# HABILITATION THESIS

## Fire Design of Civil Engineering Structures

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# A. ABSTRACT

The present thesis summarizes a part of the research activity of the candidate after defending the PhD Thesis at the Politehnica University of Timişoara in February 2000. The selected activity was considered to be relevant in terms of originality and importance, in order to anticipate an independent development of the further research and teaching career.

The presentation of the post-doctoral activity is developed within the main thematic direction: "Fire design of civil engineering structures". A secondary direction is also considered: "Design assisted by testing".

The candidate was involved in the main topic "Fire design of civil engineering structures" since 2000, after the defense of the PhD thesis, when he obtained a research grant of one year, offered by the Services for Scientific and Cultural Affairs of the prime Minister of Belgium. The research was lead by Prof. Jean Marc Franssen from Liege University in Belgium, a world-wide recognized pioneer in the field of the fire design of civil engineering structures, with decisive contributions in the relatively recent topic of calculation of the fire resistance of structures. The research was focused on the fire behaviour of high-rise steel rack structures and a description of the main results of the research is presented in Part B section 2.5.1 "Fire resistance analysis of high-rise rack structures".

The relevance of the scientific activity and the recognition of the national and international activity in the field of the first main direction "Fire design of civil engineering structures" is emphasised by the publications of the candidate, mostly in cooperation with European researchers, but also by the involvement in two European technical committees. Another relevant aspect for the recognition of the international activity of the candidate in the field is that he was member of the Scientific Committees for the recent editions of the only two specialised international conferences dedicated exclusively to the structural analysis component of the fire engineering.

The candidate participated at two European COST projects dealing with the fire behaviour of civil engineering structures. The candidate was member of the first Working Group "Fire resistance" of COST Action C26. Within this Working Group, the candidate was responsible together with Beatrice Faggiano, PhD from University of Naples "Federico II", for the topic "Fire after earthquake". The relevant contributions of the candidate in this topic are presented in section 2.5.2 "Fire resistance analysis in extreme situations – fire after earthquake". Within the second COST Action TU0904, which deals entirely with the fire design of civil engineering structures, the candidate is Co-chairman for the Working Group 2, and member of the Managerial Committee. The candidate was one of the five members which presented the first draft of the proposal to the COST Office. One important topic within this COST Action is the validation of the numerical models for the advanced fire analysis of structures. Part of the activity of the candidate on this topic is presented in Part B section 2.3 "Validation of the advanced calculation models for fire design".

The candidate was responsible for the Politehnica University of Timisoara for three European projects dealing with the first main thematic direction. All these projects, with European partners including universities, research centres and industry, were coordinated at European level by a strong industrial partner, ArcelorMittal Luxembourg. The responsible for these projects was Olivier Vassart, PhD, Head of Structural Long Products R&D at ArcelorMittal and Invited Professor of Steel and Composite Structures at Université Catholique de Louvain, a leader European researcher in the field of fire engineering. Following this cooperation, the candidate published some papers with Olivier Vassart, regarding the fire resistance of composite steel-concrete floors. Part of the activity related to this topic is presented in Part B section 2.4 "New recommendations for fire design".

The implementation of the fire design principles is still an on-going process in Romania. However, in the last decade, the candidate calculated the fire resistance of the structural elements for some structures built in Romania. This was a premiere in Romania and, up to this moment, no other similar design cases exist. Part of this activity is presented in Part B section 2.2 “Application of modern fire design in Romania”.

The research in support for new design recommendations was one another activity of the candidate, described in Part B section 2.4 “New recommendations for fire design”.

The cooperation with Prof. Jean Marc Franssen from Liege University continued after the conclusion of the research grant obtained by the candidate in 2000-2001. One part is represented by the research work performed in collaboration with the team lead by Prof. Jean Marc Franssen at Liege University, highlighted by the topics presented throughout Part B section 2 “Fire design of civil engineering structures”. The second part is represented by the publication of two books. The first book (2006) was a world premiere on the topic of fire design of steel structures, being not only a background of the actual Eurocode 1993-1-2, but also a design guide for the engineers, including an introduction into the general topic of fire design. The second book (2009), represented an improved and extended edition of the first one, and included also the relevant fire design provisions from the USA codes. The other author of this book is Prof. Venkatesh Kodur, from Michigan State University, one of the top researchers and teachers in structural fire design in North America. The relevance of these books and their international impact is highlighted by the high number of citations in conferences and journal papers (from which four in journals with high relative impact factor, relevant for the habilitation thesis).

The development of the experimental facilities in CMMC laboratory of the Politehnica University of Timisoara was, since 1994, one of the continuous activities of the candidate. The relevant post-doctorate activity in the secondary thematic direction “Design assisted by testing” is presented in Part B, section 3 of the Habilitation Thesis.

The candidate participated to three major grants in which the CMMC laboratory was substantially transformed by acquisition of new equipments (two as part of the managing team and one as director). Within a “Capacities” type grant, for which the candidate was director, together with the acquisition of new equipments, the laboratory of structures was extended with a new building, which houses a unique facility in Romania and in the Eastern Europe (and one of the few in Europe): an experimental stand for static, dynamic and pseudo-dynamic testing of structural details and real scale buildings. A description of the new experimental facility within CMMC laboratory is presented in Part B section 3.3.

The candidate applied for this grant following the experience gained in the field of pseudo-dynamic testing of real scale building structures within a research grant of two years, wined by competition at the European Commission’s Laboratory for Structural Assessment – ELSA, of the Joint Research Centre –JRC, located in Ispra, Italy, which represents the biggest real scale pseudo-dynamic testing facility in Europe. This activity is presented in section 3.2 “Pseudo-dynamic test of a real scale flat-slab reinforced concrete struture”.

All the scientific, professional and academic future development plans of the candidate deal in principal with the main thematic direction “Fire design of civil engineering structures”.

Even if the main direction for further development is fire engineering, the experience and the activity in the second thematic direction “Design assisted by testing” will allow the candidate to follow also this direction, especially by the realisation of new experimental facilities in the CMMC laboratory, linked to the topic of structural fire research.

A first plan in this direction is to develop the CMMC laboratory capabilities by the acquisition of an electrical system for the local heat of structural elements. This would lead to a unique experimental facility in Romania, which would have the possibility to test a real-scale structure, heated to a given level of temperature and then loaded with external loads (within the new reaction wall-strong floor experimental facility).

Some research directions were identified for short and middle term, based on actual preoccupations of the candidate and on the numerical and experimental capabilities available at CMMC department:

- Behaviour of composite steel-concrete floors with cellular beams
- Natural fire models – Localised fires
- Validation of the advanced calculation models for fire design
- Fire behaviour of steel connections
- Simple design recommendations for Slim Floor beams

One long term objective of the candidate is to properly implement the fire design into the current design practice in Romania. Further courses targeted on fire design of steel and composite steel-concrete structures (EN 1993-1-2 and EN 1994-1-2), are provided to begin from 2014 at national level for civil engineers, within a larger package on EN 1993 design rules. The candidate is in charge with the lectures on the mentioned fire parts of Eurocodes.

The first step for a proper implementation of the fire design practice in Romania is to prepare civil engineers with the knowledge of fire engineering from the university, also through diploma works targeted in this direction. Since 2009, Bachelor and Master Thesis on fire engineering are performed in double coordination by the candidate and by Prof. Jean Marc Franssen, through an ERASMUS agreement with Liege University in Belgium. This agreement was prolonged this year and it is the intention of the candidate (and of Prof. Jean Marc Franssen) to continue this collaboration on long term.

The candidate is attested by the Ministry of Public Works for expertise and verification of the civil engineering buildings projects for fire safety (verification for projects since 2009, Expert since 2009) and intends to continue this activity, related to the authorisation of civil engineering buildings.

The candidate is member of the professional Romanian association AICPS (Asociatia Incginerilor Constructiori Proiectanti de Structuri). Through the conferences organised by this associations, or by publications in the dedicated review, the candidate promoted the fire design through presentations and articles on the principles of fire design, or on the particular fire design study cases that he conducted. This kind of activities will continue.

In 2009 was founded the Romanian Association of Engineers for Fire Safety (Asociatia Romana a Inginerilor pentru Securitate la incendiu – ARISI). The candidate was among the founder members and is the president of the Timis Region branch of this association and will continue to support the implementation of the fire design in Romania through this association. It is the intention of the candidate to propose further application guides for Eurocodes EN 1993-1-2, EN 1994-1-2 and EN 1999-1-2 for fire design.

Through the actual research activity related to the localised fires in which the candidate is involved and through its participation in the European Committee for Standardization - Technical Committee CEN/TC 250/SC 01/WG 04 "Actions on structures exposed to fire", the candidate will participate to the improvement of the natural fire models in the Eurocodes.

The involvement of the candidate in some national and international grants as director or managing team member provided the relevant skills and competences on management of such projects. One important aspect in the further development of the career of the candidate is to build a research team focused in the direction of fire engineering at home university. It is the intention of the candidate to recruit further potential PhD students among the students involved in Master Thesis on the topic of fire engineering, especially from the ones which gain an international experience by performing double coordination thesis within the mentioned collaboration with Liege University. It has to be mentioned that the candidate already trained one young researcher fom CMMC department in the field of fire engineering, by involving him into the COST IFER activity (including participation to 2 STMS – Short term Scietific Missions) and on the related research of the candidate in the field, but also in the teaching activity, by leading the seminars on fire engineering at the three Master courses.

# B. SCIENTIFIC, PROFESSIONAL AND ACADEMIC ACHIEVEMENTS

## 1. INTRODUCTION

The present thesis summarises a part of the research activity of the candidate after defending the PhD Thesis at the Politehnica University of Timişoara in February 2000. The selected activity was considered to be relevant in terms of originality and importance, in order to anticipate an independent development of the further research and teaching career.

The presentation of the post-doctoral activity is developed in two main thematic directions: "Fire design of civil engineering structures", presented in Chapter 2, and "Design assisted by testing", presented in Chapter 3.

The candidate was involved in the first topic "Fire design of civil engineering structures" since 2000, after the defense of the PhD thesis, when the candidate obtained a research grant of one year, offered by the Services for Scientific and Cultural Affairs of the prime Minister of Belgium. The research was focused on the fire behaviour of high-rise steel rack structures and a description of the main results of the research is presented section 2.5.1 "Fire resistance analysis of high-rise rack structures". The activity presented in this section is sustained by the seventh publication in the list selected by the candidate to be relevant for the professional achievements after he obtained his PhD:

**07 - Zaharia R.,** Franssen J.M. (2002) "Fire design study case of a high-rise steel storage building". Stability and Ductility of Steel Structures - SDSS 2002, Editor: Ivanyi M., Akademiai Kiado, Budapest, Hungary, ISBN 963-05-7950-2

One condition imposed to the applicants for this research grant was to bring the proof of a relevant research in the field of structures made of thin-walled cold-formed steel elements (the rack structures being realized with this particular type of structural elements). The candidate fulfilled this condition, considering its research work done within the PhD thesis, dealing with the safety of the civil engineering structures made by thin-walled cold-formed steel elements. The thesis focused on the behaviour of the bolted connections of such structures.

The research was lead by Prof. Jean Marc Franssen from Liege University in Belgium. Prof. Jean Marc Franssen is a world-wide recognized pioneer in the field of the fire design of civil engineering structures, with decisive contributions in the relatively recent topic of calculation of the fire resistance of structures.

Indeed, util the last decades of the last century, few countries had normative regulations for the fire design of civil engineering structures. Moreover, the existing regulations were generally incomplete and unable to cover many of the complex aspects of the fire design. There was a huge lack of information in terms of fire behaviour of different materials used in civil engineering structures and in terms of proper methodologies to calculate the fire resistance of different possible topologies of structural elements. Another problem was the lack of information in terms of fire action, which was limited to some nominal temperature – time curves, used in the fire tests (in special the standard ISO fire curve), but which did not considered any physical parameter which may affect the development of the fire in a building. The problem of the fire resistance of civil engineering structures is a very important feature in the process of authorization of the civil engineering projects and was traditionally in the hand of the authorities (firemen), generally in base of some poor and conservative assumptions based on observations from tests. Traditionally it was (and generally it is still !) considered that concrete structures provide a very good fire resistance, while the steel structures may be used only if they are fire protected.

The last decades of the XX$^{th}$ century have seen important research endeavours devoted to the calculation of the behaviour of building structures submitted to fire (Zaharia & Franssen, 2006). This activity was particularly significant in Western Europe where numerous research reports, PhD theses and scientific papers were published. Among the first internationally recognised codes of practice are, for steel elements, the recommendations of the ECCS "*European Convention for Constructional Steelwork*" (ECCS 1983) and, for concrete elements, the recommendations of the CEB/FIP "*Comité Euro-International du béton / Fédération Internationale de la précontrainte"* (CEB 1991).

The ambition that prevailed at the redaction of the Eurocodes was to publish a set of documents that would form a common basis for the design of structures made of various materials. This was also the aim of the fire parts of the Eurocodes when they were first presented in Luxemburg in 1990. These documents have since evolved from a code of practice to a more formal text when they were taken on board by the CEN "*Comité Européen de Normalisation"* and transformed first in ENV's, or provisory European norms, in the mid 90's, then more recently in EN's, or European norms, in the first years of the XXI$^{rst}$ century.

During this evolutionary process, research activity kept going on and was even boosted by the fact that the texts would eventually end up in a more official and more permanent format: the EN's. It was indeed desirable that as few questions as possible remain unanswered when the final drafts would be accepted. The documents have therefore been significantly modified in the last decade and a lot of recent research results were introduced.

However, there is still a gap between the experience that exist in some European countries between the researchers and the designers, which are able to provide a modern fire design and the fire authorities, which in many cases are not accepting such calculations, or, at least, are not accepting some aspects related to the modern fire engineering (as, for example, the natural fire concept).

In Romania, the document OM 620/29.04.2005, issued by the Romanian Ministry of Transportation, Civil Works and Tourism, regarding the implementation and use of the structural Eurocodes, states that for the design problems which are not covered by the existing national regulations, Eurocode provisions can be applied, based on CEN approved documents, even before national implementation (this includes fire design as well).

It has to be underlined that for fire design of steel structures, two normative documents based on ENV1993-1-2 have been already published in Romania in 2000 (Romanian norm and guide of application):

-"Normativ pentru verificarea la foc a elementelor structurale ale constructiilor din otel" – NP046/ 2000

-"Ghid pentru verificarea la foc a elementelor structurale ale constructiilor din otel" – GP055/ 2000"

These documents were elaborated at the home university of the candidate, in the CMMC – "Steel Structures and Structural Mechanics" department of the Politehnica University of Timisoara, under the supervision of the actual Head of Department, Prof. Dan Dubina, Corresponding Member of the Romanian Academy, in collaboration with INCERC Bucharest.

More recently, the document OM 130/2007 issued by the Romanian Ministry of Administration and Internal Affairs, regarding the methodology of elaboration of the fire safety scenarios, allows to determine the fire resistance of the main resistant elements of the building, based on the fire parts of the Eurocodes. However, there is still a gap between the prescriptive rules existing in the national norm for fire safety of buildings, P118-99 (P118, 1999) and the performance based fire design. The Romanian norm for fire safety of buildings P118-99 is based on prescriptive rules and the requirements of fire resistance consider isolated elements with specified fire duration function of the occupancy, surface, height and fire risk, and no reference is made to the possibility to calculate the fire resistance of the structural elements. This norm is subjected to revision, and will include the concept of fire design. The candidate is part of this revision process.

The candidate was also part in the process coordinated by ASRO in Romania, for the translation of the fire parts of the Eurocodes EN 1993-1-2, EN 1994-1-2 and EN 1999-1-2, dealing with the fire design of steel, composite steel-concrete and aluminium structures, together with the corresponding National Annexes. These documents were elaborated at the CMMC department, under the supervision of Prof. Dan Dubina.

The implementation of the fire design principles is still an on-going process in Romania. However, in the last decade, the candidate calculated the fire resistance of the structural elements for some structures built in Romania. This was a premiere in Romania and, up to this moment, no other similar design cases exist. Part of this activity is presented in section 2.2 “Application of modern fire design in Romania”, and is sustained by the first publication in the list of publications selected by the candidate to be relevant for the professional achievements:

**01 - Zaharia R.,** Pintea D., Dubina D. (2007) “Fire analysis and design of a composite steel-concrete structure”, Steel and Composite Structures, Editors: Wang Y.C., Choi C.K., Taylor & Francis Group, London, U.K., ISBN 978-0-415-45141-3

The candidate organized three seminars on the fire design of civil engineering structures at the Politehnica University of Timisoara, at which participated engineers, architects, but also authorities involved in the process of the authorization of constructions. The scope of the seminars was to disseminate the new concepts of fire design, which are, as mentioned above, not yet fully implemented in Romania. These seminars where unique of this kind in Romania, on the topic of fire design:

- „Calculul structurilor pentru constructii la actiunea focului/ Design of civil engineering structures for fire action”, Timisoara, december 2008;
- „Evaluarea rezistentei la foc a planşeelor compuse protejate parţial/ Fire resistance of partially protected composite floors” Timisoara, september 2011;
- „Efectul de membrana in evaluarea rezistentei la foc a planseelor compuse otel-beton, realizate din grinzi cu inima plina sau ajurate/ Membrane effect in the evaluation of fire resistance of composite floors with solid and cellular beams”, Timisoara, december 2012.

The research in support for new design recommendations was one another activity of the candidate, described in section 2.4 “New recommendations for fire design”. Part of this activity is sustained by the following publications from the list selected by the candidate to be relevant for the professional achievements:

**04 - Zaharia R.,** C. Vulcu, O. Vassart, T. Gernay, Franssen J.M. (2013). "Numeric analysis of partially fire protected composite slabs". Steel and Composite Structures 14(1). ISSN 1229-9367

**05 - Zaharia R.,** Franssen J.M. (2012). "Simple equations for the calculation of the temperature within the cross-section of slim floor beams under ISO fire". Steel and Composite Structures 13(2). ISSN 1229-9367.

**06 - Zaharia R**., Vassart O. (2012). "Fire analysis of slim floor systems using Cofradal floor units". International Conference on Advances on Steel Concrete Composite and Hybrid Structures - ASCCS 2012, July 2-4, Singapore, Edited by J. Y. Richard Liew and Siew Chin Lee, ISBN-13: 978-981-07-2615-7, ISBN-10: 981-07-2615-5

The relevance of the scientific activity and the recognition of the national and international activity in the field of the first main direction “Fire design of civil engineering structures” is emphasised by the publications of the candidate, mostly in cooperation with European researchers, but also by the involvement in two European technical committees:

- expert for Romania in Technical Committee 3 “Fire Design”of ECCS (European Convention for Constructional Steelwork);
- expert for Romania in the European Committee for Standardization - Technical Committee CEN/TC 250/SC 01/WG 04 "Actions on structures exposed to fire".

Another relevant aspect for the recognition of the international activity of the candidate in the field is that he was member of the Scientiffic Committes for the recent editions of the only two specialised international conferences dedicated exclusively to the structural analysis component of the fire engineering: “Applications of Structural Fire Engineering”, which traditionally is held in Prague Czech Republic (editions 2009, 2011, 2013) and “Structures on Fire” (2012’th edition, held at Zurich, Zwissland).

The candidate is also involved as expert for evaluation of European research projects within the European Commission, Directorate General for Research and Innovation, G5 – Research Fund for Coal and Steel – RFCS.

The candidate participated at two European COST projects dealing with the fire behaviour of civil engineering structures:

- Cost Action C26 “Urban habitat constructions under catastrophic events” 2006-2010;
- COST Action TU0904: “Integrated Fire Engineering and Response – IFER”, 2010-2014.

The candidate was member of the first Working Group “Fire resistance” of COST Action C26. Within this Working Group, the candidate was responsible together with Beatrice Faggiano, PhD from University of Naples “Federico II”, for the topic “Fire after earthquake”. The relevant contributions of the candidate in this topic are presented in section 2.5.2 “Fire resistance analysis in extreme situations – fire after earthquake”. Part of this activity is sustained by the following publications from the list of publications selected by the candidate to be relevant for the professional achievements:

**08** - Faggiano B., Esposto M., **Zaharia R.**, Pintea D. (2008) “Risk management in case of fire after earthquake – Structural analysis and design in case of fire after earthquake”. Urban Habitat Constructions under catastrophic Events, Editors: Mazzolani F., Mistakidis E., Borg R.P., Byfield M., De Matteis G., Dubina D., Indirli M., Mandara A., MUseau J.P., Wald F., Wang Y., Malta University Publishing, Malta, ISBN 978-9909-44-40-2

**09 - Zaharia R.,** D. Pintea (2009). "Fire after earthquake analysis of steel moment resisting frames". International Journal of Steel Structures (INT J STEEL STRUCT). 9(4). ISSN 1598-2351

Within the second COST Action TU0904, which deals entirely with the fire design of civil engineering structures, the candidate is Co-chairman for the Working Group 2, and member of the Managerial Committee. The candidate was one of the five members which presented the first draft of the proposal to the COST Office. One important topic within this COST Action is the validation of the numerical models for the advanced fire analysis of structures. Part of the activity of the candidate on this topic is presented in section 2.3 “Validation of the advanced calculation models for fire design”. Part of this activity is sustained by the following publications from the list of publications selected by the candidate to be relevant for the professional achievements:

**02 - Zaharia R.,** Dubina D. (2012) “Case study: Fire design validation”. Integrated Fire Engineering and Response – Case Studies, Edited by Wald F., Burgess I., Rein G., Kwasniewski L., Vila Real P., Horova K., COST Office, CTU Publishing Production, Prague, Czech Republic, ISBN-978-80-01-05004-0

**03 - Zaharia R.,** Gernay Th. (2012). "Validation of the advanced calculation model SAFIR through DIN EN 1991-1-2 procedure". International Conference on Advances on Steel Concrete Composite and Hybrid Structures - ASCCS 2012, July 2-4, Singapore, Edited by J. Y. Richard Liew and Siew Chin Lee, ISBN-13: 978-981-07-2615-7, ISBN-10: 981-07-2615-5

The cooperation with Prof. Jean Marc Franssen from Liege University continued after the conclusion of the research grant obtained by the candidate in 2000-2001. One part is represented by the research work performed in collaboration with the team lead by Prof. Jean Marc Franssen at Liege University, highlighted by the topics presented throughout chapter 2 "Fire design of civil engineering structures". The second part is represented by the publication of the following two books:

- Franssen J. M., **Zaharia R.** (2006) "Design of steel structures subjected to fire – Background and design guide to Eurocode 3", Editions de l'Universite de Liege, Liege, Belgium, ISBN 978-287-456-0279;
- Franssen, J. M., Kodur, V., **Zaharia, R**. (2009), "Designing steel structures for fire safety", CRC Press, Taylor & Francis Group, London, UK, ISBN 978-0-415-54828-I.

The first book was a world premiere on the topic of fire design of steel structures, being not only a background of the actual Eurocode 1993-1-2, but also a design guide for the engineers, including an introduction into the general topic of fire design. The second book, represented an improved and extended edition of the first one, and included also the relevant fire design provisions from the USA codes. The other author of this book is Prof. Venkatesh Kodur, from Michigan State University, one of the top researchers and teachers in structural fire design in North America. The relevance of these books and their international impact is highlighted by the high number of citations in conferences and journal papers (from which four in journals with high relative impact factor, relevant for the habilitation thesis).

The candidate was responsible for the Politehnica University of Timisoara for three European projects dealing with the first main thematic direction:

- "Dissemination of structural fire engineering knowledge throughout Europe DIFISEK+", European Commission - RFCS Contract No RFCS-CT-2007-00030, 2007-2008;
- "Membrane action in fire design of composite slab with solid and cellular steel beams - MACS+" European Commission - RFCS Contract No RFS2-CT-2011-00025, 2011-2012;
- "Temperature assessment of a vertical steel member subjected to localised fire – LOCAFI" European Commission - RFCS Contract No RFSR-CT-2012-00023, 2012-2015.

All these projects, with European partners including universities, research centres and industry, were coordinated at European level by a strong industrial partner, ArcelorMittal Luxembourg. The responsible for these projects was Olivier Vassart, PhD, Head of Structural Long Products R&D at ArcelorMittal and Invited Professor of Steel and Composite Structures at Université Catholique de Louvain, a leader European researcher in the field of fire engineering. Following this cooperation, the candidate published some papers with Olivier Vassart, regarding the fire resistance of composite steel-concrete floors. The activity related to this topic is presented in section 2.4 "New recommendations for fire design".

The candidate is attested by the Ministry of Public Works for expertise and verification of the civil engineering buildings for fire safety (verification for projects since 2009, Expert since 2009).

In 2009 was founded the Romanian Association of Engineers for Fire Safety (Asociatia Romana a Inginerilor pentru Securitate la incendiu – ARISI). The candidate was among the founder members and is the president of the Timis Region branch of this association.

At the Politehnica University of Timisoara, lectures of fire design of civil engineering structures are lead by the candidate at three master programs.

The development of the experimental facilities in CMMC laboratory of the Politehnica University of Timisoara was, since 1994, one of the continuous activities of the candidate. The relevant post-doctorate activity in the second main thematic direction “Design assisted by testing” is presented in Chapter 3.

Regarding this activity it must be first mentioned that, since 1994, the laboratory of structures of CMMC department developed continuously its research capabilities, by acquisition of modern equipments, in a first period within the TEMPUS projects lead by the actual Head of Department, Prof. Dan Dubina, Corresponding member of the Romanian Academy. Further, by means of research grants or by means of dedicated projects for the development of the existing experimental research facilities, the acquisition of equipments continued, in order to build a modern laboratory, competitive within the international state-of-art of the experimental analysis in the field of civil engineering.

The candidate has a good experience in the domain of experimental techniques, by the research activity with a strong component of experimental analysis, developed within the Excellence Centre CEMSIG – Timisoara, lead by Prof. Dan Dubina (the single Excellency Centre in Romania in the domain of Civil Engineering), but also by his research stages done at the Liege University, Belgium in 1997, University of Trento, Italy, 1999 and the Joint Research Centre –JRC of the European Commission in Ispra, Italy between 2003 and 2005. The thesis defended by the candidate in 2000 also contained three experimental programs in order to assess the behaviour of cold-formed steel connections. It is also to be mentioned that the candidate leads the course “Design assisted by testing” at the Master program of studies held at the Faculty of Civil Engineering from the Politehnica University of Timisoara.

The candidate participated to three major grants in which the CMMC laboratory was substantially transformed by acquisition of new equipments (two as part of the managing team and one as director):

- " STOPRISC - Sisteme constructive si tehnologii avansate pentru structuri din oteluri cu performante ridicate destinate cladirilor amplasate in zone cu risc seismic - Structural systems and advanced technologies for structures made by high strength steel in strong seismic areas - STOPRISC", CEEX MATNANTECH No 29/2005, 3 partners, coordinated by the Politehnica University in Timisoara (Director Prof. Dan Dubina - the candidate was part of both managing and research teams), 2005-2008;
- “Centrul de studii avansate si cercetare in ingineria materialelor si structurilor - CESCIMS” within the CNCSIS grant “Platforme/ laboratoare de formare si cercetare interdisciplinara” No CNCSIS 42/2006, (Director Prof. Dan Dubina – the candidate was part of the managing team), 2006-2008;
- “Dezvoltare laborator pentru incercari la scara mare – INSTRUCT” - PN II Modul I Capacitati, 90 CP/ I/ 14.09.2007, (Director Assoc. Prof. Raul Zaharia), 2007-2010.

Within the last grant, for which the candidate was director, together with the acquisition of new equipments, the laboratory of structures was extended with a new building, which houses a unique facility in Romania and in the Eastern Europe (and one of the few in Europe): an experimental stand for static, dynamic and pseudo-dynamic testing of structural details and real scale buildings. In European Union, pseudo-dynamic testing is used at the ELSA Laboratory of European Commission in Ispra (Italy), University of Trento (Italy) and at the University of Porto (Portugal). A description of the new experimental facility within CMMC laboratory is presented in section 3.3.

The candidate applied for this grant following the experience gained in the field of pseudo-dynamic testing of real scale building structures within a research grant of two years, wined by competition at the European Commission’s Laboratory for Structural Assessment – ELSA, of the Joint Research Centre –JRC, located in Ispra, Italy. ELSA laboratory represents the biggest real scale pseudo-dynamic testing facility in Europe. During this period, the candidate was involved in both numerical and experimental research within a project initiated by the European Commission, aimed to study the seismic behaviour of reinforced concrete flat-slab buildings. The experimental programme included the pseudo-dynamic test of a real-scale building of thys type. This activity, presented in section 3.2 “Pseudo-dynamic test of a real scale flat-slab reinforced concrete struture" is sustained by the last publication in the list of publications selected by the candidate to be relevant for the professional achievements:

**10 - Zaharia R.,** Taucer F., Pinto A., Molina J., Vidal V., Coelho E., Candeias P. (2006) "Pseudodynamic earthquake test on a full-scale RC flat-slab building structure". European Commission, Joint Research centre, Institute for the Protection and Security of the Citizen, European Laboratory for Structural Assessment – ELSA, Publication No EUR 22192EN, European Communities, Printed in Italy

**Relevant publications**

The 10 publications selected by the candidate, considered to be relevant for the professional achievements obtained after he obtained his PhD and which sustain the activity presented in the Habilitation Thesis, are listed bellow, in the order of appearance in chapters 2 and 3:

**01 - Zaharia R.,** Pintea D., Dubina D. (2007) "Fire analysis and design of a composite steel-concrete structure", Steel and Composite Structures, Editors: Wang Y.C., Choi C.K., Taylor & Francis Group, London, U.K., ISBN 978-0-415-45141-3

**02 - Zaharia R.,** Dubina D. (2012) "Case study: Fire design validation". Integrated Fire Engineering and Response – Case Studies, Edited by Wald F., Burgess I., Rein G., Kwasniewski L., Vila Real P., Horova K., COST Office, CTU Publishing Production, Prague, Czech Republic, ISBN-978-80-01-05004-0

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# 2. FIRE DESIGN OF CIVIL ENGINEERING STRUCTURES

## 2.1 Introduction

According to the Directive of the European Commission issued on 21 December 1988, the construction works must be designed and built in such way that in the event of an outbreak of fire:

- the load bearing capacity of the construction can be assumed for a specific period of time;
- the generation and spread of fire and smoke within the works are limited;
- the spread of fire to neighbouring construction works is limited;
- occupants can leave the works or be rescued by other means;
- the safety of rescue teams is taken into consideration.

In order to reach these objectives, passive and active protection measures are necessary. The active measures are generated by all the devices activated in case of fire, as for example the automatic extinguishing systems. Concerning the passive measures, the following may be highlighted: the adequate number of escape routes function of the specific and particularities of the building, the elimination and the protection of the potential fire sources, the limitation of fire spread through an adequate partitioning of the building and, last but not least, the appropriate design of the resistant structure of the building, in order to maintain its strength and stability under elevated temperatures, for a specified period of time.

The basic principle in determining the fire resistance of a structural element is that the elevated temperatures produced by the fire reduce the materials strength and stiffness until possible collapse. As example, Fig. 2.1.1 shows the reduction factors for the stress-strain realtionship of carbon steel at elevated temperatures. When the temperatures on the cross-section of a structural element produce the reduction of the element resistance bellow the level of the effect of actions for fire design situation, it is considered that the element has lost its load-bearing function under fire action.

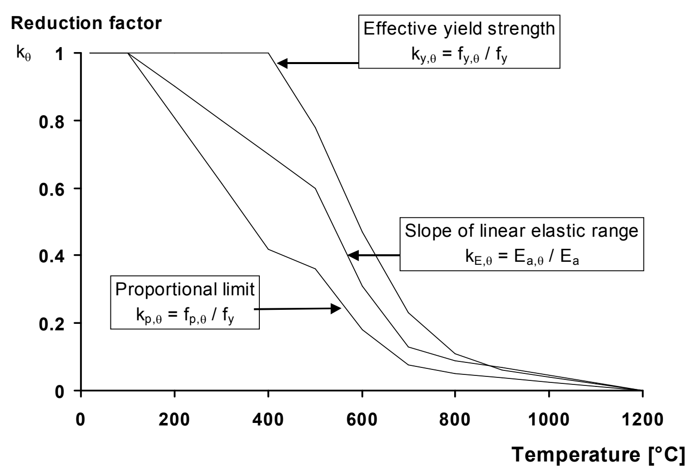



Fig. 2.1.1. Reduction factors for carbon steel at elevated temperatures

The fire resistance of steel, concrete or composite steel-concrete structures is calculated according to EN 1993-1-2 (EN 1993-1-2, 2005), EN 1992-1-2 (EN 1992-1-2, 2005) and EN 1994-1-2 (EN 1994-1-2, 2005) respectively. Three methods are available in order to evaluate the fire resistance: the tabulated data method, the simple calculation models and the advanced calculation models.

The tabulated data method is based on observations resulted from experimental study. It is the easiest to apply, but it is limited by the geometrical conditions imposed to the composite cross-section. The tabulated data method is available for concrete, composite steel-concrete and mansonry, but not for steel structures.

The simple calculation models compute the ultimate load of the element by means of formulas or design charts, established on the basis of experimental data, or by means of analytical equations derived from the corresponding equations at ambient temperature, corrected in order to consider the degradation of matherial characteristics function of temperature.

The advanced calculation models consider advanced numerical analysis of the elements or of the entire structure under fire, using specialized software for the mechanical analysis of structures under elevated temperatures. A word-wide used special purpose program for the analysis of structures under ambient and elevated temperature conditions is SAFIR (Franssen, 2005). The program, developed at the University of Liege by Prof. Jean Marc Franssen, accommodates various elements for different idealization, calculation procedures and various material models for incorporating stress- strain behaviour under elevated temperatures. The advanced analysis of a structure exposed to fire consists generally of two steps. The first step involves predicting the temperature distribution inside the structural members, referred to as "thermal analysis". The second part of the analysis, termed the "structural analysis" is carried out to determine the structural response due to static and thermal loading.

There are several fire models, accepted by the European Standard EN1991-1-2 (En 1991-1-2, 2005), which describes the thermal actions to be be considered for a structure under fire.

The nominal standard temperature-time ISO model does not take into account any physical parameter, and can be far away from reality. From the beginning, the nominal model supposes that the entire compartment is in the flashover phase and the temperature is increased continuously, without taking into account the cooling phase.

The parametric fire model considers the cooling phase and gives the temperature-time curve function of the fire load density and openings. This model is, however, limited to the surface and the height of the fire compartment considered, and supposes that the temperature is the same on the entire compartment, from the beginning of the fire.

A modern approach for the fire action is `Two Zone` and `One Zone` model. In this natural fire model, in the pre-flashover phase, the fire compartment is divided in a hot upper zone and a cold inferior one. For each zone, with uniform temperature, mass and energy equations are solved. Complex equations describe the air movement in the fire plume, the radiative exchanges between the zones and the gas movements on the openings and adjacent compartments. After the flashover, the temperature is considered uniform and is determined by solving the equations of mass and energy of the compartment, taking into account the walls and openings. In the frame of the ECSC research "Natural Fire Safety Concept" (CEC, 2001) it was considered necessary to develop a computer program for this model. This objective is now reached, a computer program called ***OZone*** is available in order to determine the temperature-time curve by means of the `Two Zone` and `One Zone` concept, and was built at the Liege University, Belgium, in collaboration with the Politehnica University of Timisoara(Cadorin et al 2003).

The fire is considered an accidental situation which requires, with some exceptions, only verifications against the ultimate limit state. The combinations of actions for accidental design situations are given in the European Standard for basis of structural design EN1990 (EN 1990, 2004) by the following equations:

$$G_k + P_k + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i} \quad \text{or} \quad G_k + P_k + \sum_{i \geq 1} \Psi_{2,i} Q_{k,i}$$

$G_k, P_k, Q_k$ are the characteristic values of the permanent, variable and prestressing action. According to the European Standard for actions on structures exposed to fire EN 1991-1-2 (EN

1991-1-2, 2005) the representative value of the variable action $Q_1$ may be considered as the quasi-permanent value $\Psi_{2,1}Q_{k,1}$, or as an alternative the frequent value $\Psi_{1,1}Q_{k,1}$.

The following verifications are possible for a steel, concrete or composite steel-concrete structure, substructure or element, in fire situation:

- Verification in the time domain, in which is has to be verified that the time of failure $t_f$ is higher than the required fire resistance time $t_{req}$:

  $t_f > t_{req}$

  The failure time is the time for which the resistance of the structure (or substructure, or element, as considered) under elevated temperatures reach the effect of actions for the fire design situation, considering the combinations of actions presented above.
- Verification in the load domain, in which it has to be verified that the resistance of the structure (or substructure, or element, as considered) for the fire design situation at time $t_{req}$, $R_{fi,d}$, is higher than the effect of actions for the fire design situation, $E_{fi,d}$.

  $R_{fi,d} > E_{fi,d}$ for $t = t_{req}$

  This is the standard verification proposed in both EN1993-1-2, EN1994-1-2 [5,6].
- As an alternative, the verification may be carried out in the temperature domain (only for steel structures), in which it has to be verified that, at the corresponding required fire resistance time $t_{req}$, the temperature of the structure $\theta_a$ (or substructure, or element, as considered) is lower than the temperature that leads to the failure, called critical temperature $\theta_{a,cr}$.

  $\theta_a < \theta_{a,cr}$ for $t = t_{req}$

  This verification may be performed for cases in which deformation criteria or stability phenomena have not to be taken into account.

## 2.2 Application of modern fire design in Romania

Since 1990, a growing demand for steel structures was emphasised in Romania, especially for industrial and commercial objectives, where erection speed is critical in the choice of the structural solution. The main problem of steel structures is their low fire resistance. Composite steel-concrete solutions have the advantage of increased resistance of the structural element, in the case of fire, as well as in normal conditions. The fire resistance of steel and composite structures may be determined using simplified methods, based on analytical formulas or tables, provided in the corresponding Eurocodes for fire design. For special situations or for complex structures, it may be necessary to perform an advanced analysis, using special purpose programs for the analysis of structures under ambient and elevated temperature conditions.

This section presents two examples of fire design (among others performed by the candidate) for composite steel-concrete buildings in Romania, using the design methods provided in the Eurocodes, including advanced calculation models. The fire design done by the candidate constitutes a premiere in Romania and, up to this moment, no other similar studies were performed in this country. The two examples deal with reinforced concrete filled CHS composite cross-sections and cross sections made of crossed hot rolled steel profiles, partially encased in reinforced concrete. The second example is sustained by the first publication in the list of publications selected by the candidate to be relevant for the professional achievements obtained after he obtained his PhD:

**01 - Zaharia R.**, Pintea D., Dubina D. (2007) "Fire analysis and design of a composite steel-concrete structure", Steel and Composite Structures, Editors: Wang Y.C., Choi C.K., Taylor & Francis Group, London, U.K., ISBN 978-0-415-45141-3

The first example presents the calculation of the fire resistance for the columns of a three-storey framed building structure for the LINDAB-Romania Company Headquarters, in Bucharest. The office structure has three levels (ground floor, 1st floor, and an attic floor), two spans of 6m each, and 7 bays of 5m, with a total area of 1308 m2. Taking into account the specific of LINDAB – Romania (systems of steel industrial buildings) the special demand was that the resistance structure must be visible steel, made by circular columns. Because for this type of building, according to Romanian fire regulations, the columns must have 2 hours of fire resistance, the solution of reinforced concrete filled CHS columns was chosen.

A global view of the structure is presented in Fig. 2.2.1. The office building structural scheme is composed of interior moment-resisting frames and exterior eccentrically braced frames. The lateral force resisting system was considered to be composed of both moment-resisting and eccentrically braced frames, through the diaphragm effect of the r.c. slabs and roof braces. The steelwork is fabricated from European hot-rolled profiles, and partially from built-up sections (steel quality S235). The beams have I cross section, the eccentric braces are RHS sections and the horizontal braces are round bars. Structural elements are joined on site by bolts only, avoiding site welding. Reinforced concrete slabs are fabricated in classical solution, with secondary steel beams and corrugated sheet lost formwork. The columns cross-section is presented in Fig. 2.2.2. The columns are composed of two sections, with a splice at +6.80 m. Concrete filling is accomplished after erection of each of the column sections, by pumping.

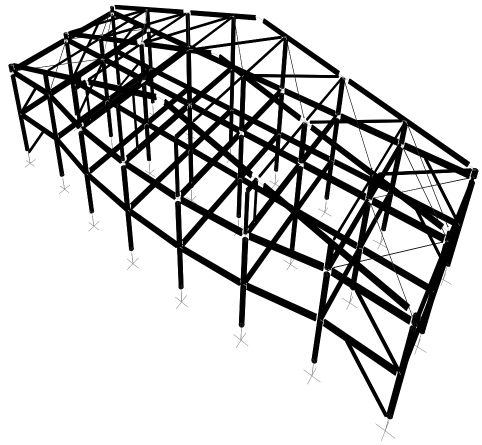


Fig. 2.2.1 - Office building structural scheme

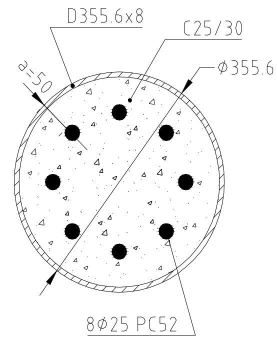



Fig. 2.2.2 - Column cross-section

The tabulated data method for reinforced concrete filled CHS columns, Table 4.7 of EN1994-1-2 (2005) was first considered. There is one condition that is not satisfied, the minimum percentage of reinforcement. Meanwhile, the other conditions are satisfied far away from limitations and the value of the load level represents less than 50% of the lowest one provided in the table.

