STRUCTURAL BEHAVIOUR OF PRESTRESSED CONCRETE HOLLOW CORE FLOORS EXPOSED TO FIRE

STRUCTURAL BEHAVIOUR OF PRESTRESSED CONCRETE HOLLOWCORE FLOORS EXPOSED TO FIRE

Wim Jansze (The Netherlands)

Arnold Van Acker (Belgium)

Bruno Della Bella (Italy)

Ronald Klein-Holte (The Netherlands)

Gösta Lindström (Sweden)

Jean-Paul Py (France)

Matthieu Scalliet (France)

Andreas Nitsch† (Germany)

Hermann Benhöfer (Germany)

ISBN 978-90-8891-812-4
Structural behaviour of prestressed concrete hollow core floors exposed to fire
Jansze, W., van Acker, A. et al., 226 pages
Published 24 January 2014

BIBM
info@bibm.org
Brussels, Belgium

IPHA
info@hollowcore.org
Brussels, Belgium

Uitgeverij BOXPress
Weerdskampweg 13-15
5222 BA ’s-Hertogenbosch
The Netherlands
info@boxpress.nl
+31 73 51 30 704

Executive Summary

The precast concrete hollow core slab is a widely applied and successful floor construction product. The product has been in high demand for the last decades due to its highly efficient design, structural efficiency and lean production method. Every year, around 25 million square metres of precast concrete hollow core floors are built in Europe. The estimated total stock of hollow core floors currently installed in Europe is 1,000 million square meters. The product has been tested intensively on many aspects, including its fire resistance. All tests confirm that floors consisting of hollow core slabs have outstanding load bearing capacity and excellent resistance to fire.

In the years 2000s a few fire tests with premature shear failure of hollow core slabs attracted the interest of the academic world. In late 2007, the extreme fire in the just completed car park with hollow core floors in The Netherlands generated questions on the fire resistance of the product from both clients as well as from regulatory institutions in some European countries.

In order to re-assure these stakeholders, the European project ‘Holcofire’ was therefore initiated in order to gain a complete understanding of the behaviour of concrete hollow core slab floors under fire conditions. The Holcofire project consists of state-of-the-art laboratory fire tests, statistical analyses over 162 standard fire test results, dynamic finite element simulations on fire development and calculations on the load bearing capacity by recognised experts in the field of precast hollow core floor construction and fire testing. The results of the extensive research analyses are presented and detailed in the various chapters of this publication. The methodologies and results have been peer reviewed.

The ‘Holcofire’ study concludes that the proven track record of more than 1,000 million square metres of installed hollow core floors in Europe plus the extensive testing of hollow core slabs in fire laboratories and analysis of the fire in the Rotterdam incident confirm once again that hollow core floor systems meet all regulatory, quality and safety requirements. The Holcofire lessons learned are, firstly, that the product meets regulations and requirements; secondly, that the product performs well when exposed to fire; and thirdly, that, in specific cases, fires in car parks are more severe than standard fires. Based on the knowledge and experiences gained in this European project carried out by experts and reported on in this book, there is no need for further fire testing and modelling. The product performs well under fire conditions, even under extreme fire conditions. The results justify the conclusion that society can continue to rely fully on the solid structural performance of floors consisting of hollow core slabs.

Brief Contents

Acknowledgements

The following experts are gratefully acknowledged for their contribution to the Holcofire project in the form of in-depth discussion and/or review of specific research topics:

- Mr Ch. Baloche, Centre Scientifique et Technique du Bâtiment, France
- Prof. dr J.-M Franssen, Université of Liège, Belgium
- Prof. dr D. Hosser, IBMB, Technische Universität Braunschweig, Germany
- Dr J.F. Jensen, Alectia, Denmark
- Dr N. Kerkeni, H+P Ingenieure, Germany
- Dr G. Rein, Imperial College London, United Kingdom
- Dr E. Richter, IBMB, Technische Universität Braunschweig, Germany
- Dr F. Robert, CERIB, France
- Prof. A.C.W.M. Vrouwenvelder, TNO Delft, The Netherlands
- Prof. dr J.C. Walraven, Emeritus Delft University of Technology, The Netherlands

The following BIBM member organisations are gratefully acknowledged for their participation in the “Holcofire” project as part of the “Hollow Core” BIBM product group:

