predominant type of stress. The confined concrete would be most active in the case of compressive stress; longitudinal strains activate transverse strains and increase the confinement effect. The predominant compressive stress is assumed for short columns, which is why confinement by a CFRP sheet has a markedly higher effect on a short column's resistance. When calculating a strengthening action, CFRP sheet confinement can be included to increase the concrete's strength. CFRP strips act most efficiently if they are in tension; this case occurs on the tension side of a cross-section of a column strengthened with CFRP laminate strips in longitudinal grooves in a concrete cover along a column's axis. The type of stress is removed to the predominant bending region by slender columns, where the second order effects cause an increase in the bending moment at the same value of the compressive force. CFRP strips in grooves have a greater effect on the resistance of a slender column. For calculating strengthening actions, CFRP strips in grooves can be included as an additional reinforcement with related material properties.

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STRUCTURAL ANALYSIS OF BEAMS BY NEW MODEL CODE

Ľudovít Fillo

Abstract

The presented paper deals with structural analysis of hyperstatic concrete beams based on the New Model Code clauses [2]. Non-linear behavior of structural materials (a stress-strain relationship of concrete and steel), creation of cracks and the creep phenomena are reasons of a non-linear relationship between a load and bending moment distribution along a span of the beams.

Redistribution of bending moments is a first step how to take into account the above mentioned behavior of structural materials. The paper begins with analysis of redistribution formulas which are function of reinforcement, its area and distribution over a cross-section. The redistribution of continuous beams has an advantage only by different ratios of a live and a dead load by a given location of the live load.

The plastic analysis is another possibility to the realistic behavior of hyperstatic beams by increased loads and for evaluation of their reserves which depends on a required and allowed rotation of a plastic hinge.

Keywords: Reinforced concrete; Redistribution; Plastic analysis; Rotation capacity.

1 Redistribution of bending moments

Possibility to use redistribution by design of hyperstatic concrete structures was anchored in *fib* Bulletin 56*: Model Code 2010* [2] in clause 7.2.2.4.2. The bending moments at the ULS calculated using a 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 a ratio of the lengths of adjacent spans in the range of 0.5 to 2. In this case the following relations should apply:

$$\delta = 0{,}44 + 1{,}25\left(0{,}6 + \frac{0{,}0014}{\varepsilon_{cu2}}\right)\frac{x}{d} \quad \text{for concrete strength classes} \leq C50/60 \tag{1}$$

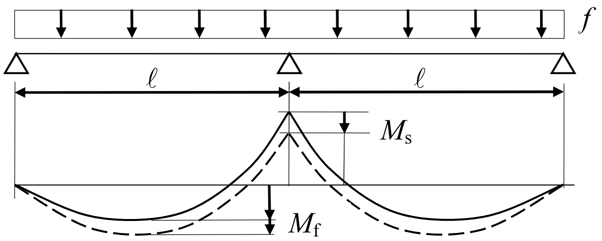
$$\varepsilon_{cu2} = 0{,}0035 \tag{2}$$

$$\delta = 0{,}54 + 1{,}25\left(0{,}6 + \frac{0{,}0014}{\varepsilon_{cu2}}\right)\frac{x}{d} \quad \text{for concrete strength classes} \geq C55/67 \tag{3}$$

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

Analysis of the redistribution formulas (1) and (3) 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. It is obvious 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 in a ratio $A_{s2} = 0{,}5\ A_{s1}$ is the possible redistribution with $\delta = 0{,}7$ under the reinforcement ratio $\rho = 0{,}031$. It is necessary to take into account also other conditions, especially the reinforcement classes.

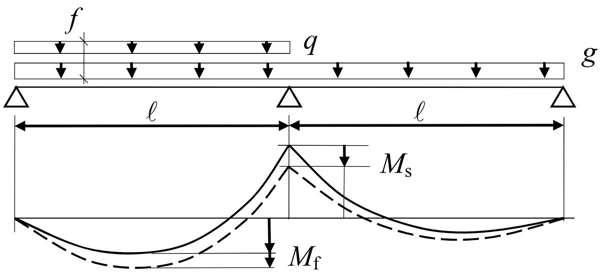
The redistribution for continuous beams has an advantage only by different ratios of live and dead load by a certain location of the live load. On the **Fig. 1** are distributions of bending moments on two span continuous beam after redistribution to $\delta = 0{,}7$. A support bending moment has decreased from $0{,}125f\ell^2$ to $0{,}088f\ell^2$ and a field moment has increased from $0{,}070f\ell^2$ to $0{,}085f\ell^2$. It does not mean significant savings of reinforcement by redistribution if any.



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

Fig. 1 Bending moments of continuous beams after redistribution

The above-mentioned advantage of the redistribution for continuous beams is obvious from **Fig. 2**. By different ratios of live and dead load and for a certain location of live load only in the first field the bending moment M_f has increased from $0{,}070f\ell^2$ to $0{,}085f\ell^2$. There are the same design field bending moments by redistribution with $\delta = 0{,}9$ and by the ratio $q/g = 0{,}25$. So there is the possibility to save with the support redistributed moment $M_s = 0{,}113f\ell^2$.



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

Fig. 2 Bending moments of continuous beams by different ratios of ***q/g***

2 Plastic analysis

The possibility to use plastic analysis by design of hyperstatic concrete structures was anchored in fib Bulletin 56: Model Code 2010 [2] in clause 7.2.2.4.3. When applying methods based on the theory of plasticity it should be ensured that the ductility of critical sections is sufficient for the envisaged mechanism to be developed. The bending moment for which a hyperstatic continuous beam is designed can yield and form the plastic hinge prior to failure and must be able to rotate sufficiently for creation of other plastic hinges for the envisaged mechanism.

Plastic analysis without any check of the rotation capacity may be used for the ultimate limit state if all the following conditions are met:
– the area of tensile reinforcement is limited to such a value that at any section
$x/d \leq 0.25$ for concrete strength classes ≤ C50/60 (5)
$x/d \leq 0.15$ for concrete strength classes > C50/60 (6)
– reinforcing steel is either Class B or C
– the ratio of the intermediate support moment to the moment in the span is between 0.5 and 2.

Analysis of conditions (5) and (6) by relationship of a mechanical reinforcement ratio ω with a ratio of *x/d* comes from a possible concrete strength class and from an area and a location of reinforcement over a cross-section. It is obvious that by the concrete strength class C50/60 the plastic analysis is possible without any check of the rotation capacity by a reinforcement ratio ρ = 0,013 (by taking into account only A_{s1}). By tension and compression reinforcement in the ratio A_{s2} = 0,5 A_{s1} it is possible under the reinforcement ratio ρ = 0,041. It is necessary to take into account also other conditions, especially the reinforcement class B – C.

If hyperstatic beams do not meet the conditions for which no check of rotation capacity is required (see (5) and (6)), a simplified procedure can be used. This procedure is based on a control of the rotation capacity. The rotation capacity is determined over a length of approximately 1.2 times the depth of the section by continuous beams. It is assumed that these zones undergo a plastic deformation (formation of yield hinges) under the relevant combination of action. The verification of the plastic rotation in the ultimate limit state is considered to be fulfilled, if it is shown that under the relevant combination of actions the calculated rotation θ_s is smaller than the allowable plastic rotation, see **Fig. 3**.

In regions of yield hinges, *x/d* should not exceed the value 0.45 for concrete strength classes less or equal to C50/60, and 0.35 for classes higher than or equal to C55/67. The rotation θ_s should be determined on the basis of design values for action and materials.

In accordance with last sentences there are presented results of a required and allowed rotation analysis of the plastic hinge versus different relative neutral axes depth of the critical section ***a*** by bending resistance (**Fig. 3)**. The beam is analyzed by uniformly distributed load. The critical sections of the beam ***a*** and ***b*** were reinforced with the equal area of high ductile steel bars B 500C. A relationship between the allowable plastic rotation $\theta_{pl,d}$ and *x/d* was defined in *Model Code 2010* [2] Figure 7.2-5.

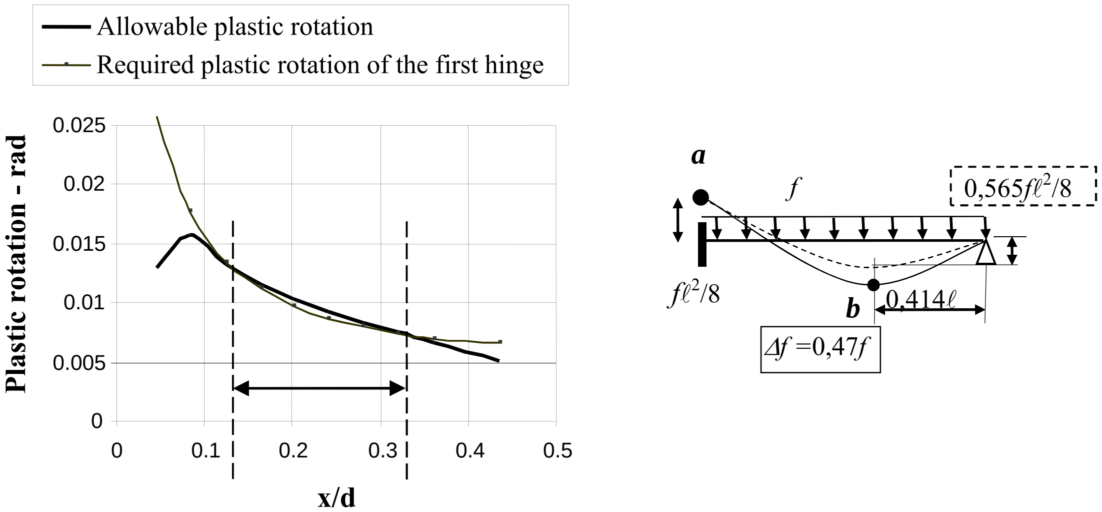


Fig. 3 The required and allowable plastic rotation in node ***a*** to creation of a plastic mechanism

On the **Fig. 3** are presented ranges of ***x/d*** where required plastic rotation of the hinge ***a*** is under allowable value and therefore the plastic mechanisms can be fully developed in the hyperstatic beam.

It is to add that for the sake of comparison the same area of reinforcement steel bars were used in sections ***a*** and ***b***.

3 Conclusions

The paper deals with redistribution analysis of hyperstatic concrete beams based on the New Model Code [2] clause 7.2.2.4.2 and plastic analysis – clause 7.2.2.4.3.

The paper begins with analysis of redistribution formulas. This redistribution for continuous beams has an advantage only by different ratios of a live and dead load by a possible location of the live load (see **Fig.1 and 2**).

The plastic analysis without any check of the rotation capacity may be used for the ultimate limit state if the conditions (5) and (6) are met. Other it should be shown that under the relevant combination of actions the calculated rotation θ_s is smaller than the allowable plastic rotation. On the **Fig. 3**, there are presented results of a required and allowed rotation analysis of the plastic hinge versus different relative neutral axes depth of the critical section a by bending resistance.

This outcome has been achieved with the financial support of the research project granted by Slovak Grant Agency VEGA 1/0857/11. All support is gratefully acknowledged.

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BIAXIAL BENDING OF SLENDER COLUMNS

Ľudovít Fillo [1], Vladimír Benko[2]

Abstract

Paper deals with columns subjected to axial loads accompanied by bending about two perpendicular axes. Corner columns in a frame or a bridge pillar are typical examples.
Analyzed are methods which take into account the second order effects to fulfil principles of EN 1992-1-1. Verified are methods recommended by EN with another appropriate and with non-linear general methods. Presented is verification of real bridge pillar.

Keywords**:** concrete slender column, biaxial bending, second order effects.

1 Introduction

The slender columns which are subjected to axial force and bending moments are also influenced by second order effects provided that the columns overcome the slenderness criterion. Special care should be taken to identify the section along the member with the critical combination of the moments. Simplified methods are always necessary to verify software calculations. In following parts are presented two simplified methods for design of columns subjected to biaxial bending with final comparison of results illustrated on an example of the bridge pillar**.**

2 Simplified methods for biaxial bending

2.1 Design in both directions

The general method [1] 5.8.6, based on non-linear analysis, including geometric non-linearity i.e. second order effects, may also be used for biaxial bending. The same principles as in uni-axial bending are applied, although the complexity of the problem increases. The following provisions can be applied when simplified methods are used:
- Separate design in each principal direction, disregarding biaxial bending, may be made as a first step.
- Imperfections need to be taken into account only in the direction where they will have the most unfavourable effect.
- No further check is necessary if the slenderness ratios satisfy the following condition:

$$0{,}5 \le \lambda_y / \lambda_z \le 2 \tag{1}$$

And also if the relative total eccentricities e_y/h and e_z/b (see Figure 1) satisfy the condition (2)

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$$5{,}0 \le \frac{e_y / h}{e_z / b} \le 0{,}2 \qquad (2)$$

where: $e_z = M_{Edy} / N_{Ed}$; eccentricity along z-axis
$e_y = M_{Edz} / N_{Ed}$; eccentricity along y-axis
M_{Edy} is the design moment about y-axis, including second order moment
M_{Edz} is the design moment about z-axis, including second order moment
N_{Ed} is the design value of axial load in the respective load combination

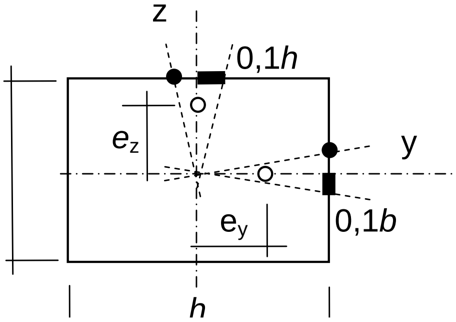

Fig. 1 Relative eccentricity criterion

Comment: The e_z and e_y are total eccentricities including second order effects.

If the condition of (1) and (2) are not fulfilled, biaxial bending should be taken into account including the 2nd order effects in each direction – unless they may be ignored according to [1] 5.8.2 (6) or 5.8.3. In the absence of an accurate cross section design for biaxial bending, the following simplified criterion may be used:

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \le 1{,}0 \qquad (3)$$

where: $M_{Edz/y}$ is the design moment around the respective axis, including a 2nd order effects;
$M_{Rdz/y}$ is the moment resistance in the respective direction;
a is the exponent; for circular and elliptical cross sections: a = 2

for rectangular cross sections:

N_{Ed}/N_{Rd}	0,1	0,7	1,0
a =	1,0	1,5	2,0

N_{Ed} is the design value of axial force
$N_{Rd} = A_c f_{cd} + A_s f_{yd}$, design axial resistance of section.
where: A_c is the gross area of the concrete section
A_s is the area of longitudinal reinforcement

The stiffness and curvature methods can be used separately for each direction if the resulting moments fulfill criterion (3), no further action is necessary.

2.2 Design by fictive eccentricity

By this simplified method the biaxial bending is transformed for axial bending with increased "fictive" eccentricity. This eccentricity is a function of both eccentricities e_z and e_y and of relative parameter of normal force (4).

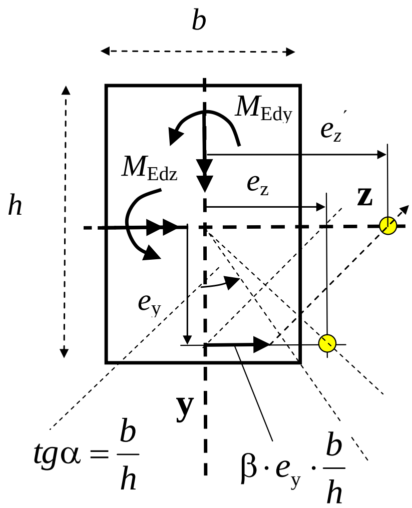

Fig. 2 The fictive eccentricity

$$\nu = \frac{N_{Ed}}{b \cdot h \cdot f_{cd}} \qquad (4)$$

The "fictive" eccentricity in z or y direction (7),(8):

$$\nu \le 0{,}33 \Rightarrow \beta = 0{,}6 + \nu \tag{5}$$

$$\nu > 0{,}33 \Rightarrow \beta = 1{,}131 - 0{,}609 \cdot \nu \tag{6}$$

$$\frac{e_z}{e_y} \ge \frac{h}{b} \Rightarrow e_z' = e_z + \beta \cdot e_y \cdot \frac{h}{b} \tag{7}$$

$$\frac{e_z}{e_y} < \frac{h}{b} \Rightarrow e_y' = e_y + \beta \cdot e_z \cdot \frac{b}{h} \tag{8}$$

Comment: In the case of mechanical ratio of reinforcement $\omega > 0{,}6$, it is to increase coefficient β about 0,1. In the case of mechanical ratio of reinforcement $\omega < 0{,}2$, it is to decrease coefficient β about 0,1.

3 Comparison of results by illustrative example

3.1 Geometry and loads

R_{Ed} = 9283 kN
R_{Eqp} = 5620 kN
G_d = 1299 kN
N_{Ed} = R_{Ed} + $G_d/3$= 9716 kN
H_{wd} = 308 kN
H_{trd} = 132 kN
$M_{0Edy} = R_{Ed}\,(e_{i1}+e_{i2}) + G_d/3.e_{i1} + H_{wd}.\ell$ = 5399 kN
$M_{0Edz} = H_{trd}.\ell$ = 5565 kN
$M_{0Eqpy} = R_{Eqp}\,(e_{i1}+e_{i2}) + G_s/3.e_{i1}$ = 476 kN
M_{0Eqpz} = 0 kN
$e_{0z} = M_{0Edy}/\,N_{Ed}$ = 555 mm
$e_{0y} = M_{0Edz}/\,N_{Ed}$ = 573 mm

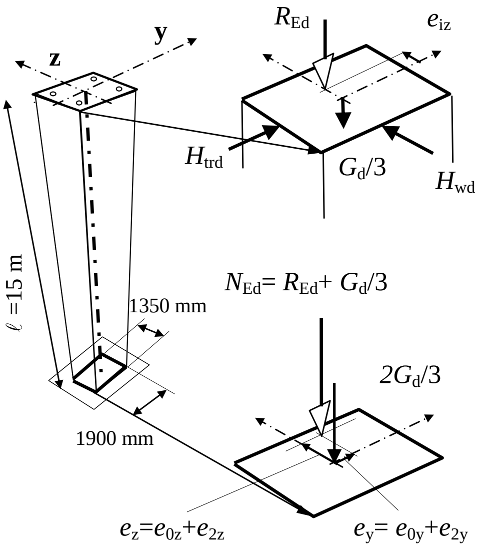

Fig. 3 Geometry and loads on bridge pillar

3.2 Materials and durability

Concrete class C 30/37; Steel B 500B; The basic creep coefficient φ =2,5;
Exposure classes XC4,XD1; Structural class S6; Concrete cover cnom=55 mm

3.3 Design in both directions

Slendernes about z axis $A_c = 2{,}565\ \mathrm{m}^2$ $I_{cz} = 0{,}772\ \mathrm{m}^4$ $\lambda_z = \frac{\ell_0}{i_z} = \frac{32{,}7}{0{,}548} = 59{,}6$

Slendernes about y axis $A_c = 2{,}565\ \mathrm{m}^2$ $I_{cy} = 0{,}390\ \mathrm{m}^4$ $\lambda_y = \frac{\ell_0}{i_y} = \frac{32{,}7}{0{,}390} = 83{,}9$

Criterion (1) $0{,}5 \le \dfrac{\lambda_y}{\lambda_z} = \dfrac{83{,}9}{59{,}6} = 1{,}41 \le 2$ satisfied ;

Criterion (2) $5{,}0 \le \dfrac{e_y / h}{e_z / b} = \dfrac{0{,}797/1{,}9}{0{,}907/1{,}35} = 0{,}624 \le 0{,}2$ not satisfied – curvature method;

Criterion (2) $5{,}0 \le \dfrac{e_y / h}{e_z / b} = \dfrac{0{,}798/1{,}9}{0{,}812/1{,}35} = 0{,}698 \le 0{,}2$ not satisfied – stiffness method;