The simple calculation model was applied using the diagrams provided in the CIDECT Design Charts for fire resistance of concrete filled hollow section columns (Twilt et al, 1994). Even if the fire resistance R120 of the column is demonstrated, taking into account the superior characteristics of the studied column in comparison with the data available in the Design Charts, no information about the fire resistance of the column under the imposed load is obtained. Therefore, an advanced analysis was considered.

Fig. 2.2.3 presents the numerical model of the concrete filled CHS column cross-section. Due to obvious reasons of symmetry, only a quarter of the cross-section was represented. Fig. 2.2.4 shows the temperature distribution after 2 hours of standardized ISO fire. It may be observed that for the CHS profile, the temperatures are superior of 1000$^{o}$C, so the steel profile exhausted its loading capacity. In the same time, the temperatures of the reinforcing bars are around 500$^{o}$C and there is an important core of concrete with quite low temperatures.

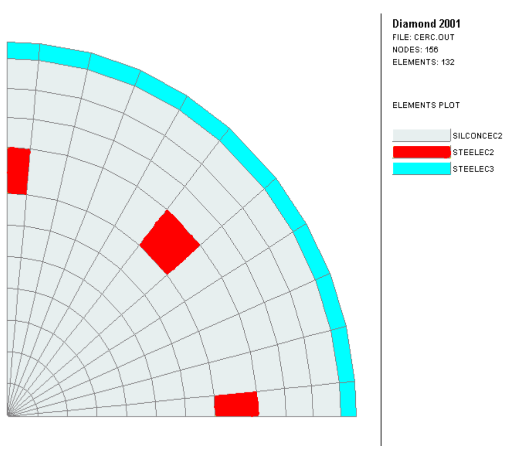



Fig. 2.2.3 - Numerical model of the composite cross-section

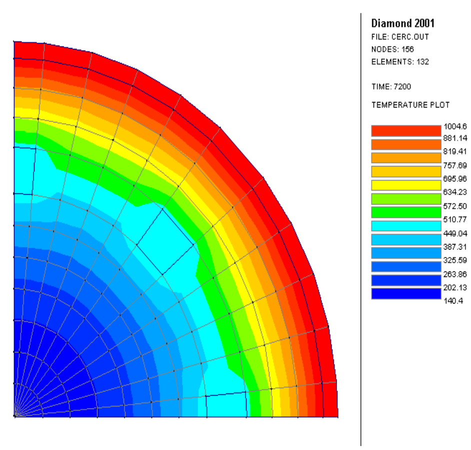



Fig. 2.2.4 - Temperature distribution on cross-section after 2h of standardized ISO fire

The column, loaded with the axial force and bending moment, corresponding to the fire situation, was modeled with 2D beam elements. Conservatively, the buckling length of the column was considered as the system length ($L_{ei}$=1.00L), i.e. the height of the relevant storey.

Equivalent imperfections according to EN1994-1-1 (EN 1994-1-1, 2005) were considered. As the characteristic time-displacement demonstrate (Fig. 2.2.5) after 2 hours of ISO fire the column is still able to resist to the imposed static loads, due to the bearing capacity reserve provided by the concrete core and the reinforcing bars. The collapse of the column is produced after around 3h of ISO standardized fire.

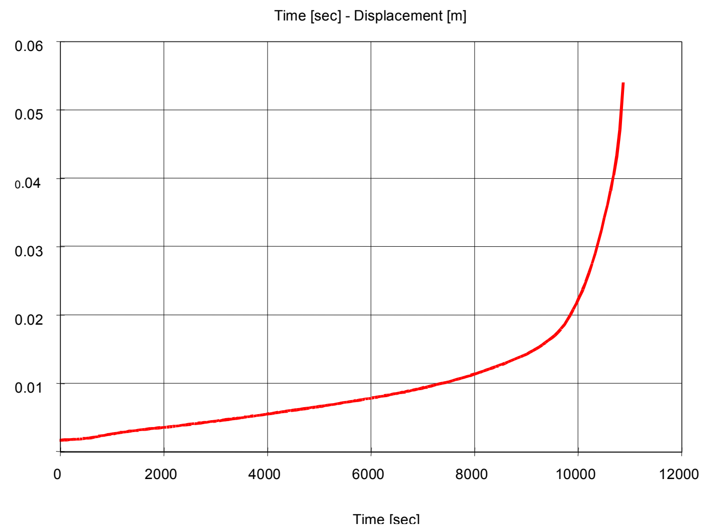



Fig. 2.2.5 - Time-displacement characteristic

The second example of application of modern fire design in Romania by the candidate is the verification of the fire resistance for the composite columns of „Bucharest Tower Centre“ structure, the tallest building in Bucharest at the moment of its construction (Fig. 2.2.6 - 7). The building has 3 basements, one ground floor, 21 floors, 3 technical floors at a total height of 106.3m. The building is 25.5m by 41.5m in plan and has a total construction gross area of approximately 24135m$^2$.

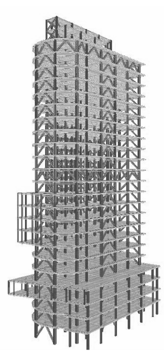


Fig. 2.2.6 - General view of the building (3D model)

Fig. 2.2.7 - Photo during construction

The building system uses steel braced and unbraced frames (dual structural configuration) (Fig. 2.2.8-10). The structure was designed by Britt Ltd, in cooperation with the Politehnica University of Timisoara, CEMSIG Research Centre. The project was awarded in 2007 with the 1$^{st}$ Prize of the Romanian Association of the Structural Design Engineers AICPS and ECCS Steel Design Award.

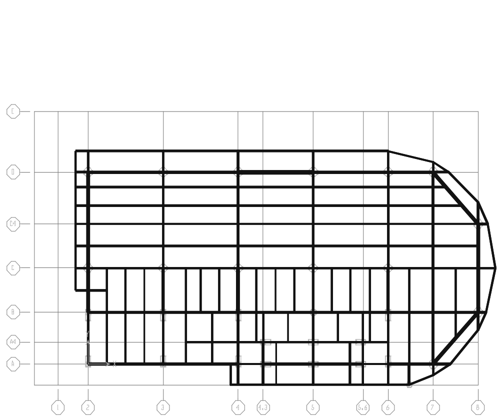


Fig. 2.2.8 - Structural system (current floor plan)

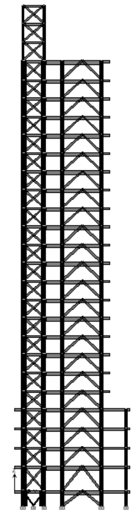


Fig. 2.2.9 - Side transversal frame

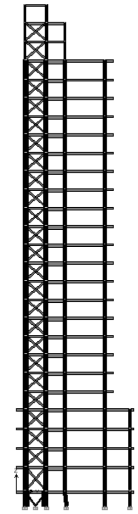


Fig. 2.2.10 - Current transversal frame

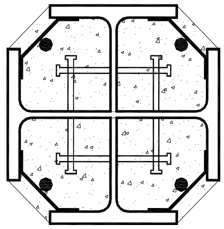
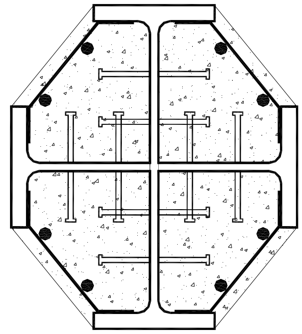
The columns are made by hot rolled steel European profiles, partially encased in reinforced concrete, in order to increase strength, rigidity and fire resistance. According to Romanian fire regulations, considering the specific and particularities of the building, the columns must have 150 minutes of fire resistance. The beams are made of IPE450 to IPE550 sections, except for the coupling beams and the beams connected to inverted V braces, which are HEB800 sections. Secondary beams are made of IPEO300 sections. Headed stud connectors were welded to the top flanges of the beams, to ensure the composite action of beams and concrete slab. In order to allow the development of plastic hinges in the moment resisting frames, no stud connectors were used in the vicinity of beams ends. Centric X braces are made of hot rolled sections, varying from HEB450 in the lower storeys to HEA340 in the upper ones. Similarly, inverted V braces vary from HEB450 to HEA340.

The seismic design was done according to Romanian seismic code P100-1/2006 (P100, 2006). The design ground acceleration is $a_g$ =0.24g, that corresponds to a reference return period of 100 years. In order to obtain a favourable plastic mechanism and to reduce the over strength requirements on the non dissipative members, S355 steel was generally used for frame members, excepting the braces designed as dissipative members, which use a S235 steel.

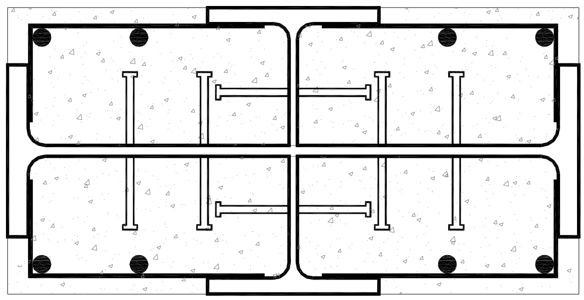
Four different cross section types were used for columns, as Fig. 2.2.11 shows (the values in parathesis represent the maximum load level, i.e. the maximum ratio to the ambient temperature capacity for each set): octagonal sections with identical steel profiles 2HEB500 (0.319), 2HEA800 (0.153), 2HEB800 (0.257), 2HE800x373 (0.303), octagonal sections with different steel profiles HEM800HEM700 (0.310), HEB800HEB700 (0.257), HEA800HEA700 (0.172), double-symmetric rectangular sections HEB1000HEB500 (0.293) and rectangular sections with one axis of symmetry HEB1000HEM500 (0.250), HEB1000 HEB500 (0.201).

The rebars have 25 mm diameter and the concrete is C30/37.

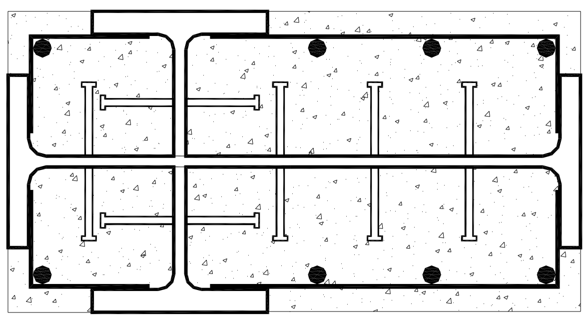
Only the 2HEB500, HEM800HEM700, HEB1000HEB500 and HEB1000HEM500 cross-sections will be further presented, as they have the lowest fire resistance under ISO fire from each type of cross section types, respectively.

a) b)



c)



d)

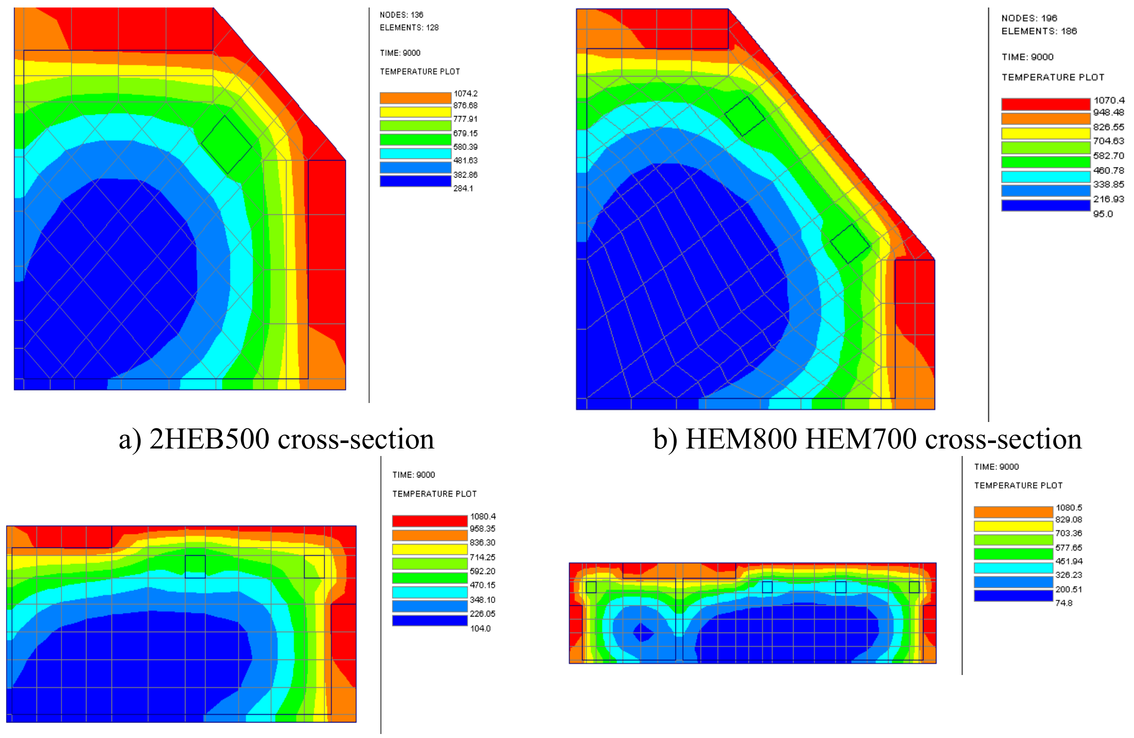
Fig. 2.2.11. - Composite cross-sections

Figure 2.2.12 shows the temperature distribution on the cross sections of the considered columns, after 150 minutes of ISO fire. Due to symmetry, only a quarter, or half of the cross-sections was modelled. The round reinforcing bars are represented by quadrilateral elements, with equivalent area. For all cross-sections, after 150 minutes of ISO fire, the steel profiles flanges exhausted practically their load capacity, having temperatures greater than 900$^{o}$C, while the profiles webs and the reinforcing bars have lower temperatures and there is an important core of concrete with quite low temperatures. Consequently, after 150 minutes of ISO fire, the sections have a reserve of load capacity.

The columns, considered as isolated elements, loaded with the axial force and the bending moments on both principal cross-section axes (efforts corresponding to the fire combination of actions), were modelled with 3D beam elements, taking into account that the internal moments have important values in both directions of the principal cross-section axes The buckling length of the columns was considered, conservatively, as the system length. Equivalent imperfections according to EN1994-1-1 (EN 1994-1-2, 2005) were imposed on both directions.

The horizontal displacement evolutions at the mid height of the columns of ground floor (which have the highest efforts in fire situation) are presented in Fig 2.2.13. As the characteristic time-displacement demonstrates, excepting for the columns with rectangular cross-section with one axis of symmetry (d), all other columns of the ground floor does not resist to the 150 minutes of ISO fire under the imposed static loads.

Similar analysis was made for the columns of the upper floors. The fire resistance grows with each floor, as the stress level in the columns decrease on the height of the building. Excepting the columns with octagonal cross section and identical profiles 2HEB500, all columns above the ground floor fulfil the 150 minutes requirement. For the 2HEB500 columns, the R150 fire resistance requirement is fulfilled only from the 11th floor forth.




a) 2HEB500 cross-section

b) HEM800 HEM700 cross-section

c) HEB1000 HEB500 cross-section

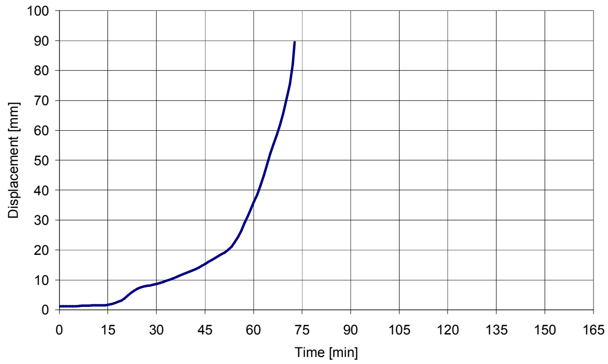
d) HEB1000 HEM500 cross-section

Fig. 2.2.12 - Temperature distribution at 150 minutes of ISO fire

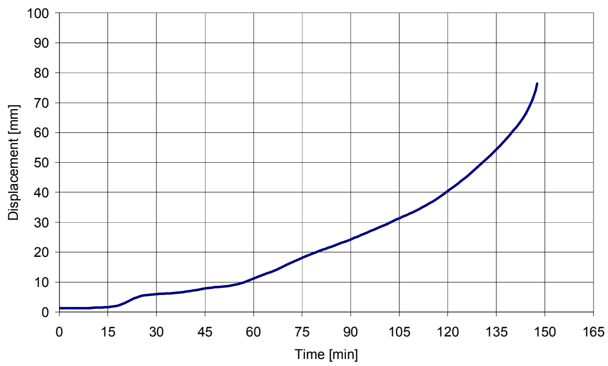
Table 2.2.1 gives the corresponding fire resistance times for all considered columns. Fire protection is needed for all the columns on the ground floor, except for the columns with rectangular cross-section with one axis of symmetry, while the 2HEB500 columns need protection up to the 11th floor.

Table 2.2.1. Fire resistance times [minutes].

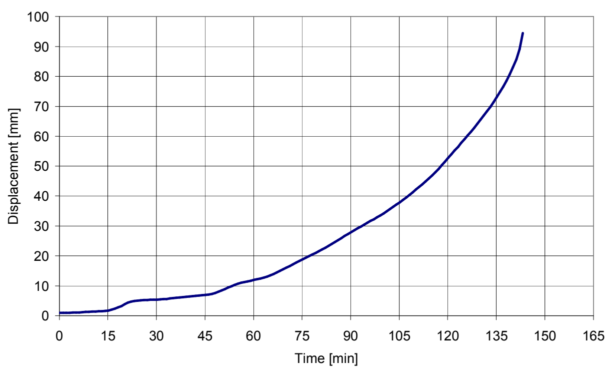
| Column | Ground floor | Floors 1-10 | Floor 11 - forth |
|---|---|---|---|
| 2HEB500 | 70 | 100-149 | >150 |
| HEM800HEM700 | 146 | >150 | >150 |
| HEB1000HEB500 | 143 | >150 | >150 |
| HEB1000HEM500 | >150 | >150 | >150 |



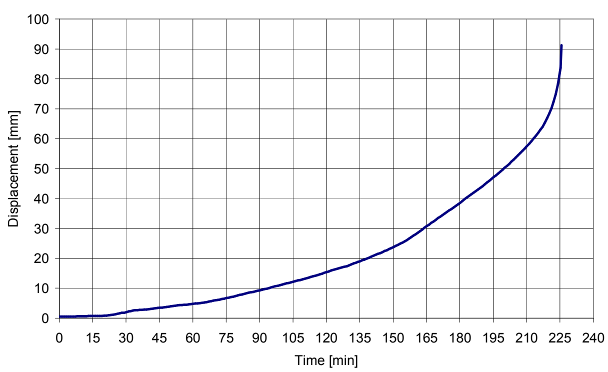

a) 2HEB500 column




b) HEM800 HEM700 column




c) HEB1000 HEB500 column




d) HEB1000 HEM500 column

Fig 2.2.13 - Displacement evolution under ISO fire.

The outstanding fire resistance obtained for the composite concrete filled circular hollow sections, as well as their increased resistance under usual conditions, confirms the efficiency of this structural solution. The 2 hours fire resistance required for columns in the office building was checked using all the three methods available in the European fire regulations EN1994-1-2. Beside the structural performance of this solution, it should be noted the simpler technology, as compared to the classical reinforced concrete columns. In the case of composite concrete filled circular hollow sections, there is no need for shuttering, leading to important savings in the cost of the structure.

The fire resistance of the composite steel-concrete columns of the „Bucharest Tower Centre“ structure was determined using advanced computation models. The fire analysis considering the ISO fire showed that the fire protection is needed for a limited number of columns of the building, in order to attain the $2^{1/2}$ hours of fire resistance.

## 2.3 Validation of advanced calculation models for fire design

Considering the limitations of the simple calculation models provided in the Eurocodes for fire design, as shown in the examples presented in 2.2, it is often necessary to use advanced calculation models in order to evaluate, at least, the temperature distribution inside the cross-section. Even the Eurocode for fire design of composite steel-concrete structures EN 1994-1-2 (2005) states, in clause 4.4.1, that "compared to tabulated data and simple calculation models, advanced calculation models give an improved approximation of the actual structural behaviour under fire calculations".

Clause 4.4.4 of EN1994-1-2 (2005) refers to the validation of the advanced calculation models, implemented in computer programs for the structural analysis under elevated temperatures.

The validation of advanced calculation models for the fire design is an important issue for computer code developers, designers and authorities. The Eurocodes for fire design state that the advanced calculation models must be validated through relevant test results. The problem which usually arises in such exercises is that, generally, the number of available experiments that may be found in literature is limited. The usable results of the tests that may be collected in the literature is another issue, due to the incomplete information about the input data, support conditions, or even of the results. It may also happen that the numerical simulation reveals significant disagreements in comparison with the experiment; in such situations, the reasons for this should be sought in both sides (Kwasniewski, 2009). In principle, the validation of an advanced calculation model is based on the comparison with experimental results. On the other hand, there is always the possibility of verification for a numerical model, through the comparison to analytical solutions or to other computer codes.

Finite element codes, suitable for structural-fire analyses, can be broadly divided into two categories (Gillie et al., 2008): commercial general-purpose codes, such as ABAQUS, ANSYS, etc. and research-based codes such as VULCAN or SAFIR, to cite just two of the most known ones. In literature, there are many references regarding the comparisons of the numerical analyses, which use these programs against fire tests on steel-concrete members or even structures.

The numerical investigation of the structural results from a compartment fire test, conducted in 2003 on the full-scale multi-storey composite building constructed at Cardington, U. K. (STC, 1999), was performed by Foster et al. (2007), using the computer program VULCAN, developed at the University of Sheffield, which offered good results in comparison with the experimental ones. More recently, a three-dimensional eight-node brick element, capable of representing the performance of composite structures subjected to 3D stress conditions at ambient and high temperatures, has been incorporated in the same research based code VULCAN. The model was validated against the results from a number of tests on composite structures subjected to 3D stress conditions, at both ambient and elevated temperatures (Yu et al., 2010).

Fire design using advanced calculation models may consider the fire action applied to the structural members in a separate thermal analysis, following a nominal or a natural fire model, as given in EN 1991-1-2 (2005). Another modern approach is to use an integrated fire dynamics and thermo-mechanical modelling framework, as done by Choi et al. (2010), who validated their numerical simulations using ABAQUS, through one of the six Cardington compartment tests on the full-scale steel-concrete composite structure, above mentioned.

In order to assess the behaviour of steel or composite connections in fire, parametric studies are necessary. A parametrical experimental study would be an expensive solution, and, therefore, the typical approach is to perform a parametric numerical analysis, after an appropriate calibration and validation of the numerical model against available experimental results. Jones and Wang (2008) made such an approach, by validating a numerical model in ABAQUS through an experimental program for welded fin-plate connections of hollow and concrete filled tubular columns, at ambient and elevated temperatures.

The computer program SAFIR (Franssen, 2005) is widely used, and is internationally recognised as a special purpose computer program for the structural analysis in fire conditions, for both research and design purposes. SAFIR satisfies the conditions of the fire parts of Eurocodes for an advanced calculation model, and implements the thermal and mechanical characteristics of the materials provided in these norms. In the last two decades, SAFIR proved its capability to reproduce the fire behaviour of steel, concrete and steel-concrete composite structures against experimental results. Just a few references may be mentioned here: a 2D analysis to model an early Cardington fire test performed in 1987 on a full-scale steel frame (Franssen et al., 1995), and some more recent 3D analyses, to model the behaviour of cellular composite beams tested as isolated elements (Nadjai et al., 2007) or the membrane effect of slabs in fire, using a full scale test on a composite steel-concrete slab under natural fire (Vassart et al., 2011).

Clause 4.4.4 of EN 1994-1-2 states that the validity of any computer program that uses an advanced calculation model for the analysis of structures under elevated temperatures, shall be verified on basis of relevant test results. The keyword, in this clause, is “relevant”. When using a computer program for calculating the fire resistance of a structural member or of a structure, the designer should be aware that the program is able to reproduce the behaviour of a specific category of applications (for instance composite steel-concrete structures). Once the computer program was validated for a specific problem it is obvious that it is not necessary to perform a fire test for the same type of structural element, in a given design situation. The validation is made for a calculation model (computer program), so that it can be applied for a range of applications. In fact, this is the reason to use a computer program for fire design; otherwise, a fire test for each member of the analysed structure would be made, with the corresponding length, cross-section dimensions and loading. This would be not only uneconomical, but also practically impossible.

Clause 4.4.4 of EN 1994-1-2 also mentions to perform a sensitivity analysis, in order to verify that the calculation model complies with the sound engineering principles. There are some examples for the critical parameters of this sensitivity analysis, such as buckling length, size of elements and load level.

Even if a special purpose computer program that implements an advanced calculation model for fire structural analysis has already been validated for a category of applications, it is, of course, possible to perform a supplementary validation of the program and a corresponding sensitivity analysis, if experimental tests on elements that are similar to a particular design situation exist.

The Romanian authorities (General Inspectorate for Emergency Situations) requested such a particular validation, in order to demonstrate the ability of the advanced calculation model SAFIR to reproduce the fire behaviour of partially concrete encased columns with crossed I-sections. As shown in the previous section 2.2, the candidate used SAFIR to determine the fire resistance of the columns of this particular type of columns of “Bucharest Tower Center” building situated in Bucharest, Romania. Because the validation of advanced numerical models used for fire safety engineering has always been a concern of the authorities and because a fire test of a column with cross-section similar to the columns of the “Bucharest Tower Center” was available, the verification of the results given by SAFIR against this test was required.

In the following section 2.3.1 are presented the results of the numerical fire resistance assessment made by the candidate for the composite steel-concrete columns of “Bucharest Tower Center” building and the validation of the advanced calculation model SAFIR for this particular design situation. An extended journal paper about this topic is in review (Zaharia & Dubina, 2013). Validation of the advanced calculation models for fire design is one important topic of the COST Action TU0904: Integrated Fire Engineering and Response – IFER, 2010-2014, in which the candidate is Co-chairman for Working Group 2 and member of the Managerial Committee. The activity presented in section 2.3.1 was presented within the COST Action and is sustained by the second publication in the list of publications selected by the candidate to be relevant for the professional achievements obtained after he obtained his PhD:

**02 - Zaharia R.,** Dubina D. (2012) “Case study: Fire design validation”. Integrated Fire Engineering and Response – Case Studies, Edited by Wald F., Burgess I., Rein G., Kwasniewski L., Vila Real P., Horova K., COST Office, CTU Publishing Production, Prague, Czech Republic, ISBN-978-80-01-05004-0

The German National Annex of EN 1991-1-2 (2010) offers an alternative to the validation through experimental results. This document presents a series of validation examples assembled in Annex CC, concerning: heat transfer for different sections and material properties, temperature induced expansion for different material laws, internal forces and stresses induced by thermal action and ultimate bearing capacity of elements. These examples include steel, concrete and composite steel-concrete sections. Each example offers a set of results and the acceptable tolerances for the results. In the following section 2.3.2 is presented the simulation of the examples of DIN performed with the computer software SAFIR and ABAQUS (for one example). The aim of this study, was double: to validate the computer program SAFIR and to have a critical view on the examples of DIN, which could be profitable for future validation exercises of other programs. The activity presented in section 2.3.2 is sustained by the third publication in the list of publications selected by the candidate to be relevant for the professional achievements obtained after he obtained his PhD:

**03 - Zaharia R.,** Gernay Th. (2012). "Validation of the advanced calculation model SAFIR through DIN EN 1991-1-2 procedure". International Conference on Advances on Steel Concrete Composite and Hybrid Structures - ASCCS 2012, July 2-4, Singapore, Edited by J. Y. Richard Liew and Siew Chin Lee, ISBN-13: 978-981-07-2615-7, ISBN-10: 981-07-2615-5

**2.3.1 Validation of the advanced model used for the fire resistance assessment of the composite columns of "Bucharest Tower Center" building**

Partially concrete encased sections with crossed steel I-sections, used for the columns of "Bucharest Tower Center" building were promoted since 1987 by ARBED Luxembourg, in order to obtain high fire resistance times without supplementary fire protection (REFAO, 1987). Therefore, the validation of the advanced calculation model SAFIR requested by the authorities considered a composite column with cross section similar to the cross sections of the "Bucharest Tower Center" columns, from the experimental report REFAO (1987), which presents some fire tests conducted on composite beams and columns made by steel profiles embedded in concrete. The experimental research summarised in this report consisted of fifteen full-scale ISO fire tests of composite beams and columns, performed by Arbed Luxembourg since 1982 up to 1985 and sponsored by C.E.C., the Commission of the European Community.

Fig. 2.3.1.1 shows the experimental set-up of the composite column considered for the validation.

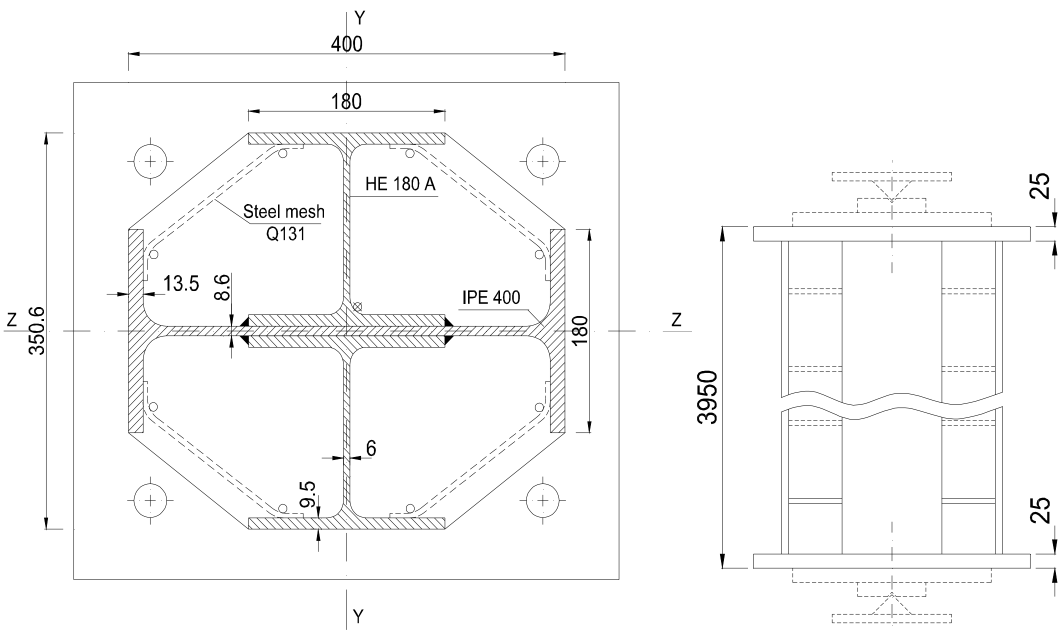



Fig. 2.3.1.1 Composite column (REFAO, 1987)

The column was composed of three hot-rolled H profiles, welded together and concreted between the flanges. Two HEA180 profiles were welded on the web of an IPE400 profile, as Fig. 2.3.1.1 shows. The composite section did not contain any supplementary longitudinal reinforcing bars. A steel mesh was provided at the outer concrete parts between the steel flanges, as shown in Figure 2.3.1, in a similar manner as the welded stirrups on the steel profile flanges provided in case of the columns of "Bucharest Tower Center".

The characteristics of concrete were determined on cubic samples of 200 mm, tested before the fire test. The obtained values for resistance were used for the computation of the resistance on cylindrical samples of 150mm diameter and 300mm height, resulting a compressive strength of 51.90N/mm$^2$. In order to prevent the spalling of concrete during the fire test, all composite test elements were concreted four to five months in advance. The age of the concrete at the test date was of 168 days. The water content of concrete for the tested specimen was evaluated to 80 l/m3, but no moisture was considered in the numerical analysis.

The yield strength of steel of 299.90N/mm2 was determined on samples taken from the profiles used in the fire test.

The tested column had no measurable initial imperfections on its length and was loaded with a compressive force of 1600 kN, with a zero eccentricity. However, the numerical simulation considered a small initial sinusoidal imperfection of the element, with a maximum value of 1mm.

The octagonal composite column behaved quite well in the fire test, as the resistance time attained 172 minutes, though four visible unprotected steel flanges were exposed directly to the fire action. Few spalling occurred, so that the concrete mass remained intact up to the failure of the column. This was due to the presence of the steel mesh placed between the steel profiles flanges (REFAO, 1987).

Figure 2.3.1.2 shows the comparison between the measured temperatures within the composite cross section of the column (REFAO, 1987) and the temperatures obtained from the numerical analysis, at the failure time of the tested column of 172 minutes. It has to be mentioned that the resultant emissivities considered in the numerical model were the ones given in the test report for the surfaces of steel elements (0.3 and 0.5) and for the concrete (0.45), as shown in Fig. 2.3.1.3. These values are lower than the surface emissivities given in EN1994-1-2 for steel and concrete (0.7), which were used for the fire resistance assessment of "Bucharest Tower Center" columns.

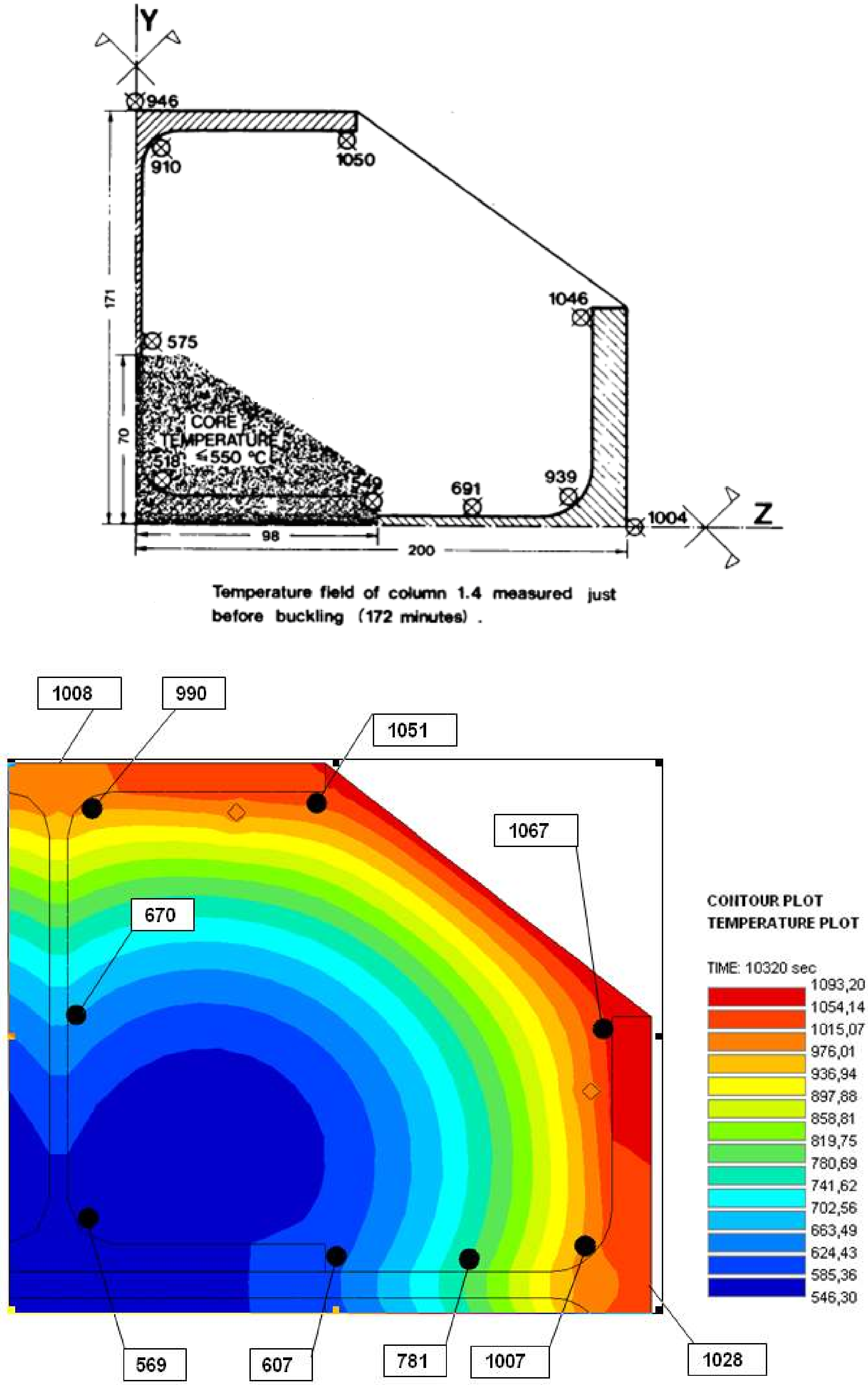



Fig. 2.3.1.2. Temperature distribution on cross-section:
experimental (REFAO, 1987) and numerical

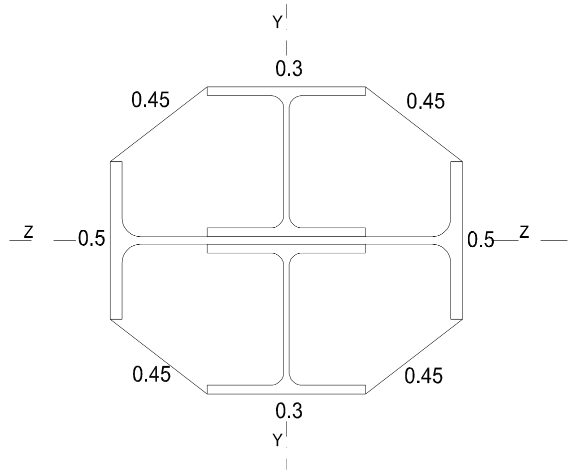



Fig. 2.3.1.3 Resultant emissivity for profile flanges and for concrete (REFAO, 1987)

The numerical thermal analysis offers good results, with values which are close to the calculated temperatures, in the points from the cross section where the thermocouples were placed.

Considering a buckling length equal to the length of the column (both ends are pinned, as supposed to be in the test) the failure time given by the numerical analysis is of 132 minutes. This is a very conservative result, in comparison with the failure time of the experimental specimen of 172 minutes.

If a buckling length of 50% from column length is considered (both ends are fixed), the failure time obtained by numerical analysis is of 188 minutes, higher than the failure time obtained in the test. If a buckling length of 70% from column length is considered (intermediate situation between fixed and pinned supports), the failure time given by the numerical analysis is of 164 minutes. This suggests that for the tested column it was not possible to realize perfect pinned ends and there was a degree of rotational restraint at the supports.

The values of fire resistance times presented above were determined for a small initial global imperfection of 1 mm introduced in the numerical analysis, even if the test report states that no initial imperfection was emphasised after measurements.

Compared to the test, SAFIR gives good results, in the safe side, for both thermal and mechanical analysis, if all the parameters measured experimentally are considered. If the Eurocode specifications for the initial imperfections and resultant emissivities (as used for the calculation of the fire resistance of the columns of “Bucharest Tower Center” building), are introduced in the numerical analysis, more conservative results are obtained. As expected, the sensitivity analysis showed that the computer program SAFIR offers appropriate results, in accordance with the engineering principles. Following this procedure of validation, the advanced calculation model SAFIR was accepted as a reliable tool to evaluate the fire resistance of the composite columns of “Bucharest Tower Center” building.

### 2.3.2 Validation of the advanced calculation models through DIN EN 1991-1-2 Procedure

Annex CC of DIN EN1991-1-2 contains eleven validation examples for advanced calculation models to be used in fire design.

Example 1 analyses the heat transfer in the cooling process of a square section with given material properties (see figure and results in Table 2.3.2.1). The initial temperature of the section is 1000 °C. Three sides of the cross-section represent an adiabatic boundary, while the other side exchanges heat with a medium of which temperature is and remains equal to 0 °C, by linear convection and radiation. The limit deviations for the calculated temperatures in a selected point were satisfied for a 64 quad elements mesh, as given in the cross-section model figure of Table 2.3.2.1 (which presents the temperature distribution at 1800s).

Example 2 analyses the heat transfer in the heating process of a square section with given material properties (see figure and results in Table 2.3.2.2). Similar to the above example, the initial temperature of the section is 0°C and the section is plunged into a medium having 1000°C. The limit deviations for the calculated temperatures in the center point of the section were satisfied for a 576 quad elements mesh.

Example 3 analyses the heat transfer in a steel hollow section, filled with a material for which the thermal properties are known, with an initial temperature of 0°C, plunged into a medium having 1000°C (see figure and results in Table 2.3.2.3). The limit deviations for the calculated temperatures in the center point of the section were satisfied for a 324 quad elements mesh.

Example 4 analyses the thermal induced expansion Δl of a steel element with given dimensions, at different values of homogenous temperature in the cross-section (see figure and results in Table 2.3.2.4).