- Austria Verband Österreichischer Beton - und Fertigteilwerke, VÖB
- Belgium Fédération de l’industrie du béton, FEBE
- Denmark Betonelement-foreningen
- Finland Finnish concrete industry association
- France Fédération Française de l’industrie du béton, FIB
- Germany Bundesverband Spannbeton-Fertigdecken e.v., BVSF
- Italy Assobeton
- Netherlands Bond van fabrikanten van betonproducten in Nederland, BFBN
- Norway Betongelementforeningen
- Portugal Associação Nacional dos Industrias de Prefabricação em Betão, ANIPB
- Sweden Svensk betong, Swedish Concrete Federation

As well as:

- The international prestressed hollow core association, IPHA

Preface

It is my pleasure to present the analyses and results of the “Holcofire” project. After the initial preparations that started back in 2009 and presentation of the project in Brussels in May 2010, the European industry as a whole acknowledged its responsibility and launched the “Holcofire” project. An expert project team and steering group were established and many meetings held with in-depth discussions on the challenges and findings of the various subjects of the project in order to achieve the goals formulated. After four years of hard work, the project was finalised in December 2013. This book presents all the technical knowledge gained during the execution of the project, the goal of which is to disseminate the lessons learned from the Holcofire project to a broader public.

The Holcofire project team was the driving force behind the project. I would like to sincerely thank Arnold van Acker, Bruno Della Bella, Ronald Klein-Holte, Gösta Lindström, Jean-Paul Py, Matthieu Scalliet, Hermann Benhöfer and Andreas Nitsch, who regretfully became ill and died in June 2013, for their valuable contributions and great efforts. It was a pleasure working with all of you and learning from you. A special thanks to Farida Maibeche and Fabienne Robert, who participated in the project team in its early stages.

The Holcofire steering group was responsible for ensuring that the project met the intended goals. I would like to express my genuine appreciation for the excellent guidance of the two chairmen during the execution of the Holcofire project: Stef Maas (until August 2012) and Axel Bauman (from September 2012). I am very grateful for the contributions by the following steering group members between 2010 and 2014: Luc Bresse, Ruud van Groesen, Bert Jongsma, Matthias König, Olli Korander, Eduard van der Meer, and Esko Salo. The coordination and support by Alessio Rimoldi from BIBM is gratefully acknowledged.

I am indebted to the technical staff at Cerib, headed by Cedric Collignon, who performed the experimental work with great enthusiasm. The notable contributions by Andreea Muntean, Manhal Said, and Tomas Plauška from Consolis Technology in parts of the project are also greatly appreciated.

Wim Jansze
6 January 2014

About the Authors

Wim Jansze (The Netherlands) is the project manager of BIBM's project Holcofire. He holds the position of Technology Development Director Building Solutions at Consolis. He is responsible for research, developments and innovations in the business area of residential and non-residential precast buildings including hollow core flooring. Wim Jansze studied at Delft University of Technology the design of concrete and steel structures (MSc, 1993) and specialized in Delft (PhD, 1997) on the shear resistance of externally strengthened concrete structures. Up to 2002 he worked in the fields of precast tunnel linings at Van Hattum and Blankevoort, and precast bridges and precast buildings at Spanbeton. From 2003 up to 2007 he lived in Suriname (South America) where he lectured the design of concrete and steel structures at the Anton de Kom University. Also, he managed the University's research institute INTEC and coordinated the inter-university project VLIR with Flemish universities. Moreover, he assisted Suriname based construction companies Van Kessel Contractors and Spanbeton in developing further prefabrication of bridges and hollow cores. In Suriname he pursued his MBA (2006) on strategy and innovation from the Maastricht School of Management. In 2008 he returned to The Netherlands and started to work for Consolis. Since 2012, Wim Jansze has been a member of the board of IPHA and chairman of the IPHA Technical Committee. Since 2013 he has been a member of the Task Force Fire Safety of the European Concrete Platform.