- **Curvature method – design moments - reinforcement**

Eccentricity of second order $e_{2z} = \dfrac{\ell_0^2}{r.c} = \dfrac{32{,}711^2}{270{,}3.11{,}2} = 0{,}352\ m$

Design moment in critical cross section

$$M_{Edy} = M_{0Edy} + N_{Ed}.e_{2z} = 5399 + 9716.0{,}352 = 8811\ \text{kNm}$$

Eccentricity of second order $e_{2y} = \dfrac{\ell_0^2}{r.c} = \dfrac{32{,}711^2}{400.12} = 0{,}224\ m$

Design moment in critical cross section

$$M_{Edz} = M_{0Edz} + N_{Ed}.e_{2y} = 5565 + 9716.0{,}224 = 7741\ \text{kNm}$$

$$\left(\frac{M_{\text{Edz}}}{M_{\text{Rdz}}}\right)^a + \left(\frac{M_{\text{Edy}}}{M_{\text{Rdy}}}\right)^a = \left(\frac{7741}{18168}\right)^{1{,}125} + \left(\frac{8811}{13450}\right)^{1{,}125} = 1{,}004 \approx 1{,}0$$

$$a = 1 + \frac{n_R - 0{,}1}{0{,}6} = 1 + \frac{0{,}175 - 0{,}1}{0{,}6} = 1{,}125$$

$$n_R = \frac{N_{Edtot}}{A_c f_{cd} + A_s f_{yd}} = \frac{10581}{2{,}565.17 + 0{,}0386.435} = 0{,}175$$

Reinforcement design $A_s = 38604\ \text{mm}^2$

- **Stiffness method – design moments - reinforcement**

Design moment in critical cross section

$$M_{Edy} = M_{0Ed1y}\left(1 + \frac{\beta_{const}}{\dfrac{N_B}{N_{Ed}} - 1}\right) + M_{0Ed2y}\left(1 + \frac{\beta_{tri}}{\dfrac{N_B}{N_{Ed}} - 1}\right) = 7889\ \text{kNm}$$

$$M_{Edz} = M_{0Ed2z}\left(1 + \frac{\beta_{tri}}{\dfrac{N_B}{N_{Ed}} - 1}\right) = 7748\ \text{kNm}$$

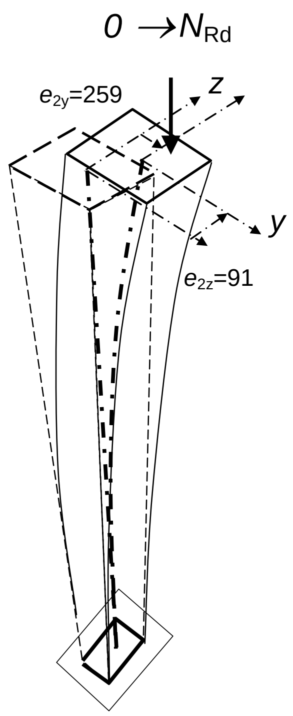

Fig. 6 General method

$$\left(\frac{M_{\mathrm{Edz}}}{M_{\mathrm{Rdz}}}\right)^{a}+\left(\frac{M_{\mathrm{Edy}}}{M_{\mathrm{Rdy}}}\right)^{a}=\left(\frac{7748}{16937}\right)^{1,11}+\left(\frac{7889}{12728}\right)^{1,11}=1,008\leq 1,0$$

$$a=1+\frac{n_R-0,1}{0,6}=1+\frac{0,166-0,1}{0,6}=1,11$$

$$n_R=\frac{N_{Edtot}}{A_c f_{cd}+A_s f_{yd}}=\frac{10581}{2,565.17+0,034482.435}=0,166$$

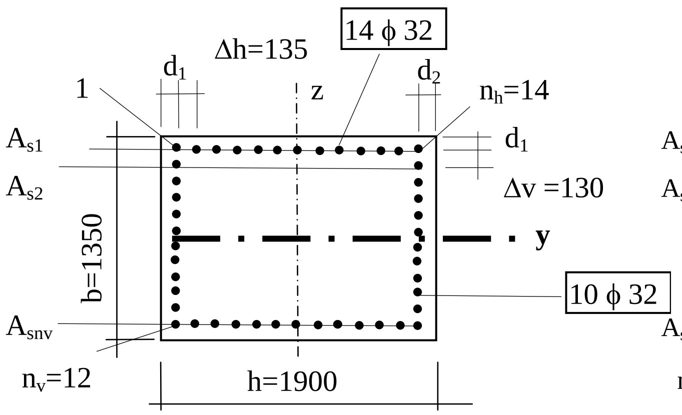

Fig. 4 Reinforcement design - curvature method

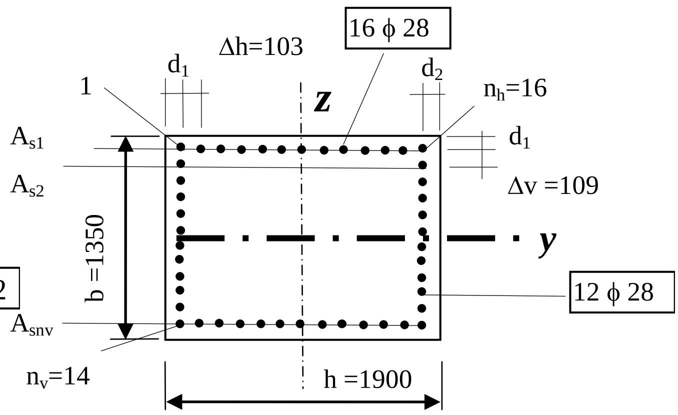

Fig. 5 Reinforcement design - stiffness method

3.4 Design by fictive eccentricity

- **Curvature method – design moment - reinforcement**

$$\nu=\frac{N_{\mathrm{Ed}}}{b\cdot h\cdot f_{\mathrm{cd}}}=\frac{9716}{1,35.1,9.17}=0,223<0,33\Rightarrow\beta=0,6+\nu=0,6+0,223=0,823$$

$$\frac{e_z}{e_y}=\frac{0,907}{0,797}=1,138\geq\frac{b}{h}=\frac{1,35}{1,9}=0,71\Rightarrow\ e_z^{'}=e_z+\beta\cdot e_y\cdot\frac{h}{b}=0,907+0,823.0,797\frac{1,35}{1,9}=1,373$$

$$M_{\mathrm{Edy}}=N_{\mathrm{Ed}}.e_z^{'}=9716.1,373=13340\ \mathrm{kNm}\qquad\Rightarrow\qquad A_s=32032\ \mathrm{mm}^2$$

- **Stiffness method – design moment - reinforcement**

$$\nu=\frac{N_{\mathrm{Ed}}}{b\cdot h\cdot f_{\mathrm{cd}}}=\frac{9716}{1,35.1,9.17}=0,223<0,33\Rightarrow\beta=0,6+\nu=0,6+0,223=0,823$$

$$\frac{e_z}{e_y}=\frac{0,812}{0,798}=1,018\geq\frac{b}{h}=\frac{1,35}{1,9}=0,71\Rightarrow\ e_z^{'}=e_z+\beta\cdot e_y\cdot\frac{h}{b}=0,812+0,823.0,797\frac{1,35}{1,9}=1,280$$

$$M_{\mathrm{Edy}}=N_{\mathrm{Ed}}.e_z^{'}=9716.1,28=12420\ \mathrm{kNm}\qquad\Rightarrow\qquad As=28336\ \mathrm{mm}^2$$

3.5 Comparison of design methods

Tab. 1 – Biaxial bending with axial force

Method	$A_{s,req}$ [mm] - total area of longitudinal reinforcement
Design in both directions – curvature m.	38604 - 48 ϕ 32
Design in both directions – stiffness m.	34482 - 56 ϕ 28
General method	24550 - 50 ϕ 25
Design by fictive eccentricity– curvature m.	32032 - 52 ϕ 28
Design by fictive eccentricity– stiffness m.	28336 - 46 ϕ 28

3.6 Detailing

Rules of longitudinal and transverse reinforcement detailing are in [1] 9.5.2

The total amount of longitudinal reinforcement should not be less than

$$(A_{s,min} = \frac{0{,}1 N_{Ed}}{f_{cd}} = \frac{0{,}1.9{,}716}{435} = 0{,}0022 \text{ or } 0{,}002\, A_c)$$

$$A_{s,min} = 5643\, mm^2 \quad A_{s,max} = 0{,}04 A_c = 102600\, mm^2$$

The spacing of the transverse reinforcement along the column should not exceed:
$s_{cl,tmax}$ =min(20ϕ; 400 mm; b=1350mm).

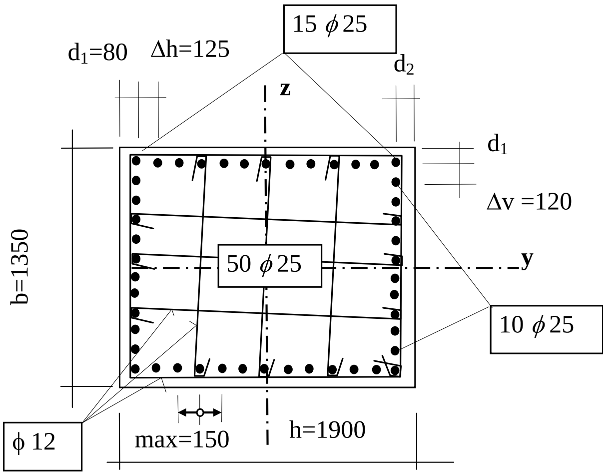

Fig. 7 Detailing of cross-section

No bar within a compression zone should be further than 150 mm from a restrained bar.

Conclusions

The paper presents the use of auxiliary, check methods for design of slender columns subjected to axial force and biaxial bending moments taking into account second order effects. The simplified methods are compared also with general method – illustrated on example of concrete bridge pillar.

Acknowledgement

This contribution has been prepared with the financial support of Slovak Grand Agency VEGA 1/0857/11.

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FLAT SLABS - MOMENTS DISTRIBUTION AND DESIGN OF REINFORCEMENT

Andrej Bartók[1]

Flat slabs have a large field of application in buildings and civil engineering. There are various methods to analyse of such structures, more or less complex. This paper deals with two flat slabs analysis topics.

Keywords: flat slab, twisting moments, Wood-Armer moments, Baumann's method, equivalent frame method, results along a section

1 INCORPORATION OF TWISTING MOMENTS TO SLAB REINFORCEMENT DESIGN – DESIGN MOMENTS

Predominant internal forces for flat slabs are bending moments m_x, m_y and twisting moments m_{xy}. Different structural analysis methods give different set of results.

Since the advent of computer era simplified methods of flat slab analysis were used, giving usually bending moments only. Common way of reinforcement design in such cases is design to pure bending and add reinforcement according to code recommendations at regions, where twisting moments are expected.

Computer era brought development of finite element method analysis of structures, that gives both bending and twisting moments. The problem emerged, how to incorporate twisting moments to slab reinforcement design.

Solution introduced R. H. Wood in 1968 for orthogonal reinforcement and G. S. T. Armer for skew reinforcement. [1] Their equations are called Wood-Armer method, Wood-Armer moments.

Another solution was introduced in 1972 by Theodore Baumann [2]. Baumann's method is used mainly in Germany (DIN).

Both methods determine the moments, reinforcement should be designed for by the normal analysis of a section in bending. These moments are usually called the design moments m_x* , m_y* (not in the meaning of characteristic/design load). Since the action of twisting moments may be positive or negative, design moments are given for top and bottom reinforcement separately, providing the most safe values.

Another methods were introduced later, but most commercial structural analysis software packages use one of mentioned methods, sometimes modified. Almost all of them only offer one method. Applied method should be explained in software documentation.

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2 INTERPRETATION OF THE FINITE ELEMENT ANALYSIS RESULTS OF FLAT SLABS

Finite element method is very powerful tool for analysis of structures, but interpretation of results may be complicated.

There are a lot of options to visualize results of analysis. Most common are contour lines. It looks effective, but need not be proper for practical reinforcement design. Negative bending moments are concentrated around supports (columns). There are some ways, how to make these peak values more real. Furthermore, it is necessary to convert rather complicated bending moments distribution to simply, regular arrangement of reinforcement.

More suitable is to display results along a defined section. Most of programs enable to sum results over a strip of defined width, perpendicular to section direction. Such type of results display cuts down local extremes over supports.

Wide spread method for analysis of flat slabs is equivalent frame method. Structure is divided into frames, running through the building. Frames consist of columns and corresponding parts of slab. Frames are analysed and total bending moments are distributed across the width of the slab into column and middle strip using distribution coefficients, recommended by design codes. Fig. 1 displays distribution coefficients according to Eurocode 2 [3].

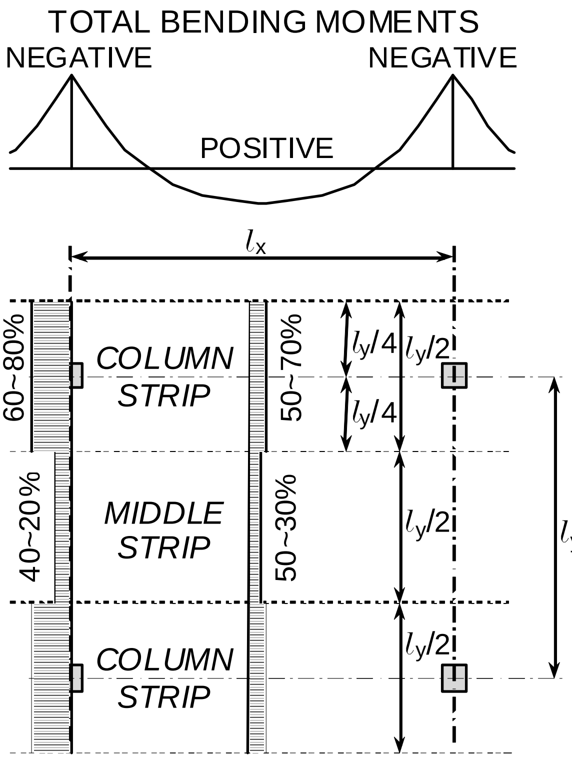


Fig. 1 Equivalent Frame Method distribution coefficients

Next two figures display results of analysis of flat slab - bending moments m_x. Fig. 2 shows contour lines, Fig. 3 results along a two sections at centres of column and middle strips. Dotted line represents results along a section line without any summation, solid line is sum of results over a whole column/middle strip 3,75m. Moments per unit *m* show, that summation considerable cut down local extremes of negative moments over the columns. Percent relationships between moment at column and middle strip - 57:43 for positive moment, 83:17 and 81:19 for negative moments - are consistent with distribution coefficients of equivalent frame method – Fig. 1.

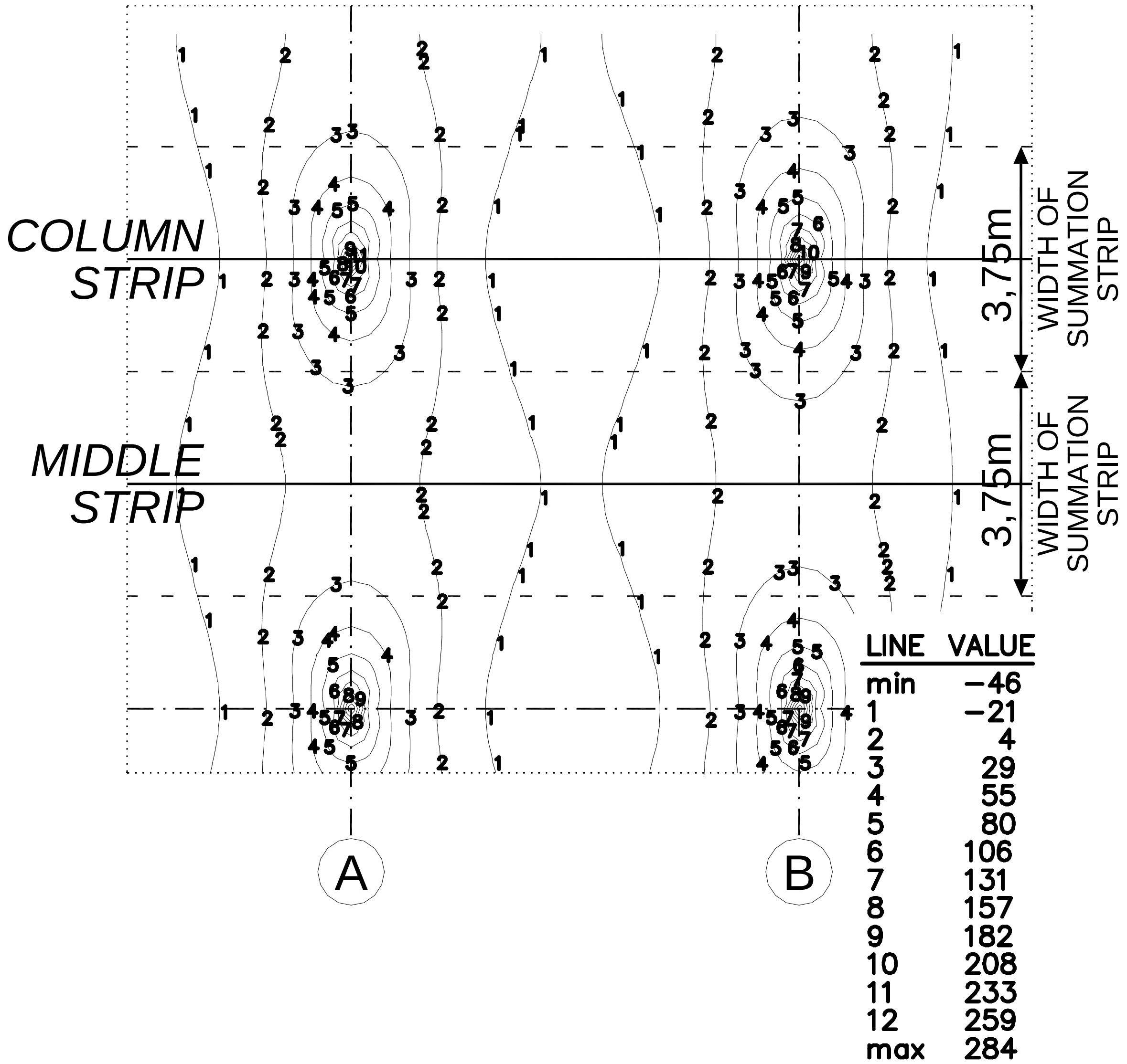
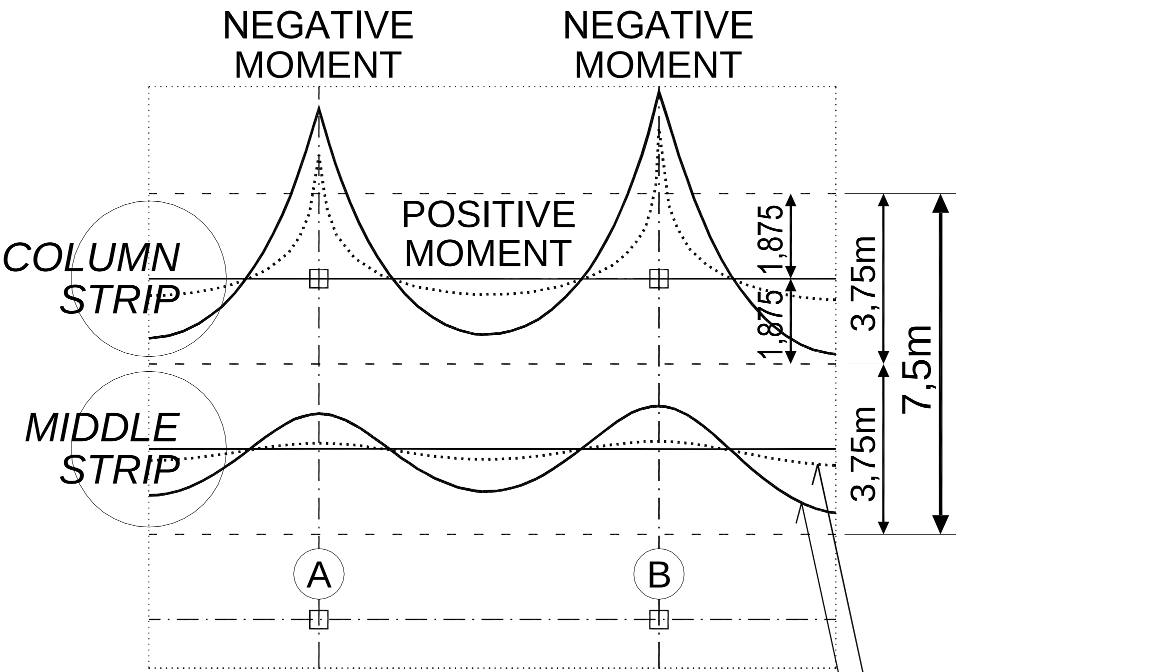


Fig. 2 m_x contour lines



STRIP:	M	$m=\frac{M}{1}$	%	M	$m=\frac{M}{1}$	%	M	$m=\frac{M}{1}$	%
COLUMN	-272	-272	95	36	36	61	-331	-331	95
MIDDLE	-13	-13	5	23	23	39	-17	-17	5
Σ	-285	-285	100	59	59	100	-348	-348	100

STRIP:	M	$m=\frac{M}{3{,}75}$	%	M	$m=\frac{M}{3{,}75}$	%	M	$m=\frac{M}{3{,}75}$	%
COLUMN	-374	-100	83	124	33	57	-411	-110	81
MIDDLE	-77	-20	17	92	25	43	-94	-25	19
Σ	-451	-120	100	216	58	100	-505	-135	100
	NEGATIVE MOMENT			POSITIVE MOMENT			NEGATIVE MOMENT		

Fig. 3 m_x along sections

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BRIDGE APROACH SLABS DESIGN FOR MOTORWAY AND HIGHWAY IN SLOVAKIA ACCORDING TO EN 1992-2

Viktor Borzovič[1], Ján Laco[2]

Abstract

Approach slab is a construction element of transition areas, which compensates height difference between settlement of bridge abutment and road body. Geometry of transition slab, that means length and thickness, is based on that settlement. For reinforcing of this concrete element designers usually use simple manuals. In Slovakia is based on the analysis under the Czech and Slovak national standards. Design rules, reinforcing design for ultimate limit state and fatigue verification according to Eurocodes is a subject of this article.