Example 5 analyses the elongation of a cantilever with a height of 10 cm and a square cross-section made of steel or concrete, for different uniform temperature distributions at some given stress-strength ratios (see figure and results in Table 2.3.2.5). The mechanical properties of concrete and steel under elevated temperatures are those given in the corresponding Eurocodes (EN 1992-1-2, 2005; EN 1993-1-2, 2005). For the steel cantilever, all results fit well within the prescribed deviation limits. The results presented in Table 2.3.2.5 are only for the concrete cantilever and some deviations from the limit values are emphasized. It must be noted that the results are of very small values, the problem here being the number of digits that the software can provide. The higher deviations from the limit are obtained for the values of the elongations that are very close to 0 (E-02 milimeters, which corresponds to the number of digits that SAFIR offers for the values of displacements in millimeters) for which the opposite thermal and mechanical deformations are almost the same. For the cases in which the limit deviation is not satisfied, the difference between the calculated and reference values of the displacements are given in Table 2.3.2.5. It may be observed that the differences in terms of absolute values are very small, the maximum value being 5E-03 millimeters. The same problem related to the number of digits that the software can provide for the values of the displacements in millimeters may be observed in Example 4, but in this case all calculated values were within the deviation limits, because the values of the elongations were higher.

The aim of Example 6 is to determine the load bearing capacity at different temperatures of the structural steel and concrete elements of Example 5. For steel, a perfect fit of the reference and calculated values was obtained. For concrete, only a slight difference for one case was obtained, which fit well in the limits, see Table 2.3.2.6.

In Example 7, for a fixed steel beam subjected to thermal loading and having a square cross-section of 100x100 mm, the calculation of the internal forces N and M as well as of the stress σ are demanded (see figure and results in Table 2.3.2.7). Two cases of temperature distribution are considered: uniform temperature distribution of 120 °C in the section and linear variation of temperature on the height of the cross-section, of 20/220 °C in the top/ bottom fiber, respectively. An analysis of the results presented in Table 2.3.2.7 reveals an amazing situation: only the axial force calculated by SAFIR for one of the load cases does not fit within the deviation limits, while all other values offer deviations which are extremely low compared to the limits (including for the stress, which considers the effect of the axial force).

The results in Table 2.3.2.7 are based on the hypothesis that the temperatures are imposed only on the top and bottom fiber of the cross-section. A new gradient of temperature was imposed in SAFIR, by dividing the height of the cross-section in 200 elements, and by imposing in each element the corresponding temperature in order to obtain a real linear distribution (a difference of 1$^{o}$C from one element to another, on the height of the section). Considering this new approach, all results for the gradient of temperature of 20/220$^{o}$C fit within the deviation limits, as the values presented in Table 2.3.2.8 demonstrate.

Example 8 analyses a weakly reinforced simply supported concrete beam loaded with uniform distributed load, subjected to fire on three sides. The cross-section of the beam is given in Fig. 2.3.2.1. The purpose is to determine the necessary area for the two rebars S500, in order to reach the fire resistance class of 90 minutes. Fig. 2.3.2.2 shows the mesh adopted in the numerical model in the neighborhood of the rebars. A temperature of 544 $^{o}$C is obtained in the reinforcement, which presents a deviation of -3.2% from the reference value of 562 $^{o}$C. It must be emphasized that the limitation of the deviation refers only to the area of the reinforcement and, as shown in Table 2.3.2.9, this criteria is fulfilled.

Example 9 is similar to Example 8, with the difference that in this case, the same concrete beam is strongly reinforced (see Fig. 2.3.2.3). Fig. 2.3.2.4 shows the mesh in the area of the rebars and the temperature values in these rebars. For this case also, the criteria refers only to the area of the reinforcement, and this criteria is fulfilled, as shown in Table 2.3.2.10. The calculated temperatures in the rebars present a deviation that ranges between -0.54% and -4.76%, compared to the reference values.

Example 10 analyses a reinforced concrete column loaded with a vertical load having an eccentricity and a uniformly distributed horizontal load (see figure and results in Table 2.3.2.11). The reference results, for which a limit deviation is provided, are: the fire resistance time, the top horizontal displacement and the bending moment at the base of the column after a fire time t = 90 minutes. The criteria are fulfilled for all requested results. The maximum deviation of the calculated temperatures in the rebars is of -3.33% from the reference ones.

The last example analyses a composite column with partially encased steel section, subjected to fire on four sides (see figure of the cross-section and results in Table 2.3.2.12). The column is centrically loaded and a parabolic imperfection with peak value of l/1000 is considered. The reference results, for which a limit deviation is provided, are: the fire resistance and the horizontal displacement at the mid-span of the column at t = 30 and 60 minutes. The deviation of the calculated temperatures from the reference ones, given at 90 minutes, is of -4.11% for the rebars and of +1.79% for the center of the steel profile.

The criterion is not fulfilled for the displacement corresponding to 60 minutes of ISO fire (see results in Table 2.3.2.12). It must be emphasized that this deviation is not consistent with the other results. The displacement at 60 minutes is lower than the reference value (-8.18% compared to the deviation limit of +/-5%), while for 30 minutes the displacement is slightly higher (+0.82%, within the accepted limits of +/-5%). It would be expected that lower displacements lead to a higher failure time, which is not the case here, the calculated failure time is lower than the reference one.

The numerical analysis of Example 11 was considered also with ABAQUS program. Figure 2.3.2.5 shows the numerical 3D model of the composite column. Due to the symmetry, only half of the cross-section was considered.

As shown in Table 2.3.2.13, using ABAQUS the criteria are not fulfilled for two results: the failure time and the displacement corresponding to 60 minutes of ISO fire. It may be observed that, as in case of SAFIR, the failure time is lower than the reference time, but the displacement corresponding to 60 minutes given by ABAQUS is much higher than the reference value. However, the results given by ABAQUS are consistent: the model offers a lower failure time, together with higher displacements, in comparison with the reference values.

It is interesting to observe that, even if the two programs offer high differences for the displacements at 60 minutes, the temperatures at the level of the reinforcements and in the centre of the steel profile are close to the reference results for both programs. As shown in Table 2.3.2.14, both SAFIR and ABAQUS offer temperatures that are higher than the reference ones for the steel profile and lower than the reference ones for the steel reinforcement. The maximum deviations obtained for the temperatures in the reinforcements and are of 3.2% for SAFIR and 4.3% for ABAQUS. It is to be noted that, even if DIN EN 1991-1-2 offers the reference temperatures at 90 minutes, these values are not imposed with a corresponding deviation for validation purposes. Figure 2.3.2.6 shows the mesh of the cross-sections and the temperature distribution of both SAFIR and ABAQUS numerical models, at 90 minutes of standard ISO fire.
It has to be mentioned that, even if there are differences at the level of the displacement obtained for 60 minutes, both SAFIR and ABAQUS offer conservative results for the fire resistance time (lower values than the reference one), which is, in fact, the most important result of such a numerical simulation. Indeed, in the Eurocodes for fire design, with some rare exceptions, only the Ultimate Limit States are verified, i.e. the fire resistance time. This time corresponds to the moment in which the design resistance of the element subjected to fire equals the design effect of actions in the fire situation, and thus the collapse of the element is produced.

**Table 2.3.2.1: Results for Example 1**

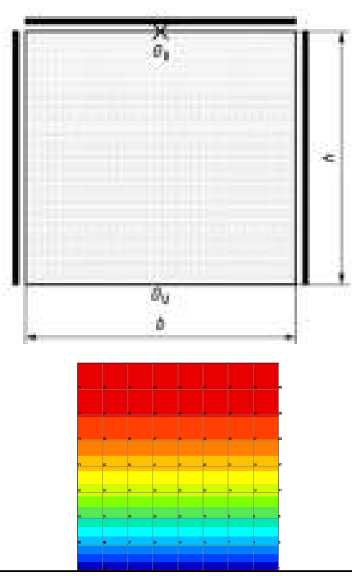
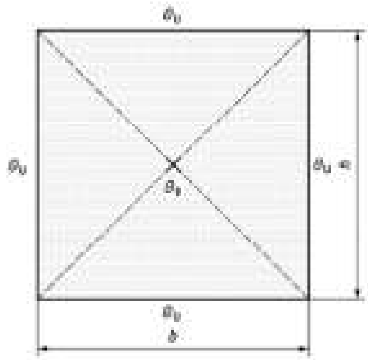
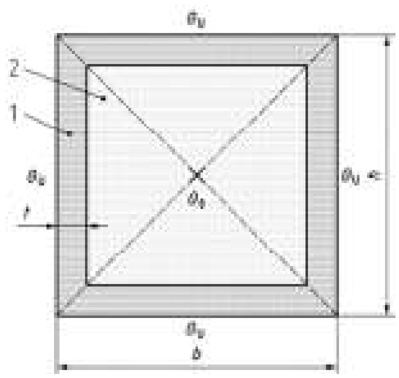
|  | **Time** [s] | **Calculated Temperature** [°C] | **Deviation** [%] / | [K] | **Limit** |
|---|---|---|---|---|---|
| | 0 | 1000 | 0 | 0 | ± 1 % and ± 5.0 K |
| | 60 | 998.6 | -0.07 | -0.7 | |
| | 300 | 892.5 | 0.07 | 0.6 | |
| | 600 | 719.1 | 0.20 | 1.4 | |
| | 900 | 576.4 | 0.26 | 1.5 | |
| | 1200 | 461.9 | 0.33 | 1.5 | |
| | 1500 | 370.1 | 0.38 | 1.4 | |
| | 1800 | 296.3 | 0.34 | 1.0 | |

**Table 2.3.2.2: Results for Example 2**



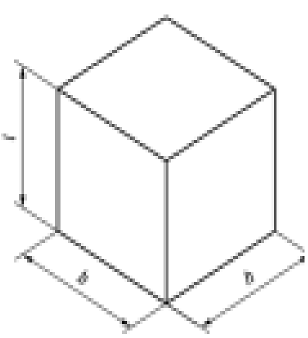
| Time [min] | Calculated Temperature [°C] | Deviation [%] / [K] | | Limit |
|---|---|---|---|---|
| 30 | 32.5 | -11.9 | -4.4 | for t ≤ 60 min |
| 60 | 132.5 | -3.6 | -4.9 | |
| 90 | 241.6 | -1.2 | -3 | ± 5.0 K |
| 120 | 362.6 | 0.4 | 1.5 | for t > 60 min |
| 150 | 469.6 | 0.7 | 3.4 | |
| 180 | 559.7 | 0.9 | 4.9 | ± 3 % |

**Table 2.3.2.3: Results for Example 3**



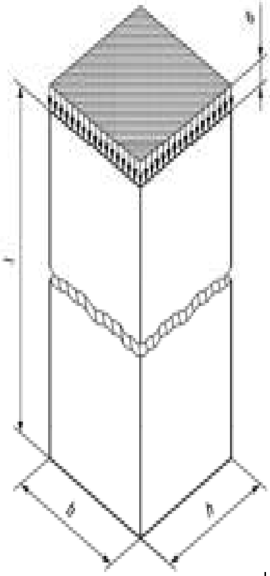
| Time [min] | Calculated Temperature [°C] | Deviation [%] / [K] | | Limit |
|---|---|---|---|---|
| 30 | 337 | -1.0 | -3.5 | ± 1 % |
| 60 | 721.7 | 0.6 | 4.6 | |
| 90 | 885.3 | 0.4 | 3.7 | and |
| 120 | 952.7 | 0.2 | 2.1 | |
| 150 | 980.5 | 0.1 | 1.2 | ± 5.0 K |
| 180 | 992 | 0.0 | 0.3 | |

**Table 2.3.2.4: Results for Example 4**



| Θ [°C] | Reference $\Delta l$ [mm] | Calculated $\Delta l'$ [mm] | Deviation [%] / [mm] | | Limit |
|---|---|---|---|---|---|
| 100 | 0.09984 | 0.10 | 0.16 | 0.00016 | for Θ ≤ 300 °C |
| 300 | 0.37184 | 0.37 | −0.49 | −0.00184 | |
| 500 | 0.67584 | 0.68 | 0.61 | 0.00416 | ± 0.05 mm |
| 600 | 0.83984 | 0.84 | 0.02 | 0.00016 | |
| 700 | 1.01184 | 1.01 | −0.18 | −0.00184 | for Θ > 300 °C |
| 900 | 1.18000 | 1.18 | 0.00 | 0.00000 | ± 1 % |

**Table 2.3.2.5: Results for Example 5**



| Θ °C | Stress/ Strength | Reference $\Delta l$ [mm] | Calculated $\Delta l'$ [mm] | Deviation [%]/ $\Delta l'$- $\Delta l$ | Limit [%] |
|---|---|---|---|---|---|
| 20 | 0.2 | −0.0334 | −0.03 | **−10.18/ -0.0034** | ± 3 |
| | 0.6 | −0.104 | −0.10 | **−3.85/ -0.004** | |
| | 0.9 | −0.176 | −0.18 | +2.27 | |
| 200 | 0.2 | +0.107 | −0.11 | +2.80 | |
| | 0.6 | −0.0474 | −0.05 | **+5.48/ +0.0026** | |
| | 0.9 | −0.2075 | −0.21 | +1.20 | |
| 400 | 0.2 | +0.356 | +0.36 | +1.12 | |
| | 0.6 | +0.075 | +0.07 | **−6.66/ -0.005** | |
| | 0.9 | −0.216 | −0.22 | +1.85 | |
| 600 | 0.2 | +0.685 | +0.69 | +0.73 | |
| | 0.6 | −0.0167 | −0.02 | **+19.76/ +0.0033** | |
| | 0.9 | −0.744 | −0.74 | −0.53 | |
| 800 | 0.2 | +1.066 | +1.07 | +0.37 | |
| | 0.6 | +0.365 | +0.36 | −1.35 | |
| | 0.9 | −0.363 | −0.36 | −0.82 | |

**Table 2.3.2.6: Results for Example 6 (concrete)**

| Θ [°C] | Calculated $N_{R,fi,k}$’ [KN] | Deviation [%] / [KN] | Limit |
|---|---|---|---|
| 20 | -20.0 | 0 | ± 3 % and ± 0.5 KN |
| 200 | -19.0 | 0 | |
| 400 | -15.0 | 0 | |
| 600 | -8.99 | -0.11/0.01 | |
| 800 | -3.0 | 0 | |

**Table 2.3.2.7: Results for Example 7**

| Temperature load case | | Calculated value | Deviation [%] | Limit [%] |
|---|---|---|---|---|
| 120/ 120 | N [KN] | −2587 | +0.08 | N: ± 1 M: ± 1 σ: ± 5 |
| | M[KNm] | 0 | 0 | |
| | σ [N/mm²] | −258.73 | +0.09 | |
| 20/ 220 | N [KN] | −2457 | **−2.15** | |
| | | | | |
| | M [KNm] | −40.35 | +0.12 | |
| | σ [N/mm²] | −478.60 | −0.08 | |

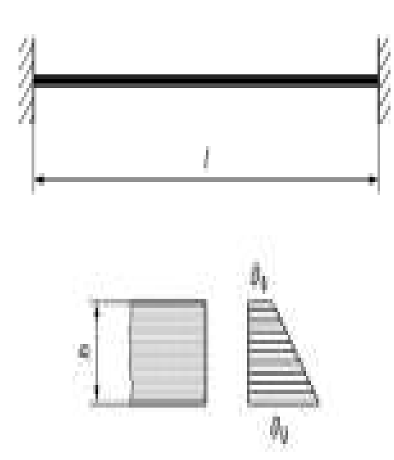


**Table 2.3.2.8 Example 7 – real linear distribution of temperature**

| Temperature load case | | Calculated value | Deviation [%] | Limit [%] |
|---|---|---|---|---|
| 20/ 220 | N [KN] | −2510 | -0.039 | N: ± 1 M: ± 1 σ: ± 5 |
| | M [KNm] | −40.50 | +0.496 | |
| | σ [N/mm²] | −479.00 | 0.00 | |

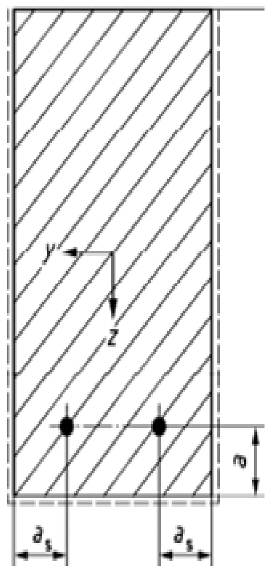



Fig. 2.3.2.1 Example 8

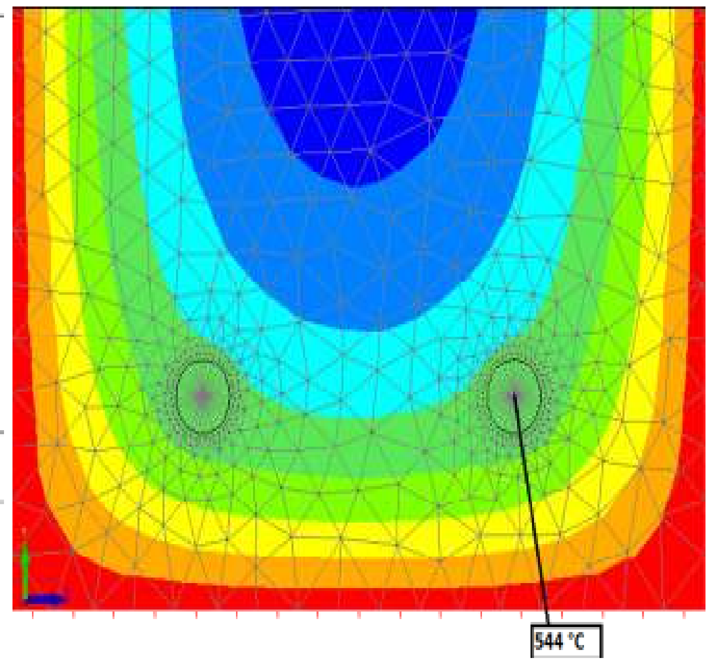



Fig. 2.3.2.2 Temperature distribution at 90 min

**Table 2.3.2.9: Results for Example 8**

| **Fire resistance** | **Reference value** ***As*** **[cm²]** | **Calculated value** ***As'*** **[cm²]** | **Deviation [%]** | **Limit [%]** |
|---|---|---|---|---|
| R90 | 3.56 | 3.77 | +5.90 | ± 10 |

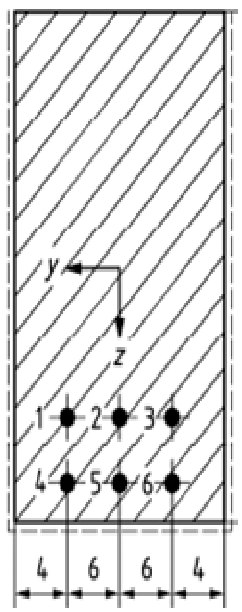



Fig. 2.3.2.3 Example 9

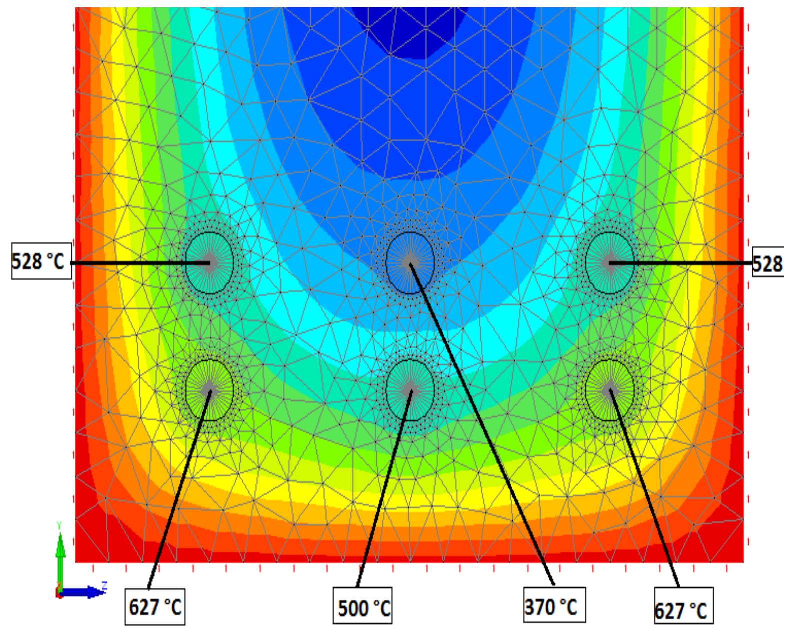



Fig. 2.3.2.4 Temperature distribution at 90 min

**Table 2.3.2.10: Results for Example 9**

| Fire resistance | Reference value *As* [cm²] | Calculated value *As'* [cm²] | Deviation [%] | Limit [%] |
|---|---|---|---|---|
| R90 | 9.76 | 9.28 | -4.91 | ± 10 |

**Table 2.3.2.11: Results for Example 10**

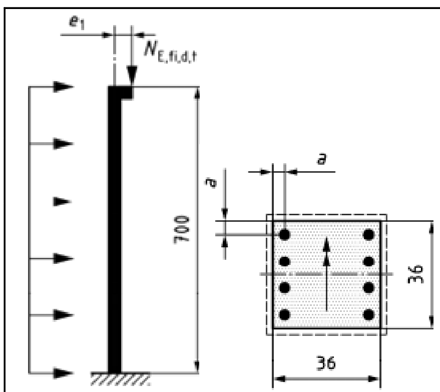
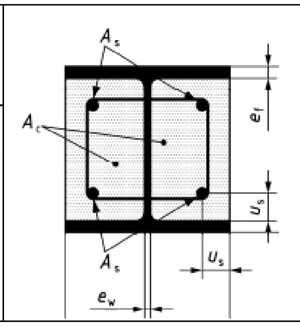
| | **Requested results** | **Calculated value** | **Deviation** [%] | **Limit** [%] |
|---|---|---|---|---|
|  | Failure time [min] | 92 | -1.08 | ± 3 |
| | Displacement [mm] | 405 | +6.30 | ± 15 |
| | Bending Moment [kNm] | 74.93 | -0.75 | ± 5 |

**Table 2.3.2.12: Results for Example 11 - SAFIR**

| Requested results | | Reference value | Calculated value | Deviation [%] | Limit [%] |   |
|---|---|---|---|---|---|---|
| Failure time | | 92 | 88 | −4.35 | ± 5 | |
| Displ. [mm] | 30 min | 4.40 | 4.44 | +0.82 | | |
| | 60 min | 5.50 | 5.04 | **−8.18** | | |

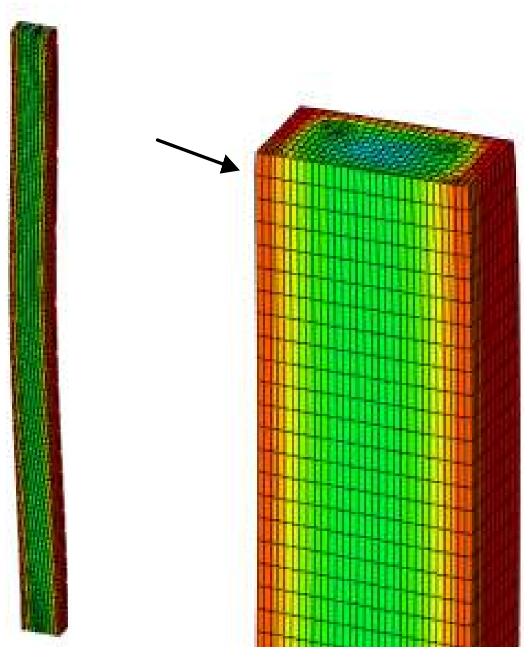


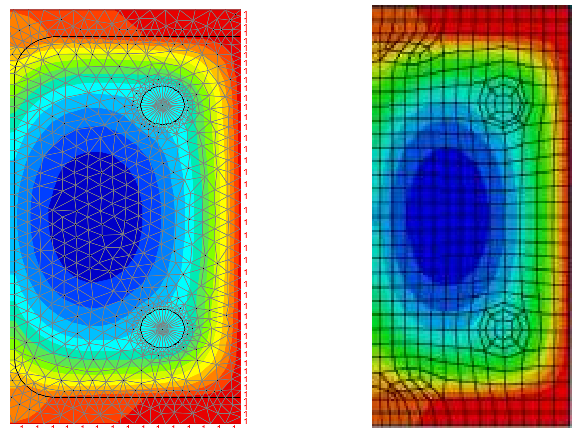
Fig. 2.3.2.5 Column model using solid elements in ABAQUS

**Table 2.3.2.13: Results of ABAQUS - Example 11**

| Requested results | | Reference value | Calculated value | Deviation [%] | Limit [%] |
|---|---|---|---|---|---|
| Failure time | | 92 | 87 | **−5.43** | ± 5 |
| Displ. [mm] | 30 min | 4.40 | 4.56 | +3.52 | |
| | 60 min | 5.50 | 7.82 | **+42.16** | |

**Table 2.3.2.14: Temperature for structural steel and rebar**

| Temperature | Abaqus [$^0$C] | Safir [$^0$C] | DIN [$^0$C] |
|---|---|---|---|
| HEB300 | 452 | 459 | 447 |
| Rebar | 512 | 518 | 535 |



a) b)

Fig. 2.3.2.6 Temperature distribution on the half of cross-section at 90 minutes : a) SAFIR b) ABAQUS

It may be concluded that, excepting for one result of Example 11, which provides a value slightly lower than the criterion given in DIN EN1991-1-2 procedure, SAFIR gives appropriate results which comply with the criteria and tolerances allowed by the DIN EN 19911-2 procedure.

For this example, the 3D analysis made using ABAQUS software provided contradictory results; using ABAQUS higher differences are obtained in comparison to the criteria of DIN EN1991-1-2 procedure, not only for the displacement at 60 minutes, as in case of SAFIR, but also for the fire resistance time. It must be however underlined that both SAFIR and ABAQUS offer conservative values for the most important result of the numerical simulation, which is the fire resistance time.

It would be helpful to identify the origin of the DIN EN1991-1-2 procedure results and to have more details for the input data for Example 11. It is also important to find out if the results given in DIN EN 1991-1-2 procedure for Example 11 where obtained using a 2D numerical model (similar to the fiber model implemented in SAFIR, attached to a beam element) or a 3D one (similar to the ABAQUS model used in this paper). At the moment of the elaboration of this thesis, a discussion about this topic is in progress among the European partners within the COST Action IFER.

## 2.4 New recommendations for fire design

Section 2.4.1 presents a numerical investigation, done with the computer program SAFIR, in order to obtain simpler finite element models (in order to be used for the fire design considering advanced calculation models, one of the three methods for fire design of structures considered in the Eurocodes) for representing the behaviour of the partially protected composite steel concrete slabs in fire situations, considering the membrane action. Appropriate understanding and modelling of the particular behaviour of composite slabs allows a safe approach, but also substantial savings on the thermal insulation that has to be applied on the underlying steel structure. In order to obtain a proper calibration of the numerical models, the results of the numerical analyses are compared with the results of three full scale fire tests on composite slabs that have been performed in recent years. The activity presented in section 2.4.1 is sustained by the fourth publication in the list of publications selected by the candidate to be relevant for the professional achievements obtained after he obtained his PhD:

> **04 - Zaharia R.,** C. Vulcu, O. Vassart, T. Gernay, Franssen J.M. (2013). "Numeric analysis of partially fire protected composite slabs". Steel and Composite Structures 14(1). ISSN 1229-9367

Section 2.4.2 proposes a method to calculate the bending capacity under ISO fire, for Slim Floor systems using asymmetric steel beams, with a wider lower flange or a narrow upper flange welded onto a half hot-rolled profile. The temperatures in the cross-section are evaluated by means of empirical formulas determined through a parametrical analysis, performed with the special purpose non-linear finite element program SAFIR. Considering these formulas, the bending capacity may be calculated, using an analytical approach to determine the plastic bending moment, for different fire resistance demands. The results obtained with this simplified method are validated through numerical analysis. The activity presented in section 2.4.2 is sustained by the fifth publication in the list of publications selected by the candidate to be relevant for the professional achievements obtained after he obtained his PhD:

> **05 - Zaharia R.,** Franssen J.M. (2012). "Simple equations for the calculation of the temperature within the cross-section of slim floor beams under ISO fire". Steel and Composite Structures, 13(2). ISSN 1229-9367.

Section 2.4.3 paper presents a numerical exploratory investigation to determine the fire resistance of Slim Floor systems using proprietary Cofradal floor units (property of ArcelorMittal Luxembourg), considering the composite steel-concrete action. The advanced numerical analysis of such systems is still reserved to experimented researchers and designers and, therefore, the aim of the study is to offer to the designer's simple "rule of thumb" recommendations for the assessment of the fire resistance, based on the results of the resistance verifications of the elements at ambient temperature. An extended study is in progress, in cooperation with ArcelorMittal Luxembourg. The activity presented in section 2.4.2 is sustained by the sixth publication in the list of publications selected by the candidate to be relevant for the professional achievements obtained after he obtained his PhD:

> **06 - Zaharia R**., Vassart O. (2012). "Fire analysis of slim floor systems using Cofradal floor units". International Conference on Advances on Steel Concrete Composite and Hybrid Structures - ASCCS 2012, July 2-4, Singapore, Edited by J. Y. Richard Liew and Siew Chin Lee, ISBN-13: 978-981-07-2615-7, ISBN-10: 981-07-2615-5.

### 2.4.1 Recommendations for the fire design of partially fire protected composite steel-concrete slabs using advanced calculation models

Observations of actual building fires and large-scale fire tests conducted in a number of countries have shown that the fire performance of composite steel framed buildings with composite floors is much better than indicated by standard fire resistance tests on composite slabs or composite beams as isolated structural elements. It is clear that there are large reserves of fire resistance in modern steel-framed buildings and that standard fire resistance tests on single unrestrained members do not provide a satisfactory indicator of the real performance of such structures.

Results of fire tests made during 1995-1996 on full-scale steel concrete composite building constructed at the Building Research Establishment laboratory Cardington, U.K., indicated that the stability of composite steel-concrete framed buildings, where some of the steel beams are unprotected, can be maintained even when the temperature of the unprotected beams exceeds 1000ºC (STC, 1999), (Bailey et al., 1999). Wang (1996) used tensile membrane action which develops in the composite steel-concrete slabs to explain the excellent fire behaviour of the composite building in Cardington full-scale tests.

Another full-scale test on a fire compartment of the same composite building was performed in Cardington in 2003, within a research project aimed to study the tensile membrane action and the robustness of structural steel joints under natural fire. An assessment of the temperature development within the fire compartment and structural elements, and of the behaviour of the structure exposed to fire is presented by Wald et al. (2005, 2006) and Simões da Silva et al. (2005). The predicted local collapse of the structure was not reached during the test, in which the columns, the external joints and the edge beam of the composite slab were fire protected.

Recent full-scale fire tests on composite steel-concrete slabs confirmed a good fire performance when exposed to a long ISO fire (Zhao et al., 2008; COSSFIRE, 2006) even if some of the interior supporting beams of the slab panel are not protected.

The load transfer mode in the slab relies essentially on bending on short spans, with small deflections, at normal temperature. In fire situations, due to the large deflections, the load transfer changes to membrane behaviour along a larger slab panel. Considering this, a design method for composite steel concrete floor slabs in fire was developed by Bailey (2004), allowing designers to specify fire protection to only a portion of the steel beams within a given floor plate. The approach represents a further development of a previous simplified design method (Bailey and Moore, 2000), (Bailey, 2001), based on the experimental work at Cardington. In Bailey design method, the final displacement of the slab is calculated considering the yield-line model. An alternative method to calculate the load capacity of simply supported composite slabs considering the membrane action, by dividing the slab into one center-elliptic part and four rigid parts around, at the limit stage of load capacity, was presented by Li et al. (2007).

Numerical modelling of six full-scale fire tests carried out in 1995-1996 on the composite frame constructed at BRE laboratory at Cardington was performed by (Huang et al, 2002), using the computer program VULCAN, developed at the University of Sheffield. The study highlighted that for low temperatures of the steel unprotected beams, the influence of the concrete slabs on the structural behaviour of the building is small, but when the steel beam temperatures are higher than 500$^{o}$C, the slab became increasingly influential as part of the load-carrying mechanism.

Another finite element modeling (Moss and Clifton, 2004) of the Cardington test of 2003 as an assemblage of linked composite members, showed the three-dimensional nature of the floor system response, involving two-way action of the floor system, consistent with Bailey model.

A comparison of the Bailey design method with non-linear finite element modelling was performed by (Huang et al, 2004). One conclusion was that the simple design method may predict a greater enhancement of fire resistance due to tensile membrane than is apparent from finite element analyses, in particular for the case of the highly reinforced square slabs, for which the simple method predicts very large enhancement. Cases with reinforcement quantities which are typical for anti-crack meshes, as well as the more rectangular slabs, show less enhancement, and the disparity is less apparent.

In order to evaluate the fire resistance of a composite slab considering the membrane effect, a finite element analysis may nevertheless be required in particular situations. Yet, a complete and detailed numerical modelling of the membrane effect is quite complex and CPU time consuming, due to the simultaneous presence of beams and of orthotropic shells. If such a numerical simulation can be done in research centres and universities, it is not practically applicable for real projects that have to be analyzed in shorter time.

The FE numerical analyses presented in this section simulate three full scale tests that have been performed in recent years: two have been performed by CTICM in France, FRACOF (Zhao et al., 2008) and COSSFIRE (2006), and one by the Czech Technical University of Prague, in Mokrsko, the Czech Republic (Chlouba & Wald, 2009), (Wald et al, 2010). The numerical analyses have been performed with the advanced calculation model SAFIR, developed at the University of Liege (Franssen, 2005).

The first objective of the study is to derive the simplest possible models for representing the partially protected composite floors in fire situations that, based on simplifications and approximations, would nevertheless yield a sufficiently close to reality representation of the structural behaviour and a safe estimate of the load bearing capacity. This is why, the simple beam connections were modelled as hinges and the nominal values for material properties were used, as would be the case for a real design situation. This “a priori” predictive simulation approach (Beard, 2000) was considered, in the same manner, to determine the natural fire temperatures of Mokrsko test, by means of the Ozone program (Cadorin and Franssen, 2003, Cadorin et al. 2003), rather than to consider the measured temperatures. The second objective is to highlight the influence of some critical parameters on the behaviour and fire resistance of composite slabs.

**Numerical models**

The transient temperature distribution across the section of the beam and on the thickness of the shell finite elements is determined by means of linear triangular or quadrangular conductive elements. Thermal properties are temperature dependent and evaporation of moisture is considered by means of modified specific heat. Convection and radiation heat transfer are considered at the boundaries.

Uniaxial mechanical properties of steel used in the beam elements and for the re-bars that are present in the shell elements follow the proposal of EN 1993-1-2 (2005); the stress-mechanical strain relationship is non-linear until 2%, with a horizontal plateau from 2 to 15% and a descending branch thereafter; unloading is plastic.

The plane stress constitutive model used for the concrete of the shell elements is an associate plasticity model with isotropic hardening. Tension is considered by a tension Rankine surface and is implemented in a smeared cracked model. Transient creep is implemented in an implicit manner.

Strength and stiffness of steel recover during cooling whereas this is not the case for strength in concrete. The fact that thermal expansion also recovers during cooling, although partially in concrete, is another factor that explains why the deflection reduces after an hour in the Mokrsko test, see section 5.

Connection between the concrete slab and the steel beams is modelled as perfect, having both types of elements sharing the same nodes. Connections between the unprotected steel beams and the protected edge beams are modelled as hinges. The choice of simple connections being used is motivated by the objective of obtaining a simple model that could be used in everyday practice as opposed to more complex models based on springs implemented in the component based method.

The transient geometrically non-linear behaviour of the structure is calculated in a dynamic analysis; acceleration forces are considered explicitly while numerical damping is used. The convergence criteria used in the iterations at each time step is based on the relative norm of the energy produced by out of balance forces multiplied by iterative displacements.

There is no criterion that would lead to "failure" of the simulations. The simulations keep on running until convergence is not possible, which is an indication of a possible run away failure with displacements increasing at a very fast rate, and it is up to the user to decide whereas the displacements reached at the last converged time step are acceptable or whereas a displacement criteria must be applied. Run away failure, with a vertical asymptote in the time-displacement curve, was observed in most simulations presented in this study.

**FRACOF test and numerical simulation**

A typical composite steel-concrete slab, shown in Fig. 2.4.1.1 (Zhao et al., 2008), was adopted for this test. The slab of the designed test specimen covered an area of 7.35 m by 9.53 m, layed on 6.66 m by 8.735 m steel structure. The slab comprised four secondary beams, two primary beams, four short columns and a 155 mm thick floor slab realised with trapezoidal steel sheet of 0.75 mm thickness (height of the ribs of 58 mm). Normal weight concrete C30/37 was adopted in the design. The reinforcing steel mesh of 7 mm used in the slab was realised with S500 steel grade and had a grid size of 150x150 mm. The axis distance of the steel reinforcement from top of the slab was 50 mm. S235 steel grade was used for secondary beams and S355 for main beams. All steel beams were linked to the concrete slab with the help of headed studs, and to the columns with two common types of steel joints (flexible end plate and double angle web cleats).

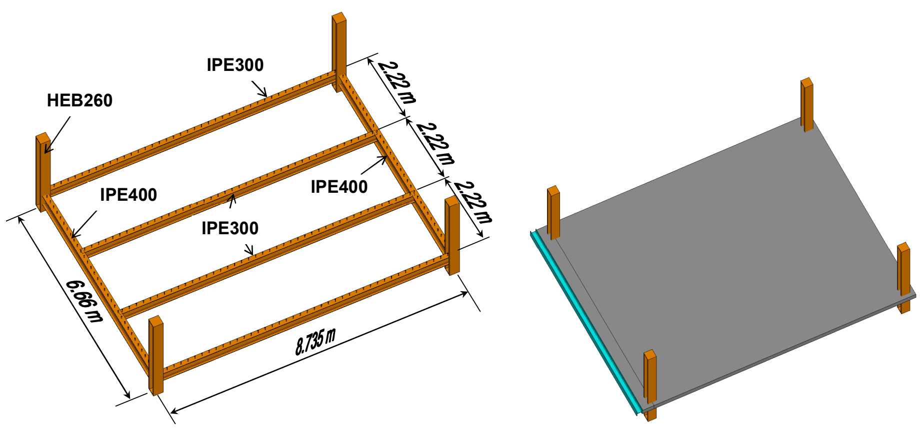



Fig. 2.4.1.1. Tested structure (Zhao et al., 2008)

During the fire test, the mechanical loading of the floor was applied using fifteen sand bags distributed over the floor leading to an equivalent uniform load of 3.87 kN/m². The two secondary beams and the composite floor were unprotected, while all the edge beams of the floor, namely all beams in direct connection with columns, were fire protected with fiber-based insulation to ensure a global structural stability under fire situations. The ISO fire exposure lasted up to 123 minutes.

Because the edge beams were placed on the boundary of the slab, the fire exposure was just on two sides. The properties of the insulation material that have been used in the simulation were the nominal ones (those given by the producer). The fiber-based insulation applied on the edge beams was characterised by: 128 kg/m$^3$ specific mass, 0 kg/m$^3$ water content and a temperature dependent thermal conductivity from 0.04 W/mK at 20°C, to 0.48 W/mK at 1200°C. The yield strength for secondary beams was 235 N/mm$^2$, while for the main beams was 355 N/mm$^2$. The yield strength of the rebars was 500 N/mm$^2$. The variation of the thermal conductivity, thermal elongation and specific heat of steel function of temperature was considered as given in EN 1993-1-2 (2005). Other parameters considered for steel are: 7850 kg/m3 specific mass and 0.7 surface emissivity. A siliceous type of concrete was considered, with mechanical properties characterised by 30 N/mm$^2$ compressive strength and 2 N/mm$^2$ tension strength. The upper limit of the thermal conductivity, according to EN 1992-1-2 (2005) was considered for concrete. Other parameters considered for the concrete within the composite slab are: 2400 kg/m3 specific mass, 46 kg/m3 water content and 0.7 surface emissivity. Due to the ISO fire exposure, for all materials, the convection coefficient on heated surfaces was considered 25 W/m2K, while the convection coefficient on unheated surfaces was 9 W/m2K.
For the unprotected beams, the fire exposure was considered on three sides (without the top flange). Fig. 2.4.1.2 shows the comparison between the calculated temperatures and the measured ones by means of thermocouples, at mid-height of the web of the secondary unprotected beams.

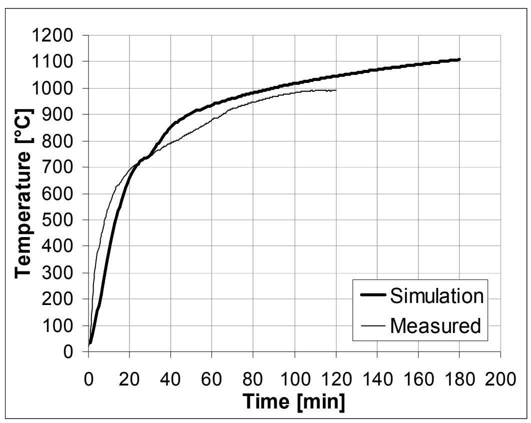



Fig. 2.4.1.2. Temperature in the secondary unprotected beams (mid-height of the web)

In the structural analyses, only the concrete located above the trapezoidal sheets has load bearing capabilities. However, the presence of the ribs is important for the temperature distribution in the concrete and in the rebars. For the thermal distribution, in order to obtain a simple numerical model, the cross section of the slab containing ribs has been replaced by a section with an equivalent thickness calculated according to EN1994-1-2 Annex D (EN1994-1-2, 2005). Fig. 2.4.1.3 shows a good agreement between the evolution of the measured and simulated temperature in the slab at the rebar level, considering this simplification.