Arnold Van Acker (Belgium) is Master of Science in Civil Engineering, University Ghent, Belgium (1961). He worked during 40 years in research and development of precast concrete structures at the R&D departments of CBR-Ergon (from 1962) and Partek/Addtek (from 1987). He has been a member of the Belgian standardisation committee for the design of concrete structures, member of the Drafting Committee of EN 1992 (Eurocode 2), convenor of CEN TC 229-WG1, and member of the Belgian NAD committee for EN 1992-1.2 (Structural fire resistance). Retired since 2001, he is still very active in prefabrication. He is an expert of the Belgian Precast Concrete Federation (FEBE), and active in the education of Architecture and Engineering students. In this context he is a board member of the Technical High school for Architects in Brussels. He has written a model lecture course on the design of precast concrete structure. He is a member of the International Concrete Federation fib – Commission on Prefabrication – since 1978 of which he has been chairman from 1986 till 2002. He has been awarded with the FIP medal for outstanding contributions to the development of prestressed and precast concrete. He received also awards from the "International Federation of Precast Concrete" BIBM, from the "European Committee for Standardisation" CEN, and from other Belgian and International organisations. On the subject of fire resistance of prefabricated structures and hollow core slabs, he advised in various studies, amongst others in Belgium, UK, Denmark, Poland, and international projects like Hiteco (fire resistance high/ultra-high strength concrete). Arnold van Acker is internationally recognized through his numerous publications and conferences all over the world.

Bruno Della Bella (Italy) graduated by Politecnico di Milano in 1968, joined in 1975 the ICN Company, now Group Centro Nord. Since 1981, as R&D Manager and Technical Director of the Group, he supported and developed researches also relevant to fire design and new applications of hollow core floor slabs in Italy through ASSAP (International Association of Hollow Core Producers), and participated to the design and start-up in 2003 of the innovative production plant of Group Centro Nord near Verona, for hollow core with depth up to 1000 mm. Member of board of ASSOBETON (Italian Precasters Association), ICMQ (Italian Institute of Certification and Quality), Italian delegate in BIBM and CEN Working Groups of TC-229 WG1 and WG4 and member of fib Commission 6 "Prefabrication" participates since 1990 to the preparation of Technical and Normative documents in the field of the prefabrication and in particular EN Standards and FIB Recommendations relevant to hollow core slabs.

Ronald Klein-Holte (The Netherlands) is manager of the R&D unit (research area: product and application) at VBI, a leading producer of prestressed hollow core elements. Ronald finished his study Civil Engineering at HAN University in Arnhem in 1981. Until 1986 he worked for 3 years at ABT, a consultancy in structural engineering. After this period he started as a structural engineer at the R&D department of VBI. In the last 25 years he specialized in the design and application of hollow cores floors, such as calculation methods, cross section design and research on camber of hollow cores. He was, among other memberships, member of the Dutch standardisation commission VC53 (design of hollow core floors on non-rigid supports). His main current memberships are: Technical Commission of the BFBN; standardisation commission VC20 (concrete structures); fib commission 6 – prefabrication; Dutch permanent commission on prefabrication (Stufib); convenor of CEN TC229/WG1/TG1 (hollow core slabs); chairman of the working group fire of the hollow core suppliers on the Dutch market. In the years 2009-2011 Ronald’s main activities were the works and research due to the Rotterdam fire and the resulting Dutch temporarily design rules for the application of hollow core floors with regard to the fire resistance.

Gösta Lindström (Sweden) holds the position of technical director in Strängbetong, the largest and leading precasting manufacturer in Sweden. He is responsible for R&D, quality, safety and environmental aspects in the company operations. Strängbetong is a member of Consolis group, the largest manufacturer of prefabricated concrete elements in Europe. A part of the work consists of exchange, share and spread of knowledge within the Consolis group regarding building system solutions. Experience from national and international R&D development projects. Active participation in the European standardisation work for precast concrete products. Member of the working group for the Swedish concrete design code and also in the work for selection of national coefficients for Eurocode. Member on European level in the working group attached to the update of the fire part of Eurocode for concrete structures. Several presentations of design considerations in national and international seminars and conferences. Specialised in structural analysis and he has a Doctor of Science degree in the field of Structural Mechanics and Engineering from the Royal Institute of Technology in Stockholm. Long experience of advanced nonlinear static and dynamic FE-analysis as well as a long interest of fire related aspects of the concrete material. Before involvement in the project of fire resistance of hollow core floor he has been participating in reference group work for the Swedish Concrete Association to develop guidelines for concrete and fire specially addressing the problem of explosive spalling. He has also experimental experience from fire testing of concrete containers for nuclear waste as well as concrete wall elements exposed to fire.