Keywords: approach slab, transition area, European standards

1 Introduction

The designer should to verify on the contact of road body and bridge construction transition area and design finishing, which prevents inadequate height difference from non-uniform settlement of the road body and bridge abutment. Transition finishing is represented generally with transition slab, which is designed if the road body and abutment are higher than 3.0 m.

2 Geometry of approach slabs

Length of transition slab depends on value of height difference *h* at the end of bridge construction, which is also difference of total settlement Δs of road embankment and settlement of bridge Δm, seefig.1. For motorways and highways designed on speed higher than 80 km/h is allowed grade change of roadbed level 0,4%, for other roads it is 0,8%, see fig.2. Width of a slab must be at least same as access width of the bridge, or same as length within hard shoulder of the road. Thickness corresponds to the length and for its determination standard [3] offer the chart shown on fig.3.

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3 Static behaviour

Transition slabs are designed dipped in angle from abutment with grade 1:10 to 1:15. Top surface is connected to abutment and should be approximately in same level as a bridge deck. Slab is pinned to abutment unmovable in any direction. On free end it is considered simple supporting on gravel layer. Transition slabs are usually executed monolithically on of plain concrete with minimal thickness 50 mm. From top surface are isolated against ground moisture.

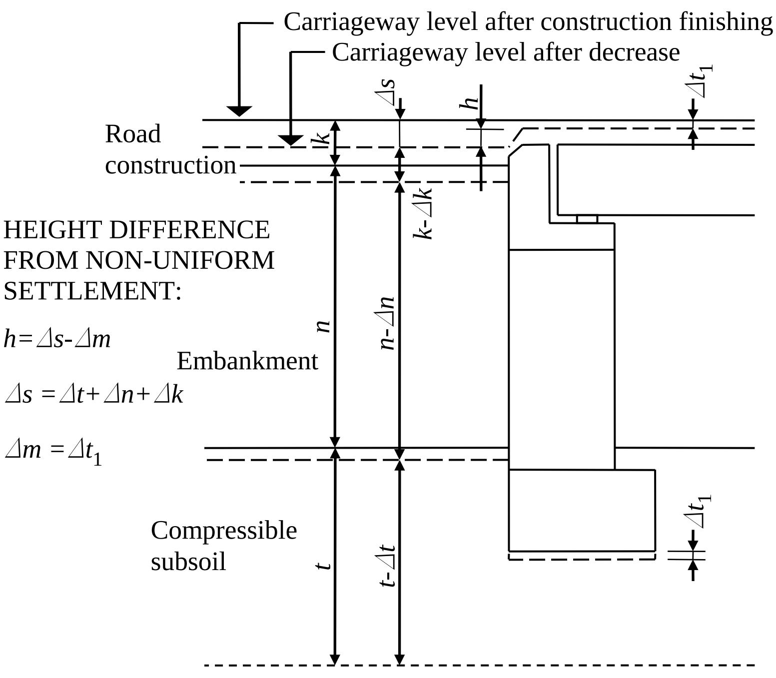


Fig. 1 Schema of non-uniform settlement

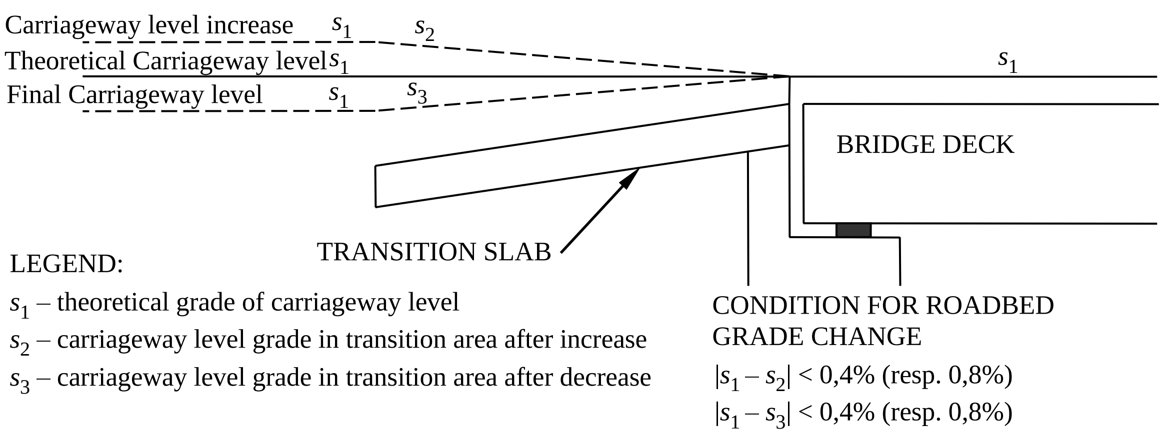


Fig. 2 Schema of roadbed level change in transition area

According to [4] exposure and concrete strength class of transition slab is determined on C25/30-XC2, XF1(SK). For top surface can be applied same rule as for top surface of the bridge deck isolated against water and there for is environment XC3. National annex to STN EN1992-1-1considers for defined indicative strength classes for durability concrete class C25/30, which is one class lower as recommended class by standards. That means the transition slab would be from concrete class C30/37. From environment class, concrete strength class and monolithically execution method can be determined reinforcement cover as: $c=c_{min,dur}+\Delta c_{dev}$=30+10=40mm.

Original standard from 80‘s [3], which was reclassified on technical prescription (OTN), prescribes cover for top surface 20 mm and only 10 mm for bottom surface. Reinforcement design on bottom surface in longitudinal direction is based bending moments chart (see fig.3), depending on length of transition slab. Intensity of bending moments is independent from subsoil stiffness. From that can be expected extreme static schema, where the embankment in the central part underneath the slab is totally consolidated and the slab behave as simple beam supported on both ends. In lot of manuals and literature we can find reinforcement schema of transition slabs probably based on bending moments curve. At bottom surface is reinforcement according to table 1.

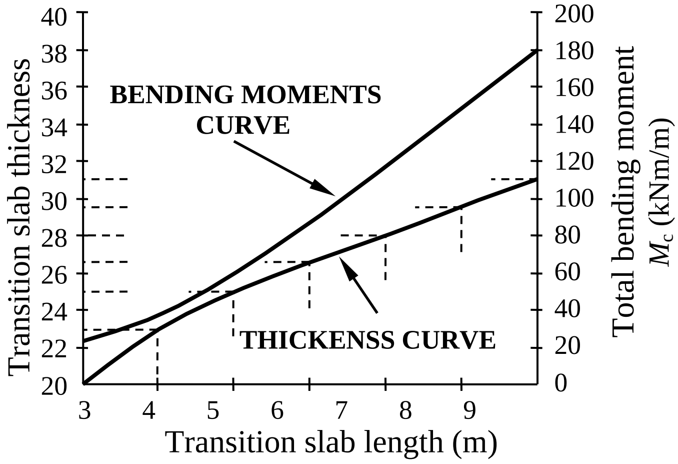


Fig. 3 Chart for thickness determination and total bending moments according to [3]

Tab. 1 Concrete reinforcement of transition slabs at bottom surface.

Transition slab length [m]	3	4	5	6	7	8	9
Reinforcement diameter [mm]	12	12	14	16	18	20	22
Distance between bars [mm]	110						

The first question is, if the reinforcement in published schemas is sufficient also for enlarged cover from 10 mm to 40 mm. Given to date of chart with bending moments curve, reinforcement verification was designed on allowed stresses theory. ($\sigma_{c,dov}$=15,5 MPa; $\sigma_{s,dov}$=280 MPa). Result of that comparison is on fig. 4. We can declare, more reinforcement caused with thicker cover is not necessary in accordance with reinforcement from tab. 1.

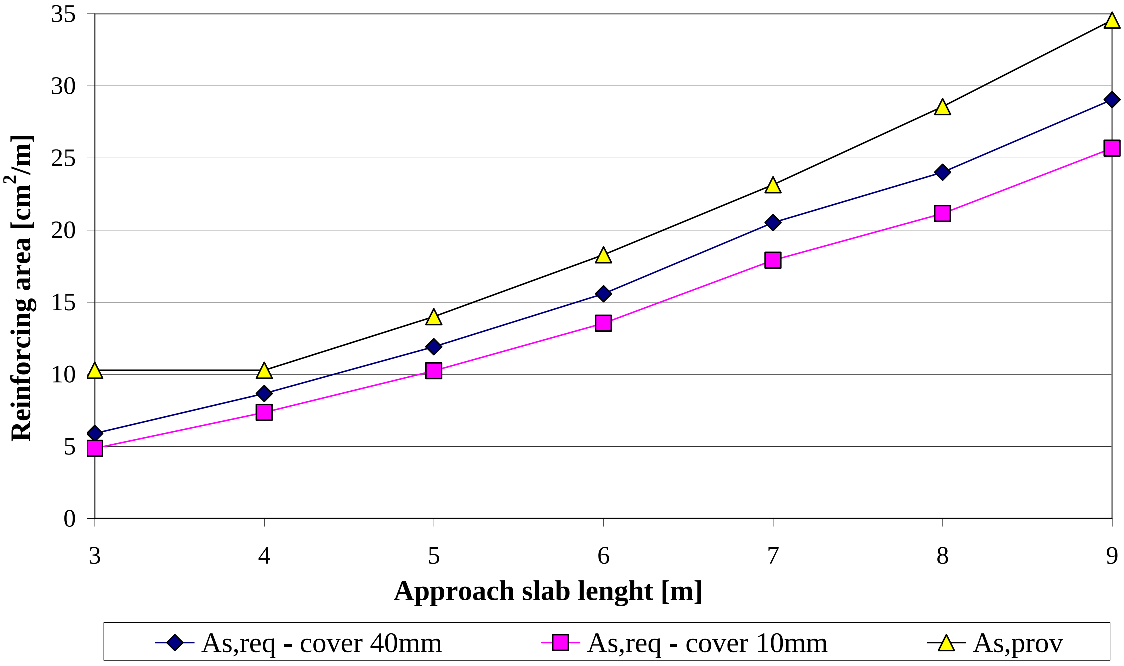
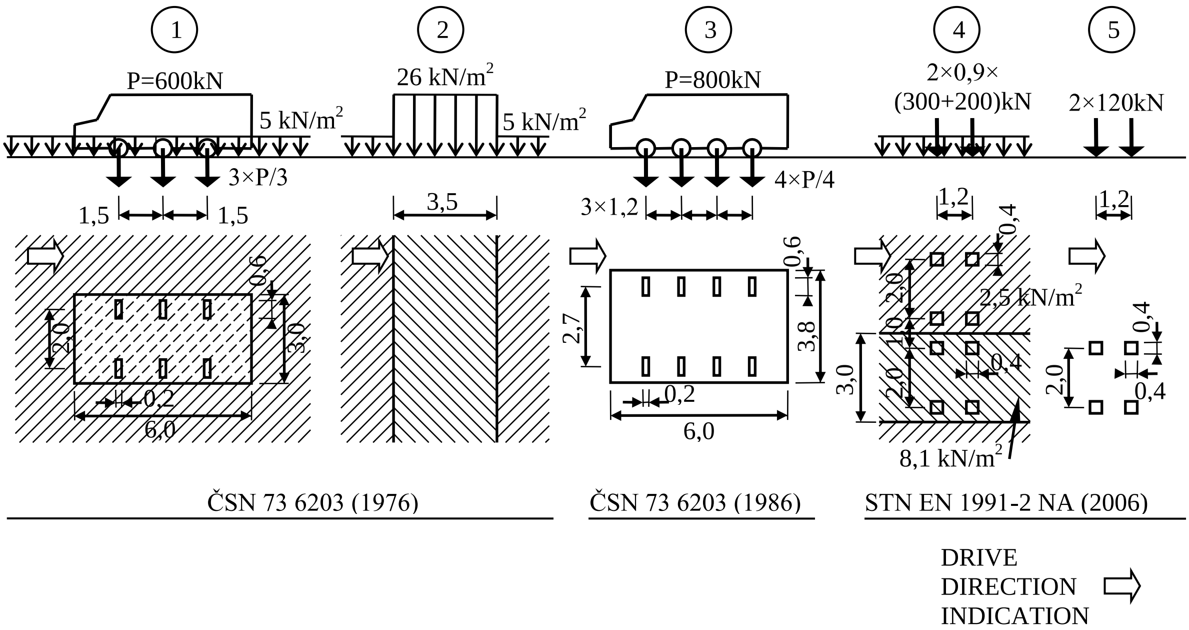


Fig. 4 Comparison reinforcement area according to allowed stresses theory

4 Traffic load

Since technical prescription [3] created in 1981 for transition areas design was changed loading standard [5] in 1986. Another significant modification was change from allowed stress method to limit state method with different loading standards according to Eurocodes. Therefore the contribution in the next section deals with comparison of these different approaches in designing the transition slabs, keeping the same action provided under OTN. On fig. 5 are shown loading models, which where compared. Model 1 – ČSN 73 6203/1976 is a three axle vehicle with weight 60t, which is combined with distributed load 1,25×4kN/m^2 for loading area with width less than 10 m. Standard allows considering of ground pressure behind abutment with equivalent uniform load with intensity 26 kN/m^2 in stripe with width 3,5 m – Model 2. Model 3 – ČSN 73 6203/1986 represents four axle vehicles with weight 80t. Nowadays is loading model represented with Model 4 – STN EN 1991-2NA/2007, which consist from two axle concentrated loads (tandem systems) and from uniform distributed load (UDL). Model 5 – STN EN 1991-2 is fatigue load model - FLM3, which was used for fatigue limit state verification. This model consists from two couples of axels, each with weight 120 kN, distance between axels is 6 m. For verification of transition slabs with length less than 9 m is the second part of load outside of verified element.



LEGEND:
1 – three axle vehicle with weight 60t;
2 – uniform distributed load of embankment behind abutment with intensity 26kN/m^2 on lenght 3,5m;
3 – four axle vehicle with weight 80t;
4 – two axle vehicle (TS) with uniform distributed load (UDL);
5 – fatigue load, part of the model FLM3, which causes extreme action of the slab.

Fig. 5 Load models

Concentrated axle load distribution from individual load entering to the calculation of bending effects was considered:

- Effective width method,
- Load distribution associated with finite element method.

Effective width method was used for its simple application and supposedly use in standard [3]. Method was used for models 1, 3, 4 and 5. Calculation of bending moments with FEM was used for comparation in model 4 and also in model 5.Bending moment from load model 2 was calculated with formulas from elementary static.

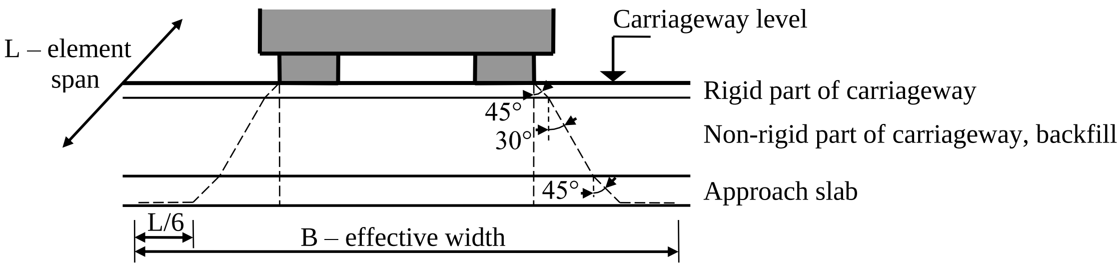


Fig. 6 Concentrated load distribution and method to determination of effective width

5 Comparisons

Comparison of bending moments from each load model is shown on fig. 7. In the comparison of bending moments from live load can be seen increase of values by models defined in valid European standards. In area of curve described as ČSN 73 6203, can be declared the best match with curve of bending moments from standard [3] reaches simplified Model 2. This model corresponds in values and also in shape of the curve. That declares the curve was probably diverted from this model.

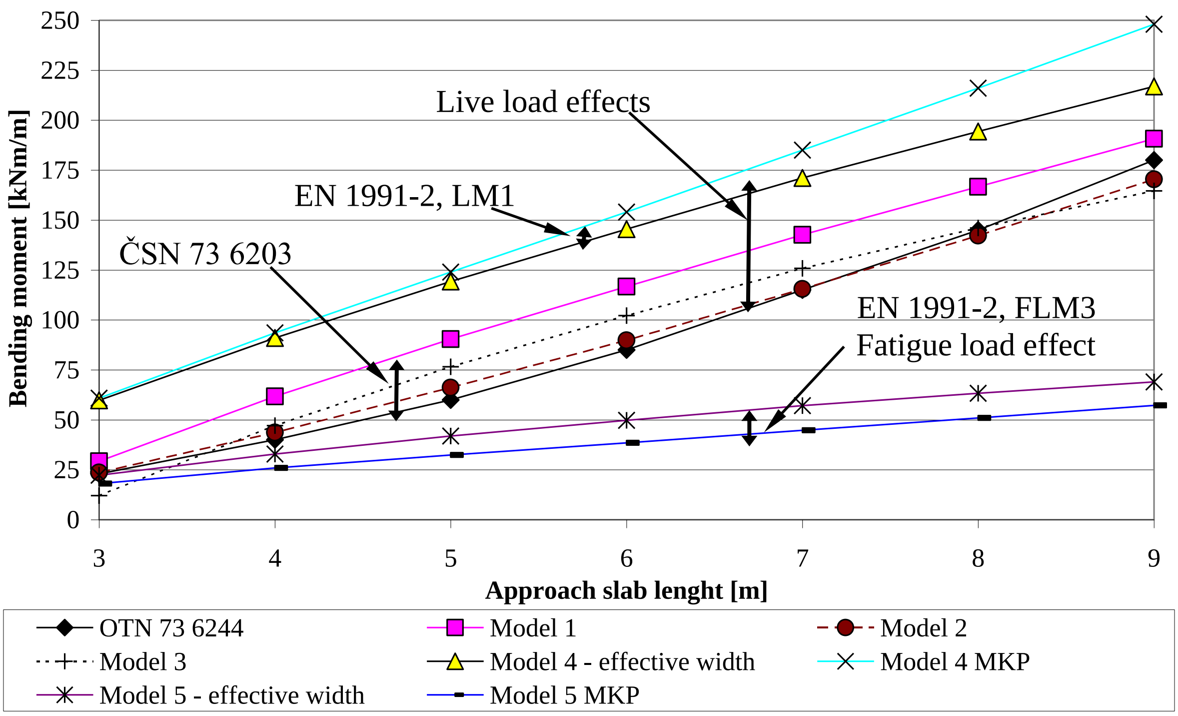


Fig. 7 Bending moment comparison from live and fatigue load.