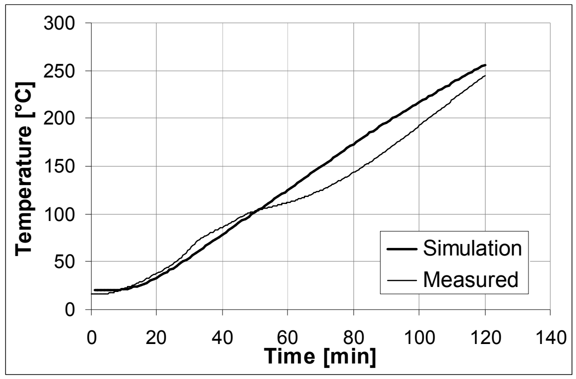
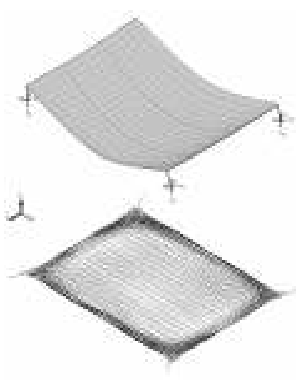



Fig. 2.4.1.3. Temperature evolution in the slab at rebar level

The primary and secondary beams have been idealised using beam elements, and the slab using shell elements. For the structural analysis, only the concrete that is present above the corrugated steel profiles was considered, while the concrete underneath only forms a thermal protection equivalent to the protective effect of the ribs. According to the joints details from the test, the beam-to-column and beam-to-beam connections were modelled as pinned. The rebars have been modelled as an equivalent steel layer on the thickness of the shell element, in amount of 256 mm²/m. Even though at the test the load was "concentrated" by using sand bags, in the simulation the load was considered uniformly distributed. For the material properties, the nominal values have been used, not the measured ones.

In Fig. 2.4.1.4, the calculated deformed shape and the membrane stresses of the slab are shown, at 165 minutes. At this moment, in the simulation, the structure failed due to large deflections of the secondary edge beams. The membrane action, characterised by the equilibrium between the compression of the concrete on the edges of the slab and the tension in the rebars from the middle of the slab, was overreached, and the slab could not uphold the load any longer. The chart shows the comparison between the measured and the calculated deflection at the centre of the slab.



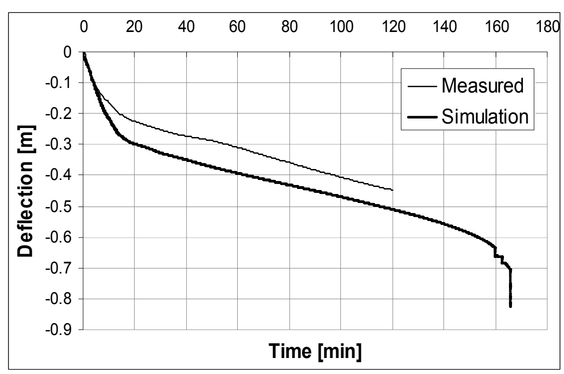



Fig. 2.4.1.4 Deformed shape and membrane forces – Deflection in the middle of the

slab

**COSSFIRE test and numerical simulation**

The concrete slab covered an area of 6.66m by 8.5m, as shown in Fig. 2.4.1.5 (COSSFIRE, 2006). The composite steel and concrete floor was made of five secondary beams, four primary beams, six short columns and a 135 mm thick slab realised with trapezoidal steel sheet of 0.75 mm thickness (height of the ribs of 58 mm). Normal weight concrete C30/37 was adopted in the design. The reinforcing steel mesh with 8 mm diameter used in the slab was realised of S500 steel grade and had a grid size of 200x200 mm. The axis distance of the steel reinforcement from the top of the slab was 35 mm. As in case of the FRACOF test, the steel beams were linked to the concrete slab with help of headed studs, and to the columns with flexible end plate and double angle web cleats.

During the fire test, the mechanical loading of the floor was applied using sand bags, distributed over the floor, leading to an equivalent uniform load of 3.75 kN/m². The interior secondary beams and the composite slab were unprotected. All the boundary beams of the floor were fire protected by fiber-based insulation, in order to ensure a global structural stability under fire situations. As for the FRACOF test, the ISO fire exposure was stopped after 123 minutes.

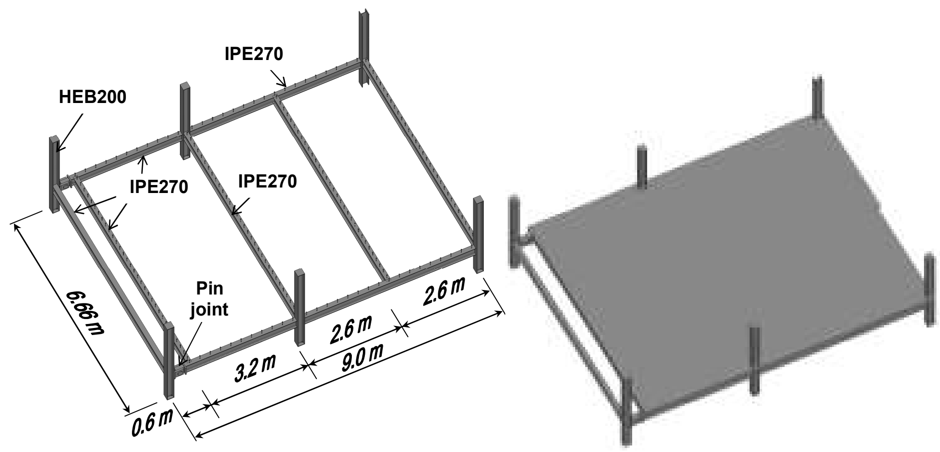



Fig. 2.4.1.5 Tested structure (COSSFIRE, 2006)

The protected beams were placed on the edges, leading to a fire exposure on two sides. For the unprotected secondary beams, the fire exposure was considered on three sides, as in case of FRACOF model. The same type of insulation as for FRACOF model was considered for the edge beams, characterised by: 128 kg/m3 specific mass, 0 kg/m3 water content and a temperature dependent thermal conductivity from 0.04 W/mK at 20°C, to 0.48 W/mK at 1200°C. The yield strength for both secondary and main beams was 235 N/mm2. The variation of the thermal conductivity, thermal elongation and specific heat of steel function of temperature was considered as given in EN 1993-1-2 (2005). Other parameters considered for steel are: 7850 kg/m3 specific mass and 0.7 surface emissivity. A siliceous type of concrete was considered, with mechanical properties characterised by 30 N/mm2 compressive strength and 2 N/mm2 tension strength. The upper limit of the thermal conductivity, according to EN 1992-1-2 (2005) was considered for concrete. Other parameters considered for the concrete within the composite slab are: 2400 kg/m3 specific mass, 46 kg/m3 water content and 0.7 surface emissivity. Due to the ISO fire exposure, for all materials, the convection coefficient on heated surfaces was considered 25 W/m2K, while the convection coefficient on unheated surfaces was 9 W/m2K.

Fig. 2.4.1.6 shows the very good agreement between the calculated temperatures and the measured ones by means of thermocouples, at mid-height of the web of the secondary unprotected beams. The FE simulation was caried for 180 min, even if the experimental program lasted for 123 min, in order to define the failure of the composite assembly.

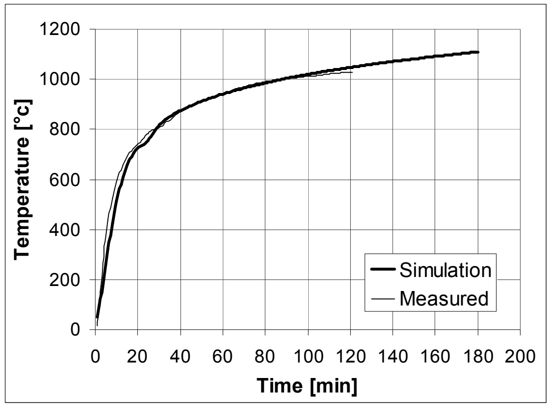



Fig. 2.4.1.6 Temperature in the secondary unprotected beams (mid-height of the web)

For the thermal distribution, as in case of the FRACOF numerical model, the cross section of the slab containing ribs has been replaced by a section with an equivalent thickness calculated according to EN1994-1-2 Annex D (EN1994-1-2, 2005). Fig. 2.4.1.7 shows the variation of the measured and the calculated temperature in the slab at the rebar level.

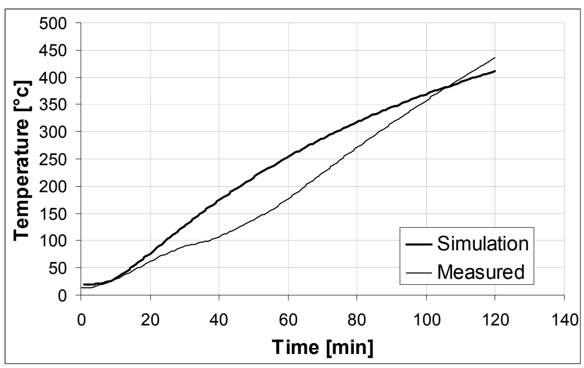



Fig. 2.4.1.7. Temperature variation in the slab at rebar level

The primary and secondary beams have been idealised with beam elements and the slab with shell elements of uniform thickness. The beam-to-column and beam-to-beam connections have been modelled as pinned. The rebars have been idealised as a steel layer in amount of 251 $mm^2/m$. In the simulation the load was considered uniformly distributed. For the material properties, the nominal values were considered.

In Fig. 2.4.1.8, the calculated deformed shape and the membrane forces of the slab after 149 minutes are shown. At this moment the composite slab failed, in the same manner as for the model of FRACOF structure, due to the large deflections of the secondary edge beam. In the chart, a comparison between the measured and the calculated deflection in the centre of the slab is shown.

After about 60 minutes a difference can be observed between the measured and the calculated deflection curves. In the test, for one of the secondary edge beams, damage of the insulation was observed, which was confirmed by an increase in temperature near the upper flange. For the mentioned edge beam, the measurements also show an increase of deflection at the middle of the span (see Fig. 2.4.1.9), affecting in this way the deflection in the middle of the slab. This effect, which could not be predicted before the test, has not been incorporated in the simulation.

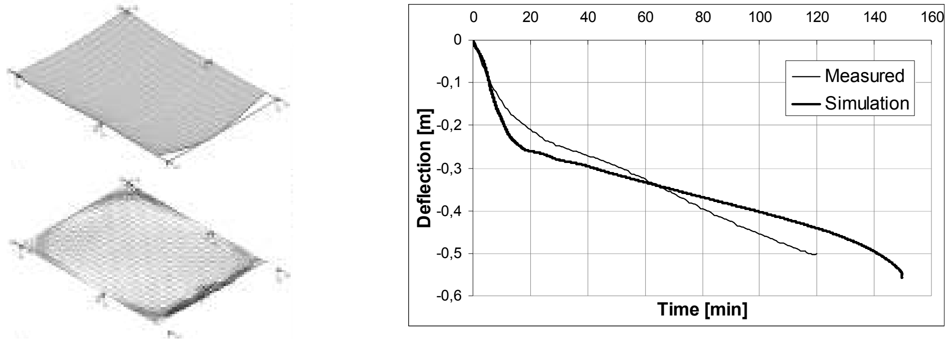



Fig. 2.4.1.8 Deformed shape and membrane forces – Deflection in the middle of the slab

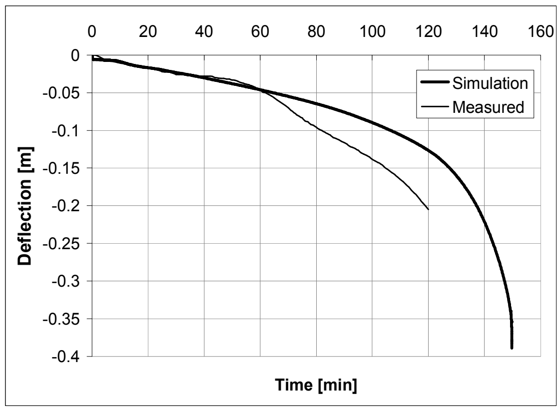
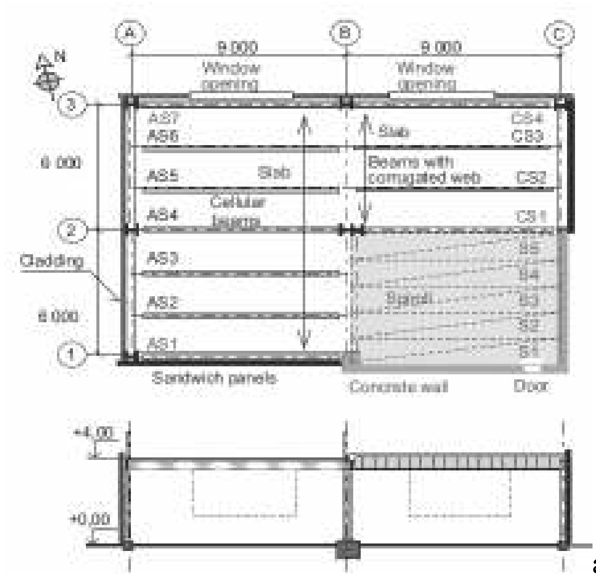



Fig. 2.4.1.9. Deflection of the secondary edge beam

**MOKRSKO test and numerical simulation**

The experimental structure, shown in Fig. 2.4.1.10a (Chlouba & Wald, 2009, Wald et al, 2010), represents one floor of an administrative building of 18 x 12 m. The composite slab on the castellated beams was designed with a 9 to 12 m span and on beams with corrugated webs with a 9 to 6 m span. The composite floor has 120 mm thickness with trapezoidal steel sheet of 0.75 mm thickness (height of the ribs of 60 mm). The concrete used for the composite slab was classified as C30/37, and was reinforced by a smooth mesh of ø5 mm 100/100 mm with 500 N/mm² strength. The axis distance of the steel reinforcement from the top of the slab was 40 mm. The height of the castellated Angelina beams with the sinusoidal openings, design by Arcelor Mittal (ASI-AS7), made of IPE270 section from S235 steel grade, was 395 mm. As shown in Fig. 2.4.1.10b (Chlouba & Wald, 2009; Wald et al, 2010), the particular cross-section of Angelina beams is obtained from cutting and re-welding of a common hot-rolled profile. The geometrical parameters which define uniquely the shape of the opening, are governed by functional requirements. The edge beams were from IPE400 sections and S235 steel grade. The beam to beam and beam to column connections were designed as header plate connections. The fire protection of the columns, primary and edge beams was designed for 60 minutes fire resistance, by board protection.




a)

b)

Fig. 2.4.1.10 Tested structure (Chlouba & Wald, 2009; Wald et al, 2010)

The dead load of the tested structure reached 2,6 kN/m². The variable load of 3.0 kN/m² was simulated by sand bags. The fire load consisted of wooden piles of 35.5 kg/m² simulating a fire load density of 620 MJ/m². The openings of 2.54 m height and a total length of 8.00 m with 1.0 m parapet ventilated the compartment. To allow a smooth development of fire, no glazing was installed.

Under the composite floor with castellated beams, a temperature of 935°C, was measured after 60 minutes. The slab failed after 62 minutes, at a time which was presumably the beginning of the cooling phase of the fire, with the measured temperature of the lower flange of the beam at the mid span of 895 °C.

The numerical simulation was performed for the 9x6m$^2$ zone, where the slab is supported by Angelina beams (between axes A-B and 1-3, see Fig. 2.4.1.10). The columns and the cross braces were not modelled. Therefore, the thermal analysis was realised only for the Angelina beams, the composite slab, and two types of protected edge beams, with fire exposure on three sides and respectively on two sides.

For the numerical model, the gypsum board fire protection was considered for the edge beams,. The yield strength for all the steel beams was 235 N/mm$^2$. The variation of the thermal conductivity, thermal elongation and specific heat of steel function of temperature was considered as given in EN 1993-1-2 (2005). Other parameters considered for steel are: 7850 kg/m$^3$ specific mass and 0.7 surface emissivity. A siliceous type of concrete was considered, with mechanical properties characterised by 30 N/mm$^2$ compressive strength and 2 N/mm$^2$ tension strength. The upper limit of the thermal conductivity, according to EN 1992-1-2 (2005) was considered for concrete. Other parameters considered for the concrete within the composite slab are: 2400 kg/m$^3$ specific mass, 46 kg/m$^3$ water content and 0.7 surface emissivity. Due to the natural fire exposure, for all materials, the convection coefficient on heated surfaces was considered 35 W/m$^2$K, while the convection coefficient on unheated surfaces was 9 W/m$^2$K.

Using the details from the test related to the fire load, the openings and the boundaries of the compartment, a fire curve has been obtained with Ozone program (Cadorin and Franssen, 2003, Cadorin et al. 2003) and used further in the thermal analysis. This curve is compared in Fig. 2.4.1.11 with the measured gas temperatures in the test.

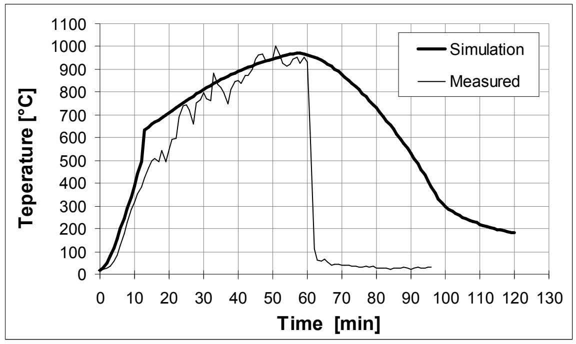



Fig. 2.4.1.11. Comparison between the Ozone fire curve and the gas measurements

The influence of web openings on the behaviour of steel beams at elevated temperatures was studied numerically by Yin and Wang (2006). It was emphasised that the presence of the openings have substantial influence on the critical temperatures of axially unrestrained beams, while for restrained beams the effect of web openings on the beam‘s large deflection behaviour is smaller. In order to simplify the numerical model, the castellated Angelina beams of the Mokrsko test were modelled using beam elements considering a minimum cross-section, for the entire length of the beam. The minimum cross-section consists of an upper and a lower T, representing the flanges, together with the parts of the web above and bellow the opening. Fig. 2.4.1.12 shows the comparison for the temperature in the lower flange of the Angelina beams. For the thermal distribution, the section of the slab containing ribs has also been replaced by a section with equivalent thickness, in a similar way as for the FRACOF and COSSFIRE numerical models.

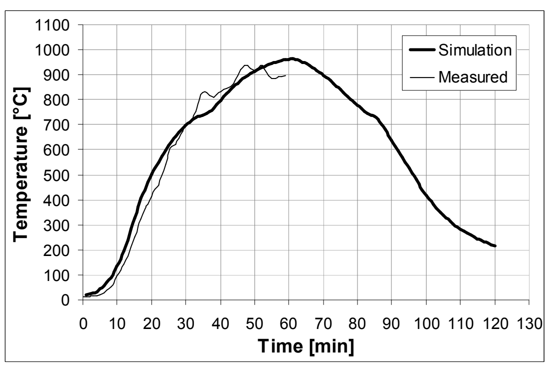



Fig. 2.4.1.12. Temperature in lower flange of the Angelina beams

In the numerical model, the edge beams and the unprotected Angelina beams were idealised using beam elements, and the slab using shell elements. Vertical supports have been used for representing the columns, and horizontal restrains for the cross braces and for the continuity of the slab.

Fig. 2.4.1.13 shows the calculated deformed shape and the membrane forces of the slab at failure, namely a concrete failure in the corner of the slab. The failure of the numerical model was produced after 56 minutes, due to lack of convergence in the concrete constitutive modeel, appearing for large strains and softening behaviour that developed at that stage. The chart in Fig. 2.4.1.13 shows the deflection curve from the simulation compared to the measured deflection from the test for the middle area of the slab.

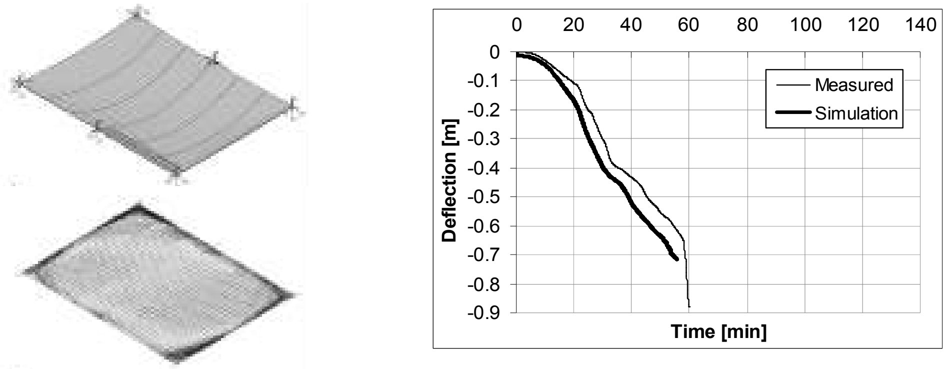



Fig. 2.4.1.13. Deformed shape and membrane forces – Deflection in the middle of the slab

**Sensitivity study**

For the three tests, a numerical sensitivity study has been performed in order to see the influence of a number of parameters on the mechanical response of a composite slab. For each parameter, supplementary simulations have been done and then compared with the reference numerical models presented above. The influence of the vertical supports on the edges, of the presence of the unprotected secondary beams, of the thickness of the slab and of the amount of reinforcement was investigated.

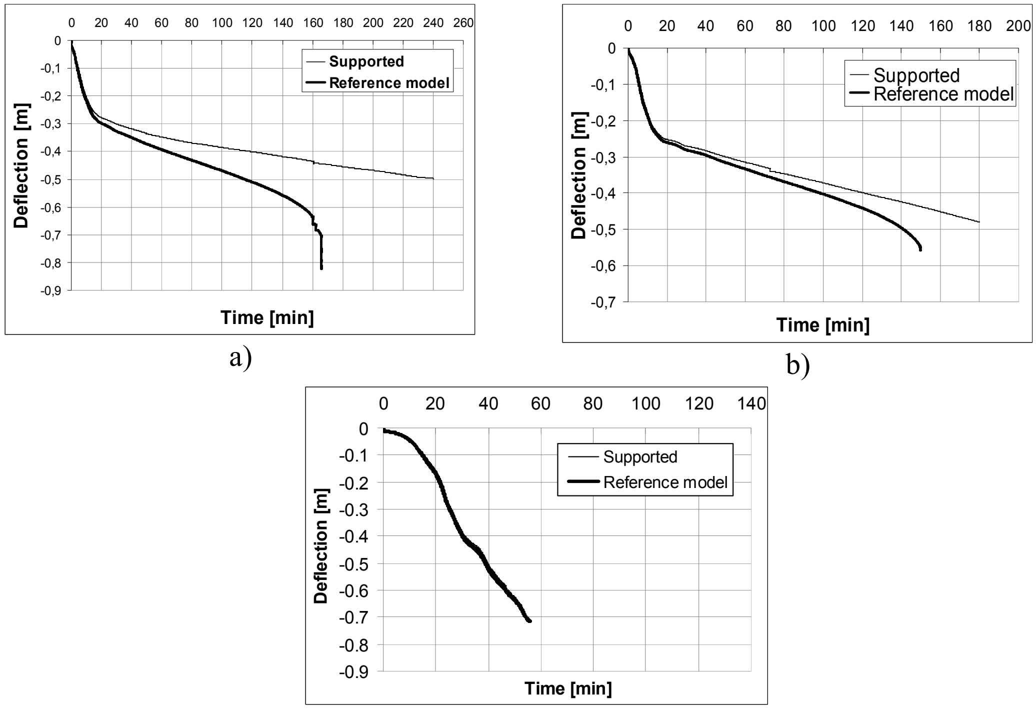
***Influence of the vertical supports on the edges***

For the three tests, a model was built in which all the edges of the composite slab were fully restrained vertically. The aim was to investigate whether this simplified model may lead to a safe estimate of the fire resistance time, thus avoiding considering the fire protection on the edge beams in the finite element model.

The influence of various degrees of edge-beam protection in the development of the membrane mechanism in composite slabs was studied by Abu et al. (2008). The numerical analyses under elevated temperatures have shown that the tensile membrane action is lost when the temperatures in the protected beams exceed about 600$^o$C. At this stage, plastic hinges form in the edge beams.

For the FRACOF and COSSFIRE tests, the slab with full vertical fixity on the edges resisted a longer time to the fire exposure than the slab with the real flexible supports, as shown in Fig. 2.4.1.14. The plastic hinge that otherwise formed in the secondary edge beams was avoided and the collapse of the slab was not reached after 4 and respective 3 hours of ISO fire exposure.

For the Mokrsko test, the deflection of the edge beams calculated for the reference case is very small; the deflection at failure is of 25 mm, compared with the deflection of 650 mm in the centre of the slab. This seems to indicate that the flexibility of the edge beams did not play a role in the failure mode. This is confirmed by the numerical simulation, in which the collapse occurred at the same time as when vertical deflections are considered on the edges of the slab; the two curves can hardly be distinguished in Fig. 2.4.1.14c.




a) b) c)

Fig. 2.4.1.14. Influence of the vertical supports on the edges: a) FRACOF b) COSSFIRE c) Mokrsko

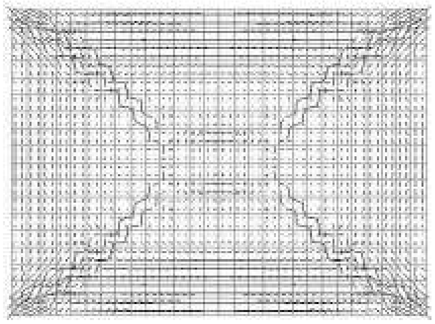
***Influence of the presence of the unprotected secondary beams***

For all tests, a model has been considered in which the unprotected secondary beams were neglected, or just a part of the section has been modelled. For the Mokrsko test, the secondary beams were castellated beams, and the question was how to model these, or whether it is really necessary to model these at all. Fig. 2.4.1.15 shows the deflection for the reference numerical models and for the models without the unprotected beams.

For the FRACOF specimen, when the unprotected beams are neglected, as shown in Fig. 2.4.1.15a, yield lines form in the slab during application of the load at room temperature, leading to the failure of the slab before the total load can be applied. For the COSSFIRE specimen, when the unprotected beams are not present, the load can be applied but the slab enters from the beginning into tensile membrane. It can be observed in Fig. 2.4.1.15b that the deflection under load at ambient temperature is very small for the reference model for which the unprotected beams are modelled, while it reaches 270 mm if the these beams are not included in the numerical model. As the fire develops, the deflection curve converges towards the same curve as the one obtained when the unprotected beams are present in the numerical model.

For the Mokrsko specimen, using the minimum section for the Angelina beams (by means of the upper and lower T, thus considering the presence of the openings for the entire length of the beams), leads to a good correlation with the test. In the case where just the upper T is considered to represent the cross-section of the Angelina beams, an early failure of the slab occurs.

This seems to indicate that considering a simpler numerical model, in which the presence of the unprotected beams is ignored (or considered in a limited way), if this model leads to stability after a significant duration, the displacements calculated with this simple model are a good approximation, on the safe side, of the displacements that would be calculated by a more complete model. If, on the other hand, the simpler model without the unprotected secondary beams leads to failure, a complete model must be considered.



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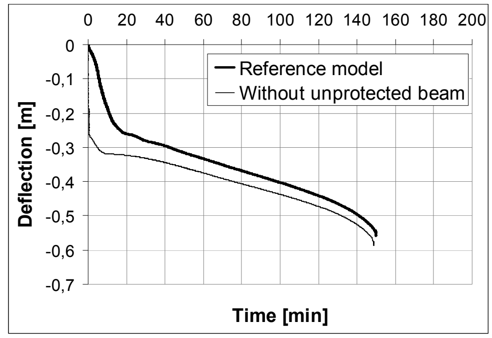


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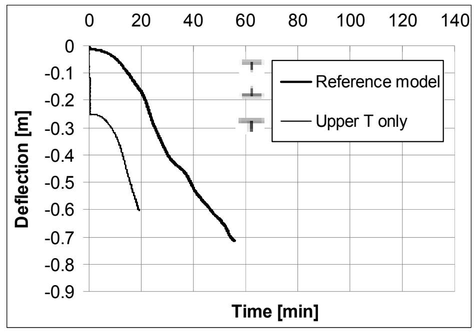



c)

Fig. 2.4.1.15 Presence of the unprotected beam: a) FRACOF b) COSSFIRE c) Mokrsko

***Influence of the thickness of the slab***

Numerical models with different thickness of concrete covering the steel trapezoidal sheets (effective slab thickness) were considered.

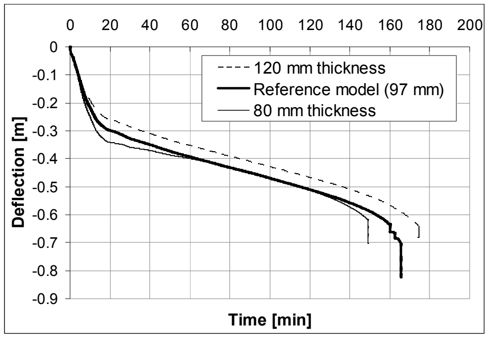
Fig. 2.4.1.16 shows that a higher thickness leads to lower deflections, and that for FRACOF and COSSFIRE tests, fire resistances over 120 minutes are achieved even with lower values of the effective slab thickness.

For the Mokrsko test, the behaviour of the slab modified completely, from failure to stability, by increasing the effective slab thickness of 60 mm for the reference model to 70 mm, while a inferior value of 50 mm leads to fire resistance times bellow 40 minutes. For the 70 mm slab, the deflection recovers due to the cooling phase, considered in the simulation of the natural fire.

The vertical displacement of the "FRACOF" composite slab with 80 mm thickness shown in Fig. 2.4.1.16a, is similar between 60 and 140 minutes with the slab of 97 mm thickness. The failure mode for the reference model, which may be observed in Fig. 2.4.1.4, was due to the large deformations of the secondary edge beams. Unlike the COSSFIRE composite slab, in Figure 2.4.1.14a it can be observed that the behaviour of the FRACOF composite slab is significantly improved by using a continuous support on the edges. Therefore, in the case of FRACOF test, the vertical displacement in the middle of the slab was influenced in a higher amount by the vertical displacement of the edge beams. A lower thickness of 80mm for the slab leads to higher deflections in the first 60 minutes of ISO fire, without a further reduction of the deflection in comparison with the slab of 97 mm thickness.

***Influence of the amount of reinforcement***

Numerical models with different quantities of reinforcements were considered with nearly unchanged results for the FRACOF and COSSFIRE tests, as shown in Fig. 17. A significant improvement is observed for the Mokrsko test, with a behaviour changed from failure to stability when the amount of steel is increased by 44%, see Fig. 2.4.1.17c.

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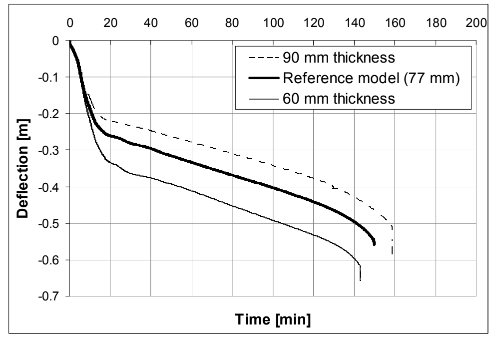


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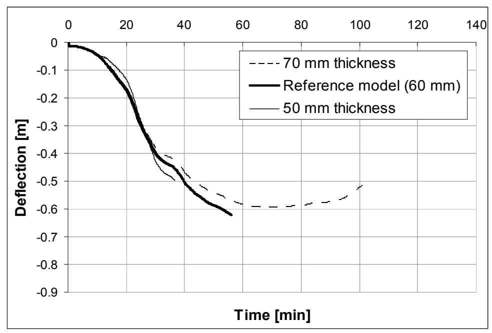
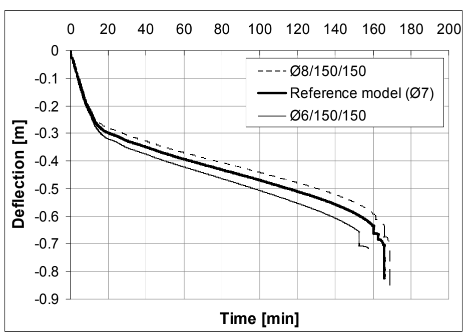



c)

Fig. 2.4.1.16. Influence of the slab thickness: a) FRACOF b) COSSFIRE c) Mokrsko

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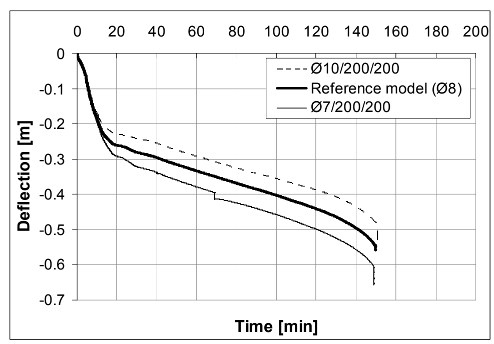


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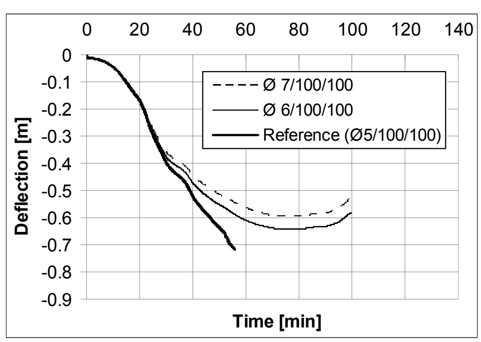



c)

Fig. 2.4.1.17. Influence of the reinforcement: a) FRACOF b) COSSFIRE c) Mokrsko

**Recommendations for design**

It may be concluded that the objective of the study, aimed to recommend a simple numerical model to be used for the fire design considering advanced calculation models, one of the three methods for fire design of structures considered in the Eurocodes, was attained. The primary and secondary beams have been idealised using beam elements, and the slab using shell elements. In order to obtain a simple numerical model, the cross section of the slab containing ribs has been replaced by a section with an equivalent thickness calculated according to EN1994-1-2 Annex D. Using this approximation, the values of the simulated temperatures in the rebars are in the safe side compared with the values of the temperatures obtained from tests. The comparison between the numerical results and the test results shows good correlation, indicating that the simplified numerical model may be used in design. Differences of time resistance for FRACOF and COSSFIRE tests could not be addressed because the fire exposure in the tests was stopped after 120 minutes. For the Mokrsko structure, the fire resistance time in the numerical simulation is almost the same as the failure time observed in the test, with a difference of 5 minutes, in the safe side for the numerical model.

An important option when the numerical model is built, is to consider or not the vertical restraints along the edge of the composite floor. For the FRACOF and COSSFIRE specimens, the failure highlighted by the numerical analysis was caused by plastic hinges forming in the secondary edge beams. When the edges of the slabs are completely restrained vertically, the plastic hinges forming in the secondary edge beams are avoided and the fire resistance times are significantly increased for these two slabs. For the Mokrsko test, the presence of the vertical restraints did not changed the failure time, which suggests that the flexibility of the edge beams did not play a role in the failure mode. Considering the results of the numerical simulations for the three cases, it is recommended to avoid the vertical restraints on the edge of the composite floor, even if this would simplify the numerical model.

The presence of the secondary unprotected beams is generally necessary in the numerical model. A simpler model, without these beams may be considered, however, if this model leads to stability after a significant duration.

There seems to be a critical thickness of the concrete slab, with lower values leading to premature failure and higher values decreasing somehow the vertical displacements but not increasing significantly the fire resistance time. In a similar manner, an increase of the amount of reinforcement does not seem to have significant effect, on the condition that the minimum amount that is required to avoid premature failure has been provided.

### 2.4.2 Recommendations for the fire design of Slim Floor beams

Slim Floor slabs are generally made of asymmetric steel beams with a wide lower flange to support prefabricated concrete elements, the gap between the steel beam and prefabricated elements being filled with concrete. The width of the bottom flange must guarantee a minimum support on both sides in accordance with the specific requirements of the slab manufacturer. One obvious benefit of this solution is that it improves the fire resistance of the composite slab, owing to the fact that the steel beam, except for the lower flange, is insulated by the concrete.
There are several asymmetric beam models which may be considered for a Slim Floor. For example, as shown in Fig. 2.4.2.1, ArcelorMittal (2008) developed three types of steel beams: IFB (Integrated Floor Beams), in which a wider lower flange or a narrow upper flange is welded onto a half hot-rolled profile, and SFBs (Slim Floor Beams), in which a wider plate is welded under the lower flange of a hot-rolled profile.

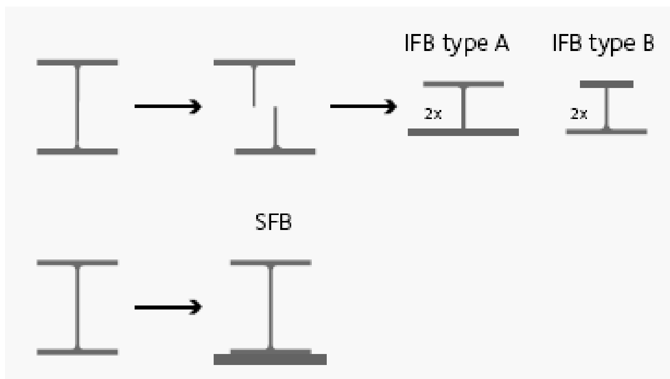



Fig. 2.4.2.1 Slim Floor beam models (ArcelorMittal, 2008)

Some of the first series of fire tests on isolated Slim Floor systems, made at ETH Zurich (Fontana and Borgogno, 1996), showed that a fire resistance of 90 minutes may be achieved for this type of slabs, without considering any fire protection. A recent experimental study on isolated Slim Floor slabs using asymmetric beams, in which the protection of the lower flange exposed to fire and the use of supplementary reinforcement were considered, concluded that the lower flange reinforcement is the most effective way to improve the fire resistance (Kim et al., 2011).

Results of fire tests made during 1995-1996 (STC, 1999), or more recently (Wald et al., 2005), (daSilva et al., 2005) on a full-scale steel concrete composite building constructed at the Building Research Establishment Laboratory Cardington, U. K., showed that the fire performance of composite steel framed buildings with composite floors is much better than indicated by standard fire resistance tests on composite slabs or composite beams as isolated structural elements. One reason for the excellent fire behaviour of the composite building in Cardington full-scale tests is the tensile membrane action which develops in the composite steel-concrete slabs (Wang, 1996). In the above mentioned large scale tests, ‘classic’ solutions were used for the composite slabs, in which the concrete slab was placed above the steel beams.

A large scale fire test on a compartment of a composite building using asymmetric Slim Floor slabs was carried out at the same laboratory in Cardington, providing for the first time a useful insight into the behaviour of this slab system in its entirety (Bailey, 2003). The experimental investigation showed that the beam-to-column connections, not assumed to transfer moment in normal design, were beneficial to the survival of the beams and of the system as a whole.

In order to investigate the behaviour of Slim Floor slabs under fire at structural level, two full-scale composite steel frames with Slim Floor slab construction were conducted more recently by Dong (2009). In one test the beam-to-column connections were protected, while in the second test both connections and steel columns were protected. The experimental study concluded that the fire resistance of frames constructed with Slim Floor slabs is at least as good as that of frames with conventional slab construction.

The composite action between the casted concrete and the steel beams is usually neglected in the design of Slim Floor beams. The beams may be calculated as pure steel elements. However, due to the presence of the concrete, the temperatures in the steel beams are not uniform, and a proper temperature distribution must be considered when calculating the fire resistance. The temperature distribution may be determined by a numerical analysis, using an appropriate program. Of course, this must be done for each particular situation, considering the dimensions of the steel beam inside the concrete and of the bottom flange exposed to fire on three sides. Anyway, for composite elements it is often necessary to use advanced calculation models in order to determine, at least, the temperature distribution inside the cross-section.

Newman (1995) used the 2D finite difference heat transfer programme TFIRE to perform the thermal analysis, for six unprotected isolated Slim Floor beams with precast concrete floor tested in fire at the Steel Construction Institute (SCI). Ma & Makelainen (2006) used ABAQUS to model the fire behaviour of Slim Floor frames, considering a combination of shell, beam and rebar elements. The numerical study highlighted that the rotational and the axial restraints on the heated beam, in the plane of the frame, have significant influence on the entire frame behaviour in fire. Ellobody (2011) developed a complex 3D finite element model in ABAQUS, in order to investigate the behaviour of unprotected composite Slim Floor slabs exposed to different fires, considering also the interface between the steel beam and the composite concrete floor. The study showed that EN 1994-1-2 (2005) predictions for composite beams at elevated temperatures may be used, being generally conservative. The fire resistance of Korean asymmetric Slim Floor slabs depending on the load ratio was numerically investigated using ANSYS (Park et al., 2011), which showed that fire resistances of 60 minutes may be achieved with the considered systems for load ratios under 0.47 without additional fire protection. The authors also set a limit temperature on the bottom flange directly exposed to fire, according to the load ratio, that indicates the fire resistance.

The temperature distribution within the cross-section of a particular Slim Floor slab, using a hat-shaped steel profile, was investigated numerically using ABAQUS software (Schaumann and Kirsch, 2011). This hat-shaped steel profile includes a cavity which must be properly considered in the numerical analysis, because neglecting the effect of the cavity radiation may lead to unsafe results. Using the results of the numerical analysis, the authors developed simple equations for the temperatures of the webs and flanges of the steel hat-shaped profile, which may be used to determine the bending capacity of the beam in fire conditions, for different fire resistance demands.