Jean-Paul Py (France) is currently responsible for studies and for aspects connected to the standardization at the R&D department at KP1 R&D, the French leading manufacturer in prefabricated concrete products. He is responsible for supervising all R&D projects developed by the company, at both residential and industrial construction levels. Jean-Paul Py specialized in urbanism and civil engineering and graduated in 1980 from the National Institute of Applied Sciences (INSA). He first started his career teaching during 2 years in an engineering school in Tunisia. After this experience (1982), Jean-Paul Py then started managing the department of structural engineering within the OTEP company and held this position for 10 years. Afterwards, he joined GIE Feder Beton (1992), where he held the R&D position of expert in standardization for 2 years. From 2004 up until now, Jean-Paul Py became part of SARET France, which then turned into KP1 R&D corporation. His main objective is to maintain the high quality values set within his department. Additionally, Jean-Paul Py is the chairman of the technical committee of the French Concrete Industry (FIB) and also expert for TG4, TG5, WG1 in the redaction of European standards.

Matthieu Scalliet (France) is a Civil Engineer, Head of Structure Design Service at Structure and Fire Section in CERIB (Centre d'Etudes et de Recherches de l'Industrie du Béton). He received his Engineering Degree in civil and urban engineering from the National Institute of Applied Sciences of Lyon (France) in 2000 and spent one year at the Polytechnic University of Valencia (Spain). He is a member of TC229 WG1 Committee, Eurocodes European Commissions, and specialist national expert group in France (GS3, Atex...). His main research interests include mechanical behaviour of precast concrete products like hollow-core slabs, linear structural elements (beams, columns…) submitted to fire or at ambient temperature. He is in charge of experimental laboratory and in field tests of products at ambient and high temperatures in the framework of European or National projects. He supervises six engineers in his department for PhD, structural design methods and trainings. Previously, he was in charge during 10 years of technical and commercial activities in a desktop named CIBO in Valencia (Spain) which was specialized in precast concrete structures for innovative projects.

Andreas Nitsch (Germany) has been a professor for concrete structures at University of Applied Science in Kaiserslautern since 2009. Since 2004 he has also been responsible for the technical work of the German association of hollow core floors (BVSF e. V.). Andreas Nitsch received his civil engineering degree from Ruhr University Bochum (1990) and his Ph.D.

from RWTH Aachen (2000). Topic of his Ph.D. was “precast und prestressed elements of high strength concrete”, both numerically and experimentally. He worked in the construction industry as designer since 1990 up to 1995, particularly on tunnelling of subway with steel and concrete structures. Since 2000 up to 2002 he was responsible for R&D&T of Imbau, former the biggest precast element company in Germany. After this he worked for the German association of precast buildings (FDB e.V.) as technical worker up to managing director in 2006. Professor Andreas Nitsch died in June 2013.

Hermann Benhöfer (Germany) is Civil Engineer and graduated form the Fachhochschule Hannover in 1975. He had various employments in Germany and abroad in the areas of structural engineering, verification engineering, and site management. Since 1985 he has been working at Brespa/DW-systembau. He started in 1985 at the Technical Department of Brespa. In 2002 he became the Production Manager Hollow core Floors at Brespa, and in 2004 at DW Systembau the Production Manager Hollow core Floors and reinforced and prestressed precast structural concrete products. Since 2008 Hermann Benhöfer has been working as Technical Manager and R&D Manager at DW-Systembau. Hermann Benhöfer is member of the board of Bundesverband Spannbeton Fertigteildecken.

Contents

Chapter One

1

Introduction

Introduction to prestressed concrete hollow core floors exposed to fire

1.1. General

The hollow-core slab has been a very successful product in precast concrete floor construction in residential and non-residential buildings, both in concrete and steel frames. This success is largely due to its highly efficient design and production methods, flexibility in use, surface finishing and structural efficiency. Every year, around 20 to 25 million square metres of hollow-core floors are built in Europe. The estimated total stock of hollow-core floors currently installed in Europe is 1,000 million square meters.

This large flooring market share was mainly gained since the development of the extrusion production methods in the 1970s. Moreover, the successful application is fuelled chiefly by thorough research and publications on the use of hollow cores under ambient conditions. For 50 years, the product has been severely tested by numerous researchers and well-established institutes and therefore meets all regulatory, quality and safety requirements.