Comparison of effective width with FEM analysis can be seen by curves (fig. 7) described as EN 1991-2, LM1 and FLM3. We can declare their relative good match. In other comparisons are therefore just results from FEM as currently most used method.

Bending moments curves from fatigue load model FLM3 – Model 5 is a separate chapter and its size was needed for fatigue failure verification of reinforcement with method of equivalent stress amplitude. Reinforcement from tab. 1 with cover 40 mm is verified. The verification may be affected by several factors (traffic volume, designed lifetime, number of driving lanes), which can be considered with coefficient λ_{s2}; λ_{s3}; λ_{s4}. The coefficient equal to 1 represents the most conservative approach for roads and highways with high proportion of trucks. This verification is shown on chart fig. 8. For comparison is in chart also curve for $\lambda_{s2} = 0.857$, which means reduced frequency of heavy vehicles, well the roads and highways with medium rate of trucks. From chart is obviously that the reinforcement in approach slabs needs increase of slab thickness especially for short elements.

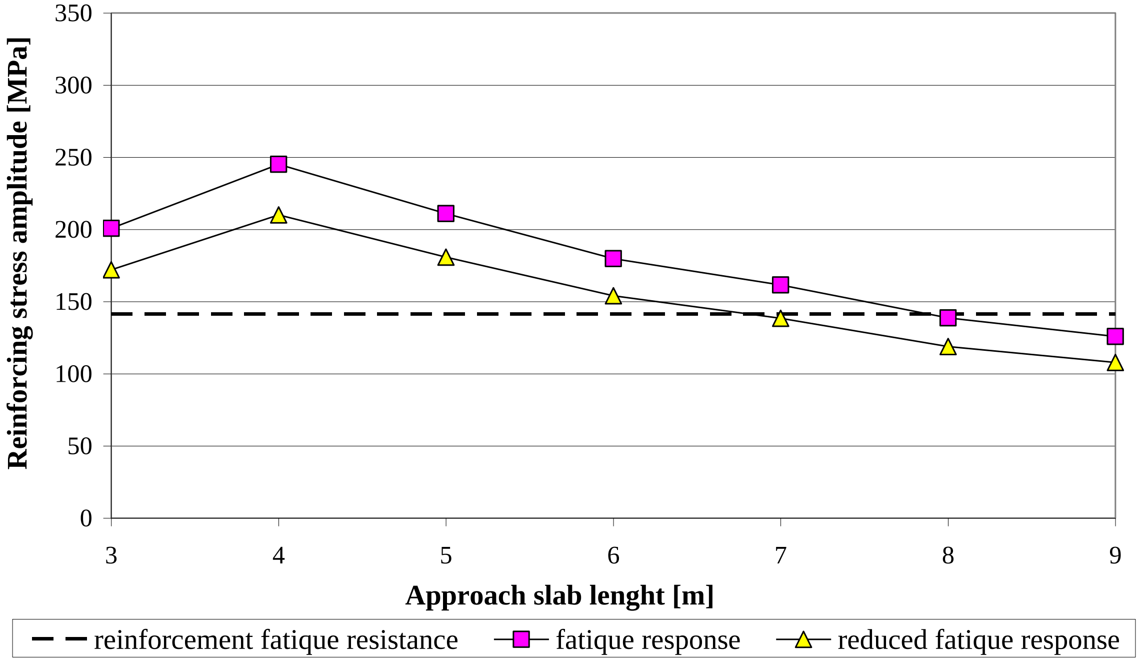


Fig. 8 Fatigue verification of concrete reinforcement

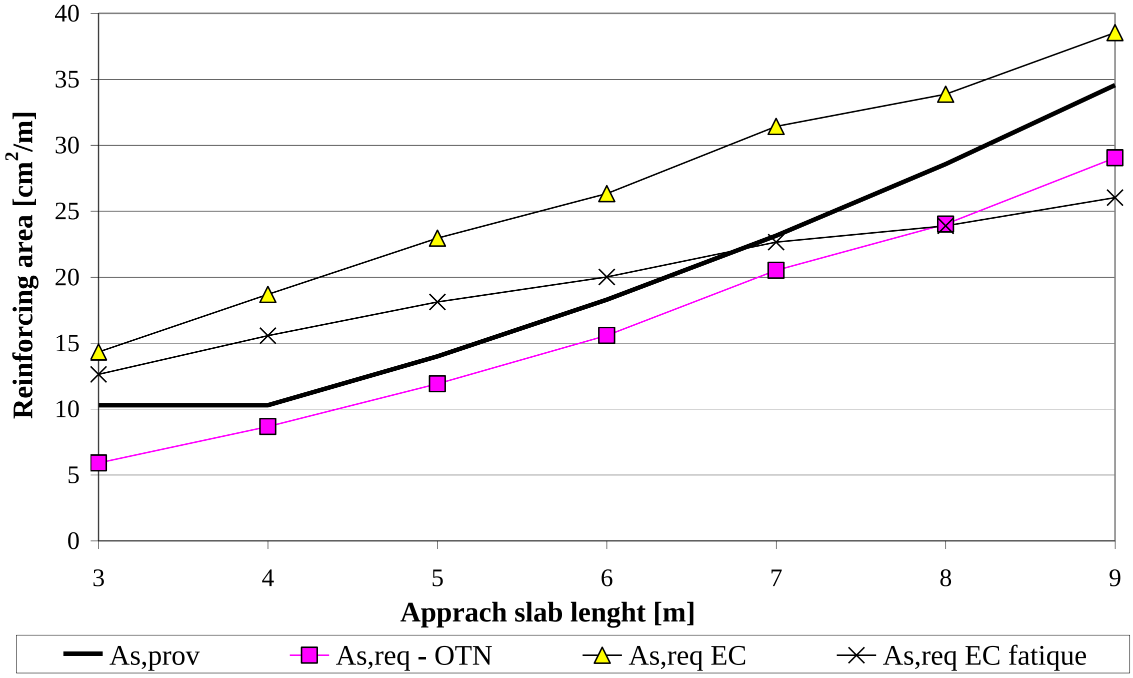


Fig. 8 Required areas of reinforcement

Also from fatigue verification is possible to find out quantity of required reinforcement. (As,req EC – fatigue), as it is shown on last comparison on fig. 9. In this comparison are required areas of reinforcement based on bending moment curve from fig. 3 (As,req OTN), and also required areas of reinforcement based on verification from currently valid European standards (As,req EC), curve of bending moments on fig. 7, Model 4 – FEM. For comparison is in chart added curve of reinforcement area from tab. 1 (As,prov).

Conclusions

From the contribution is obviously that the technical prescription for transition areas design has passed at least two significant changes. Presented analysis with results in charts highlights the need to increase the required area of reinforcement or increase the thickness of the slab. Suitable ideas for discussion would also to fatigue verification of this structural element.

Acknowledgements

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CONTINUOUS COMPOSITE CONCRETE GIRDER

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Abstract

Structures made from prestressed precast concrete girders with cast-in-place topping and support diaphragm forming continuous system are successfully applied particularly in bridge construction. The paper is focused on comprehensive experimental and theoretical analysis of long-term behaviour and behaviour under loading of two span continues girder in comparison to prediction according to EN 1992-1-1.

The long-term effects due to the shrinkage and creep in continues systems were compared with measurements on isostatic composite members that were made together with members of continuous systems.

Non-linear behaviour of girders under loading occurs on continuous statically indeterminate system and leads to redistribution of bending moments in compare with moments determined by linear-elastic analysis.

Keywords: bridge girders, prestressed precast concrete

1 Introduction

Bridge decks made from precast prestressed beams and cast in situ topping are very often used solutions for bridges with shorter spans, length up to 30 m, rarely 40 m in Slovakia. Composite precast decks were mostly designed as isostatic structures, even for multi-span bridges. Therefore only limited experience existed with continuous solutions, particularly with systems where continuity is provided by non-prestressed RC crossheads. The first continuous motorway composite precast bridges in Slovakia were build in 2004.

Continuous structures have a lot of advantages compare to the simply supported counterparts, e.g. less expansion joints, less bearings or higher stiffness, smoother passing of the cars and lorries. The maintenance of the continuous bridges is usually cheaper then of multi-span isostatic structures. Why designers have preferred application of the simply supported solutions is hidden in the complexity of design. Design of continuous composite bridges is more complex then of the isostatic members. Calculation of shrinkage and creep effects on internal stresses (composite behaviour) and on the internal forces due to restrained deformation in redundant structures requires a lot of time and is burden with high level of uncertainty. Further uncertainties are connected with a redistribution of

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internal forces due to the cracking of transverse support beams (cross heads) that are cast at an intermediate deck support and provide continuity of the structure.

Fig. 1 High-way bridge made from prestressed precast concrete girders with cast-in-place topping and support diaphragms.

Therefore since 2005 to 2010, experimental investigation of the behaviour of continuous composite girders has been performed in the laboratory of the Department of Concrete Structures and Bridges at the Slovak University of Technology in Bratislava. The experiment focused on two major problems that are connected with the above-mentioned type of structure: on the effects of shrinkage and creep on the stress state of the girders and on the effect of cracking at intermediate supports on the redistribution of internal forces.

Long-term effects of creep and shrinkage were observed on 2 continuous two-span with 2 simply supported girders. Redistribution of internal forces under instance load were monitored during loading test of 4 continuous two-span girders.

2 Description of the experimental girders

Post-tensioned 200mm x 360mm and 4500 mm long rectangular beams were cast using strength class C35/45 concrete, reinforced with steel of a characteristic strength of 500 MPa. Four alternative two straight monostrands were embedded in each beam. The prestressing units consisted of 7-wire low relaxation strands with a characteristic strength of 1800 MPa. A sectional area of a strand was 141.6 mm^2.

For long-term monitoring two sets of three beams were cast. Two beams from each set were made continuous. The third beams were tested as simply supported girders, in order to compare the behaviour of the isostatic and hyperstatic structures cast from the same concrete batch. The beams were stored for 50 days and later placed on the supports and prestressed by a four-strand jack. The reinforced concrete slab (topping) was cast after prestressing.

The cast-in-place slab of C30/37 was 90 mm thick and 800 mm wide. Continuity was provided by casting-in-place a 400 mm wide and long deck diaphragm (crosshead), see Fig.2.

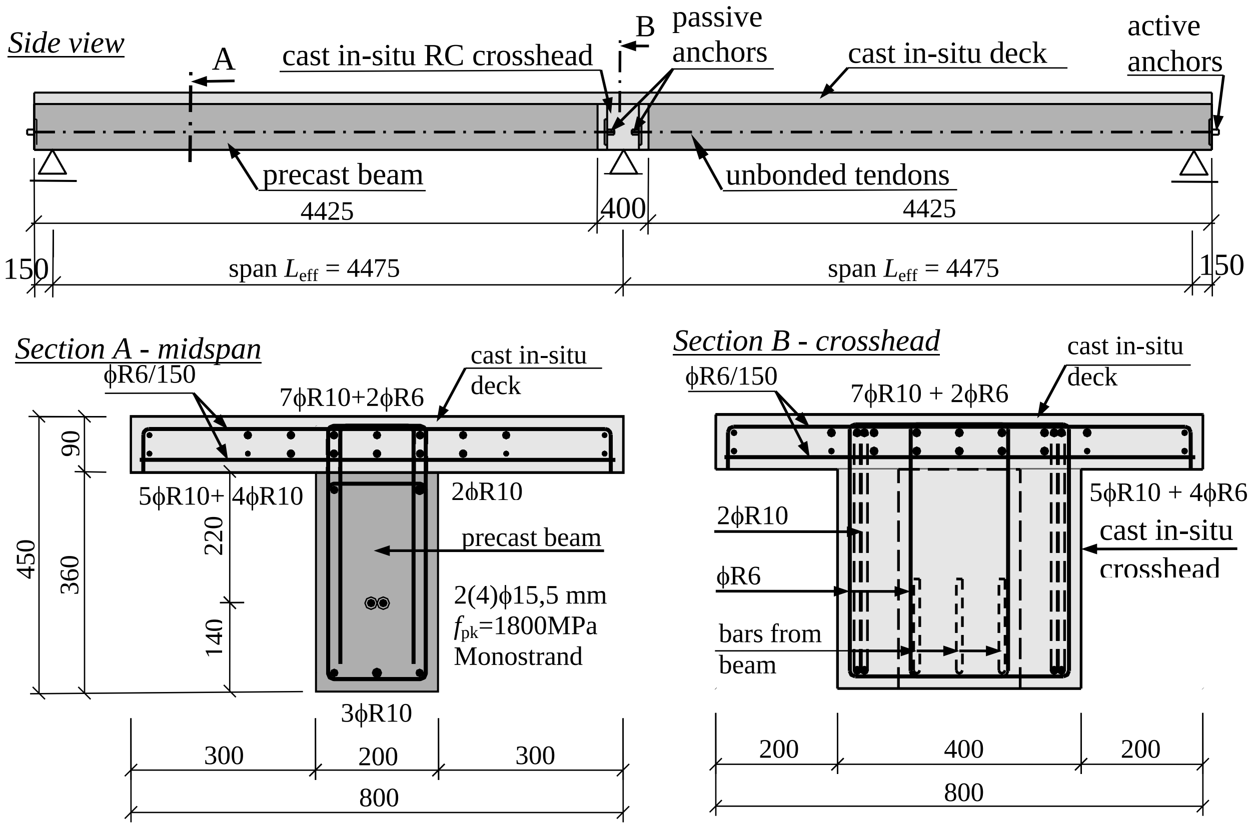

Fig. 2 Schemes of test 2 span continuous girders

3 Long-term monitoring

3.1 Concrete properties

Three concrete mixtures were used for the long-term experiment. Two concrete mixtures were used for casting the first and the second set of precast beams and the third for casting the toppings and crossheads. In order to evaluate the material models for an analysis of the experimental girders, the following material properties were tested: 28-day and 180-day compressive strength of the concrete, the modulus of elasticity, shrinkage and creep.

3.2 Measurement devices and instruments

Attached strain gauges with a base length of 400 mm (200 mm) were used for measuring the concrete strains. Five sections were located in each span, were fitted and monitored. The sixth section was located at the centre of the intermediate support.

Mechanical deflection gauges with a precision of 0.01 mm were placed in the middle of each precast beam for measuring the midspan cambers and deflections. Mechanical levels were placed at the support sections, and they served for measuring the rotations. In order to determine the effect of creep and shrinkage on the redistribution of the sectional forces in the composite continuous structures, the edge reactions were measured by dynamometers. Two optical and two mechanical dynamometers were used. The sensitivity of the optical dynamometers was 0.04 kN (4 kg) and the mechanical 0.02 kN (2 kg). The magnitude and time development of the prestressing force was measured using elasto-magnetic sensors placed on the strands at the live anchorages. The sensors were embedded in the steel tube, which transferred the prestressing force from the anchor's body to the anchorage plate embedded in the concrete, see Fig. 9. The precision of the measurement was ± 1 kN.

3.3 Results of monitoring and numerical analysis

The long-term monitoring started with the removal of the slab formwork, 18 hours after casting the slab. The frequency of the readings was two days during the first week, one week during the first month, and one month until the end of the period of long-term measuring, which took 10 months overall. Three analytical models for prediction of the long-term behaviour of the girders were prepared, and the computed values were compared with the measured ones. All the models were analysed using a general incremental step-by-step method. The first model used material properties based on EN 1992-1-1 values (strength, shrinkage, creep, relaxation). The second was based on ČSN 73 6207/93 values. Measured material properties were used for the third model. A comparison of the some measured and analytical values is presented in the next four plots. The time developments of the midspan deflection in the simply supported girder and in the continuous girder are shown in Figs. 3 and 4.

The stresses in concrete are an important quantity in the practical aspect of a design. The values of stress in concrete cannot be measured directly, especially stresses due to time-dependent effects. But they can be calculated using analytical models. If the model is well calibrated, obtained results usually provide a good image about the actual stress – strain state in the analysed structures. The theoretical time developments of stresses in the concrete are presented in Figs. 5 and 6, using all three analytical models.

3.4 Conclusions

Uncertainties are connected with the actual material models. Material properties defined in standards do not always reflect actual values. Even European standards with very comprehensive models for prediction of e.g., shrinkage or creep, do not have to provide correct values. The differences were quite visible for the modulus of elasticity of the concrete. The measured values were 11 % below the values provided by EN 1992-1-1 and even 17 % by ČSN 73 6207. Our tests showed a rapid development of the shrinkage compare with the models for shrinkage prediction, particularly for the younger concrete. The prevailing shrinkage at the early stages caused an even different indicator of measured deformations compared with the results obtained from analytical models with material properties defined in the codes, see Figs. 3 and 4. Better results (closer to the measurements) were obtained when experimentally determined material models were applied in the analytical model 3. The stress differences due to the different material models applied in the analytical models for long-term analysis were small, as opposed to the substantial difference in shrinkage.