Such a simplified approach, given in the “Model Code for Fire Engineering” (Schleich et al., 2001), is available also for asymmetric Slim Floor beams of SFB type, in order to determine the temperature distribution within the steel profile. The method presented in this document gives the temperatures of the bottom wider plate, of the lower flange and on the height of the web of the steel profile for fire resistance demands of 60 and 90 minutes. No indication is given for the temperature in the concrete, in the area of possible supplementary reinforcement in the lower flange region.

In order to offer to the designer a tool to evaluate the temperatures in a Slim Floor system of IFB type (in which a wider lower flange or a narrow upper flange is welded onto a half hot-rolled profile), exposed to ISO fire, without the need of a complex numerical simulation, a parametric study was done by the authors, based on numerical simulations using SAFIR program (Franssen, 2005), developed at the University of Liege for the analysis of structures under ambient and elevated temperature conditions. The aim was to propose simple equations for the calculation of temperature in various points in the cross-section, similar as those given in the “Model Code for Fire Engineering” (Schleich et al., 2001) for SFB Slim Floor beams, or for hat-shaped Slim Floor beams, determined by Schaumann and Kirsch (2011). The formulas available for SFB beams cannot be used for IFB beams, due to the different massivity of the components of the steel beam, especially due to the lower flange, which is exposed to fire on three sides in case of IFB beams. For SFB beams, the lower flange of the asymmetric profile has a part which is exposed on fire on three sides and a narrow part included in concrete. The study also extends the availability of such analytical method for 120 minutes of fire resistance demand and offers formulas to calculate the temperature in the area of possible supplementary reinforcement, above the lower flange of the steel profile.

For the parametric study, the steel profiles of ArcelorMittal Company for IFB Slim Floor systems (ArcelorMittal, 2008) were considered, for bottom plate thicknesses of 12 - 35 mm. The temperature on each cross-section was determined with SAFIR and some formulas have been developed, function of different parameters. Using these formulas to calculate the temperature in the bottom plate, top plate, web of the profile and inside the concrete (when supplementary reinforcement is considered) the bending capacity of the cross sections was calculated analytically, and validated through a mechanical analysis under elevated temperatures.

***Validation of the numerical model***

A fire test made at ETH Zurich in 1994, on a Slim Floor slab using a steel beam similar to the system analysed in the parametric study (IFB type A using prefabricated concrete elements), as shown in Fig. 2.4.2.2 (ProfilArbed, 1995), was considered to verify the numerical model. The floor in the test comprises 3 steel beams of 2.71 m length, with a distance between beams of 2.4 m. The asymmetric beam was realized by an half of IPE400 and a welded plate 400x12, as a wider bottom flange, supporting precast concrete units. Concrete was poured in site, up to 4 cm above the top flange of profile. An upper reinforcement mesh was considered and two rebars were placed above the bottom flange of the steel beam. The floor was subjected to 120 minutes of standard ISO fire.

Because the prefabricated concrete elements do not participate to the flexural capacity of the cross-section, they were considered with the corresponding material properties only in the thermal analysis, to determine the temperature distribution. Fig. 2.4.2.3 shows the mesh of half of the symmetric cross-section.

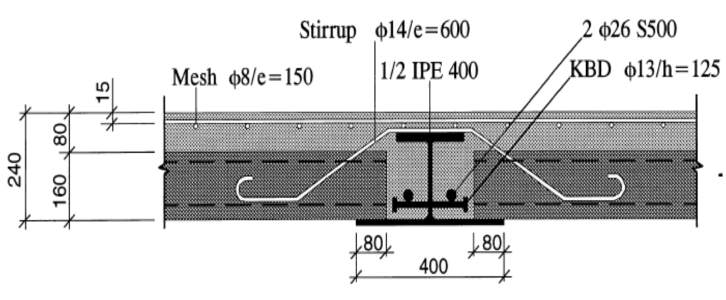



Fig. 2.4.2.2. Cross-section of the tested floor (ProfilArbed, 1995)

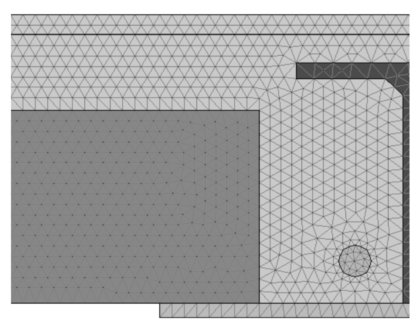


Fig. 2.4.2.3. Numerical model

As shown in Fig. 2.4.2.4, the temperatures calculated in the lower rebars are slightly higher than the corresponding temperatures recorded during the test. After 120 minutes of ISO fire, at which the test was stopped, the temperature in the rebars does not exceed 400$^o$C, which, according to EN 1993-1-2 (2005), is the temperature from which it is considered that the yield strength of steel decreases. Therefore, the rebars maintain the full yield strength for the duration of the test.

In the mechanical analysis under elevated temperatures, two situations were analysed. First, all the elements of the cross-section participate to the bending capacity (excepting for the prefabricated concrete elements), considering an effective width of 25% of the span. Second, no composite action is considered and only the steel beam with the extended bottom flange, together with the lower reinforcement, participates to the bending capacity of the slab. The results are presented in Fig. 2.4.2.5.

If the composite action is considered, Fig. 2.4.2.5 demonstrates a good agreement between the time-displacement curves resulted from the numerical simulation and from the test. Neglecting the composite action, the numerical model follows a similar path, but with higher displacements.

In order to propose an analytical model to calculate the bending capacity of IFB Slim Floor beams with precast concrete units, no composite action was considered by the authors. Therefore, for the distribution of the temperature on the cross-section, simple equations were determined only for the temperatures in the asymmetric steel beam and in the lower reinforcement.

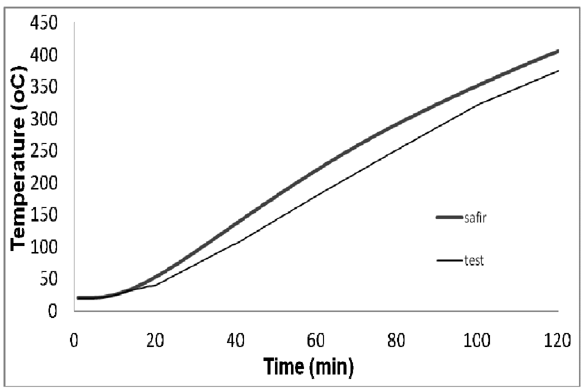



Fig.4 Temperature in rebars

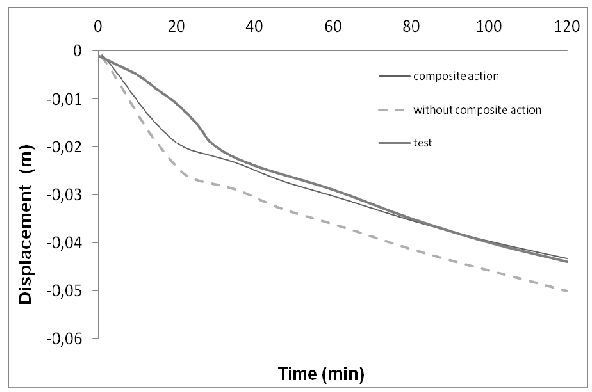



Fig.5 Displacement evolution at mid-span

**Parametric study - Temperature distribution**

For the thermal analysis, the emissivity used was 0.7 for steel and 0.8 for concrete, on heated surfaces as well as on unheated surfaces, whereas the coefficient of convection was 25 W/m²K on heated surfaces and 4 W/m²K on unheated surfaces for both materials. The upper limit of the thermal conductivity was considered for concrete.

The numerical model is shown in Fig. 2.4.2.6. The cross-section of the beam is exposed to ISO fire only from bellow, the temperature in the air on the top of the floor being considered 20$^{o}$C.

The thermal analysis was done for 30, 60, 90 and 120 minutes of fire exposure, for each cross-section considered. Temperatures from relevant points of the cross-section were extracted from the numerical analysis, and the distance from the top of the bottom flange from which the temperatures are below 400$^{o}$C was also monitored. For all cases, the 400$^{o}$C isotherm was found to be on the height of the web, even after 120 minutes of fire exposure. Therefore, the top plate retains its full strength, and the parametric study further concentrated on the temperature distribution in the bottom flange, web and concrete, in the area of possible supplementary lower reinforcement (in the hypothesis that the temperature in the reinforcement is equal to the temperature in the concrete, in the same location). Fig. 2.4.2.7 shows the points in which the temperature was monitored and the temperature distribution on the cross-section for a given case, after two hours of standard ISO fire exposure, by highlighting the 400$^{o}$C limit in the web of the asymmetric beam.

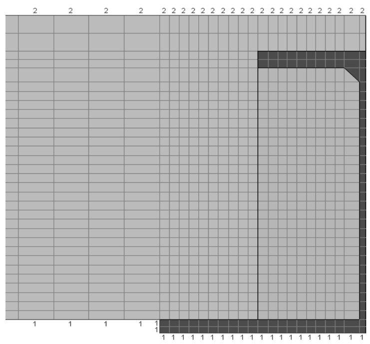


Fig. 6. Numerical model of the cross-section

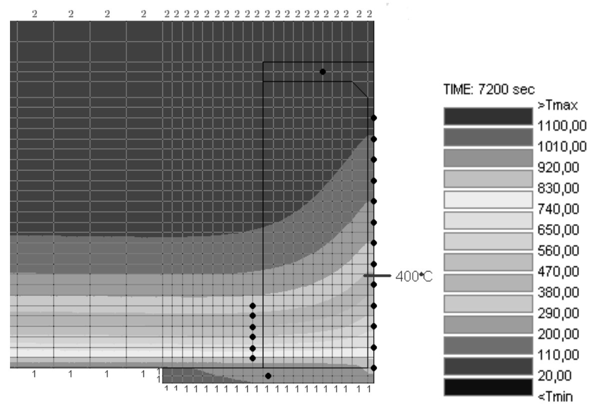



Fig. 7. Points monitored on the cross-section

***Temperature in the bottom flange***

In a first approach, the temperature in the bottom plate was calculated using the simple method presented in EN1993-1-2, table 4.2 (EN 1993-1-2, 2005), considering the section factor Am/V = (b+2t)/(bt), for the flange exposed on three sides. This leads to very conservative values of the bending capacity of the floor calculated analytically, when compared to the results of numerical simulations. For Slim Floor slabs, the temperature in the lower flange, which is directly exposed to fire, has the strongest influence on the fire resistance. Therefore, another method for the calculation of the temperature in the bottom flange was considered.

The temperature was recorded after 30, 60, 90 and 120 minutes of ISO fire, in the point shown in Fig. 2.4.2.7, situated at a quarter of the width of the bottom flange. It was found that this temperature depends essentially on the thickness of the bottom plate. Therefore, in order to derive simple formulas for the temperature evolution, the temperatures were represented as shown in Fig. 8, function of the thickness of the plate, for the different fire resistance demands. First and second order functions were found to fit better with the scatter, as Fig. 2.4.2.8 shows, and were used to represent the temperature in the bottom flange.

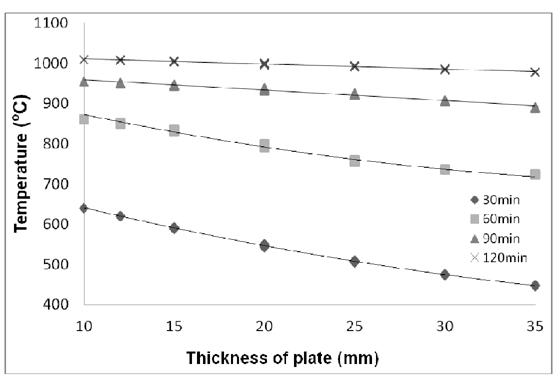



Fig. 2.4.2.8. Temperature in bottom flange function of plate thickness

The following equation is proposed to determine the temperature in the bottom flange, in which *Ti* is in °C and the plate thickness *tpl* is in mm:

$$T_i = A_i\, t_{pl}^2 + B_i\, t_{pl} + C_i$$

in which the coefficients Ai, Bi and Ci are given in Table 2.4.2.1.

Table 2.4.2.1. Coefficients for temperature calculation in the bottom flange

| Time [min] | Ai | Bi | Ci |
|---|---|---|---|
| 30 | 0.113 | -12.80 | 760 |
| 60 | 0.130 | -11.80 | 980 |
| 90 | - | -2.60 | 990 |
| 120 | - | -1.25 | 1025 |

***Temperature in the web of the steel profile***

The temperature on the height of the web is hardly influenced by its thickness, but is strongly influenced by the distance from the bottom flange and, in a smaller amount, by the thickness of the bottom flange tpl.

The temperature was recorded after 30, 60, 90 and 120 minutes of ISO fire, in the points from the web shown in Fig. 2.4.2.7. The temperatures were represented as shown in Fig. 2.4.2.9-14, function of the distance from the top of the bottom plate, for the different fire resistance demands and for a given thickness of the bottom plate. Exponential functions were found to fit better with the scatter, as Fig. 2.4.2.9-14 show, and were used to represent the temperatures in the web.

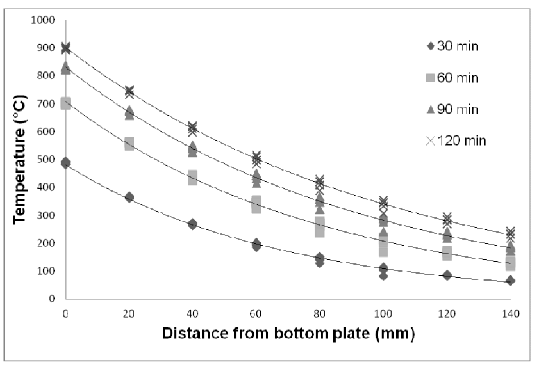



Fig. 2.4.2.9 Web temperature for 12mm bottom plate

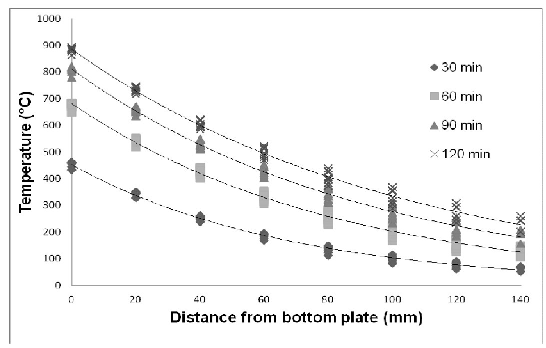



Fig. 2.4.2.10 Web temperature for 15mm bottom plate

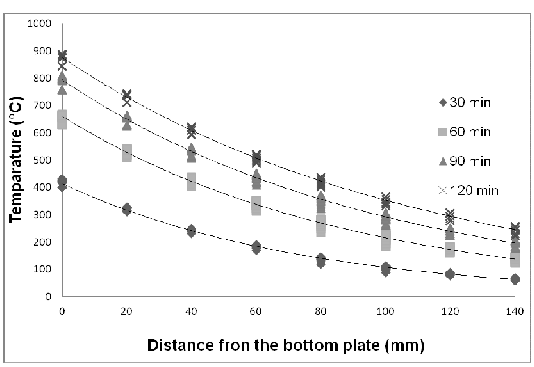



Fig. 2.4.2.11Web temperature for 20mm bottom plate

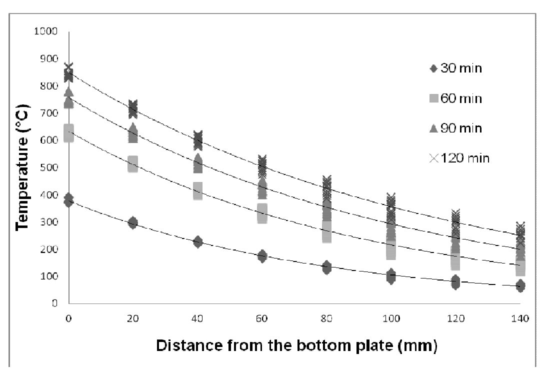



Fig. 2.4.2.12 Web temperature for 25mm bottom plate

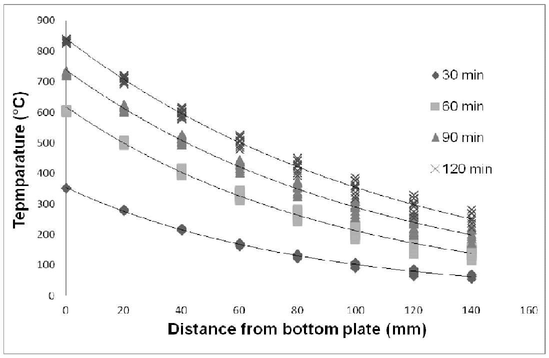



Fig. 2.4.2.13 Web temperature for 30mm bottom plate

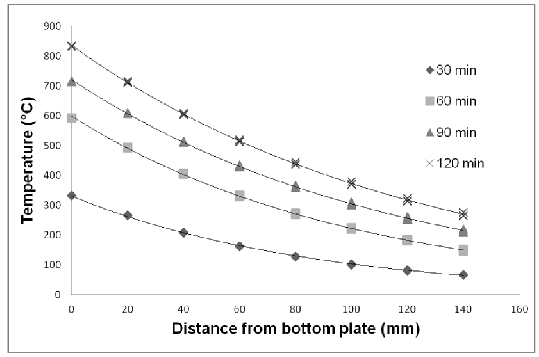



Fig. 2.4.2.14 Web temperature for 35mm bottom plate

The following equation is proposed to determine the temperature in the web, in which *Tw* is in °C, while the distance *z* along the height of the web measured from the top of the bottom flange and the plate thickness *tpl* are in mm:

$$T_w = k_1\, e^{k_2 z}$$

with

$$k_1 = A_w \ln\left(t_{pl}\right) + B_w$$

$$k_2 = C_w \ln\left(t_{pl}\right) + D_w$$

in which the coefficients Aw, Bw, Cw and Dw are given in Table 2.4.2.2.

Table 2. Coefficients for temperature calculation in the web

| **Time [min]** | **Aw** | **Bw** | **Cw** | **Dw** |
|---|---|---|---|---|
| 30 | -140.70 | 832.42 | 0.00317 | -0.0230 |
| 60 | -103.80 | 968.60 | 0.00232 | -0.0182 |
| 90 | -108.60 | 1146.70 | 0.00198 | -0.0154 |
| 120 | -70.44 | 1124.40 | 0.00158 | -0.0134 |

***Temperature in the rebars above the bottom flange***

The temperature in the rebars above the bottom flange was considered equal to the temperature of the concrete at the same location. As in case of the web temperature, the temperature on the height of the concrete is strongly influenced by the distance from the bottom flange and, in a lesser extent, by the thickness of the bottom flange.

The temperature was again recorded after 30, 60, 90 and 120 minutes of ISO fire, in the points within the concrete above the bottom flange shown in Fig. 2.4.2.7, which are located in the zone of the possible positions of the rebars. The temperatures were represented in a similar manner as for the web temperature distribution, function of the distance from the top of the bottom plate, for the different fire resistance demands, for a given thickness of the bottom plate. Exponential functions similar to the ones for web temperature distribution were found, as Fig. 2.4.2.15-20 show.

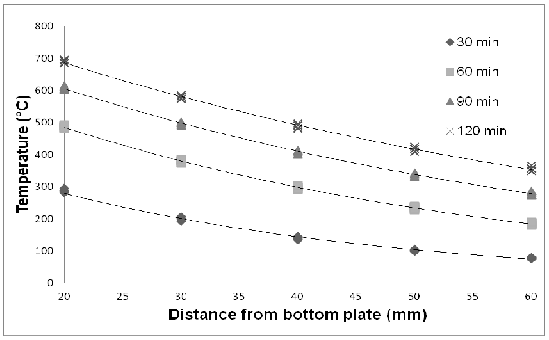



Fig. 2.4.2.15 Concrete temperature for 12mm bottom plate

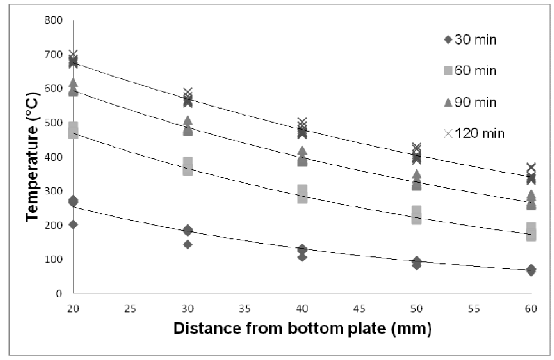



Fig. 2.4.2.16 Concrete temperature for 15mm bottom plate

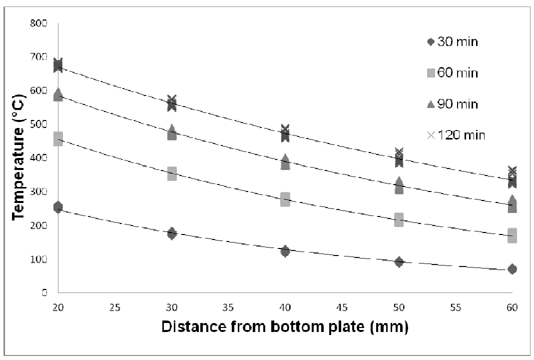



Fig. 2.4.2.17 Concrete temperature for 20mm bottom plate

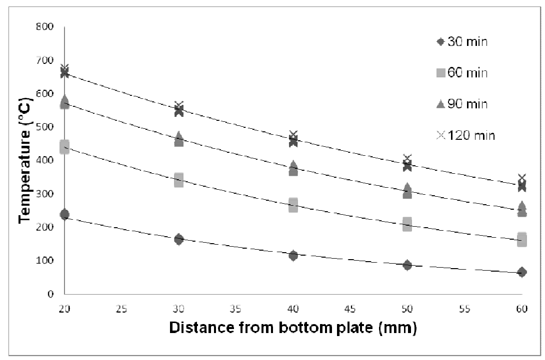



Fig. 2.4.2.18 Concrete temperature for 25mm bottom plate

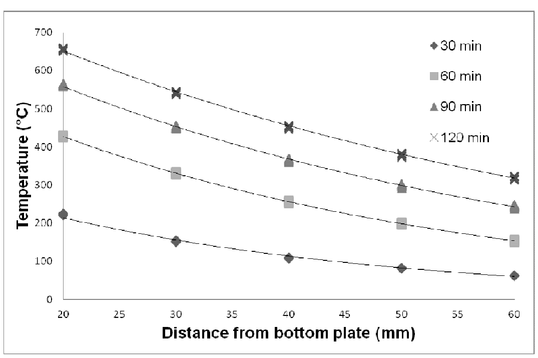



Fig. 2.4.2.19 Concrete temperature for 30mm bottom plate

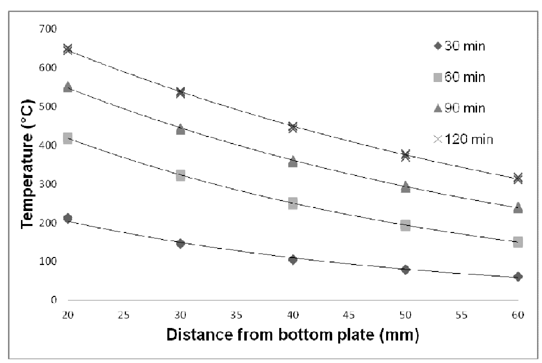



Fig. 2.4.2.20 Concrete temperature for 35mm bottom plate

The following equation is proposed to determine the temperature in the concrete (rebars), in which $Tc$ is in °C, while the distance z measured from the top of the bottom flange and the plate thickness tpl are in mm :

$$T_c = k_3\, e^{k_4 z}$$

with

$$k_3 = A_c \ln\left(t_{pl}\right) + B_c$$

$$k_4 = C_c \ln\left(t_{pl}\right) + D_c$$

in which the coefficients Ac, Bc, Cc and Dc are given in Table 2.4.2.3.

Table 3. Coefficients for temperature calculation in concrete (rebars)

| Time [min] | Ac | Bc | Cc | Dc |
|---|---|---|---|---|
| 30 | -6.90 | 612.67 | 0.00009 | -0.0342 |
| 60 | -4.06 | 834.64 | -0.00005 | -0.0240 |
| 90 | -2.71 | 970.63 | -0.00005 | -0.0181 |
| 120 | -1.37 | 1043.80 | -0.00005 | -0.0150 |

***Calculation of the bending capacity***

For the mechanical analysis under elevated temperatures, steel was considered with the following properties: E = 2.1E11 N/m$^2$, yield strength for the profile fy= 355 MPa, yield strength for reinforcement fy= 500 MPa. As stated above, the mechanical properties of the concrete were ignored (no composite action).

The bending capacity under elevated temperatures $Mpl,Rd,\theta$ was determined taking into account that a full plastic moment is developed when the section is fully yielded in bending. The following hypotheses were considered: the material behaviour is ideal rigid-plastic, the yield strength in tension is equal to the yield strength in compression, all the material of the cross-section is yielded and the cross-section carries no axial force. The plastic design bending moment $Mpl,Rd,\theta,$ was calculated by a classical integration on the depth of the cross-section, considering strips of 1 mm thickness. For each strip the stress resultant $Fi = ( Ai*fy,\theta,i )/\gamma_{a,fi}$, is calculated, where $Ai$ is area of the 'i' strip, $fy,\theta,i$ is the yield strength of 'i' strip for θ temperature and $\gamma_{a,fi}$ is a partial safety factor, considered with the unit value, as recommended in the fire parts of the Eurocodes.

To verify the calculated values of the plastic bending moment using this analytical approach, numerical simulations using SAFIR were performed, by considering for each section a simply supported Slim Floor beam, loaded with uniform bending moment. The value of the applied bending moment was determined in order to obtain the requested fire resistance demand of 30, 60, 90 or 120 minutes. The bending capacity under elevated temperatures calculated by SAFIR for a given case, is then equal to the value of the bending moment imposed to the simply supported beam in the numerical analysis, in order to obtain a given fire resistance demand.

In a first step, the simplified method provided in EN1993-1-2 (2005), using the section factor for the bottom flange was considered to evaluate the temperature in this element, together with the equation for the temperature distribution in the web of the steel profile. For all cases, the fire resistance time in SAFIR exceeded, with an important amount, the fire resistance time predicted by the analytical method. This is due to the fact that the evaluation of the temperature in the bottom flange using the EN1993-1-2 method yields to values that are too conservative, compared with the temperatures determined by the parametric numerical study. For example, Fig. 2.4.2.21 shows that all the values of the plastic bending moment $Mpl,Rd,\theta$ calculated with SAFIR are considerably higher than the corresponding values calculated using the analytical method, for the fire resistance demand of 60 minutes.

In order to increase the load bearing capacity of a given cross-section under ISO fire, in the absence of fire protection, additional reinforcement is needed. Two bars ø28 were considered as supplementary lower reinforcement. The bending capacity was again determined analytically for the fire resistance demands of 60, 90 and 120 minutes, considering for the temperature in the rebars equation, for the web equation and for the bottom plate the method based on the section factor.

Fig. 2.4.2.22 shows, for example for the fire resistance demand of 60 minutes, that all the values of the plastic bending moments Mpl,Rd,θ determined with SAFIR are considerably higher than the corresponding values calculated using the analytical method, when supplementary reinforcement is considered. Using two rebars ø28 as lower reinforcement, the bending capacity in fire increased with 25-55% in comparison with the capacity of the same sections without rebars.

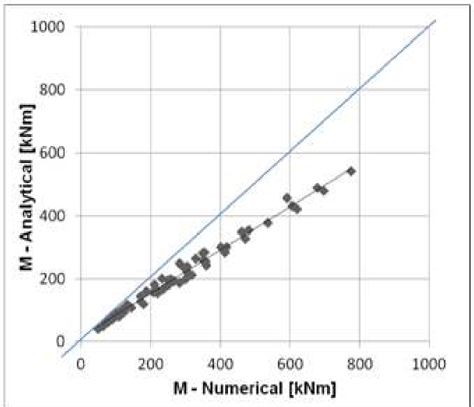



Fig. 2.4.2.21. Mpl,Rd,θ at t = 60 minutes, without reinforcement

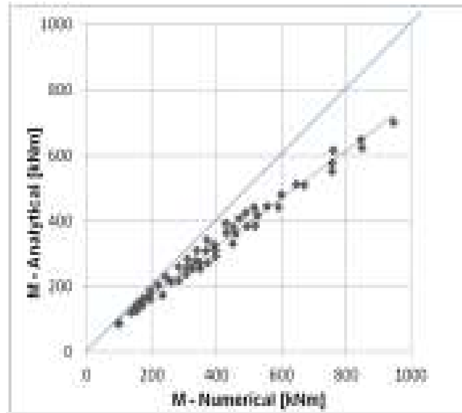



Fig. 2.4.2.22 Mpl,Rd,θ at t = 60 minutes, with reinforcement

In a second step, in the evaluation of the bending capacity of the Slim Floor beams using the analytical method, the simplified method provided in EN1993-1-2, using the section factor for the bottom flange was dropped and the first equation was considered to evaluate the temperature in this element. For the temperature distribution in the web of the steel profile, and for the temperature in the rebars, the same equations were considered.

A better fit of the results given by the analytical method and by SAFIR were obtained, in absence of the rebars, as Fig. 2.4.2.23-26 show. The results obtained with SAFIR showed that the beams can retain without reinforcement, at least, the following load ratios (defined as the ratio between Mpl,Rd,θ determined with SAFIR and the design resistance of the beam for time t=0, *Mfi,t,Rd,* determined according to EN 1993-1-2): 0.5 at 30 minutes of ISO fire; 0.17 at 60 minutes of ISO fire; 0.12 at 90 minutes of ISO fire and 0.11 at 120 minutes of ISO fire. For 60, 90 and 120 minutes of fire resistance, the following maximum load ratios are possible, respectively: 0.99, 0.43, 0.29 and 0.26. These results are illustrated in Fig. 2.4.2.27.

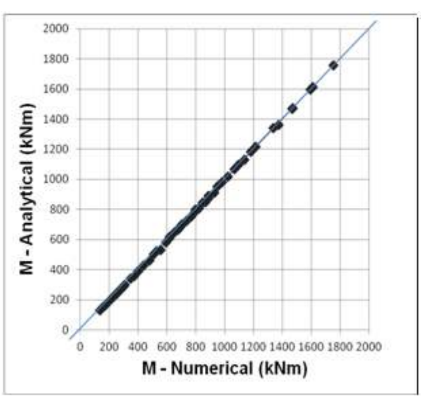



Fig. 2.4.2.23. Mpl,Rd,θ at t = 30 minutes, without reinforcement

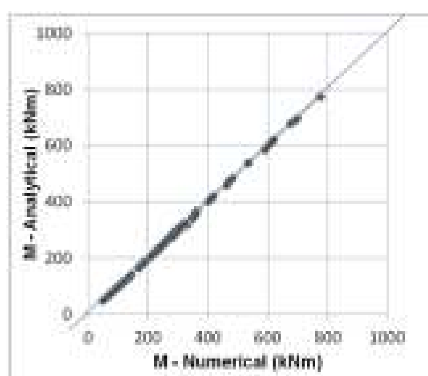



Fig. 2.4.2.24. Mpl,Rd,θ at t = 60 minutes, without reinforcement

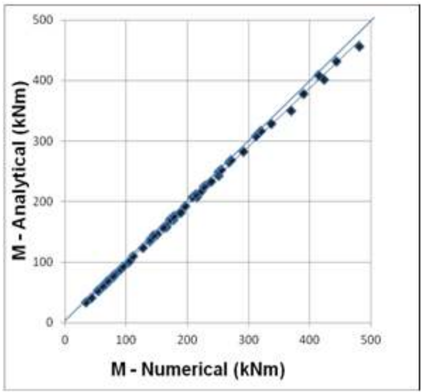



Fig. 2.4.2.25. Mpl,Rd,θ at t = 90 minutes, without reinforcement

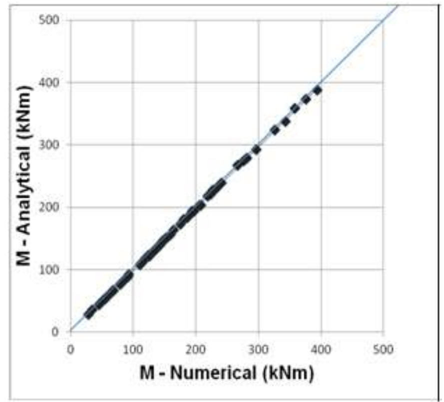



Fig. 2.4.2.26. Mpl,Rd,θ at t = 120 minutes, without reinforcement

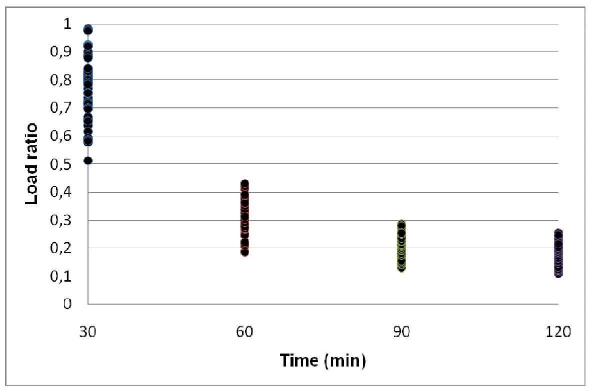



Fig. 2.4.2.27 Load ratios, without reinforcement

A better fit of the results given by the analytical method and by SAFIR were also obtained for the sections with lower reinforcement considering the first equation, as Fig. 2.4.2.28-30 show. The results show that the beams can retain with this supplementary reinforcement, at least, the following load ratios: 0.34 at 60 minutes under ISO fire; 0.25 at 90 minutes under ISO fire and 0.23 at 120 minutes under ISO fire. For 60, 90 and 120 minutes of fire resistance, the following maximum load ratios are possible, respectively: 0.60, 0.55 and 0.51. These results are illustrated in Fig. 2.4.2.31.

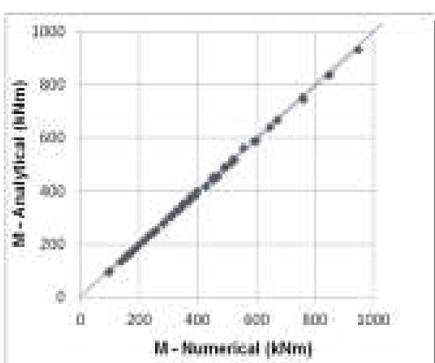



Fig. 2.4.2.28. Mpl,Rd,θ at t = 60 minutes, with reinforcement

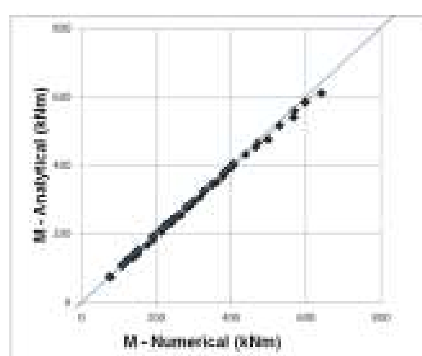



Fig. 2.4.2.29. Mpl,Rd,θ at t = 90 minutes, with reinforcement

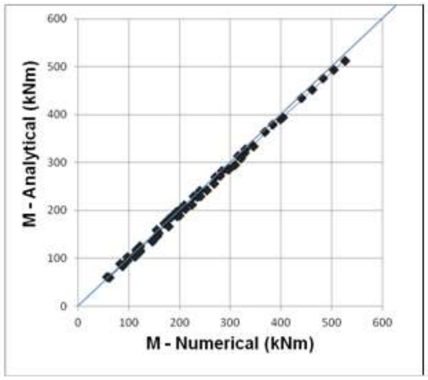



Fig. 2.4.2.30. Mpl,Rd,θ at t = 120 minutes, with reinforcement

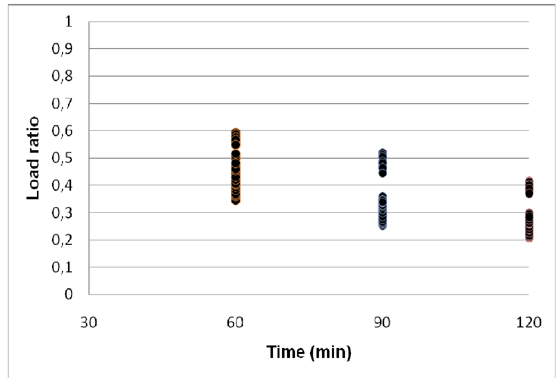



Fig. 2.4.2.31. Load ratios, with reinforcement

**Recommendations for design**

It may be concluded that the objective of the study, aimed to recommend analytical equations to be used for the fire design of Slim Floor beams was attained. In a parametric study, the temperatures on the cross-sections were determined numerically and simple equations have been developed for the temperature distribution. Using these equations, the bending capacity of the beams with or without reinforcement above the bottom flange may be calculated, by means of a classical plastic approach. The simplified analytical approach gives good results in comparison with numerical analyses at elevated temperatures.

### 2.4.3 Fire design recommendations for Slim Floor beams using proprietary Cofradal floor units

As showed in the previous section, the Slim Floor system is a fast and economical solution which combines prefabricated or casted concrete slabs with built-in steel beams. The particular feature of this system is the special kind of girder with a bottom flange which is wider than the upper flange, thus being possible to fit the floor slabs directly onto the bottom flange plate of the beam. Because the use of standard shear studs welded on the upper flange of the steel beam is not appropriate in this situation (in order to keep the reduced height of the slab), the load-bearing capacity is usually calculated, as shown in the previous section, considering only the steel profile.

The problem is that, due to the low beam height, the design of the Slim Floor beams is governed by the stiffness of the system, and therefore the spans are limited to approximately 7m (Hechler et al., 2011). However, by considering particular measures adapted to the Slim Floor systems for a controlled shear transmission between the steel section and the concrete, a composite action may be ensured, and thus the stiffness, as well as the load-bearing capacity and the fire resistance time may be increased. In 2009, ArcelorMittal developed in corporation with the University of Stuttgart, Germany, a new floor system, the CoSFB (Composite Slim Floor Beam), which combines the advantages of a composite construction with the advantages of the Slim Floor concept, by using concrete dowels, as shown in Fig. 2.4.3.1 (Hechler and Braun, 2010).

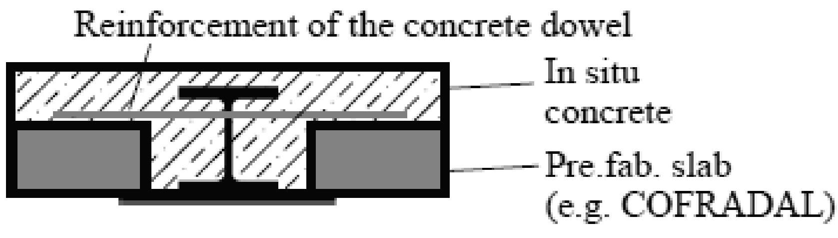



Fig 2.4.3.1: CoSFB section (Hechler and Braun, 2010)

The solution consists of a reinforcement bar introduced through a hole in the web of the steel profile, able to transfer the shear forces between concrete and steel. Sufficient load bearing capacity and ductile behaviour of this innovative shear connection has been proven recently through an experimental research performed by the Institute of Structural Design, University of Stuttgart (Hechler et al., 2011). The beam tests have shown an activation of the concrete chord even larger than the rules given in the corresponding Eurocode for the design of composite steel and concrete structures (EN 1994-1-1, 2006).

This section presents an exploratory numerical investigation for a limited number of CoSFB's with spans ranging from 6-10m, using proprietary Cofradal® floor units of ArcelorMittal Company, to emphasize the effect of the composite steel-concrete action on the fire resistance. SFB types were considered, in which a supplementary bottom plate is welded on the steel profile.

The Cofradal floor unit (see Fig. 2.4.3.2) is made of steel, rockwool insulation, reinforcement and concrete and is significantly lighter than conventional decking.

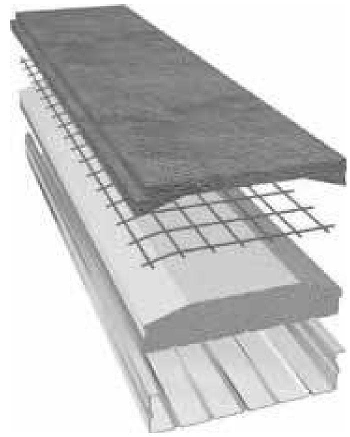


Fig 2.4.3.2: Cofradal® floor unit

The height of the Cofradal floor units decides about the size of the steel profiles that may be used. If higher steel profiles are needed, in order to respect a minimum concrete coverage above the upper flange, an additional piece of steel or timber may be added between the bottom plate of the steel profile and the Cofradal element.

Cofradal200 and Cofradal260 floor units were considered in the present study. For Cofradal200, a HEM160 S355 steel profile, with a welded lower plate of 350*25 mm S355 was considered. For Cofradal260, two steel profiles were considered: HEB200 with a welded lower plate of 400*15 mm and HEM200 with a welded lower plate of 400*25 mm (all with S355). The casted concrete was C30/37.

Three spans were considered for this numerical investigation, of 6, 8 and 10 m, with a beam distance of 6 m. The dead load was calculated considering the corresponding weights of the Cofradal units, plus an additional load of 1.00 kN/m$^2$, while the live load considered was 3.0 kN/m$^2$.

Table 2.4.3.1 gives the design value of the bending moment *MEd*, for each case, function of the span, and the design plastic moment resistance of the beam, considering only the steel profile *Mpl,SFB* or the composite action *Mpl,CoSFB*, considering an effective width of L/4 (EN 1994-1-1, 2006). The displacements calculated considering the SLS combination of actions for SFB and CoSFB situations are also given in Table 1, together with the limit values for the displacement criterion of L/250. No additional reinforcement was considered for the cases presented in Table 2.4.3.1.