An early, well known academic publication by Walraven and Mercx addressed “The bearing capacity of prestressed hollow core slabs” [1983]. Due to the lack of guidelines with respect to specific features, the first international FIP recommendation “Precast prestressed hollow-core floor” in 1988 by chairman Van Acker was greatly appreciated by designers and public authorities. It also served as a reference guide for national standards. The FIP recommendation was followed by the FIP Guide of Good Practice in 1992 “Quality assurance of hollow core slab floors” by chairman Suikka and, in 2000, by the FIB Guide of Good Practice – Bulletin 6 “Special design considerations for precast prestressed hollow core floors”, headed up once again by Van Acker, which reported on the work of Pajari. In the Holcotor project carried out in the 2000-2009 period, shear and torsion were studied by Engström, Lundgren and Broo. In 2005, CEN published the first European standard EN1168 “Precast concrete products – hollow core slabs”, prepared by Technical Committee CEN/TC229. In 2014, the EN1168 will be revisited, and a revised publication of the FIP 1988 recommendation is expected.

1.2. Introduction to fire resistance

Fires in buildings are rare events. Fires are therefore not typically considered a load during the structural design of a building. This simplification is justified on the basis of results from standard fire tests of simple building elements subjected to standard temperature-time curves. The fire safety or fire resistance of a building element is normally indicated in periods of 30, 60, 90 or 120 minutes, meaning that this is the time the product needs to resist the fire. The origins of the standard fire test date from early attempts to make a comparison between different building materials and systems to assess claims of “fire proof” construction in the late 19th century. The standard fire test thus emerged as proof of comparative performance in the most severe possible fire. The result of such tests is a “time to failure” in case of a standard fire; this is termed a fire resistance rating. The current system of fire rating has been in existence since the turn of the last century and has remained largely unchanged since its initial developments, despite major advances in both fire safety science and structural fire modelling [Gales, Maluk, Bisby (2012) Structural fire testing].

The fire resistance of a product is indicated in minutes. After the time indicated, it is acceptable for the product to be considered completely lost. Flames cannot come through within the given fire resistance time, and the product must be able to withstand the load during the required fire resistance time. But after this time, the product is allowed to collapse completely in line with the regulations set. The time a product needs to resist a standard fire is linked to the time needed to safely evacuate the building. However, other factors may also play a role in determining the fire resistance of a building. For example, if the value of the building is small in comparison to its contents, one could decide to create the building using a more fire-resistant material (e.g. concrete).

Normally, the fire resistance of a product is determined using a standard fire test. The fact that the product is totally lost and unusable after the test is accepted worldwide. The only criteria are that the flames are not flashing through and that the load is sustained for the duration of the intended period. However, the fact that a concrete structure must sometimes also be considered completely lost after a fire is not well understood. It is a wide spread misconception, that even after an intense fire - and different from steel or wooden structures - a structure made of concrete is just to be cleaned and repainted and use it as before. Even concrete is not immune to fire. In certain cases after a fire, even concrete is to be replaced completely, just like other building materials.

Fires in buildings are rare events. Therefore, as defined in Eurocode EN1990 “Basis of structural design”, 3.2(2)P and 6.4.3.3(4), fire is to be considered an accidental action. The relevant design situations and associated accidental actions of fire should be determined on the basis of a fire risk assessment. In principle, only the ultimate limit state has to be verified. This means that large deformations and important local damage are acceptable on condition that the following basic requirements are satisfied. The load bearing resistance of the structure or parts of it can be guaranteed for a specific period of time (criterion R), the generation and spread of fire and smoke within the building are limited (criterion E) and that the occupants can leave the building or be rescued by other means (criterion I). Criterion I may be assumed to have been satisfied when the average temperature rise over the whole of the non-exposed surface is limited to 140 °C and the maximum temperature rise at any point of that surface does not exceed 180 °C.

The effects of a severe fire on a concrete component and structure are highly complex due to several phenomena occurring simultaneously. Physicochemical changes in the concrete during a major temperature rise, such as dehydration of the paste and decarbonation; inward ingression of the dehydration and evaporating front, resulting in internal vapour pressure in dense concrete and risk of spalling; differential dilatations in the material itself: for various reasons, the cement paste shrinks at temperatures above 100°C, while the coarse aggregates expand and internal stresses generated inside the components due to the non-linearity of the temperature gradient over the cross-section, more or less intensified by the shape of the components. Consequently, internal stresses, cracking and spalling may occur. However, cracked concrete sections are still able to transfer, to a certain degree, stresses by aggregate interlock, provided that the cracks remain closed. Tests on furnaces reveal that components are still able to carry a major load despite significant damage to the elements. This effectively means that the assessment of a real fire or laboratory test should primarily examine the overall behaviour of the components rather than local damage, provided that the basic requirements are fulfilled.