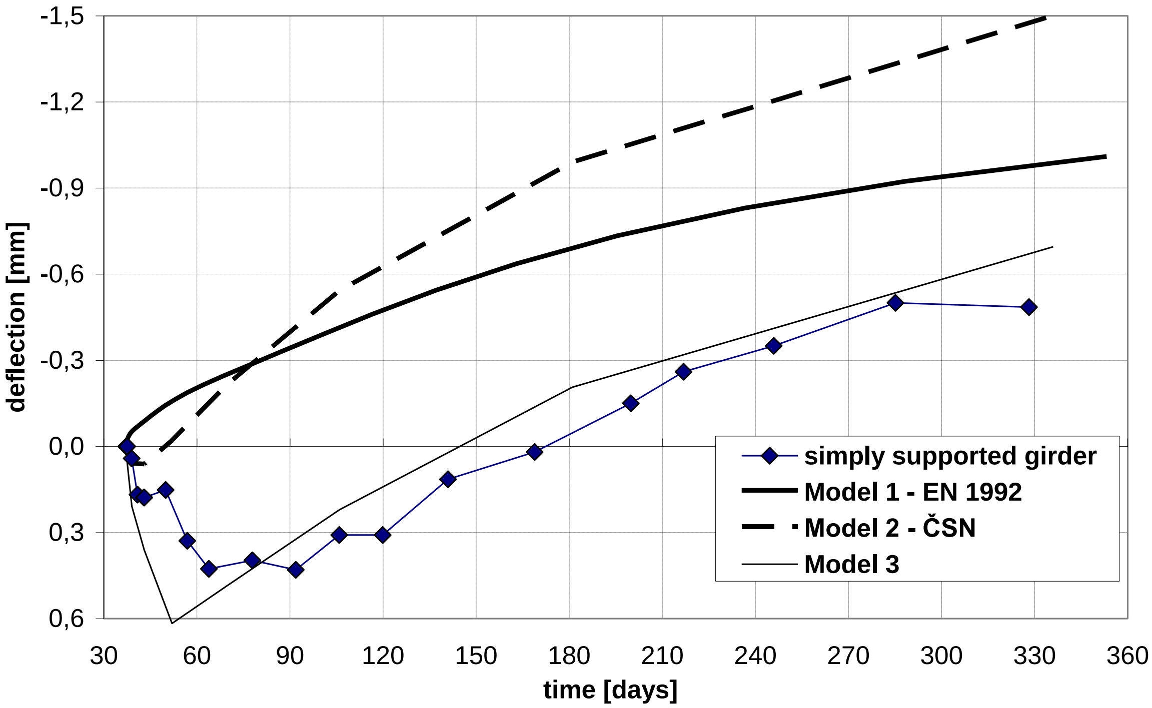

Fig. 3 Midspan deflection vs. time – simply supported girder

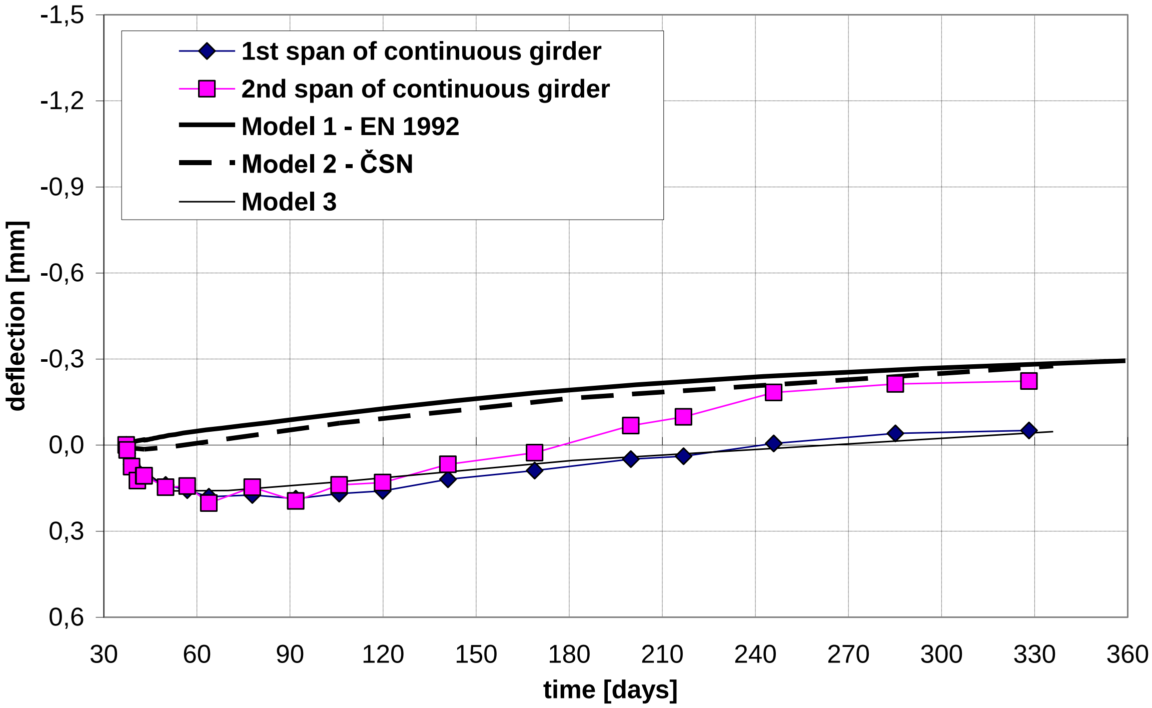

Fig. 4 Midspan deflection vs. time – continuous girder

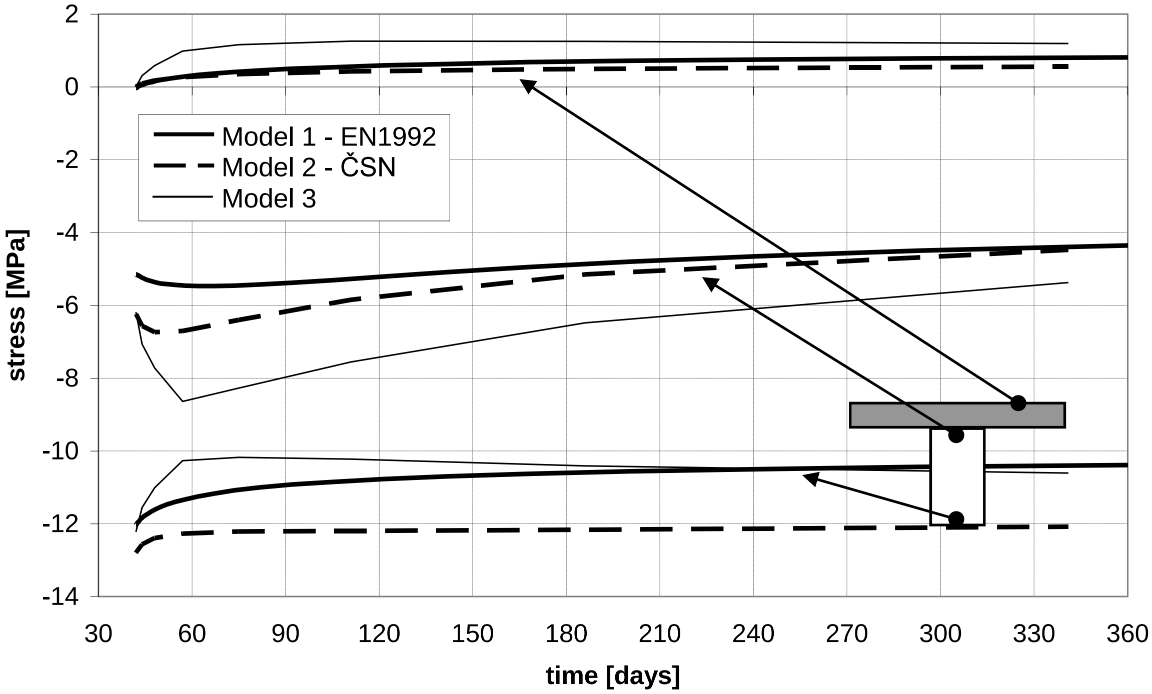

Fig. 5 Concrete stresses development in midspan section of the simply supported girder

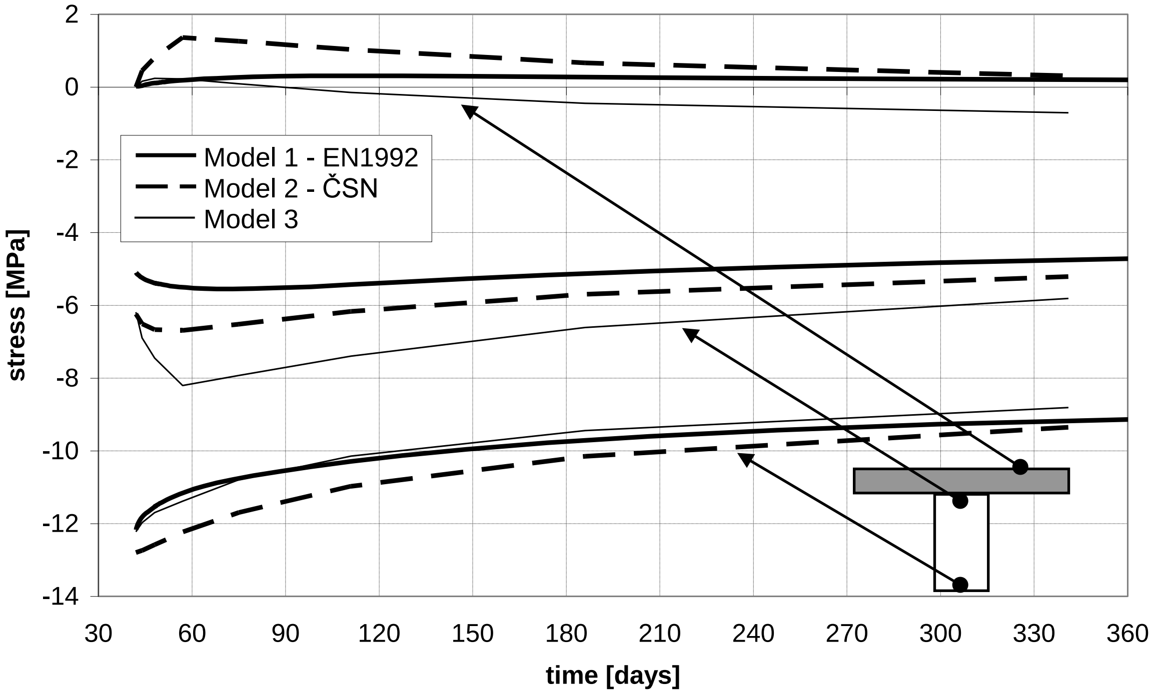

Fig. 6 Concrete stresses development in midspan section of the continuous girder

4 Loading test

Two specimens of two span continuous girders were cast for testing, see Fig.2. The continuity over intermediate support was provided by integral crosshead. The specimens were tested under monotonically increasing loading up to failure. Their response during load tests was monitored by several measure devices. One of the main goals of the loading tests was observation of non-linear behaviour of girders resulting in redistribution of bending moments. For the purpose of theoretical analysis the following properties of the used concrete were tested: compressive and tensile strength, modulus of elasticity and σ–ε diagram with descending branch. The essential properties of the used reinforcing steel such as yield strength and σ–ε diagram were tested as well.

4.1 Measurement devices and instruments

Loading of test girders was induced by means of hydraulic mangles placed in the middle of spans. The response of girders during load tests was monitored by several measure devices such as mechanical and electronic strain gauges, deflection gauges, electric dynamometers and embedded electro-magnetic sensors for monitoring of prestressing force in strands. The configuration of measuring devices was similar to long-term testing. Envelopes of redistributed bending moments during testing were determined from measured reactions in end supports.

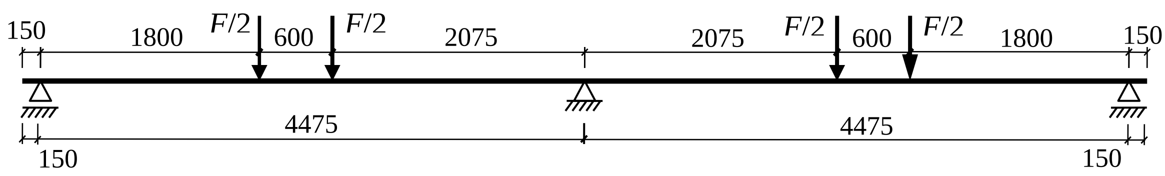

Fig. 7 Loading schemes of test girder. F – force induced by hydraulic mangle.

4.2 Results of loading tests and numerical analysis

Non-linear numerical analysis of test girders was carried out by means of software ATENA 3D v4 developed by Červenka Consulting. Parameters of the applied constitutive models for concrete and reinforcement are based on measured characteristics of used materials obtained from test of specimens.

Development of redistribution ratio δ in dependence on load level presents Fig.8. After the occurrence of the first crack above intermediate support the bending stiffness of this region suddenly decreases. It results in significant redistribution of moments into midspan region which intensifies during the crack formation in intermediate region. This tendency turns over after the occurrence of first crack at midspan region, and moment redistribution into midspan region starts to decrease. It continues until the creation of plastic hinges in critical cross-sections at midspan and intermediate region as well, when the values of ratio δ are close to 1,0. It is the consequence of application of bending moments from linear-elastic analysis for design reinforcement.

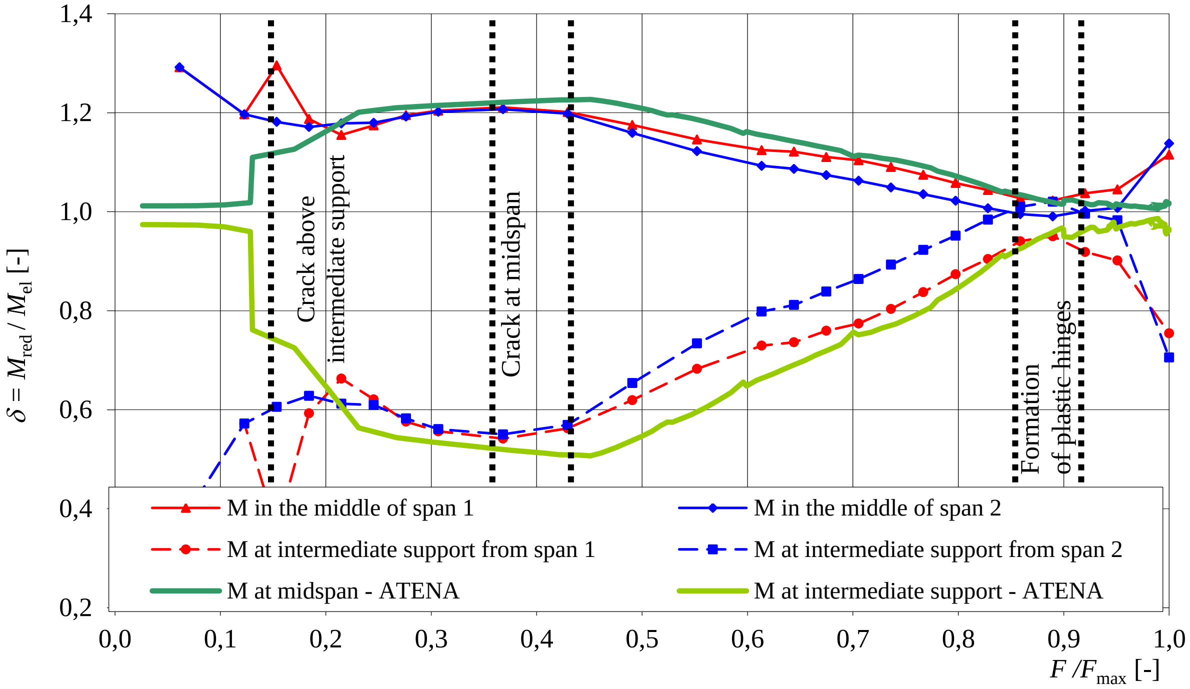

Fig. 8 Relation of ratio δ and load level F/F_{max} – comparison of results from non-linear analysis and experiment of test girder N2, where
F – force induced by hydraulic mangle, F_{max} – max. value of force, M_{red} – redistributed moment, M_{el} – moment obtained from linear-elastic analysis.

4.3 Conclusions

In case of the tested girders the degree of moment redistribution was the most significant shortly before the creation of first crack at midspan, when the ratio d for the moments above intermediate support reached values from 0,46 up to 0,56, and for the moments at midspan from 1,20 up to 1,25. For these reasons within the analysis of structures made from prestressed precast concrete girders with cast-in-place topping and support diaphragm forming continuous system it is necessary to consider moment redistribution also for service limit states. These findings show that preference of linear-elastic analysis (without redistribution) by EN 1992-1-1 for service limit states is not available in general.

Acknowledgements

The paper is a part supported by grand agency VEGA1/0857/11.

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DETAILING OF MEMBERS BY NEW MODEL CODE

Mária Bellová

Abstract

The presented paper deals with the special situations, when the cover of reinforcement of structural members exceeds 50 mm, and it may be appropriate to provide a cover reinforcement. The new Model Code 2010 [2], in Volume 2, PART III, clause 7.13.1 makes a mention about such a reinforcement, but there are no data about its' type and the area amount. However such a surface reinforcement is able to increase the bending and especially shear resistance of structural members, subjected to bending and shear. The cover reinforcement in addition to that – reduces an expansion and width of cracks.

According to Eurocode 2 [3], Part 1-1 in Annex J – Detailing rules for particular situations – there are some special cases when such a surface reinforcement should be used. It is necessary to use this reinforcement type when:

- diameter of provided bars is greater than 32 mm, or equivalent diameter of bundled bar exceeds this limit;
- cover to reinforcement is greater than 70 mm (for enhanced durability);
- crack widths without using surface reinforcement exceeds allowed limit;
- reinforced concrete structure is subject to fire and there is a danger of falling off of a concrete in the latter stage of a fire exposure according to Part 1-2 of Eurocode 2 [4].

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

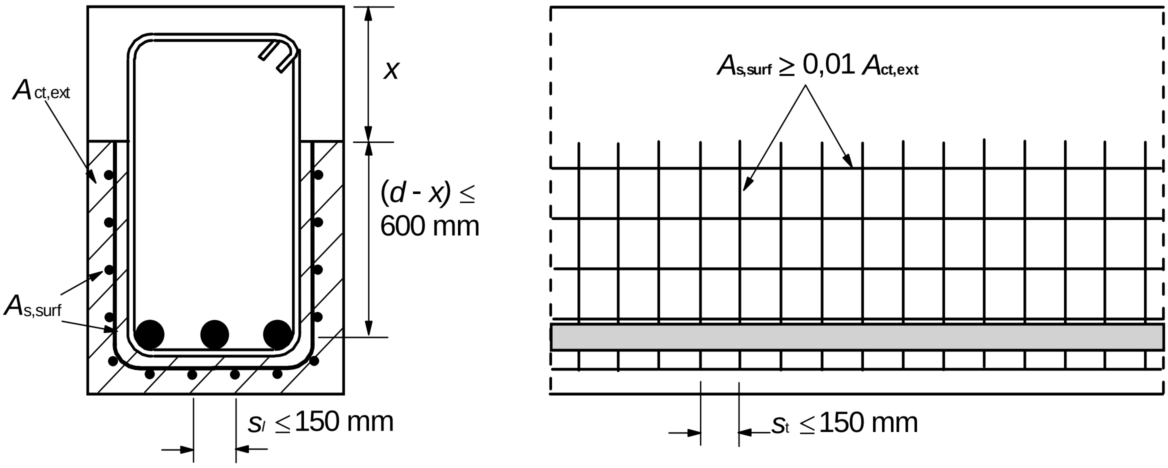
This paper analyses the increase of bending and shear resistance due to surface reinforcement of RC beams. It also includes an explanation of the effect of the surface reinforcement on expansion and width of cracks.

Keywords: Cover reinforcement, Large diameter of bars, Fire resistance, Surface reinforcement

1 When to use and how to design cover reinforcement?

The **Fig. 1** demonstrated **position** and relevant terminology of the surface reinforcement, which consists of wire mesh or small diameter bars, located in two directions, **parallel** and **orthogonal** to the tension reinforcement in the beam. The values of $A_{s,surfmin}$ will be later differentiated according to the **purpose of the surface reinforcement.**

The **longitudinal bars of the surface reinforcement** may be taken into account as **longitudinal** bending **reinforcement** and the **transverse bars** as **shear reinforcement** provided they meet the requirements for the arrangement and anchorage of these types of reinforcement.



x is the depth of the neutral axis at ULS

Fig. 1 Position and terminology of the surface reinforcement

The reasons for using of cover/surface reinforcement and design of its areas are subsequent:

2 Spalling of the concrete

Where the main reinforcement is made up of:

- bars with diameter greater than 32 mm or
- bundled bars with equivalent diameter greater than 32 mm,

the cover/surface reinforcement $A_{s,surf}$ should be used – **to avoid spalling.**

The **minimal area** of the surface reinforcement $\boldsymbol{A_{s,surfmin}}$ **in each direction, parallel** $A_{s,surfmin,l}$ and **orthogonal** $A_{s,surfmin,t}$ to the tension reinforcement in the beam is recommended (according to [3] as follows:

$$A_{s,surf} \geq A_{s,surfmin} \qquad A_{s,surfmin} = A_{s,surfmin,l} = A_{s,surfmin,t} = 0{,}01\, A_{ct,ext}\,, \tag{1}$$

where: $A_{ct,ext}$ the area of the tensile concrete external to the links (see **Fig. 1**).

3 Cover to the reinforcement c_{nom} > 70 mm

Where the cover to the reinforcement is greater than 70 mm, similar surface reinforcement should be used to enhance durability. (Diameter of the main tension reinforcement in this case is not relevant.)

The minimal area of the surface reinforcement $A_{s,surfmin}$, **in each direction, parallel** $A_{s,surfmin,l}$ and **orthogonal** $A_{s,surfmin,t}$ to the tension reinforcement in the beam may be obtained as follows:

$$A_{s,surf} \geq A_{s,surfmin} \qquad A_{s,surfmin} = A_{s,surfmin,l} = A_{s,surfmin,t} = 0{,}005\, A_{ct,ext}\,, \tag{2}$$

where: $A_{ct,ext}$ the area of the tensile concrete external to the links.