It may be observed that for SFB, the resistance criterion is fulfilled for Cofradal200 only for 6 m span, while in case of Cofradal260 with HEM200-400*25 profile this criterion is fulfilled up to 8 m span. However, from all cases, only the HEM200-400*25 profile satisfies the SLS criterion, for the 6 m span.

For CoSFB situation, the HEM200-400*25 profile used with Cofradal260 satisfies both ULS and SLS criteria, up to 1 0m span, while the other cases satisfy these criteria up to 8 m span. In all cases in which the ULS criterion is satisfied by SFB's, the SLS criterion is satisfied by CoSFB's.

The numerical fire analysis was performed using the computer program SAFIR (2005). The beams were considered as simply supported, loaded with the dead and live loads in the corresponding accidental fire combination of actions. Fig. 2.4.3.3 shows the mesh for a typical cross-section.

Table 2.4.3.2 gives the fire resistance times for all cases that satisfy the ULS criterion for SFB's and the ULS and SLS criteria for CoSFB's, as shown in Table 2.4.3.1. For the SFB situations in which the displacement criterion at normal temperature design is not fulfilled, the values of the fire resistances are marked in parentheses.

Table 2.4.3.1: ULS and SLS criteria for SFB and CoSFB

| Case No. | Section | L [m] | $M_{Ed}$ [kNm] | $M_{pl,SFB}$ [kNm] | $M_{pl,\ CoSFB}$ [kNm] | Displ. SFB [cm] | Displ. CoSFB [cm] | L/250 [cm] |
|---|---|---|---|---|---|---|---|---|
| 1 | Cofradal200 HEM160 350*25 | 6 | 231 | 351 | 690 | 2.7 | 0.9 | 2.4 |
| 2 | | 8 | 410 | | 786 | 8.8 | 2.4 | 3.2 |
| 3 | | 10 | 641 | | 847 | 21.3 | 6.3 | 4 |
| 4 | Cofradal260 HEB200 400*15 | 6 | 260 | 294 | 693 | 3.4 | 0.6 | 2.4 |
| 5 | | 8 | 462 | | 729 | 10.8 | 1.8 | 3.2 |
| 6 | | 10 | 722 | | 764 | 26.4 | 4.3 | 4 |
| 7 | Cofradal260 HEM200 400*25 | 6 | 260 | 557 | 945 | 1.8 | 0.4 | 2.4 |
| 8 | | 8 | 462 | | 984 | 5.4 | 1.2 | 3.2 |
| 9 | | 10 | 722 | | 1170 | 13.2 | 2.9 | 4 |

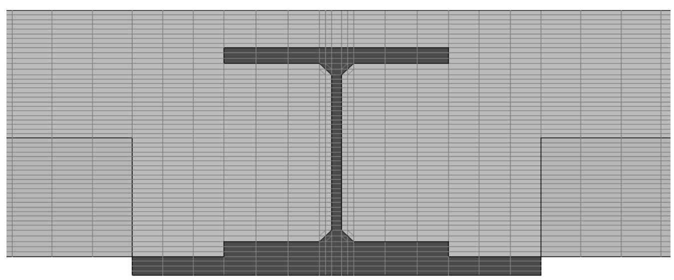


Fig 2.4.3.3: Numerical model of the cross-section

Table 2.4.3.2: Fire resistance times [min]

| Case No. | Section | L [m] | SFB | CoSFB |
|---|---|---|---|---|
| 1 | Cofradal200 HEM160 350*25 | 6 | (87) | 118 |
| 2 | | 8 | - | 79 |
| 4 | Cofradal260 HEB200 400*15 | 6 | (62) | 110 |
| 5 | | 8 | - | 61 |
| 7 | Cofradal260 HEM200 400*25 | 6 | 104 | 158 |
| 8 | | 8 | (77) | 93 |
| 9 | | 10 | - | 70 |

The following conclusions may be emphasized:

- considering the composite action, all CoSFB analysed cases respect the R60 criterion; further improvement, up to R90, may be obtained using supplementary reinforcement above the bottom plate of the steel profile;
- for all cases in which the SFB satisfies at least the resistance criterion for normal temperature design, all corresponding CoSFB cases respect at least the R90 criterion; further improvement, up to R120, may be obtained using supplementary reinforcement above the bottom plate of the steel profile;
- much higher values of fire resistance may be obtained, as case 7 demonstrates, but the section considered for this case represents an uneconomical solution for the given span of 6 m.

It may be concluded that the objective of the exploratory study, aimed to find simple "rule of thumb" recommendations for the assessment of the fire resistance of Slim Floor beams using proprietary Cofradal floor units, based on the results of the resistance verifications of the elements at ambient temperature, was attained. The results of the exploratory numerical investigation suggest that a simple rule may be applied to ensure a fire resistance time of 90 minutes for CoSFB: the design at normal temperature should be done considering only the plastic resistance of the steel profile, in which case also the displacement criterion is satisfied for CoSFB at normal temperature design. Superior fire resistances of 120 minutes may be obtained in these conditions considering supplementary reinforcement above the bottom plate of the steel profile.

Of course, the above considerations cannot be generalized based on of the very limited number of cases presented of this study. All these assumptions must be verified through an extended parametrical study, considering different spans, beam distances and profiles; further investigation will be done by the authors in this direction. This study is in progress, in cooperation with ArcelorMittal Luxembourg.

## 2.5 Advanced fire analysis using the natural fire concept

As shown in section 2.1, there are several fire models accepted by the European Standard EN1991-1-2 [3], which describe the thermal actions to be be considered for a structure under fire. The nominal models do not take into account any physical parameter, and can be far away from reality. An advanced approach is to use the natural fire concept, in which the pre-flashover phase and the cooling phase of the temperature – time curve are determined, function of the fire load density, dimensions and thermal characteristics of the fire compartment and other physical parameters. It has to be underlined that, even if natural fire models are included in the Eurocodes, the authorities does not accept easily fire models other than the standard ISO temperature-time curve. At international level, a lot of work is still performed in order to properly implement the natural fire concept in the design of civil engineering structures.

Section 2.5.1 presents the advanced fire analysis of a particular building: a high-rise rack supporting building structure. Due to the complexity of the structure, it is practically very difficult, if not impossible, to design it using the simple method in EN 1993-1-2. Therefore, an advanced analysis using specialized software was considered in this case. Moreover, taking into account the complexity of the structure, and based on previous study under standard ISO fire, which did not offered realistic results, a complex approach for the fire scenario using a natural fire scenario was considered, taking into account the pre-flashover phase and different physical parameters, concerning the combustible and the building itself. The activity presented in section 2.5.1 is sustained by the seventh publication in the list of publications selected by the candidate to be relevant for the professional achievements obtained after his PhD:

> **07 - Zaharia R.,** Franssen J.M. (2002) "Fire design study case of a high-rise steel storage building". Stability and Ductility of Steel Structures - SDSS 2002, Editor: Ivanyi M., Akademiai Kiado, Budapest, Hungary, ISBN 963-05-7950-2

Section 2.5.2 presents the evaluation of the fire resistance time for some unprotected steel moment resisting frames, in the hypothesis that they are already damaged by the earthquake, using advanced methods for earthquake and subsequent fire analysis, and using both standard and natural fire scenarios. The exceptional situation of a fire arising after an earthquake can cause substantially loss of life and property, added to the destruction already caused by the earthquake, and represents an important threat in seismic regions. The analysis under natural fire is based on a statistical approach, considering or not the possible fire fighting measures which may not be active after an earthquake. The activity presented in section 2.5.1 is sustained by the publications 8-9 in the list of publications selected by the candidate to be relevant for the professional achievements obtained after his PhD:

> **08** - Faggiano B., Esposto M., **Zaharia R.**, Pintea D. (2008) "Risk management in case of fire after earthquake – Structural analysis and design in case of fire after earthquake". Urban Habitat Constructions under catastrophic Events, Editors: Mazzolani F., Mistakidis E., Borg R.P., Byfield M., De Matteis G., Dubina D., Indirli M., Mandara A., MUseau J.P., Wald F., Wang Y., Malta University Publishing, Malta, ISBN 978-9909-44-40-2
>
> **09 - Zaharia R.,** D. Pintea (2009). "Fire after earthquake analysis of steel moment resisting frames". International Journal of Steel Structures (INT J STEEL STRUCT). 9(4). ISSN 1598-2351

### 2.5.1 Fire resistance analysis of high-rise rack steel storage structures

From the legal point of view, the classic rack systems included in a building with an independent resistance structure are usually not supposed to have an imposed fire resistance. These racks are considered only in order to evaluate the fire load density provided by the stored goods. More and more yet, high-rise rack supported building systems are constructed, as shown in Figure 2.5.1.1, in which the pallet racks are used to support the roof and walls of the building. For this type of warehouses, the racks become the resistant structure, so they have to be verified in case of fire.

The structure considered herein, is a steel storage building of this type from the Belgian company TRAVHYDRO, with a surface of $9168m^2$ for 30m high. There are 36 racks on the 160m length of the warehouse. Between the cross-aisle frames of the racks, horizontal elements are provided, in order to maintain the distance between the rails for the wagons of the automatic pallet transport system, as shown in Fig. 8.1. On the down-aisle direction, one rack has 60m length and is provided with 10 levels for pallet disposal, as Fig. 2.5.1.2 suggests.

Normally, for this type of industrial building, and accounting for the existence of the sprinkler system, a fire resistance time of 15 minutes is required. A special requirement for this building, in the hypothesis of a malfunction of the sprinkler system, was that the firemen may visit the building on their arrival, without danger. That means that, even if one rack of the structure falls down, the entire building must not present a progressive collapse phenomenon. Concerning the safety of the existing persons in the building, it must be emphasised that only 2 people are authorised to enter the building, for two hours, once a week. These people are trained for fire situations, know the building, and they may evacuate in less than 10 minutes from the moment of fire discovery.

Due to the complexity such structure, it is practically very difficult, if not impossible, to design it using the simple method in EN 1993-1-2. Therefore, an advanced analysis using specialized software was considered in this case. The mechanical analysis of the warehouse was realised with the SAFIR computer program.

A mechanical analysis under elevated temperatures, considering the standardised ISO fire in the entire fire compartment, was realised first, and showed that the 15 minutes of fire resistance criteria imposed for this type of industrial building is difficult to obtain. First, this is due to the low thermal massivity of the cold-formed profiles normally used to build rack systems. A second reason is that the partial load factors for fire accidental situation in case of storage systems are much higher than for other types of buildings. Last but not least are the important indirect effects due to dilation of such building systems, often continuous over 100m, and supposed to be uniformly heated in case of ISO fire.

Considering the complexity of the structure, a more realistic approach for the fire scenario is to consider a natural fire, taking into account the pre-flashover phase and different physical parameters, concerning the combustible and the building itself. In order to determine the temperature - time curves for the thermal analysis of cross-section, using the combined `Two Zone` and `One Zone` model, the computer program OZone, developed at the University of Liege, (Cadorin et al, 2003), was used. Function of the fire spread area and height of hot zone or temperature in the hot zone, a transition from the `Two Zone` model to `One Zone` model, with an uniform temperature in the entire compartment, may occur. The flashover moment is considered when the temperature in the compartment reaches $500^oC$.

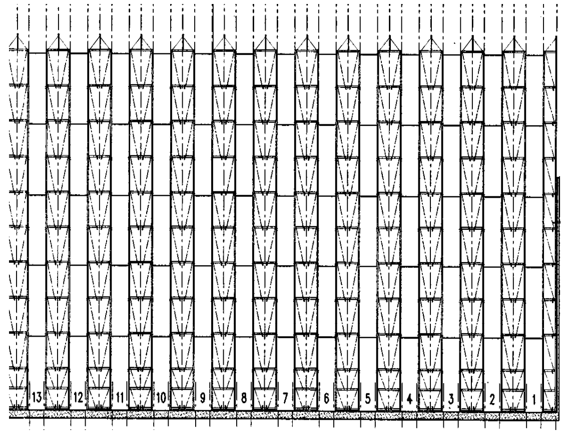


Fig. 2.5.1.1. Cross-aisle direction

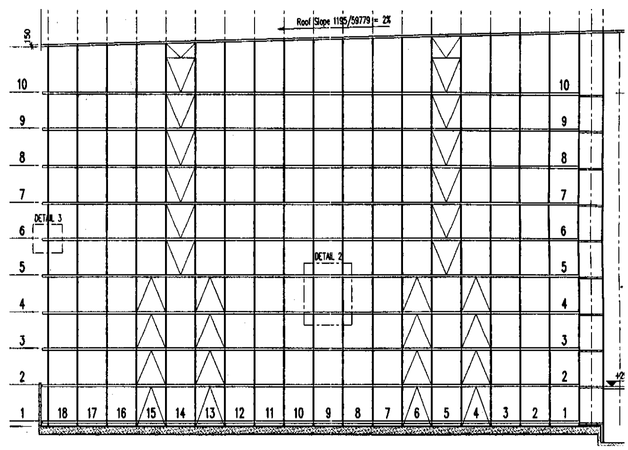



Fig. 2.5.1.2. Down-aisle direction

There are no important openings in the storage building. There are no windows, only normal doors for personnel access. Smoke outlets are provided in the ceiling, for 2% of the surface. The fire load density was evaluated to 8400 MJ/m$^2$, taking into account the combustible provided by the stored goods, on the 25 m height of the building, and the maximum rate of heat release is of 8620 kW/m$^2$, according to a previous study made by CTICM - France, Joyeux & Zhao (1999).

A very important parameter for the evolution of the temperature-time curve is the fire growth rate. Recent tests on rack fires made in The Nederland, at TNO, demonstrate that the fire growth rate is higher than 'Ultrafast', to which corresponds a constant of time of 75 seconds in a $t^2$ model. For this building, a time constant of 66 seconds was taken into account as the time to obtain 1MW of heat release, which represents the fastest fire growth rate obtained in the tests.

Considering these parameters, the temperature-time curve for the hot zone obtained with the ***OZone*** program is shown in Fig. 2.5.1.3. The compartment is under a two zone situation until 17 minutes after the fire beginning, when the interface of the two zones reaches less than 20% of the building height, so one of the criteria for one zone transition is fulfilled. The flashover phase is reached after 24 minutes.

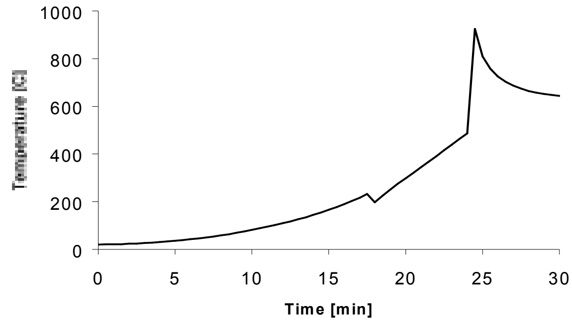



Fig. 2.5.1.3 Natural fire : Temperature - time curve for upper zone

After 10 minutes of fire, according to the ***Ozone*** results, the smoke height is around 20m and the temperature in the lower zone is less than 30$^{O}$C, so the smoke and the temperature do not endanger the people evacuation.

For the rack where the fire starts, the fire scenario supposes that a column and the adjoining rails are subjected to localised fire. The rest of the rack is under the influence of the two zones of temperature distribution, as Fig. 2.5.1.4 suggests.

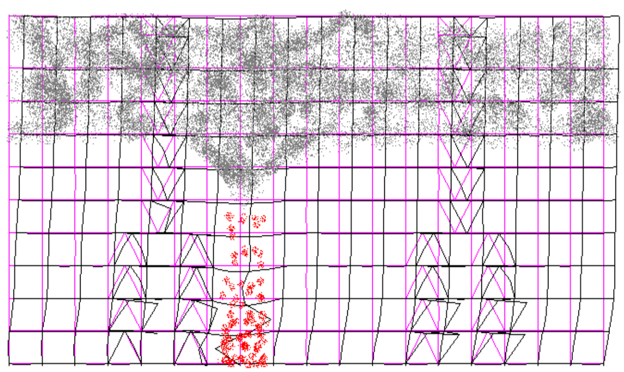


Fig. 2.5.1.4. Collapse of one rack under natural fire

Local buckling of elements from the bracing system is emphasised after less than 3 minutes, but this does not lead to the collapse of the entire rack. The numerical analysis demonstrates that the global collapse is produced after around 6 minutes, just after the collapse of the upright under localised fire.

For a fire scenario considering that the fire starts on the rack in the middle of the building, after this time it can be considered that the building is cut in two parts, as shown in Fig. 2.5.1.5. After the collapse of the first rack, it can be considered that the fire spreads to the two adjoining frames, and they are under the influence of a localised fire. The working hypothesis is that the collapse of a rack produces the breaking of the horizontal elements between cross aisle frames so it does not produce the collapse of adjoining frames. Therefore, in order to ensure that the chain collapse is avoided, it is recommended to design the horizontal elements connections to handle the necessary compression and tension efforts, due to the forces induced by the wind and by the movement of the wagons of automatic transport system, but to have poor resistance for relative displacements between frames in both vertical and horizontal directions.

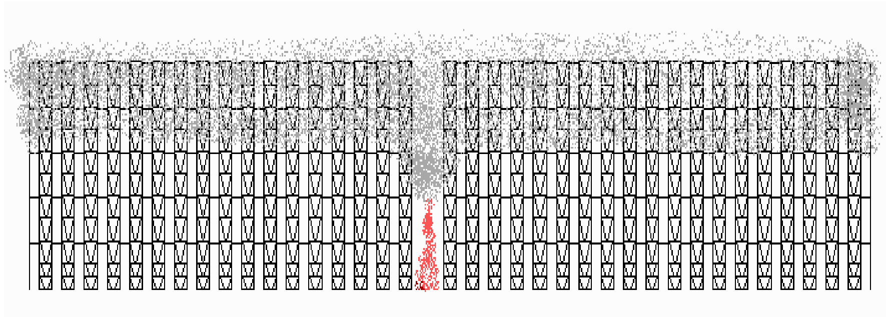


Fig. 2.5.1.5. Cross-aisle configuration after collapse of the first rack

Still concerning the horizontal elements between the cross-aisle frames, it must be taken into account that, after the collapse of the first rack, due to the temperature evolution in the upper zone, these elements expand, thus introducing horizontal forces in the remaining racks in the middle of the building. In the same time, these elements also begin to exhibit buckling, due to restraint caused by the increasing temperature, thus reducing the horizontal forces on the racks. An analysis of the cross-aisle direction of the building is then necessary, considering the expansion and the progressive suppression of the horizontal elements. The analysis using Two zone fire model for the entire building combined with the localised fire acting on the racks in the middle of the building emphasised that the collapse of the rack in localised fire, for the cross-aisle direction is produced still in around 6 minutes.

Thus, after 6 minutes from the ruin of the first rack, it may be considered that the collapse of the other two racks in the middle of the building is also produced. The localised collapse of the racks in the middle of the building may continue in the same manner, without producing progressive collapse, considering the breaking of the horizontal elements.

The global collapse of the entire structure corresponds to the moment of collapse of all other current racks, not under localised fire, but under the influence of the increasing temperature in the two zones. The time of the global collapse is more than 20 minutes.

It may be observed that the modified structure survives long enough to reach the time corresponding to the flashover phase. After this limit, it would be difficult to obtain more structural resistance, taking into account the high level of temperatures in the compartment.

The collapse of the first rack under localised fire, after 6 minutes, does not present a danger, because normally at this moment the persons which evacuate the building cannot be in the area of engulfed fire.

Considering the rescue service safety, which may arrive in less than 15 minutes after the fire beginning, the main risk for firemen who enter the building is the progressive collapse of the racks. The present study tends to indicate that no progressive collapse has to be feared as long as the flash over does not occur. When and if this happens, the structure is certainly in great danger of immediate collapse but, at this time, this is not anymore a real issue for the firemen, who should normally have evacuated the compartment.

The flashover may be avoided by the intervention of the fire brigade, which is supposed to limit the fire development. The fire scenario considered in the natural fire study does not consider the sprinkler system or the firemen action after their arrival.

### 2.5.2 Fire resistance analysis in extreme situations – fire after earthquake

Fire may be produced by many situations, but international experience confirms that earthquake is a major initiator of fires (Wellington Lifelines Group, 2002). As shown in (Buchanan, 2001), the loss resulting from fires developing after the earthquake are comparable to those resulting from the earthquake itself. Fires following earthquakes were highlighted in the last decades by the major earthquakes of Northridge, Los Angeles, USA (1994) and Kobe, Japan (1995). The Northridge earthquake from 1994 resulted in fires which challenged the resources of the fire service due to the number of fires, disruption of fire supply and damage to fire protection systems within buildings (Todd et al., 1994). The Los Angeles City Fire Department responded to over 2200 emergencies in the day of earthquake (David et al., 1997).

An appropriate risk management is therefore necessary for a fire after earthquake situation. The general approach with regard to catastrophic events has consisted mainly in operating on relief programs. In recent years, however, considering the catastrophic consequences of a fire following earthquake, for example, relief measures after impact have become increasingly inadequate to protect personal or community assets, safeguard and economic investments. A new approach is felt, which should be aimed at identifying problems before they happen, by means of systematic process of analysis of risk and decisions about its acceptability (Faggiano et al., 2008a). The fire following earthquake risk management requires an approach at two different scales: a local scale, referred to the building itself (building scale), and a global scale, referred to a whole region (regional scale).

Concerning the regional scale of the problem, models for post-earthquake fire hazard in urban regions were developed since 1986 by Scawthorn (1986, 1987) and by Scawthorn et al. (1991). These models are applicable to both specific earthquakes and for determining annual expected losses on a probabilistic basis, including parameters as building density, wind velocity, deterioration of fire-fighting response and seismic intensity.

A significant research was done by Cousins et al (1991), concerning the losses due to post-earthquake fires in central New Zealand. More recently, a model of the spread of post-earthquake fires was elaborated for Wellington City, by Cousins et al. (2002). A study done by Botting (1998) suggested various ways in which the fire protection and the fire engineering measures may reduce post-earthquake fire losses in urban regions. The research was based on the analysis of fifteen major earthquakes and recommendations were given on the fire brigade response, urban water supplies and urban macro-scale fire protection.

The life-safety risk for post-earthquake fires from the perspective of gas and electricity distribution systems was investigated by Williamson and Groner (2000). The study based on fire scenarios relating to the various aspects of gas and electricity distribution systems, concluded that the old residential buildings susceptible to structural damage and potential collapse pose an increased risk due to the concurrent fire hazard. In relation with the lifelines services, a study by Robertson and Mehaffey (2000) recommends that performance based building codes should contain a framework to prevent undue reliance on sprinkler and other life safety systems that are dependent on seismically vulnerable water and electrical services.

A book published by ASCE (Scawthorn et al, 2005) gives the most comprehensive view on fire following earthquake in urban regions, covering the history of past post-earthquake fires, models for fire spread in post-earthquake urban environment and cost effectiveness of various fire after earthquake mitigation strategies.

Concerning the building scale aspect of the problem, the fire safety aspects related to a building must be first emphasised. According to the Directive of the European Commission issued on 21 December 1988, the construction works must be designed and built in such way that in the event of an outbreak of fire, the load bearing capacity of the construction can be assumed for a specific period of time, the generation and spread of fire and smoke within the works and to neighbouring construction works are limited, occupants can leave the works or be rescued by other means and the safety of rescue teams is taken into consideration.

When an earthquake occurs, it may cause structural and fire protection damage, and thus the building is more vulnerable to a fire. In the same time, after an earthquake, the loss of water supply or the low water pressure, combined with multiple independent fires, traffic congestion and the limited resources which are not able to respond promptly to all fires, allow to the fire to spread. According to a study made by All-Industry Research Advisory Council (1987), the experience of the impact of fire following earthquake indicates that at wind speeds above 9m/s, the fire spread and associated loss increase dramatically.

After an earthquake, it is supposed that the people already evacuated the building, as usually happens in case of such an event, or are more prone to evacuate in case of a fire resulting after the earthquake. On the other hand, concerning the rescue teams, in case of a fire after an earthquake, for the reasons shown above, the increased time needed for firemen to reach the building in fire, associated to the possible lack of the active fire measures and to the increased vulnerability of the building may affect their security.

Even in case of a latter post-earthquake fire (not immediately after the seismic event), the effect on the structure must be adequately taken into account, since the earthquake-induced damage makes the structure more vulnerable to fire effects than the undamaged one. Modern seismic design accept a certain level of damage of the non-structural and structural members of a steel structure, often non-visible after an earthquake, and only an appropriate post-earthquake expertise may evaluate this aspect .

The design of structures prone to fire following earthquake hazard is oriented at integrating structural fire safety into the design of structural systems. This goal can be successfully pursued in a performance based design framework. A performance based procedure applied to single buildings was proposed by Chen et al (2004) and consists in four major steps: hazard analysis, structural and/ or non-structural analyses, damage analysis and loss analysis. Three kinds of damage should be considered: damage of the structure, damage of the fire protection of structural members and damage of fire protection of non-structural members. Re-evaluating the fire hazard is also very important, because the damage to the fire protection systems may affect the development of the fire hazard.

An experimental study by Collier (2008) showed that the post-earthquake performance of some passive fire protection systems is definitely reduced when subjected to a design level earthquake. The amount of reduction in fire resistance of a 60 minute plasterboard lined wall (non-structural member) can be as much as 50%.

The effect of the damage by mechanical actions (impacts, earthquake, explosions) of the fire protection on the fire resistance of steel structural members was studied by means of experimental and numerical research by Ryder et al (2002), Fontana and Knobloch (2004), Pessiki et al. (2006), Li and Wang (2008). Considering the fire protection demands for steel buildings in seismic areas, Vayas et al. (2006) showed that very considerable cost savings may be achieved for seismic designed steel buildings, if the fire protection is designed in accordance to the overall structural integrity.

In recent years, some studies on the behaviour of steel structures with no fire protection, damaged by earthquakes and exposed to fires, have been carried out. A state-of-the-art of was presented by Faggiano et al. (2008b).

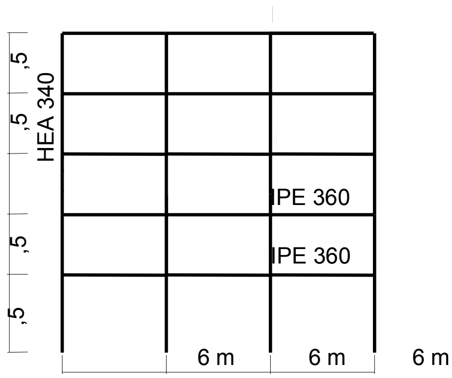
The evaluation of the fire resistance rating reduction of some unprotected steel frames function of the seismic intensity was evaluated by Della Corte et al. (2003), by means of standard ISO fire analysis on the structural configurations, geometrically distorted function of the maximum residual inter-storey drift resulted from the seismic analysis.

Using a numerical tool able to consider the thermal and mechanical aspects of the problem, similar studies on simple unprotected portal frames were made by Faggiano et al. (2007). The study evaluated the fire resistance of the portal frames in relation with the dimensions of the structure and overstrength ratios for the steel.

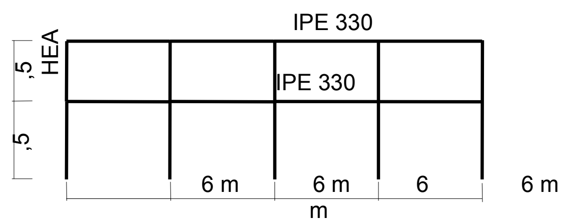
In the candidate’s study presented in the following, the fire resistance of some unprotected moment resisting framed steel structures already damaged by the earthquake is evaluated, using advanced methods for earthquake and subsequent fire analysis, including the natural fire approach. The influence of the damage level induced by the earthquake on the fire resistance is emphasized.

**Analysed structures**

The moment resisting steel plane frames considered for the present study have the dimensions given in Fig. 2.5.2.1 a,b. The structures are made using European steel profiles of S235 steel grade and all beam-to-column connections are considered rigid. Both frames were dimensioned for the same fundamental load combinations of actions (4 kN/m$^2$ for dead load and 2 kN/m$^2$ for the live load of the current storey, 3.5 kN/m$^2$ for dead load and 1.5 kN/m$^2$ for the live load of the top storey, 0.5 kN/m$^2$ for the wind action).




a) Frame A




b) Frame B

Fig. 2.5.2.1 Frame characteristics

The frames were further verified for two seismic regions in Romania, with different ground motions: a near–field type (Banat region) and a far-field type (Vrancea region). The design was made according to the Romanian seismic code (P100-1/2004, 2005), adapted from the corresponding European standard EN1998 (2005). The elastic spectral analysis was applied considering the response spectrum for the Romanian Banat region (moderate seismic area with the design peak ground acceleration ag=0.16g and control period Tc=0.7 seconds) given in Fig. 2.5.2.2, and for Vrancea region (severe seismic area with the design peak ground acceleration ag=0.32g and control period Tc=1.6 s) given in Fig. 2.5.2.3.

The design of the Frame A - Banat structure was governed by the fundamental load combination (no changes in elements dimensions after the seismic design verification). For all other cases (Frame A – Vrancea and Frame B – Banat and Vrancea) the design of the structures was governed by the seismic combination. Fig. 2.5.2.1 shows the steel sections of both frames. The values in parenthesis represent the profiles used for Vrancea structures, which resulted with stronger beams for some levels and with stronger columns on the height of the building, due to the higher seismic demand.

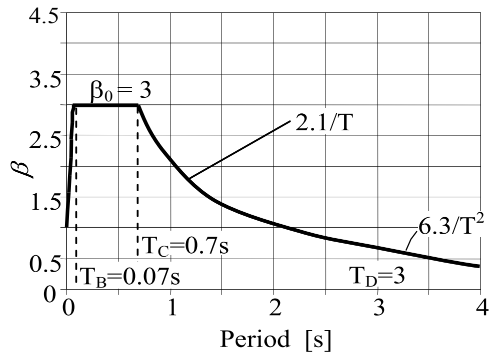



Fig. 2.5.2.2. Normalised elastic response spectrum for Banat region

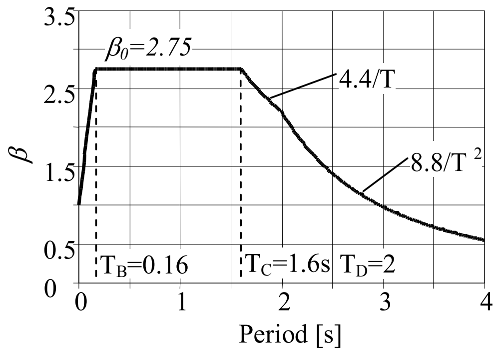



Fig. 2.5.2.3. Normalised elastic response spectrum for Vrancea region

The seismic response of the structures was evaluated using a pushover analysis, while the displacement demand under the corresponding seismic event was determined using the N2 method, developed by (Fajfar 2000), and implemented in EN1998 (2005). This method combines the push-over analysis of a multi-degree of freedom model (MDOF), with the response spectrum analysis of a single degree of freedom system (SDOF). The push-over analysis was performed based on the assumption that the response is governed by the fundamental mode of vibration.
According to the N2 method, the seismic demand spectrum is determined for an equivalent SDOF system. The push-over curves were obtained for the MDOF systems, and therefore it is necessary to determine the simplified force – displacement characteristic for the equivalent SDOF systems. The performance point for each SDOF system is defined by the intersection of the capacity curve and the demand spectrum. The displacement demand for the SDOF model, Sd, is then transformed into the top displacement Dt of the MDOF system, called the target displacement.

Figures 2.5.2.4 to 7 show in a graphical form the procedure used to determine the displacement demand of the SDOF systems. The Banat Frame A remains elastic after the occurrence of the corresponding earthquake. The Banat frame A was dimensioned from the fundamental load combination, being sensitive to the horizontal wind action, and the steel sections remained the same after the verification for the code seismic action. Frame A - Vrancea responded to the seismic motion in inelastic range, experiencing maximum interstorey drifts of 2.7%, slightly larger than the 2.5% limit corresponding to “Life safety” performance level according to the informative classification given by (FEMA 356, 2002). This means that the structure is expected to present important damages of non-structural elements and moderate damages of structural elements, but the safety of the people is guarantied. The same performance level was attained for Frame B (1.8% maximum drifts for Banat frame and 2.2% maximum drifts for Vrancea frame).

Consequently, after the earthquake code, the Frame A - Banat structure remains undamaged, while for the other structures in fire analysis, two hypotheses will be considered:

- a lower intensity earthquake occurs and the structure remains undamaged;
- an earthquake with the intensity given by the Romanian code for Banat and Vrancea regions occurs and the structures suffer the damage determined by the above procedure.

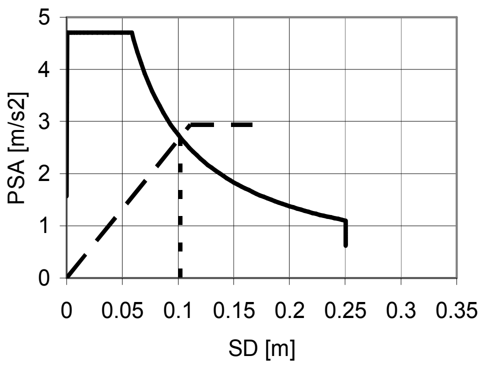


Fig. 2.5.2.4. Seismic demand spectra vs. capacity diagram for Frame A - Banat

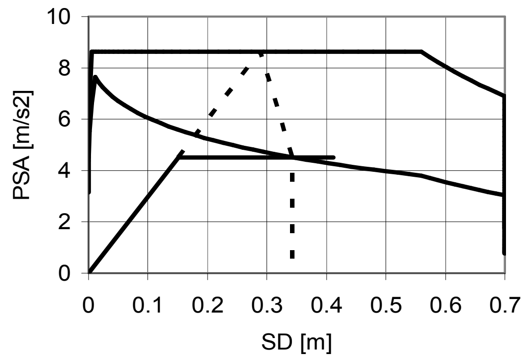


Fig. 2.5.2.5. Seismic demand spectra vs. capacity diagram for Frame A - Vrancea

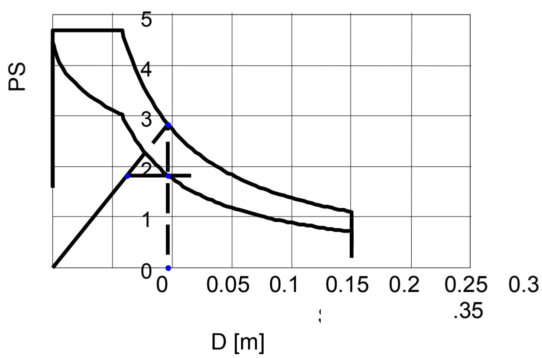


Fig. 2.5.2.6. Seismic demand spectra vs. capacity diagram for Frame B - Banat

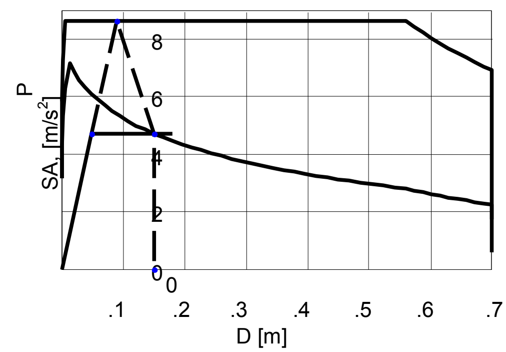


Fig. 2.5.2.7. Seismic demand spectra vs. capacity diagram for Frame B - Vrancea

**Fire analysis**

The fire scenario for a post-earthquake fire may consider a combined `Two Zone` - `One Zone` natural fire model, with the design fire load density calculated considering or not the active fire fighting measures, which could be all available in a normal fire situation, but could be partially available immediately after the occurrence of an earthquake. For the calculation of the fire resistance of the considered structures, the SAFIR computer program (Franssen, 2005) was used.

***Thermal analysis under ISO fire***

The fire was applied only for the unprotected columns and beams of the first storey of the frames, in the hypothesis that the ground floor represents the fire compartment. On the beams, the fire was applied on three sides (the top being protected by the concrete slab). In the structural analysis, the collaboration between the steel beam and the concrete slab was not considered. Figure 2.5.2.8 shows the temperature distribution in ground floor IPE400 beams exposed to fire on three sides after 15 minutes of ISO fire. It may be observed that the zone with highest temperature is the web, which is has the lowest thickness on the cross section. The lower flange exposed on all sides to the fire has temperatures around 600$^o$C, while the upper flange, exposed on three sides has lower temperatures, around 500$^o$C.

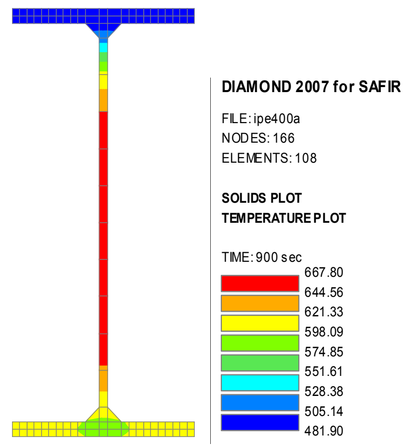



Fig. 2.5.2.8. IPE400 Beam exposed on 3 sides – ISO fire

Figures 2.5.2.9-10 show the temperature distribution in ground floor columns of Banat frame A (HEA 340 profile) and of Vrancea frame A (HEA 500 profile) exposed to fire on four sides, after 15 minutes of ISO fire. It may be observed that the HEA500 column profile of Vrancea frame, having a higher massivity (lower section factor), presents lower temperatures in the flanges and in the web, compared to the temperatures in the HEA340 column profile of Banat frame, after 15 minutes of ISO fire.

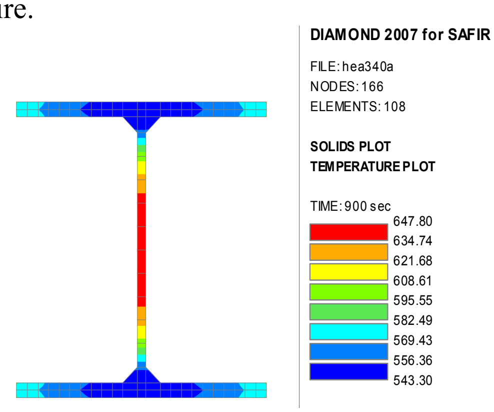



Fig. 2.5.2.9. HEA340 profile exposed on 4 sides – ISO fire

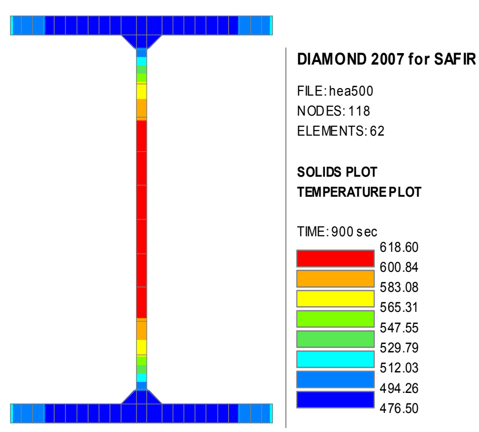



Fig. 2.5.2.10. HEA500 profile exposed on 4 sides – ISO fire

***Thermal analysis under natural fire***

Several assumptions must be made when dealing with a natural fire scenario, like the maximum fire area, the fire load, and the surface of the openings.

The most challenging is to establish how the windows will behave in a fire situation (Zaharia et al., 2007). The rate of heat release of fires is limited by the flow of oxygen available to it. In all except very rare circumstances, the flow of oxygen into a room comes largely from open doors and open windows and to a slight extent from any mechanical ventilation systems and from building leakage. Once a fire gets going, however, windows previously closed may crack and break out. The results will often be drastically different, depending on whether the windows break or not. Thus, it becomes of significant interest to be able to predict if, and when, glass may break out.

Here, an important distinction needs to be made. When a window pane of ordinary float glass is first heated, it tends to crack when the glass reaches a temperature of about 150 - 200ºC. The first crack initiates from one of the edges. At that point, there is a crack running through the pane of glass, but there is no effect on the ventilation available to the fire. For the air flows to be affected, the glass must not only crack, but a large piece or pieces must fall out.

Understanding the conditions under which pieces actually fall out has been of considerable interest to fire specialists. Since the fire ventilation openings need to be known in order for fire models to be used, glass breakage has been of special interest to fire design engineers. This has prompted a number of theoretical and simplified studies and a few empirical ones as well. It is worth to mention here the work done by Roytman (1975), Hassani et al. (1994, 1995) and Shields et al. (2001, 2002). The only probabilistically-based results concerning glass exposed to a uniform hot temperature come from the Building Research Institute of Japan (Tanaka et al. 1998). In this research, enough tests were run so that a probability graph could be plotted, and these results were used in the present natural fire study.

For the plane steel moment resisting frames presented in chapter 2, Frame A was considered as part of a structure of 18mx18m in plane, while Frame B was considered as part of a structure of 24mx24m. The walls are made out of normal concrete having a thickness of 20 cm, and the following thermal characteristics: conductivity 0.8 W/mK and specific heat of 840 J/kgK. As shown in Fig. 2.5.2.11 the windows (openings) in three adjacent walls have a sill height of 1 m and a soffit height of 3.5 m. In the fourth wall the sill height is 2m and the soffit height is 3.5m. A linear variation of the openings was considered, i.e. of the glass panes. At 300°C, 30% of the windows were considered broken, while at 500°C all the windows are broken, based on available research results mentioned above.