The fire resistance of an entire concrete structure exposed to fire is a very complex phenomenon. It involves the intensity and extent of the fire, the location of the fire within the structure and the size of the building. In addition, the structural lay-out and components of the building influence the response to a fire. And, finally, there are the dimensions of the concrete elements, concrete composition, axis distance to the reinforcement, moisture content of the hardened concrete, etc. to be considered. Fortunately, concrete structures are not only highly fire resistant, but also have great fire redundancy properties due to their robustness and great load redistribution capacity. This also applies to precast concrete hollow-core floors.

Due to its success and easy construction, the precast hollow-core slab seems to be the most frequently studied concrete element in laboratory fire tests. Many fire tests have been carried out in European laboratories; van Acker started with fire tests on hollow-core slabs in the 1970s, while Fellinger also did important work with his fire test series on shear in the 2000s. The main conclusion from these fire tests is that a hollow-core slab can achieve the required fire resistance, provided that design standards are respected and a good fire test design reflecting the actual use of the hollow-core slab floor is made.

1.3. Reasons for the Holcofire project

Many standard fire tests were carried out in small-scale furnaces in fire laboratories to study the fire resistance of hollow-core slabs. Unfortunately, the tests did not always use a correct small-scale fire test design with hollow-core slabs. A few cases of premature shear failure in standard fire tests were reported in the 2000s, leading to reluctant clients, although in practical applications shear hardly impacts floor design. The question was raised if this constitutes a real structural problem for this type of floor or whether the reason lies in a lack of understanding of the behaviour of hollow-core floors during fire, resulting in poor design, particularly for small-scale laboratory test set-ups. The discussions damaged the good image of the hollow-core slab among clients in some European countries.

As a result, European-wide coordinated actions started already in 2000 under TC229/WG1/TG1 to draft a new standard for shear under fire conditions. Only recently, in 2011, did the European Standardisation Institute CEN publish rules in EN1168:A3 Annex G, the product standard for hollow-core slabs. Amendment A3 provides a formula to design for shear and anchorage for single span hollow-core slabs without shear reinforcement exposed to fire.

The heavy car fire in the just completed Lloydstraat car park in Rotterdam that took place on 1 October 2007 rekindled the interest of regulatory authorities in the fire resistance of hollow-core floors in the Netherlands. In the open car park, which was situated under a 12-storey residential building, six cars were involved in a fire within a short period of time. Within a relatively short time after this fire and the extinguishing activities involved, the bottom flange, or parts thereof, of several concrete hollow-core floor slabs located directly above the seat of the fire came down. It should be noted that the floor as a whole did not collapse. However, more rumours about the poor performance of hollow-core slabs under fire conditions were spreading through Europe. And although no people died and the floor did not collapse in this fire incident, gaps in the knowledge on the behaviour of hollow cores under fire conditions and poor communication of the industry meant that no answers were provided, and clients and public authorities in some countries questioned the suitability of hollow cores under fire conditions.

1.4. Objective of the Holcofire project

The European “Holcofire” project was initiated in 2010 in order to gain a complete understanding of the behaviour of prestressed concrete hollow-core slab floors under fire conditions that would lead to full acceptance in Europe of the use of hollow-core slabs under fire conditions. A European approach would facilitate pooling state-of-the-art knowledge, as well as effective communication, between individual countries. Two series of fire tests combined with desk research were designed to cover the gaps identified and answer questions in order to gain a full understanding of the behaviour of hollow-core slab floors under fire conditions.

1.5. Research approach

The research approach consists of a desk study designed to analyse secondary data, combined with running fire tests and simulations to collect primary data. The desk study for the analysis of secondary data involves collecting and analysing a database with previous fire tests, collecting state-of-the-art knowledge on flexible supports and citing expert opinions on hollow-core slabs on flexible supports under fire conditions. The fire test programme in the newly developed Promethee furnace of CERIB laboratory for primary data comprises two fire test series, namely test series G to check the shear formula proposed in EN 1168+A3 Annex G and test series R to analyse the Rotterdam fire case and influence of restrained conditions. The FEM simulations for primary data cover analyses of the Rotterdam fire using FEM software and building a simple frame model for cross-sectional analysis concerning web cracking and bottom flange spalling.