It means that in this case **the area of the surface reinforcement is equal to one half of the previous value.**

4 Crack widths control

For bars with a **diameter larger than ϕ_{large},** crack control can be achieved either by using surface reinforcement or by calculation. The recommended value for $\phi_{large,}$ is 32 mm.

In this case **the area** of the **surface reinforcement is not equal in both directions**, parallel and orthogonal to the main tension reinforcement.

The **area of the surface reinforcement** in the direction **parallel to large diameter bars** $A_{s,surfmin,l}$ should satisfy:

$$A_{s,surfmin,l} \geq 0{,}02\, A_{ct,ext} \tag{3}$$

and the area of the surface reinforcement in the direction orthogonal to large diameter bars $A_{s,surfmin,t}$ should satisfy:

$$A_{s,surfmin,t} \geq 0{,}01\, A_{ct,ext} \tag{4}$$

where: $A_{ct,ext}$ the area of the tensile concrete external to the links.

To include the area of the surface reinforcement in the consideration for increase of the resistance of RC beams subject to bending ($A_{s,surfmin,l}$) and shear ($A_{s,surfmin,l}$) the cover to the surface reinforcement c_{nom} has to satisfy all criteria.

The **transverse bars with the shape of a letter "U"shall be properly anchored into the compression part** of the RC member's cross section **!!!**

5 Fire performance

If the axis distance to the reinforcement is 70 mm or more and tests have not been carried out to show that falling-off does not occur, then the surface reinforcement should be provided. The surface reinforcement mesh should have a spacing not greater than 100 mm and a diameter not less than 4 mm [4].

Specific area of such a surface reinforcement mesh in each direction (parallel and also orthogonal to the main reinforcement) per 1,0 m is independent of the area of the main reinforcement.

6 Contribution of the surface reinforcement by verification of bending and shear design and by control of SLS

Tab. 1 Ultimate characteristic uniformly distributed load assuming equality in the reliability conditions – [kN/m] **rectangular cross-section**

	$M_{Ed} = M_{Rd}$	$V_{Ed} = V_{Rd}$	$y_{qp} = y_{lim}$	$\sigma_c = \sigma_{clim}$	$w_k = w_{lim}$
Without the surface reinforcement	122	90	119	94	157
With the surface reinforcement	124	124	124	97	167
increase [%]	2	38	4	3	6

The **Table 1** presents **the influence of the surface reinforcement** when the **longitudinal (parallel) bars as bending reinforcement and the transverse (orthogonal) bars as shear reinforcement are taken into account.** There are results of the analysis for simple supported reinforced concrete beam with rectangular cross-section expressed by ultimate characteristic uniformly distributed load [kN/m] assuming equality in the reliability conditions.

7 Conclusions

From the **Table 1** follows, that the most significant influence of the surface reinforcement is for the **shear stress.** Using of the surface reinforcement reduces the area of provided links in the case of the **rectangular cross-section** by about **38 %.** Influence of the surface reinforcement on the rest of analysed quantities is negligible.

This outcome has been achieved with the financial support of the research project granted by Slovak Grant Agency VEGA 1/0857/11. All support is gratefully acknowledged.

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COMPOSITE STEEL – REINFORCED CONCRETE (SRC) COLUMNS – THEORETICAL DESIGN ANALYSIS

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Abstract: *The paper presents some results of theoretical analysis of steel-reinforced concrete (SRC) composite columns. The problems in the design of composite SRC columns were divided into two main sections:*

- *analysis of simplified and general methods and their differences,*
- the resistance of composite SRC columns loaded by a normal compressive force and bending moment.

Key words*: Column, composite, concrete, reinforced concrete, resistance, steel.*

1. Introduction

Composite steel-reinforced concrete (SRC) columns are a very important application of composite structures, and widespread use of them is found, particularly in high-rise buildings. A composite SRC column is defined as a composite member with components of concrete (better reinforced concrete) and structural steel. These two components act together to resist external forces. A composite column is a composite member which is mainly subjected to compression or to compression and bending.There is a wide variety of types of columns with various types of cross-sections. The most commonly used and studied are the two main types of typical cross-sections of composite columns:

- completely or partially concrete-encased steel sections,

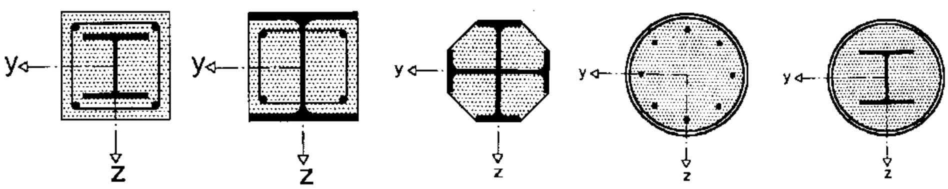

Fig. 1 Types of cross-sections of SRC columns

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[2] Civil Engineer, PhD., SCIA SK, s.r.o., Žilina, Slovakia, valach@scia.sk

Problems in the design of composite steel-reinforced concrete constructions are quite actual today. The research work was therefore directed at an analysis of the design of composite steel-reinforced concrete columns. It was based on contemporary European codes, which use the latest knowledge gained from science and investigations. The problems in the design of composite steel-reinforced concrete columns were divided into two main sections:

- analyses of simplified and general methods and their differences,
- the resistance of composite steel-reinforced concrete columns loaded by a normal compressive force and bending moment.

2. Analysis of simplified and general methods and their differences

Now we are concerned with the resistance of composite SRC columns to combined compression and bending. There are a number of design proposals that can be used to establish the load-moment strength interaction relationship. Among these are the proposals of Wakabayashi, the SSLC method, the Roik-Bergmann method, the EC4 method and others.

<u>The Wakabayashi method:</u>
This method provides the simplest equations for the ultimate strength of the failure envelope. For this, the strengths of the concrete and steel elements are found independently and are then superimposed.

<u>The American Structural Specifications Liaison Committee method (SSLC):</u>
A simplified force-moment strength interaction function has been recommended.

<u>The Roik and Bergmann method:</u>
The solution can only be applied to a cross-section that is doubly symmetrical, which is often the case in practice. The interaction curve is replaced by the A(E)CDB polygon.

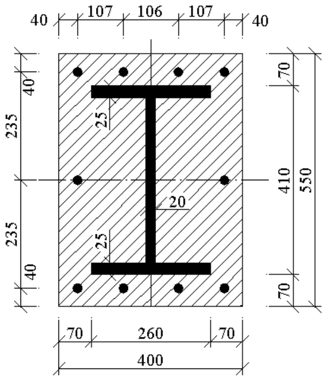
<u>The Eurocode method</u>

Here, the preference was given to the method developed by Roik, Bergmann and others at the University of Bochum, Germany. It has a wider scope, is based on a clearer conceptual model and is slightly simpler. Two design methods are provided:
- a general method, whose scope includes members with non-symmetrical or non-uniform cross-sections over the column's length and
- a simplified method for members of a doubly symmetrical and uniform cross-section over the member's length.

Through analysis in accordance with the simplified methods, we particularly focused on finding a simplified solution for determining point B in a polygonal interaction curve according to code EN 1994-1-1. This problem was solved by substituting the polygonal interaction curve with a sinusoid (Fig. 3). Fig. 2 includes the interaction functions determined for the cross-section of a SRC column with a completely concrete-encased I steel section for the calculation methods presented.

The effect of the necessity and suitability of using point E in the polygonal interaction curve for the basic types of cross-sections, which are described in code EN 1994-1-1, was also analyzed. Then we derived the approximate relations for determining the position of point E by using a linear regression (Fig. 4).

Concrete C20/25: f_{ck} = 20MPa, E_{cm} = 30MPa, γ_c = 1.5
Reinforcement 10 425(R): f_{sk} = 410 MPa, E_s = 210000 MPa, γ_s = 1.15
Structural steel S 235: f_{yk} =235 MPa, E_y = 210000 MPa, γ_y = 1.15



Method of calculation		Plastic interact i- on curve	Roik-Berg mann (EN 1994-1-1)	SSLC	Waka-bayashi ’s method
Point	**N [kN]**	**M [kNm]**			
B	0	972.6	972.6	824.8	854.4
C	1115	1004	1004	806,5	1007
D	2229	972.6	972.6	751.5	854.4
A	7477	0	0	0	0

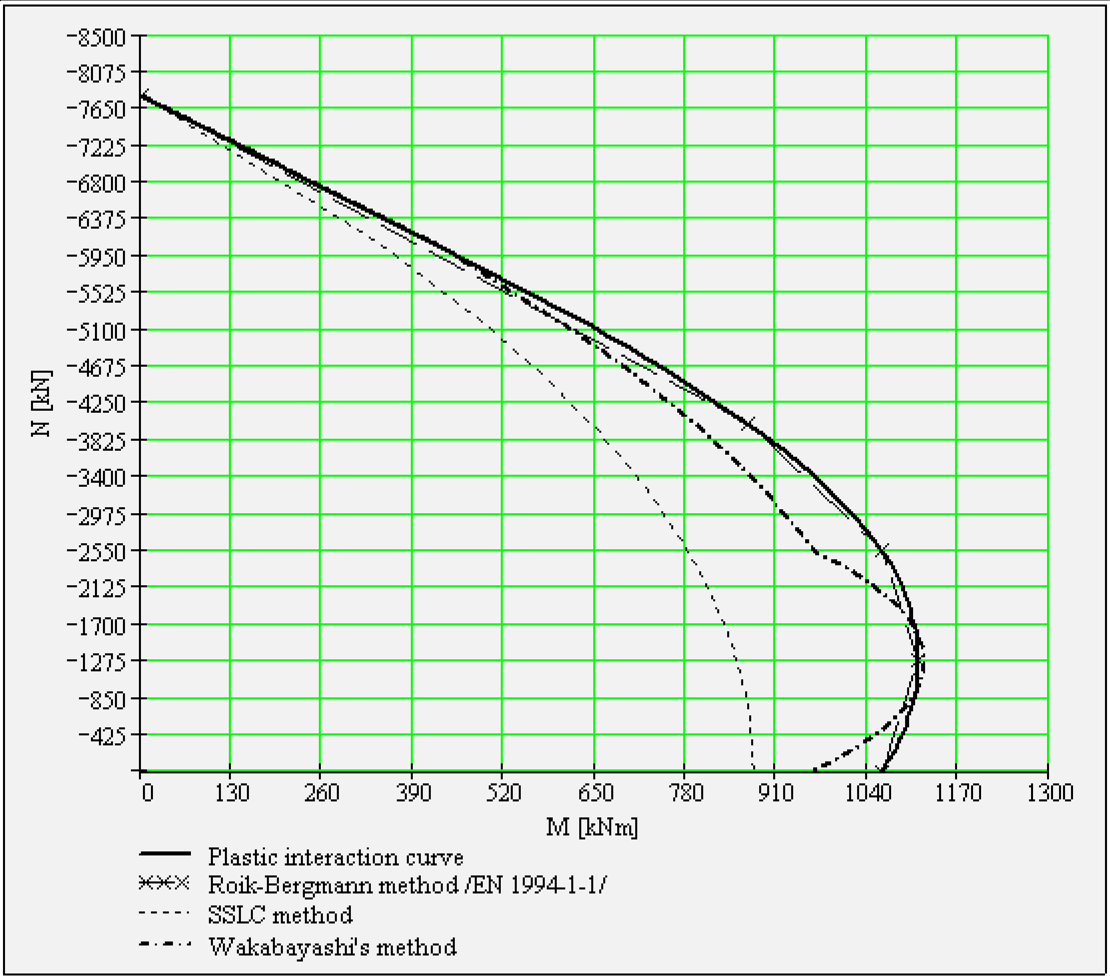


Fig. 2 Differences in the simplified method

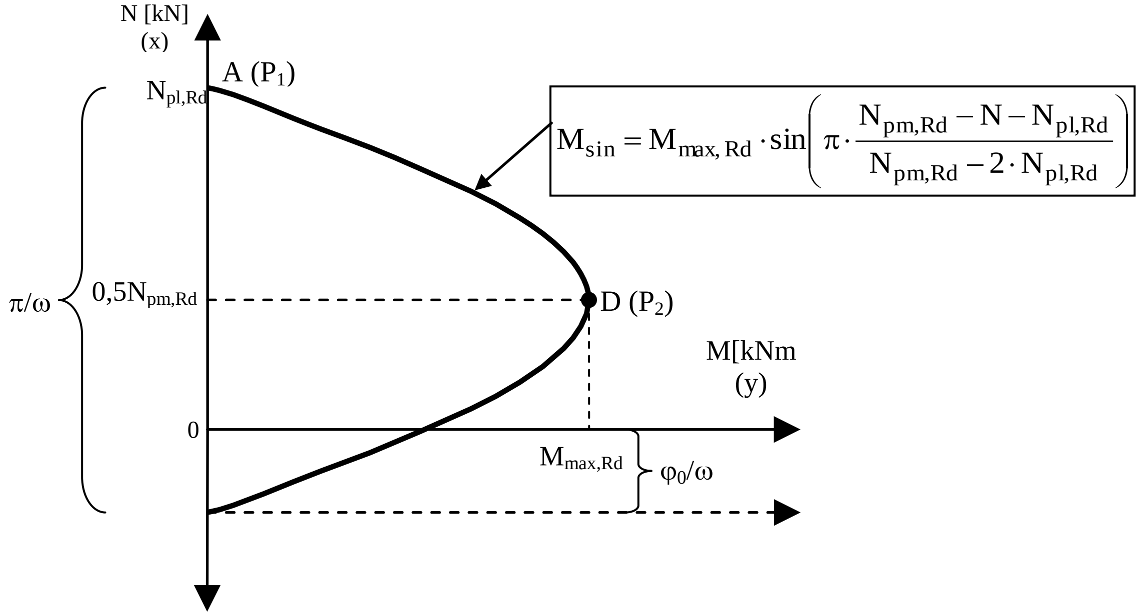


Fig. 3 The sinusoidal interaction diagram

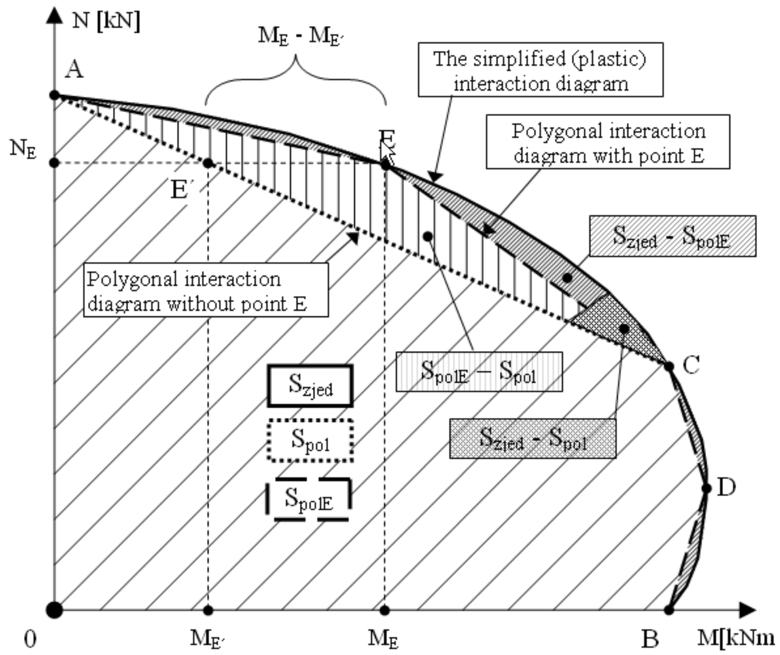


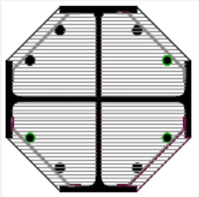
Fig. 4 The interaction diagrams and their parameters for the analysis of point E

Through analysis of the simplified methods and general method of Eurocode 4, a program in MathCAD, which permits checking a composite SRC column with an arbitrary cross-section which is symmetrical according to the vertical axis, was created. The program allows the use of an interaction curve (plastic, elastic-plastic, polygonal, sinusoidal) with or without a second-order effect. The program was used to calculate and compare 150 examples of the basic types of composite SRC cross-sections, which are defined in code EN 1994-1-1.

Table 1. presents the properties of an example of partially concrete-encased sections of SRC columns for which the sinusoidal and polygonal interaction diagram is shown in Fig. 5. Conclusions and conditions were deducted for these cross-sections, for which it is possible to

substitute a polygonal interaction curve with a sinusoid and for which cross-sections it is necessary to use point E in the polygonal interaction curve.

Table 1. The properties of an example of a partially concrete-encased section in the shape of an octagon

		**Group**	**1**	**2**	**3**	**4**
		Concrete	C50/60	C40/50	C30/37	C20/25
		Steel	S450	S355	S275	S235
		Reinforcement	10 505	10 505	10 505	10 505
		b/h [mm]	500/300	300/500	400/400	600/600
		μ_{st} **[%]**	0.42	1.16	2.49	3.21
Example in group	**1**	$\delta=(A_a \cdot f_{yd})/N_{plRd}$	0.25	0.23	0.26	0.25
	2		0.34	0.32	0.37	0.46
	3		0.51	0.49	0.54	0.61
	4		0.72	0.70	0.71	0.73
	5		0.86	0.84	0.85	0.82

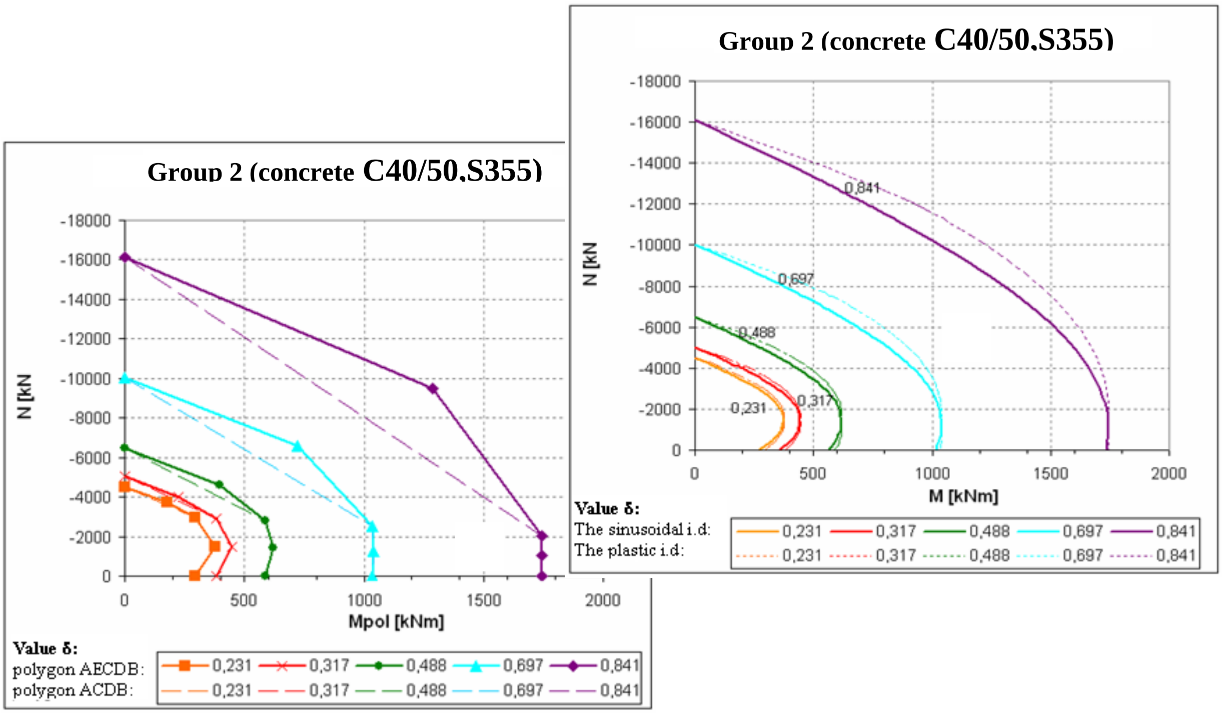


Fig. 5 Sinusoidal and polygonal interaction diagram

Conclusions

- A program for calculating and analyzing the resistance of a composite column was created according to code EN 1994-1-1, and 150 examples using this program for analysis of the simplified method were calculated.
- An equation for substituting a plastic or polygonal interaction diagram with sunisoid interaction diagram was derived.
- The suitability of using a sunisoid interaction diagram for the basic types of cross-sections which were given in code EN 1994-1-1, was analysed.
- The suitability or necessity of point E in the polygonal interaction diagram for the basic types of cross-sections, which is given in code EN 1994-1-1, was analysed.