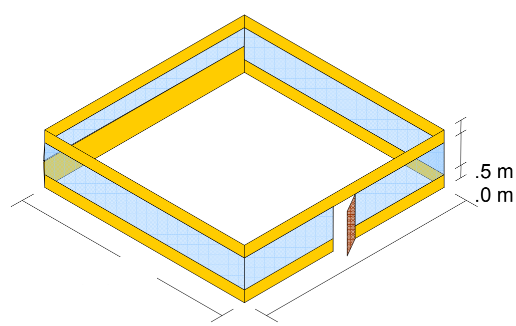



Fig. 2.5.2.11. Fire compartment

The occupancy of the fire compartment is office, with a characteristic fire load density qf,k of 511 MJ/m$^2$. The design fire load density, according to Annex E in EN1991-1-2 (2005) is

$$q_{f,d} = q_{f,k} \cdot m \cdot \delta_{q1} \cdot \delta_{q2} \cdot \delta_n$$

in which m is the combustion factor, $\delta_{q1}$ is a factor taking into account the fire activation risk due to the size of the compartment (1.51 for frame A and 1.65 for frame B), $\delta_{q2}$ is a factor taking into account the fire activation risk due to the type of occupancy (1.00 for both frames - offices) and $\delta_n = \prod_{i=1}^{10} \delta_{ni}$ is a factor taking into account the different active fire fighting measures i (sprinkler, detection, automatic alarm transmission, firemen, etc.).

Table 2.5.2.1 gives the values of the active fire fighting measure factors considered. Before an earthquake, the building being provided with sprinklers, the coefficient which takes into the account the existence of automatic water extinguishing system (δ1) and the coefficient which takes into account the possible existence of the independent water supplies (δ2) are both sub unitary. For a fire situation immediately after an earthquake, considering the possible disruptions, the sprinkler system and the automatic fire detection are no more considered, and the corresponding coefficients have the unit value. In relation with the prompt intervention of the fire brigades, which is no more possible due to the number of emergencies and traffic congestion, associated to the possible lack of the active fire measures, the coefficients δ5-9 are considered with the maximum value.

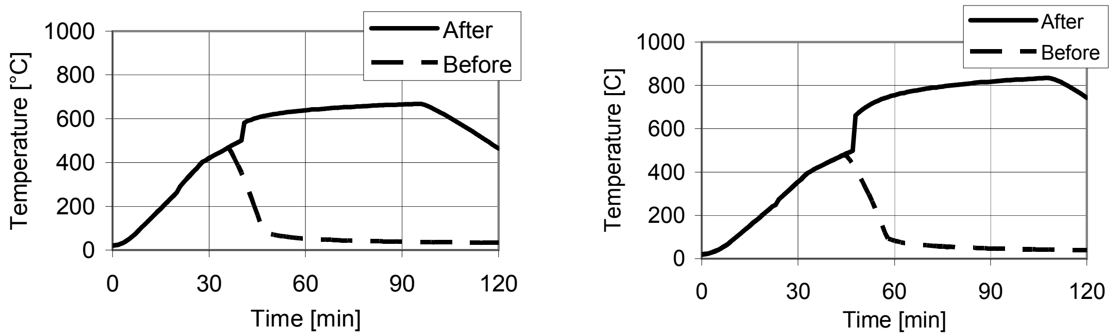
Table 2.5.2.1. Fire fighting measures before and after the earthquake

| **Fire Scenarios** | **Autom. Water Exting.** | **Indep Water Supply** | **Auto Fire Detection** | **Alarm Fire Brigade** | **Fire Brigade** | **Access Routes** | **Fire Fight Devices** | **Smoke Exhaust** | **Total** |
|---|---|---|---|---|---|---|---|---|---|
| | $\delta_1$ | $\delta_2$ | $\delta_{3/4}$ | $\delta_5$ | $\delta_{6/7}$ | $\delta_8$ | $\delta_9$ | $\delta_{10}$ | $\Pi\delta_n$ |
| Before | 0.61 | 0.87 | 0.73 | 0.87 | 0.78 | 1.0 | 1.0 | 1.0 | 0.26 |
| After | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.5 | 1.5 | 1.0 | 2.25 |

Using these parameters and running the Ozone software (Cadorin et al., 2003), two fire curves were produced for each building (Fig. 2.5.2.12). For both buildings, the “before earthquake” curves are fuel controlled, having a lower design fire load density, and no flashover occurs. The “after earthquake” curves are ventilation controlled. The peak temperatures for Frame B are higher than those of the Frame A, due to the higher design fire load density and to the size of the compartment.

These curves were used in SAFIR programme to find the temperature evolutions on each of the exposed profiles, without fire protection. As in case of standard ISO fire, the fire was applied only for the elements of the ground floor, and on the beams the fire was applied on three sides.

Some remarks regarding the application of the equation in a post earthquake situation may be mentioned. The risk of fire activation (and, hence, $\delta_{q1}$) could be significantly higher after an earthquake than in a normal situation. On the other hand, the values of this factor have been determined in a normal situation for the probability that a fire starts in the whole live of the building, whereas the duration of the "dangerous" situation after the earthquake is much shorter. Also, the factors of Eq. 4 have been calibrated in order to achieve a certain target value for the probability of failure from a fire for a normal building. It may be more realistic to accept a higher probability of failure of a building from a fire after an earthquake, considering that the structure is already damaged.




a) Frame A b) Frame B

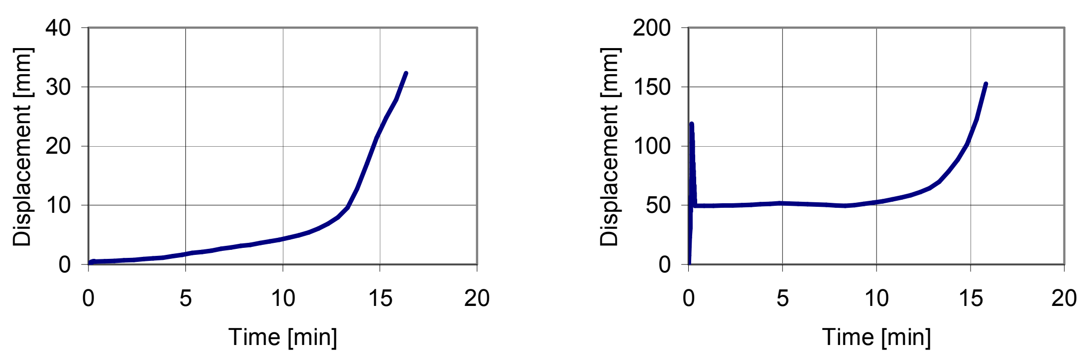
Fig. 2.5.2.12. Temperature-time evolution

**Structural analysis**

Figure 2.5.2.13 shows the response of both damaged and undamaged Frame-B Banat structure under ISO fire, in terms of top horizontal displacement – time characteristics.

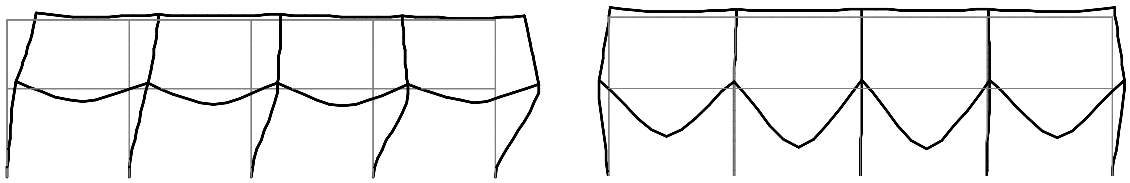
The analysis procedure for damaged structures is shown in figure 2.5.2.13b for the damaged Frame B – Banat under ISO fire. The structure, subjected to vertical loads corresponding to the fire load combination, is loaded with the lateral forces (push-over, as described previously) up to the target displacement for the MDOF system, determined using the N2 method. The structure is then discarded of the lateral loads and, because the frame responds in the inelastic range, it presents residual displacements and residual stresses. At this stage of structural damage, starts the fire analysis under vertical loads corresponding to the fire load combination.

Two types of collapse modes were observed during the fire analysis using standard or natural fire: a global mode and a mode characterised by the collapse of the beams. For all fire analyses, frame A presented a global collapse mechanism, while frame B presented both modes, as shown in figure 2.5.2.14.




a) Undamaged structure b) Damaged structure

Fig. 2.5.2.13. Displacement-time characteristics for Frame B – Banat under ISO fire



a) Global b) Beam

Fig. 2.5.2.14. Collapse mechanisms for frame B

Considering all fire fighting measures active (in a situation before an earthquake) both frames, designed for the two seismic regions, resist to the fire action. Therefore, no collapse is produced for “before earthquake” natural fire scenario, for which no flashover occurs. Table 2.5.2.2 summarizes the fire resistance times and collapse modes for the standard ISO and natural “after earthquake” fire scenarios for each frame.

It may be observed that there are differences of time resistance between the undamaged structures (before earthquake, or for an earthquake of lower intensity than the code seismic action for the corresponding region) and the damaged structures (for a code earthquake). The differences in fire resistance times between the damaged and undamaged structures are affected by the damage level. Under ISO fire, the differences are ranging from around 5% for the Banat frame B (experiencing maximum inter-storey drifts of 1.8% in the inelastic range), 11% for the Vrancea frame B (experiencing maximum inter-storey drifts of 2.2% in the inelastic range), to around 21% for the Vrancea frame A (experiencing maximum inter-storey drifts of 2.7% in the inelastic range). In case of frame B for Banat region under natural fire, the difference is lower, but it is to be also taken into account that, for the damaged and undamaged structures, the collapse mechanism is different. In case of frame B for Vrancea region under natural fire, for both damaged and undamaged structures, the collapse mechanism is local (beam) and the fire resistance time is not influenced in a significant way by the damage of the structure.

Table 2.5.2.2. Fire resistance times and collapse modes

<table>
<tr><td rowspan="2">ISO fire</td><td colspan="2">Frame A</td><td colspan="2">Frame B</td></tr>
<tr><td>Banat</td><td>Vrancea</td><td>Banat</td><td>Vrancea</td></tr>
<tr><td>Undamaged</td><td rowspan="2">16’ 10”<br>global</td><td>26’ 10”<br>global</td><td>16’ 30”<br>global</td><td>26’ 20”<br>global</td></tr>
<tr><td>Damaged</td><td>20’ 40”<br>global</td><td>15’ 40”<br>global</td><td>23’ 20”<br>global</td></tr>
<tr><td rowspan="2">Natural fire<br>(after earthquake)</td><td colspan="2">Frame A</td><td colspan="2">Frame B</td></tr>
<tr><td>Banat</td><td>Vrancea</td><td>Banat</td><td>Vrancea</td></tr>
<tr><td>Undamaged</td><td rowspan="2">74’ 00”<br>global</td><td>no collapse</td><td>56’ 20”<br>beam</td><td>71’ 40”<br>beam</td></tr>
<tr><td>Damaged</td><td>no collapse</td><td>54’ 40”<br>global</td><td>71’ 20”<br>beam</td></tr>
</table>

For both structures and under both fire scenarios, the Vrancea frame, designed for stronger seismic action, presents higher fire resistance times than the corresponding structures designed for the Banat region. Moreover, in case of frame A, for a natural fire scenario after earthquake, the stronger Vrancea frame resists the fire, even if the structure is damaged after the seismic action, while the Banat frame collapses, even if its structure remains undamaged after the code earthquake.

Therefore, it must be underlined that the structures designed for seismic action (or for stronger seismic action) have an important reserve of resistance into a fire situation.

It may be concluded that the study emphasised that there are differences in time resistance under standard and natural fire, between the undamaged structures and the damaged structures, in case of a fire after an earthquake. The fire resistance time of the damaged structures is influenced by the damage level. The highest differences in terms of fire resistance appear between the damaged and undamaged structures experiencing a global type of collapse mechanism. For the fire scenarios with all fire fighting measures available in a regular fire situation, both structures resisted to the fire action (considering all measures available, the fire does not reach flashover). The structures adapted for seismic action, or designed for stronger seismic action, have an important reserve of resistance in case of a fire after an earthquake, but also in case of a regular fire situation.

# 3. DESIGN ASSISTED BY TESTING

## 3.1 Introduction

In engineering field in general and in the civil engineering field in special, experimental tests represent an absolutely necessary investigation method in view of developing new products and solutions or to evaluate the performances of the existing ones. Despite the fact that some advanced calculation methods exist at present time, enabling to simulate numerically structural features (as for example the Finite Elements Method), the complexity of practical problems does not allow for the exclusive use of numerical methods in evaluating the structural performance. The optimum solution is generally represented by the use of both experimental methods and numerical simulation which complete each other.

A special field in civil engineering is represented by the behaviour of structures under exceptional loading, as for example earthquake load. Compared with other types of loads (dead load, live load, wind, snow) the earthquake load occurs relative rarely in structure life but with disastrous consequences on the building. This is caused by the fact that seismic the design of structures has started relatively late. The first codes for seismic design, including provisions close to the modern ones have appeared in the years 1970 (Priestley, 1997). In Romania, the first compulsory anti-seismic design code was issued in 1963 (P13-63). That is why, most of the buildings designed and erected before the introduction of modern seismic design codes are under a high risk to suffer major damage as result of a seismic action and definitely need consolidation / refurbishment. Even structures designed to modern seismic codes may suffer important damages under earthquake as a result of the fact that, by economical reasons, they are designed for lower seismic load than those corresponding to elastic behaviour. In this situation, a structure is able to survive to a major earthquake owing to its ductility, which implies large deformations in post elastic range. Some particular constructions (as hospitals, fire-stations, etc) have to remain in operation after earthquake. In many cases, special technologies are used as for example base isolation or supplementary systems to dissipate energy and diminish structural damage caused by earthquake.

In all cases hereby presented (refurbishment of existing constructions, provision of ductility for the new buildings and base isolation techniques as well as energy dissipation devices) the structural elements and the structures exhibit a complex behaviour, which compulsory implies the use of experimental techniques to evaluate existing structures performance and the development of new seismic-resistant systems. Furthermore, the experimental tests are serving as support for the development of new seismic design codes. Romania is a country with high seismic risk, so that construction of new structures, as well as rehabilitation of existing constructions cannot be accomplished efficiently without adoption of modern structural solutions, using high-performance materials and technologies.

Depending on complexity level and required technical and material resources, there are several types of experimental tests which may be performed in the laboratory, the most important being:

-static monotonic tests;
- quasi static monotonic and cyclic tests;
- pseudo-dynamic tests;
- shaking table tests.

The static monotonic tests are the simplest an generally consist in applying a force by means of hydraulic cylinders. They are generally representative for structural behaviours under actions which might be considered of static type. However, this type of testing is not adequate for seismic performance evaluation, since it is not able to describe the dynamic, cyclic and inelastic response of the structure.

The quasi-static monotonic and cyclic testing consists of a slow (quasi-static) application of the load, with deflection control, either monotonic or cyclic. In comparison with the static tests, those performed in cyclic quasi-static mode have the advantage of inducing alternating inelastic deflections in the specimens, thus reproducing more precisely the real stress level within tested elements. However, the quasi-static cyclic testing is not able to reproduce two of the most important structural features under seismic action. The first of these is the strain rate, much lower in the experimental case than in reality, and which is neglecting the dynamic effect of the seismic action. The second aspect is the deflection history, which in case of a real earthquake has a random character (aspect) and does not increase following a progressive predetermined pattern.

The shaking table is an experimental device built of a platform, moved by hydraulic actuators, which are able to induce a movement similar to that observed during earthquakes. However, some drawbacks of this solution still exist: owing to the high cost, most vibrating tables do not allow for natural scale testing, and the short time required to perform this testing makes difficult an accurate observation of structural response.

An investigating method to assess in a better way the seismic response of structures at natural scale is the *pseudo-dynamic test*. The pseudo-dynamic tests represent a modern and innovative method for experimental evaluation of seismic response of structures. The concept was proposed for the very first time by Hakuno et al (1969) followed by Takanashi et al. (1974), but only in the last decades, laboratories housing the complex and expensive infrastructure and equipments, necessary to perform such tests, were built. In principle, a pseudo-dynamic test consists of combining a quasi-static test at natural scale with computer simulation of the dynamic effect of the seismic action (see figure below) The basic hypothesis on which the pseudo-dynamic test is built consists in the possibility to describe the dynamic response of a structure based on a model with a finite number of dynamic degrees of freedom (DOF).

The equation of motion of a system with several degrees of freedom may be expressed by a differential equation of the type:

$$M \cdot a(t) + C \cdot v(t) + r(t) = M \cdot I \cdot a_g(t)$$

where M and C are the mass matrix, and damping matrix respectively, a(t) and v(t) the acceleration and velocity vectors, r(t) is the restoring forces vector while $a_g(t)$ is the accelerogram representing the seismic action.

I

In order to simulate the seismic action on the experimental model, the numerical model is analysed under the action of an accelerogram representing the seismic motion to which the structure is subjected. The result of this analysis obtained by numerical integration of the equation of motion consist into the displacements x(t) along the considered dynamic degrees of freedom. These displacements are applied on the tested structure by means of servo-controlled hydraulic actuators connected to a reaction wall (see Fig. 3.1.1). The force transducers of the actuators record the forces generated into the structure as a result of applying the imposed displacements and the information is transmitted to the computer with the purpose of integrating the equation of motion in the next step (Taucer and Franchioni, 2004). Owing to the fact that the inertial forces are numerically modelled, a pseudo-dynamic test is not performed in real time. The actuator control is performed in quasi-static regime which allows for the use of relatively small sources of hydraulic power.

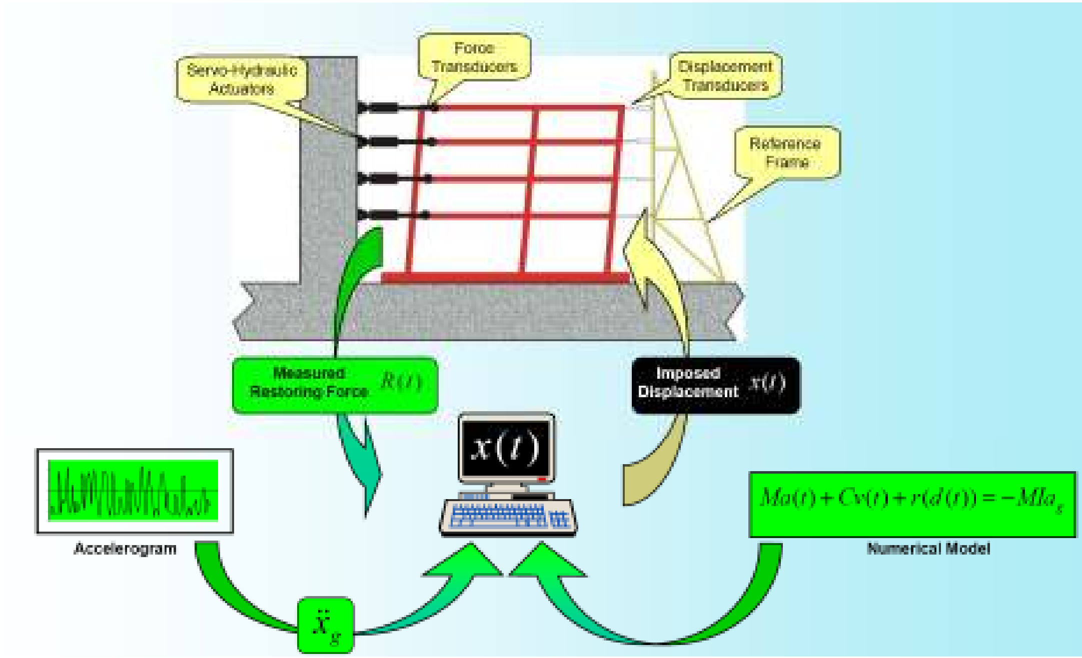



Fig. 3.1.1 Pseudo-dynamic test

In order to perform pseudo-dynamic testing, the laboratory must be equipped with a reaction wall and a strong floor. A reaction wall is an experimental device which allows performing experimental tests on structures at natural scale. This is used almost exclusively together with a strong floor. The reaction wall serves to support actuators witch apply load on the structure, while the strong floor serves to support the structure itself. The reaction wall should have a considerably higher resistance than the tested structures while its deflections should be several orders of magnitude lower than those measured on the structure itself. To allow for connecting the actuators on the reaction wall and the structure on the strong floor, these are provided with a system of holes located in a rectangular pattern at intervals between 0.25 and 1.0 m (Taucer and Franchioni, 2004). The reaction wall and the strong floor are offering a large flexibility in possibilities to perform quasi-static and pseudo-dynamic testing. Fig. 2 shows the impressive dimensions and capabilities of the pseudo-dynamic experimental facility of the European Laboratory for Structural Assessment –ELSA, of the Joint Research Centre of the European Commission in Ispra, Italy. ELSA laboratory is the biggest one in Europe for pseudo-dynamic testing and real-scale buildings testing.

The candidate won an European Commission's Research Grant of two years (February 2003- Fedbruary 2005), at this laboratory. During this period, the candidate was involved in both numerical and experimental research within a project initiated by the European Commission, aimed to study the seismic behaviour of concrete flat-slab buildings. The experimental programme included the pseudo-dynamic test of a real-scale building. A summary of this research, is presented in section 3.2. The activity presented in this section is sustained by the last publication in the list of publications selected by the candidate to be relevant for the professional achievements after he obtained his PhD:

**10 - Zaharia R.,** Taucer F., Pinto A., Molina J., Vidal V., Coelho E., Candeias P. (2006) "Pseudodynamic earthquake test on a full-scale RC flat-slab building structure". European Commission, Joint Research centre, Institute for the Protection and Security of the Citizen, European Laboratory for Structural Assessment – ELSA, Publication No EUR 22192EN, European Communities, Printed in Italy

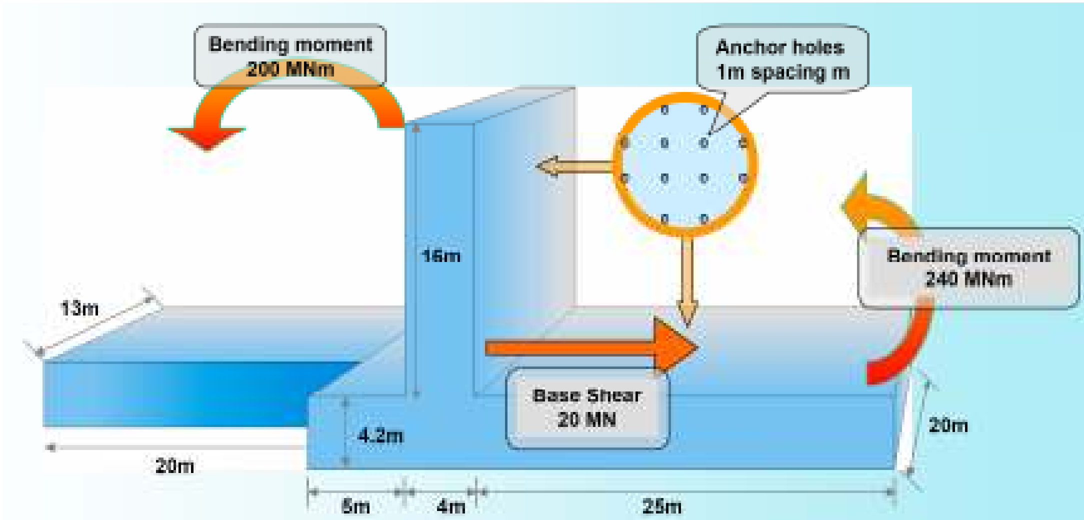



Fig. 3.1.2 The reaction wall and the strong floor of the European Laboratory for Structural Assessment –ELSA of the Joint Research Centre of ISPRA.

Following the relevant experience gained during the period spent at ELSA laboratory, the candidate applied for a national grant with the aim to built a new experimental facility winthin the laboratory of CEMSIG Research Centre within the CMMC Department at the Politehnica University in Timisoara. The grant for which the candidate was director was approved in 2007 and, at its completion in 2010, the existing laboratory of structures was extended with a new building, which houses a unique facility in Romania and in the Eastern Europe (and one of the few in Europe): an experimental stand for static, dynamic and pseudo-dynamic testing of structural details and real scale buildings. In European Union, apart of ELSA laboratory, pseudo-dynamic testing is used at the University of Trento (Italy) and at the University of Porto (Portugal). A description of this new experimental facility is presented in section 3.3.

## 3.2 Pseudo-dynamic test of a real scale flat-slab reinforced concrete structure

The construction of reinforced concrete buildings with flat slab systems has become widely used in some high seismicity European countries, both for office and residential buildings. Even though national codes may include rules for the design of these structures, this matter is not covered by the latest draft of Eurocode 8 (EN 1998, 2005) for the earthquake resistant design of structures. The behaviour of this type of structural systems with flat slab frames used as seismic resistant elements shows important drawbacks, such as the essentially non-dissipative features of their seismic response. Furthermore, flat slab building structures are significantly more flexible than traditional concrete frame/wall or frame structures, thus becoming more vulnerable to second order P-Δ effects under seismic excitations.

In the framework of the European network Safety Assessment for Earthquake Risk Reduction (SAFERR) and of the European Consortium of Laboratories for Earthquake and Dynamic Experimental Research (ECOLEADER), an experimental program was carried out at the European Laboratory for Structural Assessment (ELSA) Laboratory of the Joint Research Centre (JRC) in Ispra, Italy, for the assessment of the global behaviour of flat slab structures subjected to severe earthquakes. Two pseudo-dynamic tests, with increasing input intensity, were carried out on a full-scale three-storey reinforced concrete flat-slab building structure. The structure, representative of flat-slab buildings in European seismic regions, was tested using artificially generated accelerograms compatible with the Portuguese seismic response spectrum. The National Laboratory for Civil Engineering (LNEC) in Lisbon performed the design of the tested specimen, based on the Portuguese design code (REBAP, 1983; RSA, 1983).

### Description of the structure

The tested specimen is a reinforced concrete three storey flat-slab structure, with one bay in each direction. The general layout is show in Fig. 3.2.1. The slab was designed in order to study the failure of the slab-column connections for different situations (interior column, edge column or corner column). For this purpose, two cantilevers were provided, of 1.50 m and 1.25 m respectively.

Voids of 0.8 x 0.8 m with 0.2 m height (waffle slab) were provided in the slab, thus creating wide beams between columns with a total depth of 0.295 m. According to the Portuguese code, in order to account for punching shear, an increased number of stirrups was provided to the beams in the vicinity of slab-column connections. The columns of the first and second floor have rectangular cross sections of 0.3 x 0.5 m, while the columns of the third floor have reduced dimensions of 0.3 x 0.4 m. The columns of the South Frame (P1 and P2) are differently oriented from the columns of the North Frame (P3 and P4).

The materials considered in the design were class C25/30 concrete and class A400 reinforcing steel. Compressive strength tests on 18, 150 mm concrete cubes reference specimens, cast during construction, lead to an average strength of 37.4 MPa and a corresponding characteristic strength of 34.5 MPa. The reinforcement used in the construction of the specimen was class FeB44k Italian steel, for which the Italian norm specifies a 430 MPa yield strength, 540 MPa for the ultimate strength and an ultimate strain A5 of 14%. Tensile tests on three steel bar specimens from each diameter used in the construction of the model were performed, from which characteristic values of 502 MPa, 619 MPa and 21.9% were obtained for the yield strength, ultimate strength and ultimate strain A5, respectively.

The vertical loads, dead and live, in addition to the self-weight of structural elements, were considered to act uniformly along the slabs surface area. The total additional vertical loads for each floor were computed considering the safety factors given by the Portuguese regulations for the earthquake combination and have the following values: 4.2 kN/m2 for office floors (first and second floor) and 3 kN/m$^2$ for terraces (third floor). The self-weight of the slab, considering the voids, is 5.5 kN/m$^2$.

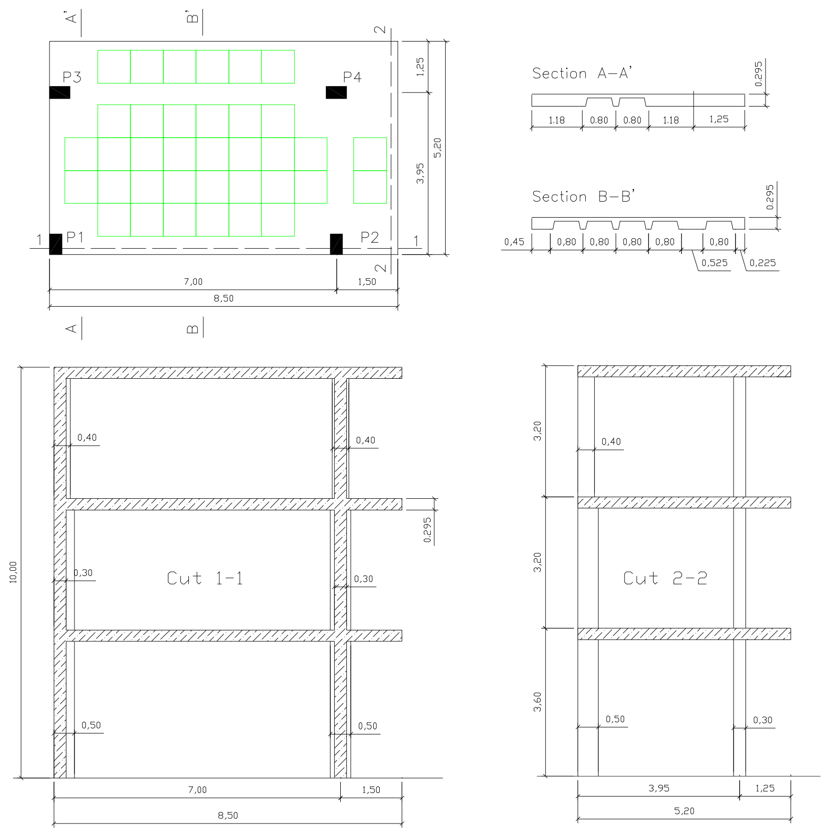
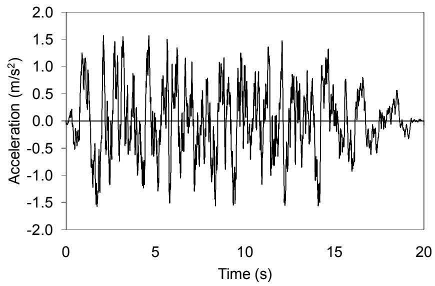



Fig. 3.2.1 Plan and elevation views of the building mock-up

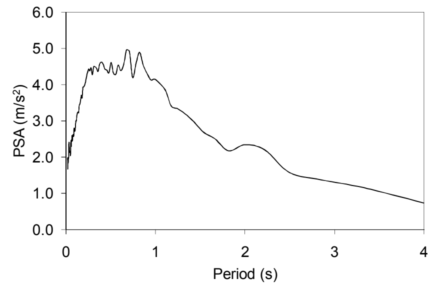
### Test set-up

Two Pseudo-Dynamic (PsD) tests of the flat-slab building, with increasing input intensity, were carried out using the reaction wall experimental facility of the ELSA laboratory. The specimen was tested using artificially generated input motions, of 20 seconds duration, corresponding to a moderate-high European hazard scenario. For the first PsD test, a 475 years return period (475 yrp) was considered, with a corresponding peak ground acceleration of 0.16g. For the second test, the 475 yrp accelerogram was scaled by a factor of 1.73, yielding a peak ground acceleration of 0.277g corresponding to a return period of 2000 years (2000 yrp). The accelerograms are compatible with the Portuguese seismic response spectrum. Fig. 3.2.2 shows the acceleration time-history and the pseudo-acceleration response spectra for the 475 yrp earthquake.

Fig. 3.2.3a shows the test set-up. The displacements were applied to the structure by means of six double-acting servo-hydraulic actuators, each of 500 kN maximum load capacity. The structural displacements were measured with respect to an exterior steel unloaded reference frame mounted on the reaction floor using Heidenhein optical transducers that provide a digital output of very high precision; the sign convention for displacements was adopted as positive when the structure displaces towards the reaction wall. For the application of horizontal loads, supplementary concrete beams were provided during construction, on each floor, at mid-span. Additional masses, to simulate permanent (other than the self-weight of the slabs) and live loads were placed on each floor by means of large water containers (Fig. 3.2.3a).




(a)




(b)

Fig. 3.2.2 475 yrp seismic input: (a) Acceleration time history; (b) Pseudo-acceleration response spectra

(a)

(b)

Fig. 3.2.3 (a) Test set-up; (b) Displacement transducers

Member rotations were measured by means of 52 inclinometers, located around the slab-column connections. In order to estimate the effective width of the slab, 40 displacement transducers were used on the first floor level around the connections of columns P1 and P3, above and below the slab, as shown in Fig. 3.2.3b.

**Pre-test numerical analysis**

In order to predict the behaviour of the flat-slab structure under the considered earthquake input motions, a pre-test time-history numerical analysis was performed. Both the South and the North Frames were considered in the numerical model, but only the sections of the primary longitudinal beams were taken into account in the analysis, using the widths given section B-B' of Fig. 3.2.1. In order to account for the stiffness contribution of the slab, the nodes corresponding to the beam-column joints of both frames were assumed to have identical displacements at each level.

The structure was modelled using the fibre/Timoshenko beam element implemented in VisualCast3M computer code (Guedes et al., 1994). This allows representing the entire reinforced concrete section of the elements, considering both the confined and unconfined parts and the location of the steel reinforcement. The behaviour of concrete is represented by a parabolic curve up to the peak stress point followed by a descending straight line in the softening zone. Confinement is taken into account by modifying the unconfined concrete curve and including an additional plateau zone. Cyclic behaviour accounts for stiffness degradation and crack closing phenomena; tensile resistance is also considered. A modified Menegotto-Pinto (Menegotto and Pinto, 1973) model with a three-stage monotonic curve (linear, plateau and hardening) represents the steel reinforcement behaviour; Bauschinger effects are taken into account and buckling effects are simulated.

The effect of punching shear in the slab-column connections was not considered in the numerical analysis. According to recent experimental data and based on some previous researches by Hawkins (Hawkins et al., 1975) and Islam (Islam and Park, 1976), Robertson (Robertson et al., 2002) concluded that in flat-slab type structures subjected to lateral loads, slab-column connections containing shear reinforcement are effective in resisting punching shear failure for inter-storey drifts ranging between 3.5% and 8% depending on the level of the vertical load. The minimal value of 3.5% is granted for a maximum gravity-shear ratio of 0.4; the gravity-shear ratio is defined as the ratio between the initial gravity load transferred from slab to column and the ACI318-99 Building Code (ACI Committee 318, 1999) direct punching shear capacity of the slab critical section. For the tested building, the maximum value for this ratio, for all connections, is lower than 0.4. The structure was also designed with an increased number of stirrups around the column-slab joints. Consequently, a conservative value of 3.5% inter-storey drift for which punching shear would not occur was considered for the tested specimen; it is expected then that failure of the connections would occur in flexure below this level of inter-storey drift. Flexural failure is characterized by a smooth decrease of the load-carrying capacity, while punching failure exhibits a sudden reduction of capacity (brittle failure) (Menetrey, 1998).

Under the 475 yrp earthquake, the maximum top displacement obtained from numerical analysis was 169 mm, corresponding to a global drift of the structure of approximately 1.7%. A maximum inter-storey drift of 2.04% was obtained at the second storey, almost equal to the maximum drift allowed by the damage limitation limit state given by clause 4.4.3.2 of Eurocode 8 (EN 1998-1, 2004), equal to 2%, for importance class II buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements.

The second earthquake input motion was introduced in the numerical analysis after the 475 yrp earthquake, with a 20 seconds zero input acceleration between the two accelerograms. Indeed, in the experimental program, the 2000 yrp earthquake was imposed to a building mock-up that had already been damaged by the 475 yrp earthquake. The 20 seconds of zero input acceleration were necessary to numerically stabilize the structure oscillations after the first earthquake. The maximum displacement obtained at the top of the structure from the numerical analysis was equal to 249 mm, corresponding to a global drift of the structure of 2.52%.

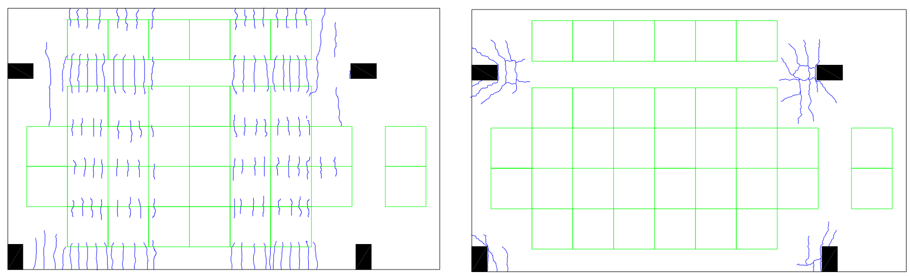
The numerical simulation of the 2000 yrp earthquake showed that the deformation is concentrated at the second storey, which reaches a maximum drift of approximately 3%. This value is lower than the conservative 3.5% limit assumed for punching shear failure. Considering also that the maximum top displacement obtained from the numerical analysis was under the displacement limit of the actuators used in the ELSA laboratory, it was decided that the 2000 yrp earthquake could be safely used for the second motion input in the PsD test.

**Experimental results**

***475 yrp earthquake test***

During the first test, thin flexural cracks developed below the slabs, mainly in the primary beams, as well as in the slab secondary beams, of the South and North Frames, as shown in Fig. 3.2.4a for the second floor. No cracks were identified at the mid-span portion of the slab: this area corresponds to the supplementary beam used for load application, which was cast together with the slab, thus creating a higher rigidity and strength. At the inferior face of the slab of the third floor important cracks were observed around column P3, due to insufficient anchorage length of the column reinforcement into the slab, which lead to initiation of detachment of the slab above the column.

Above the slabs, flexural cracks developed in front of the columns, followed by torsion cracks at approximately 45o, as shown in Fig. 3.2.4b for the second floor. Thicker cracks appeared in the exterior connections of column P1 (corner connection) and P3 (edge connection) due to torsion failure of the exterior transverse beam. Mainly flexural, and some limited torsion cracks, appeared around columns P2 and P4, due to the presence of the stronger transverse beam and to the higher concrete confinement. Some thin, flexural and shear cracks appeared in the columns as well, but only at the base.



(a) (b)

Fig. 3.2.4 475 yrp earthquake test: (a) Cracking pattern below floor 2;
(b) Cracking pattern above floor 2

The maximum top displacement recorded during the test was equal to 162 mm, which corresponds to a global drift of 1.64%. A permanent top displacement of 16 mm (0.16% global drift) was measured at the end of the test. Fig. 3.2.5 shows the maximum inter-storey drift profile. As predicted by the numerical analysis, the deformation of the structure is concentrated at the second storey, with an inter-storey drift equal to 1.87%.

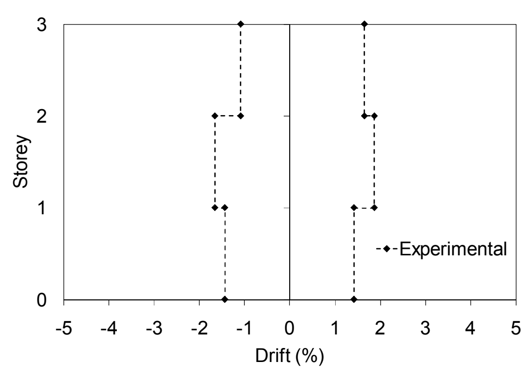


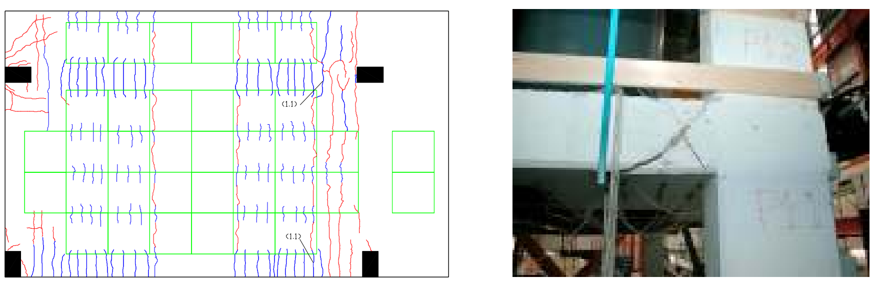

Fig. 3.2.5 475 yrp earthquake test: Maximum inter-storey drift profile

***2000yrp earthquake test***

During the second test, malfunction of one of the actuators lead to failure of the PsD algorithm and the structure was pulled towards the reaction wall until the actuators at the third storey reached their displacement limit. The structure was heavily damaged and the test had to be stopped at a point corresponding to 2.05 seconds of the accelerogram.

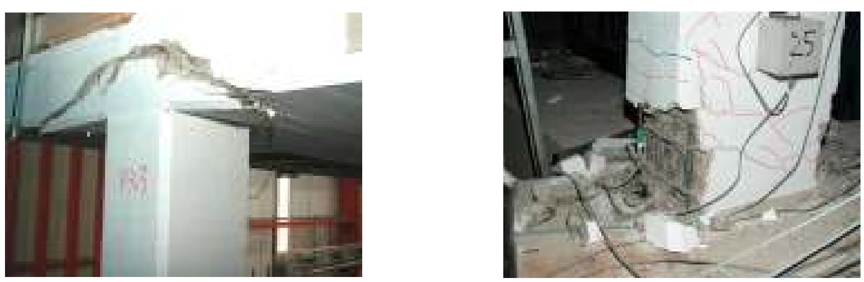
Due to the large displacements imposed during the test, important negative moments developed on the exterior slab-column connections of columns P1 and P3. The first test already damaged these connections and significant torsion cracks were present in the transverse beam connecting these two columns. Consequently, in the second test, the behaviour of the exterior slab-column was governed by torsion failure of the transverse beam. No significant supplementary cracks appeared above the slab at the slab-column connections of columns P2 and P4.