1.6. Structure of the Book

This book is a collection of six technical papers published by BIBM during the execution of the Holcofire project. Each technical paper contributes to understanding the behaviour of prestressed concrete hollow-core slabs under fire conditions. It should be noted, however, that these technical papers should not be read separately, but as a whole. Holcofire therefore included the six technical papers in this book in Chapters 2 to 7.

Chapter 2 addresses the Holcofire database. A large number of fire tests have been carried out throughout Europe. In 2010, these independent but unexplored fire tests were believed to contain a wealth of information on the behaviour of hollow-core slabs under fire conditions. The Holcofire database on prestressed hollow-core fire tests covers a period of 45 years from 1966 until 2010. This database comprises a collection of 153 fire tests resulting in 162 individual analysable fire test results. This meta-analysis compares the database with the design rules given in the European design standard EN1991-1-2:2004 and the European product standard EN1168:2005+A3:2011, and with the requirements given in the European fire testing standards EN1363-1:1999 (+ EN1363-2:1999) and EN1365-2:1999. This chapter provides an overview of the design rules and requirements and presents the conclusions of the meta-analysis on the accuracy of the design models.

Chapter 3 discusses shear and anchorage according to EN1168 Annex G and the Holcofire G series fire tests. Annex G in the newly published EN1168:2005+A3:2011 provides a formula that has been validated with 9 fire tests that failed in shear as described in the background document. Therefore, in addition to the new data from the database, new fire tests were designed to confirm the formula in Annex G to EN1168:2005+A3:2011 after its publication. The aim of test series G was to check the validity of the shear formula in Annex G to EN1168:2005+A3:2011 using new fire tests. An additional goal was to validate the standardised fire test set-up as described in Annex G.

Chapter 4 covers shear and anchorage of hollow cores on flexible supports. Due to the flexibility of the support, the hollow-core slabs follow the deflection of the support, which reduces the shear capacity of the hollow-core floor. The question is, however, whether the effect of this reduction is limited under fire conditions or whether the fire exposure produces additional stress that reduces the shear capacity of the hollow-core floors on flexible supports even more. As such, this chapter seeks to establish whether the decrease in shear capacity under flexible supports at ambient temperature is magnified by extreme fire conditions or whether the flexible support effect can be disregarded under fire conditions.

Chapter 5 of this book describes the Rotterdam case study. In the early morning of 1 October 2007, a fire broke out in the car park under the Harbour Edge apartment building in Lloydstraat, Rotterdam, the Netherlands, burning six cars and wrecking both the surface of the precast concrete facade and soffit of the hollow-core concrete floor in that area. For five years, numerous investigations were conducted into this so-called “Rotterdam fire” case, focusing especially on the hollow-core floor structure. Holcofire decided to analyse the Rotterdam fire case better. The project summarises clear facts on the Rotterdam fire case in order to inform the international reader and looks back on the research activities conducted and decisions taken, giving the international reader an understanding of the progress in this area. It also addresses how it was handled by the legislative and advisory bodies in the Netherlands. In addition, Chapter 5 provides Holcofire’s point of view on the local damage that occurred during the fire by outlining the delamination process in successive steps.

Chapter 6 of this book addresses floors with restraints and the Holcofire R-series fire tests. Fire cases with the local damage like Rotterdam are rare and the phenomena of local damage are rarely observed in fire tests. This chapter discusses four fire tests (R1 to R4) with restrained deformations conducted as part of the Holcofire R series and should be read in conjunction with Chapter 7. It is believed that these open cores and delamination are a combination of explosive spalling, buckling spalling and horizontal cracking through the webs induced by restraints under fire conditions. Blocking in span direction will have a positive effect on shear behaviour (conclusion of Holcofire G series), but a high level of restraints (in transversal direction) could produce a negative effect on the compressive stresses in the bottom flange of the hollow core. All these phenomena and influences need to be studied in more detail in order to reach valid conclusions. Hence, the aim of fire test series R is to investigate the influence of restrained conditions on spalling of the soffit and horizontal cracking through the webs in hollow-core floors under fire conditions. The restraint is simulated by horizontal transversal blocking in certain design situations, i.e. floor layout, support beam rigidity, structural topping thickness, type of edge structure, age of slabs, etc.