- The approximate equations for determining the position of point E using linear and polynomial regressions were derived.

Table 2. Approximate equations for calculating the position of point E

Cross-section	$N_E/N_{pl,Rd}$	R^2	x_{ue}/h	R^2
partially concrete-encased steel I section (direction of the web)	$-0.5\cdot\delta+0.9$	0.69	-	
partially concrete-encased steel I section (direction of the flange)	$-0.2\cdot\delta+0.8$	0.32	-	
completely concrete-encased steel I section (direction of the web)	$-0.3\cdot\delta+0.8$	0.27	$0.2\cdot\delta+0.95$	0.89
completely concrete-encased steel I section (direction of the flange)	$-0.3\cdot\delta+0.85$	0.70	-	
partially concrete-encased section in the shape of an octagon	$-0.4\cdot\delta+0.9$	0.80	$0.2\cdot\delta+0.95$	0.75
concrete-filled rectangular steel tubes	$-0.45\cdot\delta+0.80$	0.67	-	
concrete-filled circular steel tubes	$-0.40\cdot\delta+0.90^*$	0.64	-	

*the equation may only be used for a column with a relative slenderness of $\lambda < 0.25$

Acknowledgement
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BRIDGE OVER "PRIEMYSELNÁ" STREET ON R1 EXPRESSWAY SECTION "NITRA – SELENEC"

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ABSTRACT

The bridge forming a flyover along "Priemyselná" Street is a dominant structure on the R1 southern bypass of the town Nitra. Due to complexity of site conditions the bridge with the overall length of 1,165 m is formed of two separate structures built by two different construction methods: first structure 806 m long is being constructed by incremental launching method, second one with length of 359 m by cast-in-situ balanced cantilever construction method.

1. INTRODUCTION

The bridge is a part of the R1 Expressway section Nitra, West – Selenec. The structure is situated within the urban area of the town Nitra and is crossing (bridging) both the railway track and the river Nitra.

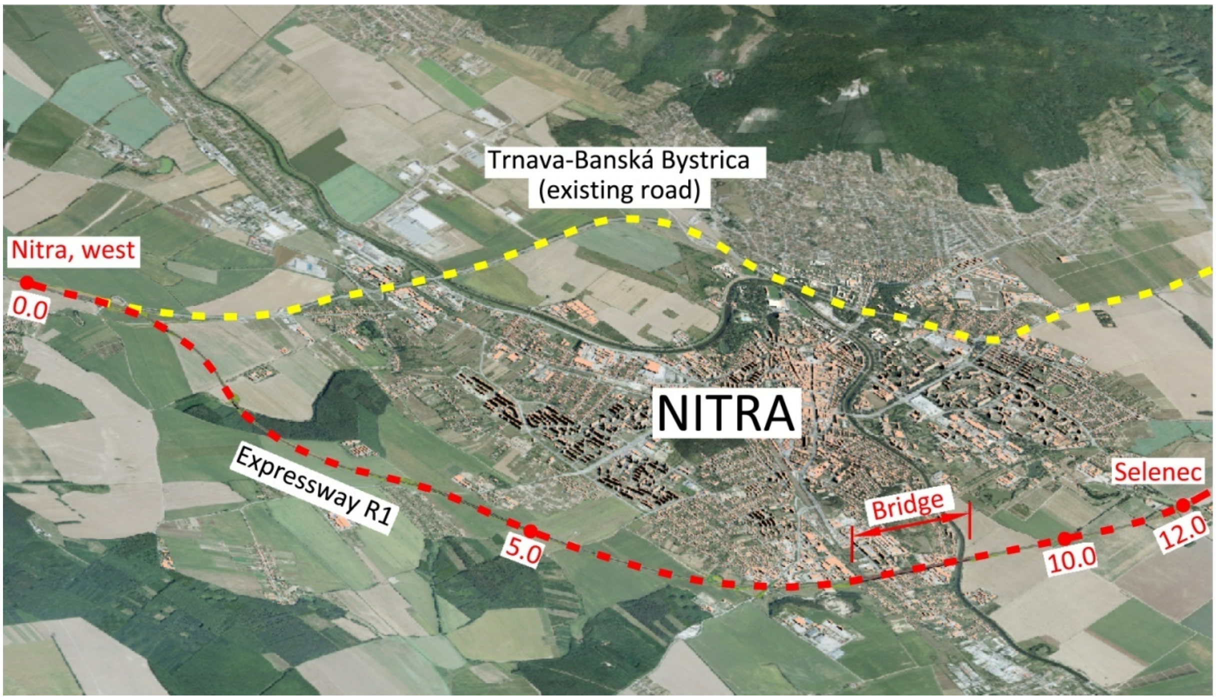

Fig.1 Bridge location

The bridge consists of two structures – construction units – with different shape and constructed by different construction methods. Construction unit N°1 (CU1) has been designed as two separate parallel bridges with single-cell box-section superstructure to be constructed by incremental launching method. Construction unit N°2 (CU2) has been designed as one superstructure for full expressway profile, i.e for both traffic directions. The CU2 cross section is formed by 3-cell structure with lower haunch above each pier. The bridge superstructure has been constructed by cast-in-situ free cantilevering.

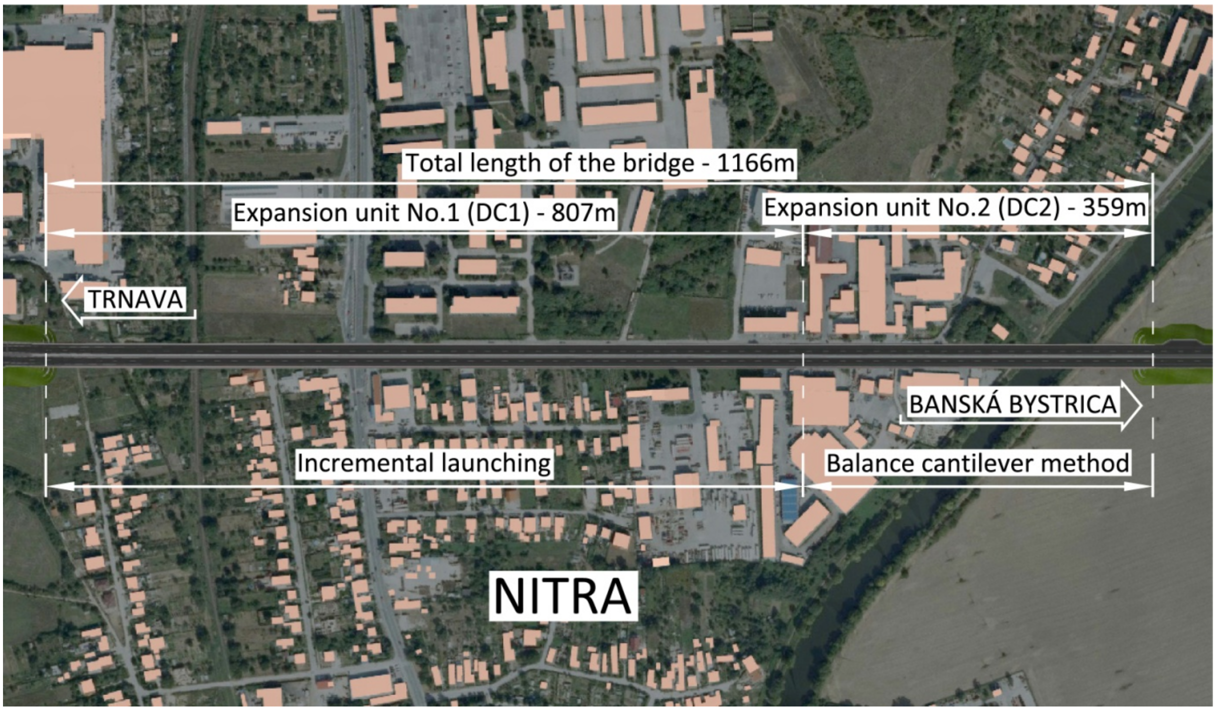

Fig.2 Bridge two superstructures

2. DESIGN CONCEPT OF BRIDGE SUPERSTRUCTURE

Final design of the bridge superstructure was significantly influenced by the site conditions. The bridge crosses the railway track, the I/64 road, various in-town roads and streets and the river Nitra. The R1 expressway in km 8.4-8.7 is situated in the town's industrial zone with a number of plants. Within the given locality the precondition was to minimize the land required, which resulted in increasing the bridge spans. Due to this reason the bridge was divided into two separate structures with different length of spans in km 8.47 of the R1 expressway.

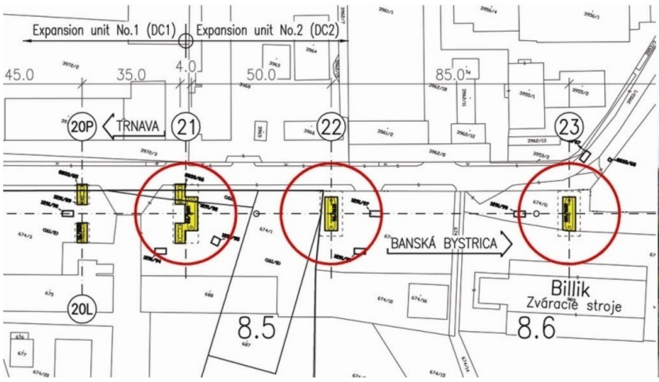

Fig.3 and 4 Land required in the industrial zone

Spacing of piers for construction unit N°1 (CU1) was influenced by the site conditions; length of superstructure spans varies from 40 to 45 m. Spacing of piers for construction unit N°2 (CU2) was given by the site conditions (the industrial zone) and the bridging of the river Nitra; the length of superstructure spans is 85 m. The form of the bridge substructure was influenced by the lack of space for the bridge foundations within the river Nitra flood protection dykes and necessity to minimize the land required within the industrial zone. The resulting permanent land required at the given locality is "only" outline of the piers.
In km 8.47 of the R1 expressway a joint pier with expansion joint is situated in order to enable movement of two different superstructures. Design of cross sections of the aforementioned structures was influenced by aesthetic reasons so as their junction would not form a "disturbing feature".

3. CONSTRUCTION UNIT N°1 (CU1)

The unit N°1 (CU1) has been designed as two separate parallel bridges. The superstructure of each of them is formed by a single-cell prestressed cast-in-situ structure with constant cross section strengthened above the piers and in the places of deviators and fixing devices. From the structural point of view the superstructure represents a continuous 20-span girder (with depth of 2.67 m) constructed by the incremental launching method altogether in 40 segments with their length varying from 12.0 – 22.5 m. The bridge substructure consists of abutment N°1, piers and the joint pier N°21. The bridge foundations are designed to be resting on piles with diameter of 0.90 m. The bridge superstructure rests on "calotte" bearings (spherical knuckle bearing).

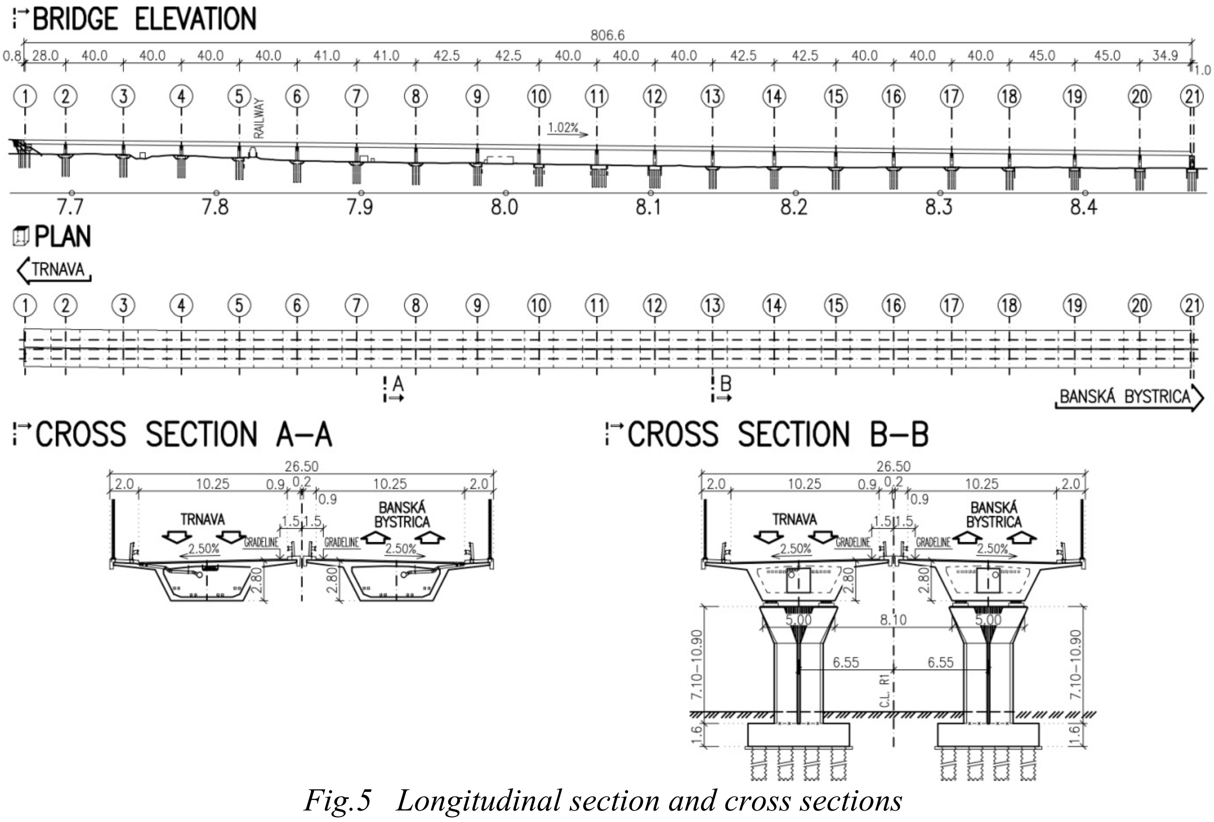

Fig.5 Longitudinal section and cross sections

Casting of superstructure (its individual segments) is carried out in the concrete-casting plant situated behind the abutment N°1. Casting is performed in two stages: the bottom slab and webs and then the top slab. The length of individual segments varies from 12.0 – 22.5 m and depends on the section position within the bridge superstructure. Due to technological reasons the length of the first two segments is 12.0 m. The incremental launching of bridge superstructure is carried out by means of two hydraulic jacks situated at the abutment N°1 and on the auxiliary construction at the pier N°11. Due to the structural and the technological reasons the 32 m long steel launching noses are firmly fixed with the first segment of bridge superstructure.

The superstructure prestressing includes 3 basic types of tendons:

I.) longitudinal bonded tendons (prestressing during superstructure launching)
II.) longitudinal unbonded tendons inside the bridge box
III.) prestressing bars and bonded tendons for fixing the launching nose.

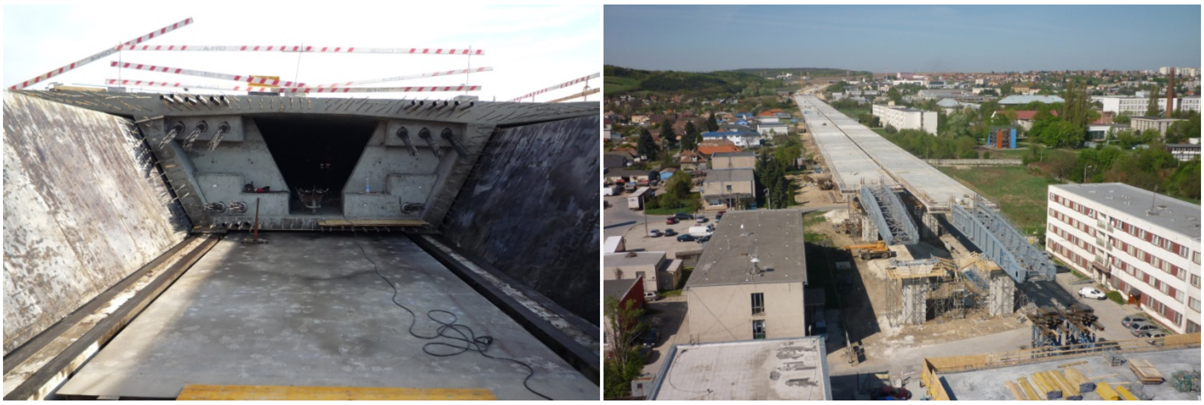

Fig.6 and 7 Bridge superstructure construction

After launching the bridge superstructure into final position the unbonded tendons inside the bridge box are prestressed as first and then the temporary slide plates are replaced with permanent bridge bearings. By the contractor's decision there are larger slide plates used on the bridge in consequence of which the superstructure is constructed 60 mm higher compared to the design level. The replacement of slide plates and seating of superstructure on permanent bearings is carried out in several stages by phased structure settlement, provided that the uneven settlement of bridge superstructure at adjacent piers shall not exceed the value of 20 mm.

When deciding on construction method for the superstructure of construction unit N°1, the time factor was crucial; due to this reason the most suitable construction method was the incremental launching. By using this method the superstructure of the unit N°1 was completed in 320 days.

4. CONSTRUCTION UNIT N°2 (CU2)

The unit N°2 (CU2) has been designed as a single bridge (single superstructure) for both R1 expressway directions. The bridge superstructure is formed by 3-cell prestressed cast-in-situ structure with variable depth and lower haunches above piers. From the structural point of view it represents a 5-span continuous girder (with the depth varying from 2.80 m to 4.50 m), constructed by the balanced cantilever construction method for the whole cross section width (26.0 m). The bridge superstructure consists of four balanced cantilevers and two end spans constructed on stationary scaffolding. The bridge substructure consists of the joint pier N°21, piers N°22-25 and abutment N°26. The bridge foundations are designed to be resting on piles with diameter of 0.90 m. The bridge superstructure rests on "calotte" bearings (spherical knuckle bearing).

The in-situ cantilever construction is being carried out by means of three pairs of form travelers (6 pcs in total); the balanced cantilevers over the river Nitra were constructed as first, then those over industrial zone of “Priemyselná” street. Each balanced cantilever is formed by 12.5 m long base segment and 7 pairs of typical segments with the length of 4.75 – 5.00 m; its overall length is 82 m, the maximum cantilever overhang during the superstructure construction is 41.50 m. Casting of individual segments is carried out completely in one pour; except for the deviators for unbonded tendons, which are being carried out subsequently through construction openings in the deck slab.