Below the slabs, supplementary flexural cracks, associated to the positive moment, developed around columns P2 and P4, new or as extension of the existing ones that resulted from the first test, as shown in Fig. 3.2.6a for the second floor. Important cracks also appeared at all floors across the entire width of the slab and all through the first voids of the east side of the building; cracks up to a width of 4.0 mm were observed in this area at the first floor. Thin cracks also appeared through the entire width of the slab, extending the existing cracks of the secondary beams up to the edges of the supplementary beam for load application. Important torsion cracks of the transverse beam developed below the slab around the exterior connections of columns P1 and P3, especially at the edge connection of column P3, as shown in Fig. 3.2.6a. Torsion failure of the transverse beam was evident at all the exterior connections of columns P3 and P1, as shown in Fig. 3.2.6b. Pronounced detachment of the slab occurred above column P3 at the third floor, as shown in Fig. 3.2.7a.



(a) (b)

Fig. 3.2.6 2000 yrp earthquake test: (a) Cracking pattern below floor 2; (b) Failure of floor 1 transverse beam at column P1



(a) (b)

Fig. 3.2.7 2000 yrp earthquake test: (a) Slab detachment at the top of column P3; (b) Damage at the base of column P3

Very thin flexural cracks were observed in the columns above and below the slabs, whilst important damage occurred, at the base of the columns. Concrete spalling was observed at all column bases and buckling of the reinforcement occurred at the base of column P3, as shown in Fig. 3.2.7b.

The maximum top storey displacement was equal to 423 mm, corresponding to a global drift of 4.29%. As shown in Fig. 3.2.8, the largest inter-storey drift equals 5% and occurs at the first storey, due to the development of plastic hinges at the column bases.

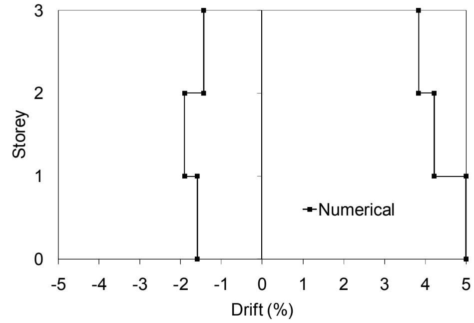



Fig. 3.2.8 2000 yrp earthquake test: Maximum inter-storey drift profile

**Effective slab width**

Different studies concerning the effective width of the slab of lateral-loaded slab-column frames have been made, in order to account for the flexural stiffness of the slab. A relatively recent research by Grossman (Grossman, 1997) evaluated several design methodologies based on flat slab-column experimental data and proposed an effective width formula, considering the effects of panel and connection geometry, as well as the effect of lateral drifts up to drift levels of 1%:

$$b_{eff} = K_d \left[ 0.3L_1 + C_1 \frac{L_2}{L_1} + 0.5\left(C_2 - C_1\right) \right] \left[ \frac{d}{0.9h} \right] K_{FP} \tag{3.2.1}$$

in which: Kd is a coefficient function of the drift level, L1 and L2, and C1 and C2, are the span lengths and column cross section dimensions in the parallel and perpendicular directions to the lateral load, respectively, d is the effective depth of the slab, h is the slab thickness and KFP is a coefficient function of the connection type (1.0 for interior connections, 0.8 for exterior edge connections and 0.6 for exterior corner connections).

In a more recent study by Hwang & Moelhe (Hwang and Moelhe, 2000), based on experimental data obtained from tests of a 0.4 scale multi-panel slab-column frame, the authors propose a different formula for exterior connections:

$$b_{eff} = \left[ C_1 + \frac{L_1}{6} \right] \beta \quad \text{where} \quad \beta = 4\frac{C_1}{L_1} \geq \frac{1}{3} \tag{3.2.2}$$

In the same way as Equation (3.2.1), Equation (3.2.2) is also geometry based, but instead of using a factor accounting for the drift level, a β stiffness reduction factor accounting for cracking is proposed. Both authors account for the case of exterior and interior connections, but the Grossman proposal distinguishes between exterior edge and exterior corner connections.

The design of flat-slab structures is not considered in Eurocode 8, however, Eurocode 2 (EN 1992-1-1, 2004) offers a methodology for buildings in the Informative annex (i.e., I.1.2: Equivalent frame analysis). The methodology consists in dividing the structure into frames consisting of strips of slabs contained between centre lines of adjacent panels of width equal to half the span along the direction perpendicular to the loading. The stiffness of these equivalent beams, computed based on the gross cross section of the slab, is reduced to 40% for horizontal loading, to reflect the increased flexibility of flat-slab structures. The same procedure is considered in the Portuguese code (REBAP, 1983; RSA, 1983), with the exception that the stiffness of the equivalent beam is reduced to 50%.

The effective widths of the slab for the exterior connections of the flat-slab building tested at the ELSA laboratory were computed from the displacements given by the transducers above and below the first floor slab (Fig. 3b) considering different inter-storey drift levels. The methodology followed consisted in computing the equivalent width considering a constant deformation equal to the deformation measured along the beam such that the same integral of deformations is obtained from the displacement transducers located at a given slab-column connection (see Fig. 3.2.3b). The procedure is explained more in detail in (Zaharia et al., 2005) and (Pinto et al., 2004).

Fig. 3.2.9 gives the values measured during the test and the trend line of the equivalent effective slab widths calculated at mid-height of the slab as a function of inter-storey drift . The effective slab width beff is represented as a function of the beff/bw ratio, in which bw represents the width calculated at mid-height of the trapezoidal section of the beam, equal to the 0.487 m and 0.601 m for the connections of columns P1 and P3, respectively . It may be observed that for the corner connection of column P1, at the maximum inter-storey drift of 5%, the effective width of the slab is practically equal to the beam width; for this connection, all the longitudinal reinforcement of the beam passes through the column core. Meanwhile, for the edge connection of column P3, the effective width of the slab is less than the width of the beam. In this case, part of the beam reinforcement passes outside the column core and consequently the ability of the beam of being fully effective in transferring bending moment relies on the torsion capacity of the transverse beam, which demonstrated a poor behaviour since the first test.

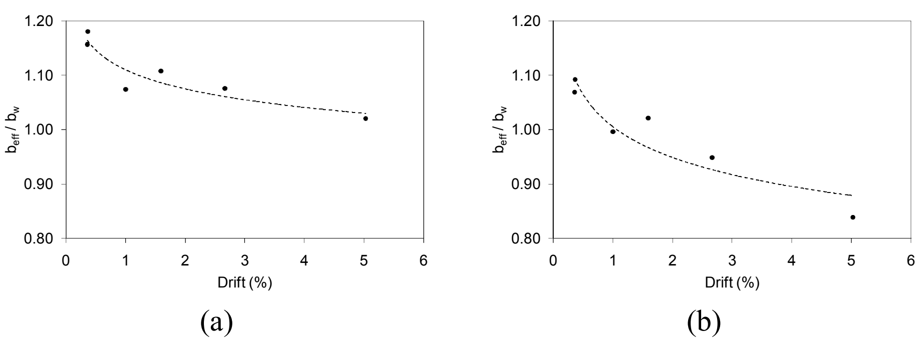



Fig. 3.2.9 Experimental slab effective widths: (a) Corner connection (column P1); (b) Edge Connection (column P3)

From the trend lines of Fig. 9, the following expression is derived for computing the beff/bw ratio of the tested structure under the action of seismic loads for inter-storey drift values dI comprised between 0.5% and 5%:

$$\frac{b_{eff}}{b_w} = A\ d_I^{-\alpha} \qquad (3.2.3)$$

where A is a coefficient equal to 1.11 and 1.01, and α is an exponent equal to 0.047 and 0.084, for the corner and edge connections, respectively.

Table 3.2.1 presents the comparison between the experimental effective widths and the results from the above mentioned design methodologies. Both minimum and maximum values of effective widths obtained from the experiment are presented, corresponding to the maximum and minimum inter-storey drifts recorded during the tests. The values presented for the Portuguese code (REBAP, 1983; RSA, 1983) methodology are computed for the slab section near the columns, considering the reduced number of slab voids and an effective width corresponding to a rectangular section with half the moment of inertia of the trapezoidal T section obtained from the equivalent slab strips described in the equivalent frame analysis of Eurocode 2 (EN 1992-1-1, 2004). The Grossman (Grossman, 1997) value for the effective width was computed for a lateral drift of 1% (Kd = 0.5). For the edge connection at column P3, which represents a typical wide beam-column connection, a supplementary design methodology by LaFave & Wight (LaFave and Wight, 1999) was considered. Based on experimental results, these authors present a simple expression for computing the effective slab width in exterior connections with wide beams for lateral drifts of up to 4%:

$$b_{eff} = 0.5\left(b_w + C_2\right) + 2C_1 \qquad (3.2.4)$$

in which bw is the width of the wide beam.

Table 3.2.1. Effective widths of slab for column P1 and P3 connections [cm]

| Experiment | | EC2 | Grossman | Hwang & | LaFave & |
|---|---|---|---|---|---|
| Min | Max | Portuguese code | | Moelhe | Wight |
| | | | Column P1 | | |
| 49.8 | 57.5 | 81.6 | 55.5 | 48 | - |
| | | | Column P3 | | |
| 50.4 | 65.5 | 115 | 74 | 54.1 | 145 |

The values from Table 3.2.1 show that the best fit with the experimental results are given by the Hwang & Moelhe expression; the value of 54.1 cm for the effective slab width of the P3 column connection can be considered acceptable when compared with the minimum value of 50.3 cm obtained from the experiment at the maximum inter-storey drift of 5% . With the exception of the Hwang & Moelhe expression, no other methodology offers conservative results for both connections. It must be underlined, however, that a wide range of design detailing rules exists for slab-column connections in flat-slab structures and that the effective width of the slab, for the case of exterior connections, is directly related to the torsion behaviour of the transverse beam.

**Numerical analysis considering the effective slab width**

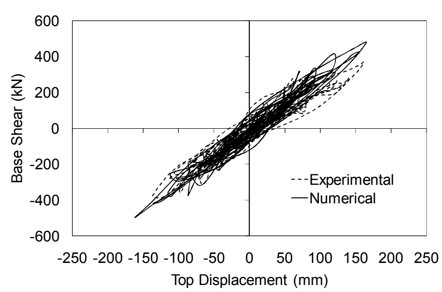
In the pre-test numerical analysis, no assumptions were made for the slab participation in the vicinity of the slab-column connections. In order to consider the slab participation in the numerical analysis, the effective slab widths for columns P1 and P3 connections as computed from the experiment were used. For the interior slab-column connection of column P4 and for the exterior connection of column P2, for which no experimental data was available, the values proposed by Hwang and Moelhe (Hwang and Moelhe, 2000) given in Table 3.2.1 were considered. The new numerical model considers also the slab-column detachment at the top of column P3 of the North Frame; in the numerical analysis, this connection was considered free to rotate.

***Comparison with experimental results for 475 yrp earthquake***

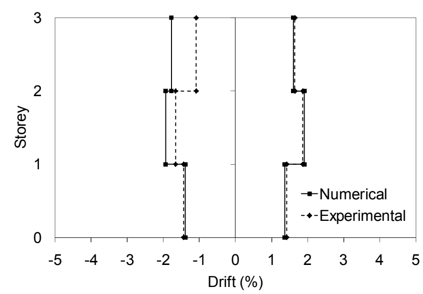
The comparison between the numerical and the experimental results is shown in Figs. 3.2.10a and 10b for the 475 yrp earthquake for the base-shear vs. top-displacement and for the maximum inter-storey drift.

The global displacements and the inter-storey drifts obtained from the numerical analysis, when compared with the corresponding values obtained from the pre-test numerical analysis, are in better agreement with the experimental values. A pronounced difference at drift level may be observed only for the third storey, when the structure is pushed away from the reaction wall: due to the complete rotational freedom considered for the connection at the top of column P3, the numerical maximum negative drift is considerably higher than the corresponding one obtained from the test. When the structure is pushed away from the reaction wall, the positive moment created at the connection of column P3 is effectively transferred to the column, due to the fact that the capacity of the connection in transferring bending moments relies on the bottom reinforcement of the beam, which is anchored into the column. On the other hand, when the structure is pulled towards the reaction wall, the transfer of bending moments to the connection relies on the top reinforcement of the beam and of the slab, which being detached from the column, results in a pinned behaviour of the connection, similar to the one assumed in the analytical model as confirmed by the positive drifts in Fig. 10b.

While good agreement is obtained for the inter-storey drifts, the inter-storey shear forces computed from the numerical model are higher than those measured during the experiment, by an important amount, as shown in Fig. 3.2.11. It must be emphasised that the numerical results are obtained from a first order dynamic analysis. Due to computer code limitations in achieving convergence resulting from the complexity of the model, it was not possible to consider second order effects in the numerical dynamic analysis, which may had given lower shear forces in better agreement with the experimental results. As an alternative and with the purpose of improving the results of the analysis, a second order static analysis is presented in the next sub-section for the 2000 yrp earthquake.




(a)




(b)

Fig. 3.2.10 475 yrp earthquake test - Comparison between experimental and numerical results: (a) Base shear versus top-displacement; (b) Maximum inter-storey drift

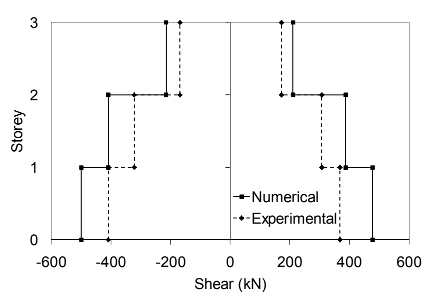
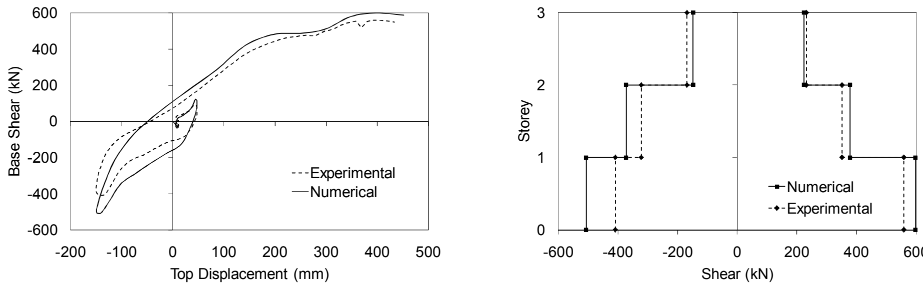



Fig. 3.2.11 475 yrp earthquake test - Comparison between experimental and numerical results: Maximum inter-storey shear

***Comparison with experimental results for the 2000 yrp earthquake***

The comparison between the results of the numerical time-history analysis of the second input motion and the data from the experimental results could not be carried out due to failure of the PsD algorithm. Nevertheless, in order to calibrate and obtain supplementary verification of the finite element model considering slab participation, the 475 yrp and the 2000 yrp earthquake tests were analysed by means of a second order static analysis by imposing to the numerical model the experimental displacement history extracted from both tests. The comparison between the numerical analysis and the data from the 2000 yrp earthquake test is given in terms of base-shear versus top-displacement (Fig. 3.2.12a) and maximum inter-storey shear profile (Fig. 3.2.12b). It may be observed that, considering second order effects, the numerical analysis gives better results for all storeys at the level of shear forces, especially for positive drifts.




(a) (b)

Fig. 3.2.12 2000 yrp earthquake test - Comparison between experimental and numerical results: (a) Base shear versus top-displacement; (b) Maximum inter-storey shear

The results show that the numerical model offers a good prediction at the level of global displacements and inter-storey drifts, and that for a good prediction of the experimental response, second order effects must be considered. Moreover, the numerical model was able to capture the plateau of the base shear vs. top displacement envelope at a displacement of approximately 250 mm, corresponding to the achievement of the maximum capacity of the structure, and the further increase of the base shear force after kicking-in of the higher mode distribution of storey forces imposed by the actuators.

The results of a push-over first and second order analysis are presented below, in order to emphasize the importance of second order effects for the analysis of flat-slab structures, and to determine the drifts at which the structure attains its maximum base shear and deformation capacity.

**Push-over analysis**

A push-over analysis of the model considering the effective width of the slab was performed in both directions, i.e., the structure being pulled towards, and being pushed away from the reaction wall. The analysis was carried out in displacement control at the top storey, with the distribution of lateral displacements through the height of the structure imposed in such a way so as to maintain a triangular distribution of lateral loads. Both first and second order analyses were considered.

The response of the structure, in terms of base shear versus global drift, is illustrated in Fig. 3.2.13. The results of the analysis show that the difference in base shear between the first and second order analysis is on the order of 15% at a drift of 3.5%. The results also show that the model gives a higher resistance when the model is pushed away from the reaction wall (negative drifts), since in this case the additional negative moments from the horizontal loads that act on the slab-column connections of columns P2 and P4 result in a slab participation much more important than in the slab-column connections of columns P1 and P3 when the structure is pulled towards the reaction wall. Likewise, the maximum strength of the structure is achieved at a global drift of approximately 2.5% before the structure starts loosing capacity, reaching a maximum global drift of approximately 3.5% corresponding to a 20% reduction of the maximum strength.

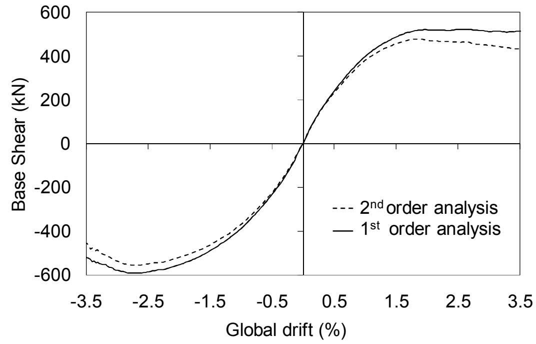


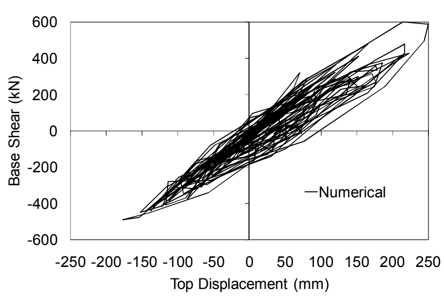

Fig. 3.2.13 Push-over analysis: Base-shear versus global drift

The results from the push-over analysis confirm the global drift value, equal to 2.5%, at which the maximum base shear was attained in the second earthquake test, and the limited capacity of the structure, exacerbated by the second order effects, in undergoing further deformations after reaching its maximum loading capacity.

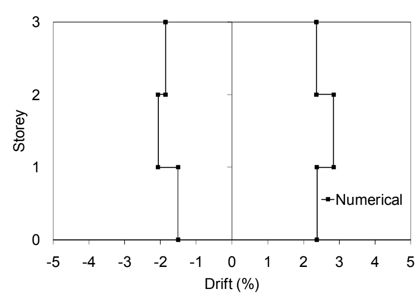
**Simulation of 2000yrp PsD test by numerical analysis**

The second PsD earthquake test, which was stopped after failure of the PsD algorithm, may be reproduced using the numerical model. A first order dynamic analysis taking into account the slab participation was performed, considering the 475 yrp and 2000 yrp earthquakes with a 20 seconds zero input acceleration between the two. Since a second order analysis could not be performed due to the inability of the numerical model in achieving convergence in dynamic analysis, the results of the first order dynamic analysis should be considered as conservative and may be corrected based on the results obtained from the push-over analysis as shown in Fig. 3.2.13

The base-shear vs. top-displacement diagram is shown in Fig. 3.2.14a and shows that for the 2000 yrp earthquake the structure reaches a top displacement of 250 mm (2.53% global drift), equal to the drift at which the structures attains its maximum capacity, as indicated by the push-over analysis. Fig. 14a also shows that the hysteresis loops are rather narrow, reflecting a limited energy dissipation capacity of the structure. The inter-storey drift profile is shown in Fig. 3.2.14b and denotes a rather uniform drift distribution, with a 2.85% maximum drift at the second storey.




(a)




(b)

Fig. 3.2.14 2000 yrp earthquake – Numerical simulation
(a) Base shear versus top-displacement; (b) Maximum inter-storey drift

## Conclusions

The results from the two experimental tests led to the following observations: the structural deformations were concentrated at the slab-column connections and at the column bases; important flexural and torsion cracks appeared around the exterior slab-column connections, with considerable damage due to the torsion failure of the transversal beam; mainly flexural and, to a lesser extent, torsion cracks, developed in the other connections, due to the presence of the stronger transversal beam and to the higher confinement of the concrete; failure of the slab-column joints, by separation of the slab from the column at the third floor, owing to insufficient anchorage length of the column reinforcement into the slab.

The evaluation of the slab effective width to be considered in the analysis of flat-slab structures is of prime importance. The test results confirm the findings of recent studies concerning the smaller slab participation under lateral loads, and the decrease of the slab participation with the increase of lateral drifts. The results also show that Eurocode 2 (EN 1992-1-1, 2004) overestimates the slab effective width, thus calling for a revision of informative annex I.1.2.

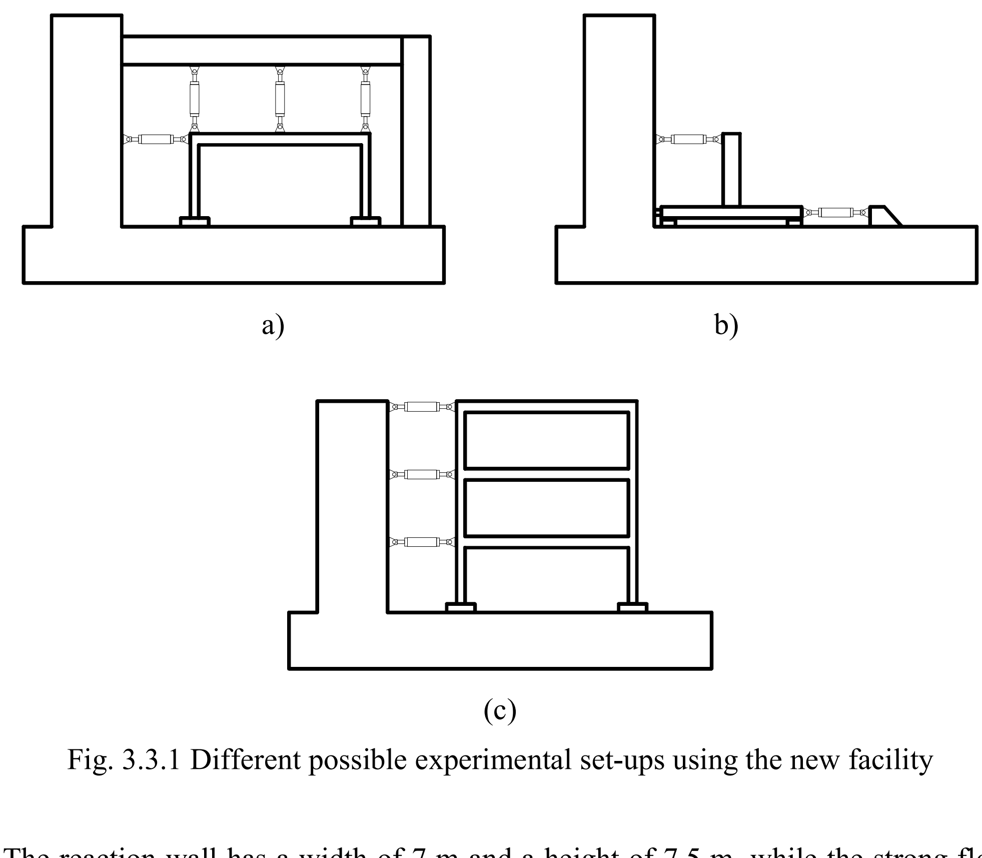
In spite of the failure of the second PsD earthquake test, important information was derived by resorting to numerical analysis, which was able to accurately capture the response of the structure during the two experimental tests. The results from the second experimental test and from the push-over demonstrate that the structure achieves its maximum load capacity at a drift of 2.5%, thereafter decreasing to 80% of its maximum load capacity at a drift of 3.5%; the hysteresis loops from the dynamic analysis confirm that the structure has a low dissipative behaviour.

The results from the push-over analysis also shows that flat-slab structures exhibit significant higher flexibility when compared to traditional frame structures, becoming more sensitive to second order effects. Thus, second order effects must be taken into account in the analysis of this type of structures. in order to limit deformation demands under earthquake excitations, combination with other stiffer structural systems as shear-walls is advisable.

## 3.3 New experimental facility

Up to year 2010, the laboratory of the research centre CEMSIG (qualified as centre of excellence by CNCIS) within the department CMMC of the “Politehnica” University of Timisoara, had the necessary equipment for static and quasi-static monotonic and cyclic tests (reaction frame, hydraulic actuators, data acquisition and control), but the existing reaction frame was limited from the point of view of loading capacity and size of specimens, whih permitted only in-plane assemblies of relatively reduced dimensions.

The project “Structural assessment laboratory for large scale tests” within the “Capacities” Program (“Dezvoltare laborator pentru incercari la scara mare – INSTRUCT” - PN II Modul I Capacitati, 90 CP/ I/ 14.09.2007, Director Assoc. Prof. Raul Zaharia, 2007-2010) aimed at extending the experimental capacity of the CEMSIG research centre, by development of a new facility for tests on large and real-scale specimens. The existing actuators were supplemented by two new large-capacity actuators, with hydraulic unit and controller. The new facility allows static, quasi-static as well as pseudo-dynamic tests to be carried on, that evaluate seismic performance of full-scale structures by combining experimental quasi-static testing with numerical evaluation of dynamic seismic forces. The new facility allows to perform testing on full-scale or close full-scale structures with 1-3 stories. A great achievement is the possibility to use this experimental facility in order to test 3D structures, but different plane experimental set-up configurations may be considered for full-scale structures, substructures or details, as shown in Fig. 3.3.1.



a) b)

(c)

Fig. 3.3.1 Different possible experimental set-ups using the new facility

The reaction wall has a width of 7 m and a height of 7,5 m, while the strong floor has a work surface of approximately 11x11 m. Full-scale tests are necessary to evaluate structural performance in inelastic range, since small scale testing performed using similitude theory are only appropriate for the elastic range. Besides testing on large structural models, the reaction wall and the strong floor allow to perform testing on independent elements and assemblies, and offer an increased flexibility in building various experimental arrangements. As shown in Fig. 3.3.2 the reaction wall and the strong floor were built as an higher extension of existing laboratory building, and was provided also with a gantry crane. Fig. 3.3.3 shows the steel structure of the reaction wall and of the strong floor. Considering the important forces acting on the floor, a 1.5m thick foundation was provided, on the entire surface of the new building, to support the reaction wall, the strong floor and the building, as shown in Fig. 3.3.4. Fig. 3.3.5 shows the structure of the reaction wall and of the building of the new laboratory.

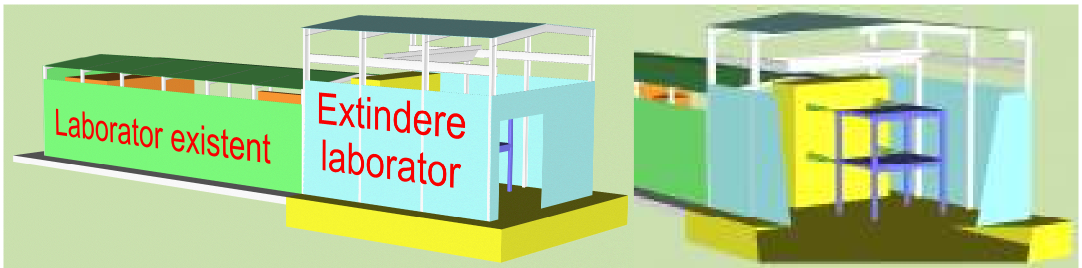



Fig. 3.3.2 Extension of the existing laboratory

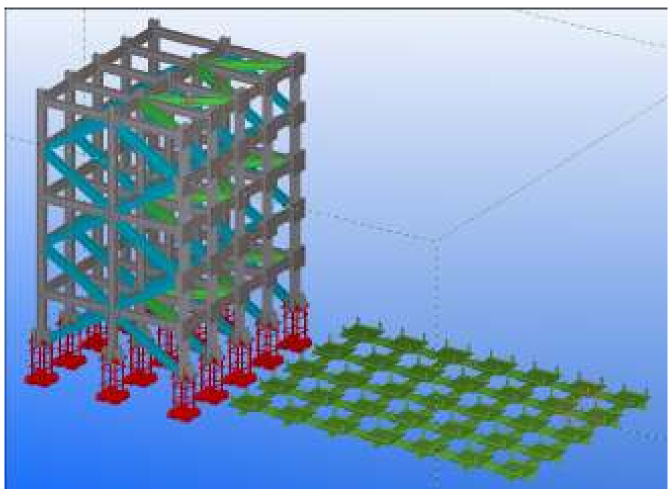


Fig. 3.3.3 Structure of the reaction wall and of the strong floor

Fig. 3.3.4 Foundation for the new experimental facility

Fig. 3.3.5 Reaction wall and external view of the new laboratory

# C. SCIENTIFIC, PROFESSIONAL AND ACADEMIC FUTURE DEVELOPMENT PLANS

All the scientific, professional and academic future development plans of the candidate deal in principal with the main thematic direction “Fire design of civil engineering structures”.

## C1. Scientific

Even if the main direction for further development is fire engineering, the experience and the activity in the second thematic direction “Design assisted by testing” will allow the candidate to follow also this direction, especially by the realisation of new experimental facilities in the CMMC laboratory, linked to the topic of structural fire research.

As shown, the CMMC laboratory has an impressive experimental capability, being able to test real-scale building structures and details under static and dynamic loads. However, there is a lack of specific equipments in order to consider the fire action on building structures. At this moment, CMMC laboratory has a temperature chamber, linked to the dynamic testing machine of 100 tons, able to heat (electrically) specimens up to 600$^{o}$C. Due to its reduced dimensions, only limited typologies of constructional details may be tested, and only under axial forces. There is another temperature chamber, of smaller dimensions, dedicated to material tests, able to heat up to 1100$^{o}$C, but dedicated only to round specimens.

A first plan in this direction is to develop the CMMC laboratory capabilities by the acquisition of an electrical system for the local heat of structural elements. This would lead to a unique experimental facility in Romania, which would have the possibility to test a real-scale structure, heated to a given level of temperature and then loaded with external loads (within the new reaction wall-strong floor experimental facility). At this moment, the single experimental facility in Romania for the fire assessment of structures is located in Bucharest at INCERC, and has two owens, able to develop the standard ISO fire curve. One of the owens is vertical, able to test structural elements as walls, but not under external loads. The other is an horizontal owen, able to test elements as beams or floors.

In the same direction, it is the intention of the candidate to complete the small temperature chamber for material tests under elevated temperatures by the acquisition of adaptors for the test of plate material specimens.

Considering the two above acquisitions, CMMC laboratory would strengthen its capabilities in the field of structural fire engineering, and, by creating a unique facility in Romania, it would be nevertheless complementary with the existing experimental facilities of INCERC Bucharest. This would create the opportunity for further collaborations with this research institute first of all at national level, but, considering the international recognition of the candidate’s activity in the field of fire research, INCERC Bucharest may be also involved in common European funded projects. The candidate is already in contact with the team of specialists of the fire laboratory of INCERC Bucharest. Some possible further collaboration through projects at national level was already identified.

The funding for the mentioned equipments is provided actually in a proposal of a POS CCE Operation 2.2.1 project (still in the evaluation process), for which the candidate is part of the management team. The project director is Prof. Dan Dubina. The project aims to bring together the existing laboratories of the Faculty of Civil Engineering of Politehnica University of Timisoara in an integrated platform of research for the assessment of the buildings under extreme actions (including fire). The project is focused on the acquisition of new equipments, in order to develop the existing experimental infrastructure, but also to create new facilities (for instance, in the CMMC laboratory is provided a shaking table, in order to complete the existing facilities for seismic research). As mentioned, the candidate is part of the management team of this project and coordinated, together with Prof. Dan Dubina, the realisation of the proposal. If the project is funded, this will represent a clear continuation of the candidate's activity in the second main theme presented in the Habilitation Thesis, but also an opportunity, after the implementation period of the project, for further research projects, considering the improvement of the existing infrastructure. If the project will not be founded, it is the intention of the candidate to obtain the necessary funds for the acquisition of the mentioned equipments, by means of further national or international funded projects.

Fire engineering represents a huge domain for further research; only in the field of fire design of civil engineering structures, there are still gaps in the knowledge of different materials behaviour (steel, concrete, masonry, aluminium, wood), fire actions and response of different structural systems and structural details to elevated temperatures. The long term development plans stick with the general thematic direction of fire engineering at national and international level. Some research directions may be identified for short and middle term, presented bellow, based on actual preoccupations of the candidate and on the numerical and experimental capabilities available at CMMC department.

*Behaviour of composite steel-concrete floors with cellular beams*

Recommendations for simple numerical models to be used considering advanced calculation models in the design of composite steel-concrete floors were presented in section 2.4. The calibration of the numerical models included the comparison with three tests, from which only two where conducted in laboratory under controlled ISO standard fire. These tests considered solid steel secondary unprotected beams. Tests on floors considering steel secondary cellular beams were performed only under natural fire. At this moment, a method is available for the design of the composite floors with solid secondary unprotected beams, considering the membrane effect. Based on one test performed under natural fire at Ulster University, a simplified model was proposed to consider in this design method the presence of cellular beams. However, dedicated fire tests under ISO fire should be performed on composite floors with cellular beams, for a better calibration of the design method. Such tests could be performed on the horizontal owen of INCERC Bucharest. A discussion on this topic was already done with both INCERC Bucharest and ArcelorMittal, which are interested in a collaboration for a national research project (through ArcelorMittal Romania) to be proposed at a further call.

*Natural fire models – Localised fires*

Localized fires are an important issue for building typologies where a generalized fire cannot develop and therefore compartment fire models are not suitable. Examples of these typologies are external structures, open car parks or large industrial halls. The candidate assessed the fire resistance of the steel beams of the Basarab bridge in Bucharest, located above an open parking, considering localised fire models, together with experimental data for car fires. A journal paper is in preparation for this study, but it was not presented in this Thesis.

The actual version of Eurocode 1 for fire engineering (EN 1990-1-2) includes an annex dedicated to localized fires. There are two models, function of the calculated length of the flame, which impacts or not the ceiling of the fire compartment. If the flame is not impacting the ceiling, it is only possible to assess the temperature of a steel element in the vertical axis of the localised fire (Heskestad model). If the flame impacts the ceiling, it is only possible to assess the temperature of a steel element in the horizontal axis at the ceiling level (Hssemi model). It is impossible, using these models, to asses the temperature or the flux received by a vertical member at a given distance of the fire source.

An European project is in progress ("Temperature assessment of a vertical steel member subjected to localised Fire – LocaFi", European Commission RFCS Project No RFSR-CT-2012-00023) with the main goal to improve the existing knowledge on the effects of the localized fires in a building compartment. A new method, developed by means of experimental and numerical research, will provide the fluxes received in any point of a building compartment subjected to a localised fire. The partners are: ArcelorMittal Luxembourg (coordinator), Centre Technique et Industriel de la Construction Métallique – France, Universite de Liege – Belgium, University of Ulster – Northen Ireland, Politehnica University of Timisoara – Romania. The candidate is director of this project for the Politehnica University of Timisoara.

*Validation of the advanced calculation models for fire design*

The validation of advanced calculation models for the fire design is an important issue for computer code developers, designers and authorities. As shown in section 2.2, this topic was one of the preoccupations of the candidate. Within the COST Action IFER, in which the candidate is Co-chairman for WG2, a database of tests and benchmarks is collected from all European countries, with the aim to offer to the designers an appropriate tool for the calibration of the numerical models. This process is still in progress, but it is the intention of the candidate to continue this activity after the conclusion of the COST Action. The possibility to propose an European funded project in order to build a stronger database to cover more materials and structural elements, including in this scope also fire tests in order to fill the gaps, is in discussion with the partners of the COST IFER Action.

*Fire behaviour of steel connections*

As shown above, CMMC laboratory is equipped with a temperature chamber able to heat up electrically up to 600$^{o}$C, which is linked to the dynamic testing machine of 100 tons. Within a national funded project, some tests on steel end-plate bolted connections components are already in progress. It is the intention of the candidate to use this experimental facility for further research, to investigate the behaviour of structural details. For example, up to this moment, there are no dedicated studies for the resistance of slip resistant joints. It is traditionally considered that this type of joints slip at elevated temperatures. However, the values of the corresponding design forces in fire conditions were not yet evaluated, and there is no information about the behaviour of these connections after a fire.

*Simple design recommendations for Slim Floor beams*

The advanced numerical analysis of structures under elevated temperatures is still reserved to experimented researchers and designers, due to the complex phenomena to be considered. On another side, when particular situations which are not covered by the Eurocodes within other calculation models have to be analysed using an advanced calculation model, the numerical model should be properly calibrated through existing tests or benchmarks. Therefore, it is useful for the designers to have simple “rule of thumb” recommendations for the assessment of the fire resistance of particular systems which may be analysed only using advanced calculation models, based on the results of the resistance verifications of the elements at ambient temperature. Such recommendations were presented in section 2.4.3, for the particular case of Slim Floor beams but were based only on an exploratory study on a few cases. More parameters were considered in a study in progress, in cooperation with ArcelorMittal Luxemburg, and studies of this type will be continued also for other proprietary systems.

## C2. Academic

The academic further development plans are also oriented in the direction of fire engineering. One long term objective of the candidate is to properly implement the fire design into the current design practice in Romania.

The candidate already leads lectures on fire design at three Master programs and already organized, in premiere in Romania, three seminars on fire design at the home university.

Further courses targeted on fire design of steel and composite steel-concrete structures (EN 1993-1-2 and EN 1994-1-2), are provided to begin from 2014 at national level for civil engineers, within a larger package on EN 1993 design rules, coordinated by Prof. Dan Dubina. The candidate is in charge with the lectures on the mentioned fire parts of Eurocodes.

The first step for a proper implementation of the fire design practice in Romania is to prepare civil engineers with the knowledge of fire engineering from the university, also through diploma works targeted in this direction. Since 2009, Bachelor and Master Thesis on fire engineering are performed in double coordination by the candidate and by Prof. Jean Marc Franssen, through an ERASMUS agreement. This agreement was prolonged this year and it is the intention of the candidate (and of Prof. Jean Marc Franssen) to continue this collaboration on long term.

## C3. Profesional

The candidate is first of all a civil engineer, and has already a professional activity of design, mainly of steel structures. It was showed that for the very first time, fire design according to the Eurocodes was considered by the candidate in the evaluation of the fire performance of some buildings in Romania.

The candidate is attested by the Ministry of Public Works for expertise and verification of the civil engineering buildings projects for fire safety (verification for projects since 2009, Expert since 2009) and intends to continue this activity, related to the authorisation of civil engineering buildings.

As mentioned within the academic further development plans, one long term objective of the candidate is to properly implement the fire design into the current design practice in Romania. The candidate is member of the professional Romanian association AICPS (Asociatia Incginerilor Constructiori Proiectanti de Structuri). Through the conferences organised by this associations, or by publications in the dedicated review, the candidate promoted the fire design through presentations and articles on the principles of fire design, or on the particular fire design study cases that he conducted. This kind of activities will continue.

In 2009 was founded the Romanian Association of Engineers for Fire Safety (Asociatia Romana a Inginerilor pentru Securitate la incendiu – ARISI). The candidate was among the founder members and is the president of the Timis Region branch of this association and will continue to support the implementation of the fire design in Romania through this association.

The candidate will continue the activity of code drafting.

The Romanian norm for fire safety of buildings P118/ 1999 is subjected to revision, and will include the concept of fire design. The candidate is part of this revision process.

The candidate was also part in the process coordinated by ASRO in Romania, for the translation of the fire parts of the Eurocodes EN 1993-1-2, EN 1994-1-2 and EN 1999-1-2, dealing with the fire design of steel, composite steel-concrete and aluminium structures, together with the corresponding National Annexes. It is the intention of the candidate to propose further application guides for these Eurocodes for fire design.

Through the actual research activity related to the localised fires in which the candidate is involved and through its participation in the European Committee for Standardization - Technical Committee CEN/TC 250/SC 01/WG 04 "Actions on structures exposed to fire", the candidate will participate to the improvement of the natural fire models in the Eurocodes.

The involvement of the candidate in some national and international grants as director or managing team member provided the relevant skills and competences on management of such projects. One important aspect in the further development of the career of the candidate is to build a research team focused in the direction of fire engineering at home university. It is the intention of the candidate to recruit further potential PhD students among the students involved in Master Thesis on the topic of fire engineering, especially from the ones which gain an international experience by performing double coordination thesis within the mentioned collaboration with Liege University. It has to be mentioned that the candidate already trained one young researcher fom CMMC department in the field of fire engineering, Assistant Ioan Both, PhD, by involving him into the COST IFER activity (including participation to 2 STMS – Short term Scietific Missions) and on the related research of the candidate in the field, but also in the teaching activity, by leading the seminars on fire engineering at the three Master courses.

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