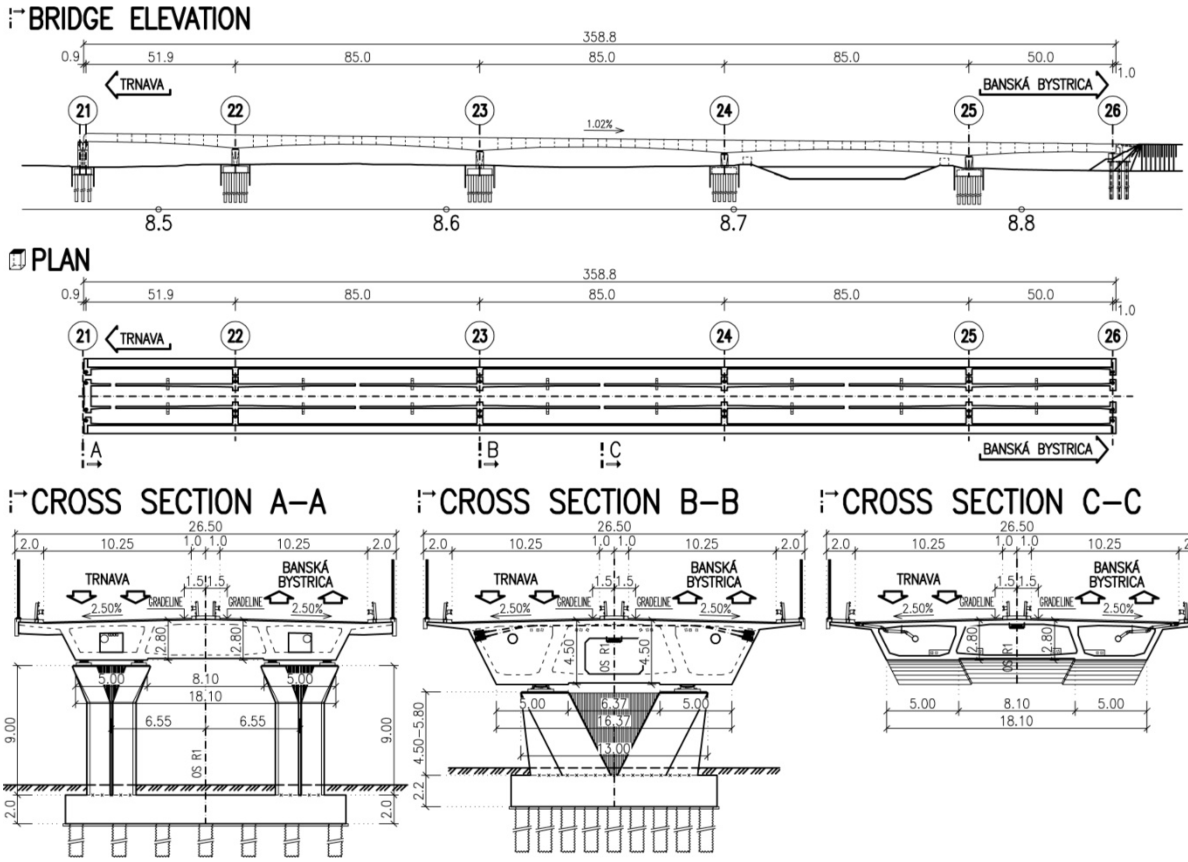

Fig.8 Longitudinal section and cross sections

The design of superstructure prestressing includes 3 basic types:

I.) longitudinal bonded prestressing (the top slab, webs and the bottom slab)

II.) transverse bonded prestressing at diaphragms over the pier

III.) longitudinal prestressing with unbonded tendons inside the bridge box cells.


Fig.9 and 10 Bridge superstructure construction

At each pier the bridge superstructure rests on the pair of bearings. The result of this is an indirect support of the outer webs; for the transfer of resulting shear forces at the diaphragm it was necessary to use the transverse bonded tendons, which were prestressed gradually in 2 stages, depending on the intensity of shear forces in the diaphragm area.

Stability of the balanced cantilevers during construction was provided for by means of temporary concrete walls erected on piers footings which will be removed after completing the superstructure.

The site conditions, the need to minimize land required during construction works and the crossing of the river Nitra are the main reasons for using the balanced cantilever construction method for construction of the unit N°2. The total time for the superstructure completion is 250 days.

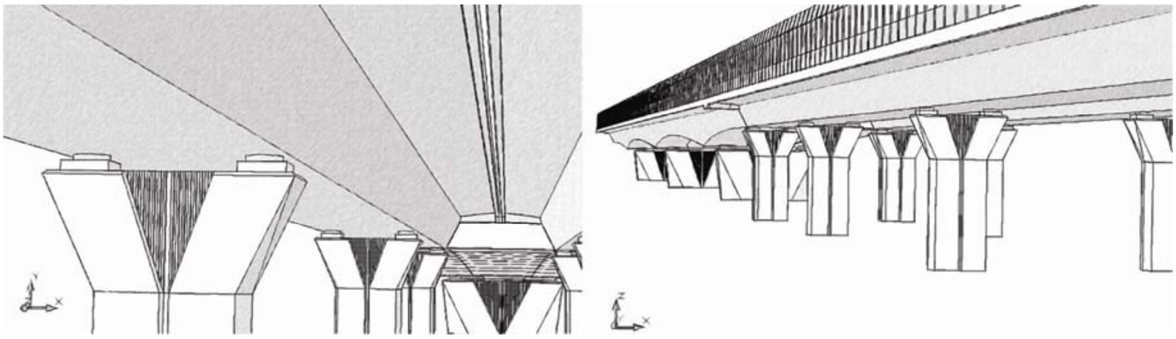

Fig.11 and 12   Visualisation

5. CONCLUSIONS

The introduced bridge is a part of the R1 Expressway in section Nitra, west – Selenec which belongs to the second package of PPP projects in Slovakia. General Contractor of the Project is GRANVIA CONSTRUCTION Ltd. Contractor for the bridge is the Hungarian company A-HÍD ÉPÍTŐ ZRT. Construction works started in 12/2009 and the bridge will be completed during the year 2011.

6. REFERENCES

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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
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
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
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

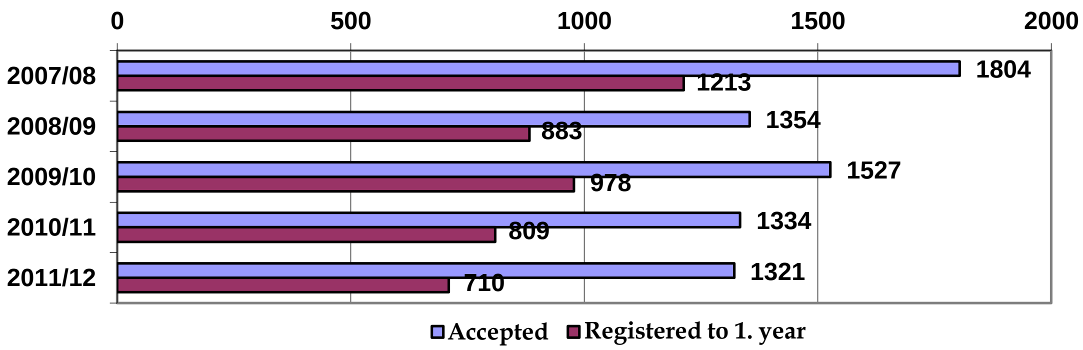
Master's degree study	Semester	Hours Per Week Lectures Seminars	Lecturer
Modeling of RC Structural Systems I	1.	2 – 2	I. Harvan
Modeling of RC Structural Systems II	1.	2 – 2	I. Harvan
Modeling of 2D RC Structures	2.	2 – 2	J. Šoltész
Modeling of 3D RC Structures	2.	2 – 2	J. Šoltész

Postgraduate courses

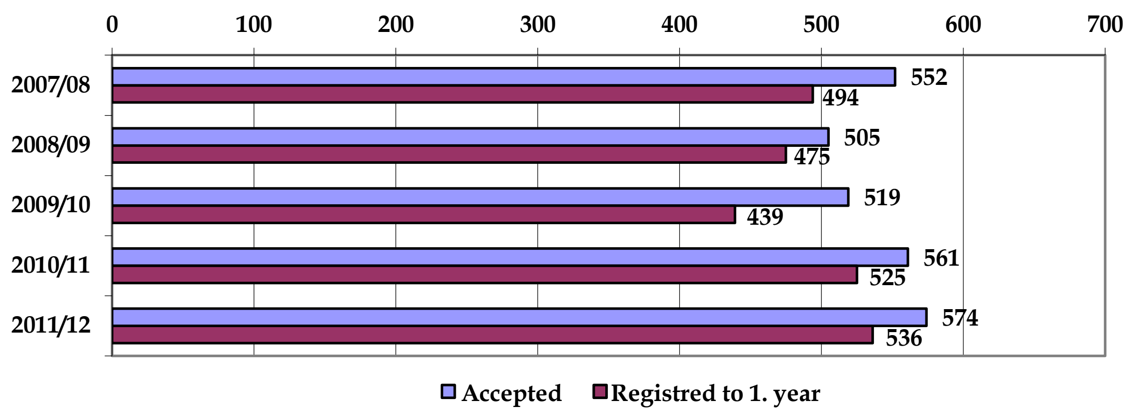
Advanced Concrete and Masonry Structures
Advanced Concrete Bridges
Reliability and Strengthening of Concrete Structures
Modeling of Concrete Structures
Structural Materials and Systems
Experimental Testing of Concrete Structures
Advanced Reinforced Concrete Structures

VIII.2 Acceptance test

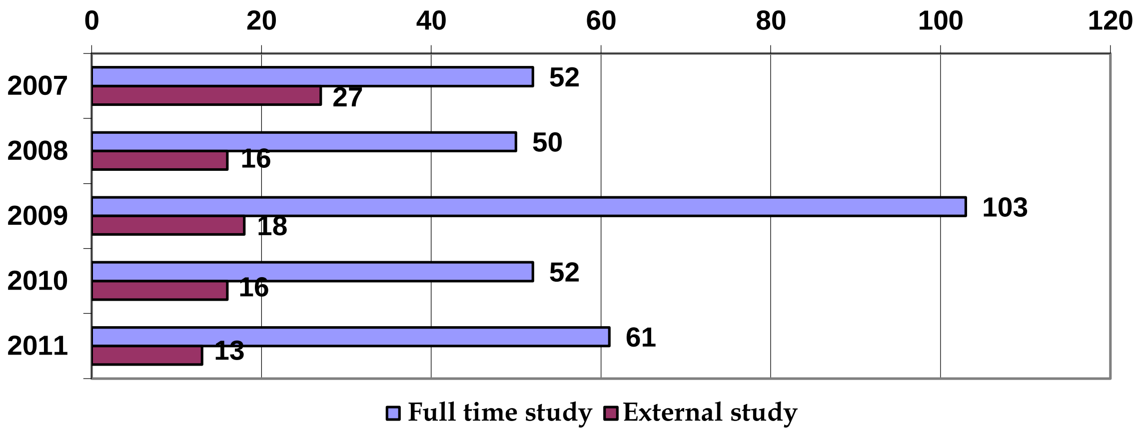
Graph VIII.1 – Number of accepted students and students registered to 1. year of Bachelor study



Graph VIII.2 – Number of accepted students and students registered to 1. year of Bachelor study

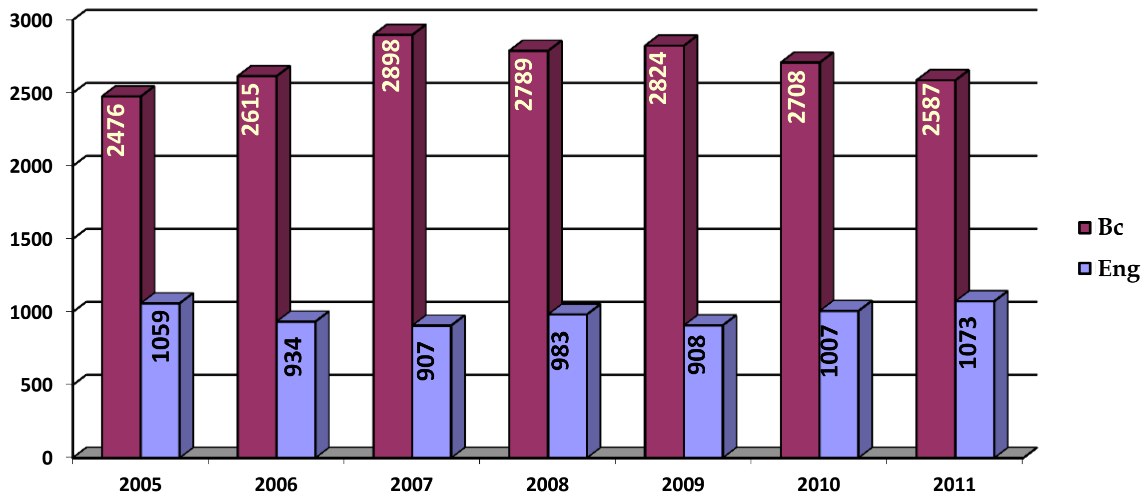


Graph VIII.3 – Number of accepted students for PhD study

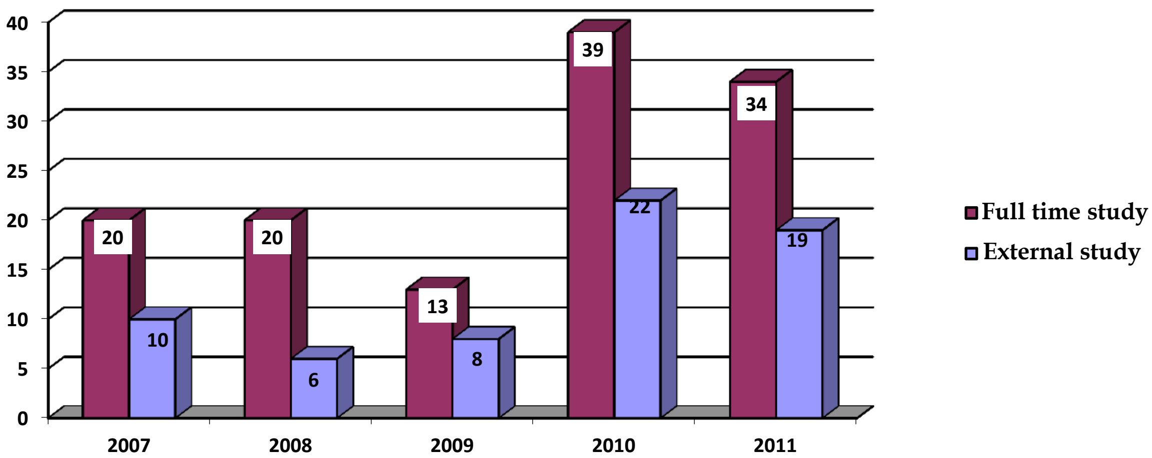


VIII.3 Results of studies

Graph VIII.4 Number of students of Bachelor and Engineer study



Graph VIII.5 Number of students successfuly finished on PhD study



IX. THESES

IX.1 Bachelor Theses:

PRÁZNOVSKÝ, M.: Ceiling structure design of one-way cast-in-place RC slab
Supervisor: Halvoník, J.

KISS, T.: Parking house – cast-in-place ceiling structure
Supervisor: Borzovič, V.

HALMO, M.: Ceiling structure of a polyfunctional building
Supervisor: Borzovič, PhD.

IX.2 Graduate Theses

BASZO, D.: Residential Building: Cast-in-Situ Reinforced Concrete Slab-Wall Structure
Supervisor: Harvan, I.

BOČEK, J.: Hotel - Staré Grunty, Bratislava: Combined Cast-in-Situ Reinforced Concrete Structure with a Stiffening Core
Supervisor: Abrahoim, I.

BREZINOVÁ, M.: Complex of Buildings for Sports: Cast-in-Situ RC Framework Structure and Halls of a Long Span
Supervisor: Harvan, I.

BUBLIŠ, M.: Multifunctional Building: Cast-in-Situ Hollow Slab Concrete Structure
Supervisor: Borzovič, V.

DUBOVÁ, A.: Office Building: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Borzovič, V.

HALAŠKA, M.: Office Building in Bratislava: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Benko, V.

KÁČEROVÁ, L.: Multifunctional Building: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Borzovič, V.

KLIMEKOVÁ, L.: Hotel “TULA” - Banská Bystrica: Cast-in-Situ Reinforced Concrete Slab-Wall Structure on a Skeleton Pedestal with a Stiffening Core
Supervisor: Harvan, I.

MÁJOVSKÝ, M.: Multifunctional Apartment House: Cast-in-Situ Reinforced Concrete Framework Structure
Supervisor: Bartók, A.

MOROZ, M.: Slovak Savings Bank Building, Bratislava: Cast-in-Situ Reinforced Concrete Structure with Flat Slabs and Stiffening Cores
Supervisor: Abrahoim, I.

NOVÁ, P.:Apartment Block: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Borzovič, V.

PASTIER, I.: Multifunctional Apartment Building: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Bartók, A.

PEŤKO, V.: Office Building of an Automobile Centre - Bratislava Rožňavská: Cast-in-Situ Reinforced Concrete Combined Bearing Structure with a Stiffening Core
Supervisor: Abrahoim, I.

RECHTORÍK, J.: High-Rise Building: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Benko, V.

SIKLENKA, M.: Shopping Centre: Large-Span Precast Concrete Structure
Supervisor: Borzovič, V.

SONNENSCHEIN, R.: “Letná" Multifunctional Building: Cast- in-Situ Reinforced Concrete Structure
Supervisor: Gajdošová, K.

VARGOVÁ, M.: Poprad Tax Office: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Harvan, I.

ÁDÁM, N.: Underground Tank of a Cooling Tower: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Hudoba,I.

ANDRÁSSY, P.: High-Rise Building: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Fillo, Ľ.

ČANIGA, I.: Concrete Road Bridge: Cast-in-Situ Concrete Structure
Supervisor: Fillo, Ľ.

DOLINAJOVÁ, K.: Bridge on Expressway R1: Nitra West - Selenec
Supervisor: Halvoník, J.

DRÁB, O.: High-Rise Building of an Office Center: Reinforced Concrete and Composite Steel and Concrete Structure
Supervisor: Gramblička, Š.

ĎURAČKA, J: High-Rise Building: Cast-in-Situ Reinforced Concrete Structure
Supervisor: Benko, V.

GÁL, A.: Underground Box Cast-in-Situ Tank
Supervisor: Hudoba, I.

GAŽIOVÁ, J.: Prestressed Cable-Stayed Bridge on Expressway R1 at km 16,044, Section Žarnovica - Šašovské Podhradie
Supervisor: Halvoník, J.

GOLEJ, M.: Supporting Structure of a High-Rise Building
Supervisor: Gramblička, Š.

GRMAN, A.: Integral Bridge over Highway D3: Kysucké Nové Mesto - Oščadnica
Supervisor: Borzovič, V.

HLÁSNY, L.: Design of a Railway Bridge Made from Prestressed Concrete
Supervisor: Fillo, Ľ.

CHRAPPA, M.: Prestressed Bridge on Highway R3 at km 1,772 over a Brook and Valley
Supervisor: Halvoník, J.

JACKOVÁ, A.: Prestressed Extradosed Bridge over Road C/11 and the River Kysuca
Supervisor: Halvoník, J.

LÖRINCZ, M.: Four-Span Bridge on Highway D3 at km 23,099
Supervisor: Halvoník, J.

MÉZSÁROS, T.: Design of the Underground Parking of the "Vtáčnik" Residential Building as a Watertight Concrete Tank
Supervisor: Šoltész, J.

IX.3 Stundent's Scientific Conference Theses:

MÁJOVSKÝ, M.: Considering of construction stages and non-uniform stresses in concrete columns by changing of elasticity modulus

GRMAN, A.: Design of integral bridge´s pillar

MATOVČÍK, Š.: Effect of uneven foundations´ sag on a frame structure

SONNENSCHEIN, R.: Parametric study of design effectiveness for reinforced concrete foundation slab

KLIMEKOVÁ, L.: Analysis of multi-storey buildings´ bearing walls on skeleton pedestal

BAZSO, D.: A calculation of crack width in reinforced concrete walls by STN EN 1992-1-1

BREZINOVÁ, M.: Accidental loading caused by an impact of road vehicle in the columns in garage area

VARGOVÁ, M.: Practical analysis of foundation slab interaction with subsoil and upper structure

IGNAČÁK, M.: Strenght verification of pipeline system in JE Mochovce

IGNAČÁK, M.: Alternative static solution of cast-in-place reinforced concrete beam by application of prestressing

VOZÁR, M.: Practical possibilities of deflection elimination in pre-stressed end span slab