Chapter 7 of this book examines the restraints of floors analysed with the Holcofire Frame Model. If properly designed and constructed, concrete structures can withstand even the most extreme fire conditions. The Rotterdam fire in 2007 and associated local damage to wall and ceiling observed sparked off a detailed technical discussion between academics and structural engineers in the Netherlands about a possible additional failure type for floors consisting of hollow-core slabs. However, a clear fact is that, in the Rotterdam fire case, the load-bearing resistance (R) was not exceeded and the integrity and the insulation (EI) criteria were met. Exploratory research pointed to a specific phenomenon in hollow-core slices and was extrapolated to floors and all follow-up research was based on the same assumptions. However, in order to deal with the cross-sectional models developed in the Netherlands and their flaws, Holcofire developed the more sophisticated Holcofire Frame Model to study the cross-sectional behaviour of a hollow-core slice under fire conditions. It should be noted, however, that a model is by definition a simplification of reality. Consequently, the Holcofire Frame model was developed to study horizontal cracking and spalling more fundamentally, but it cannot show redundancy effects as the real fire case in Rotterdam did. Furthermore, it is important to always bear in mind that the cause of this type of local damage may be found in the generic behaviour of concrete or concrete structures exposed to fire rather than in the specific behaviour of hollow-core slabs.

Chapter 8 concludes the research on the structural behaviour of prestressed concrete hollow-core floors exposed to fire with lessons learned. It outlines the overall conclusions of the research, noting that the product meets regulations and requirements, that it performed well under fire conditions and that the fire in the car park was severe.

2

Chapter Two

Holcofire Database

Meta-analysis on 162 fire test results executed between 1966-2010

Keywords: *prestressed hollow core floor, fire, failure mechanism, statistical analysis, design rules*

Abstract. *Since the 1960s many fire tests have been conducted on prestressed hollow cores slabs and floors in fire testing laboratories in order to evaluate the application of hollow cores under fire conditions. The tests not always behaved as wished and the behaviour could not always be explained fully, let alone, recalculated. This Chapter deals with the meta-analysis of the fire tests that have been gathered in the so called Holcofire database. The Holcofire database consists of 162 independent European fire test results covering a period of 45 years from 1966 to 2010. Content analysis of the database shows that in 91 fire tests a failure did not occur when the test was stopped, while in 71 fire test a failure was observed, either premature or intended. The main failure types observed under fire conditions in the database are bending failure exhibited by exceeding rate of deflection (11x); shear and anchorage failure (42x); shear-bending interaction failure (6x); explosive spalling (5x); horizontal cracking (4x), and other uncommon failure types (3x). A thorough meta-analysis on the 162 independent fire test results shows that nowadays 94.5% of the database fire test results can be explained with the design models and requirements stated in the available European standards (EN1992-1-2, EN1168, EN1363-1, EN1365-2). Statistical analysis in*

combination with the method of maximum likelihood were used to state the accuracy of the design model for the bending capacity of a hollow core cross section, the design model for anchorage and shear capacity for single slabs and floor systems, and for the shear-bending interaction capacity. 5.5% of the fire test results in the database, mainly related to explosive spalling and horizontal cracking, cannot be explained fully. The phenomena are not yet fully understood, although it becomes clear from the fire tests that moisture content, thick topping, and floor restraints are important influencing parameters. Therefore, explosive spalling and horizontal cracking remain subjects for further research in the Holcofire project, and will be reported on in the Chapters 5 to 7.

Review. *The background reports of this Chapter were reviewed by Prof.ir. A.C.W.M. Vrouwenvelder and Prof.dr.ir. J.C. Walraven, Delft, The Netherlands. The integral review text on the background reports is published in Appendix 2.D of this technical Chapter.*

2.1. Introduction

In the 1950s and 1960s the application of the prestressed hollow core slab expanded quickly in building structures throughout Europe as a result of its effective long-line production method and efficient use of (raw) materials. In order to get approval by authorities for the application of prestressed hollow core slabs under fire conditions, producers started to conduct fire tests on hollow core slabs with ISO curve regime in fire testing laboratories. A large number of fire tests were executed throughout Europe. In 2010 it was felt that these independent but unexplored fire tests must contain a huge treasure of information on the behaviour of hollow core slabs under fire conditions. Hence, one of the objectives set by the Holcofire project was to collect all these European fire tests in a database with the purpose to conduct a meta-analysis. This implies that in the Holcofire project (nearly) all original test reports had to be collected, analysed, and main parameters studied and merged into a database to enable a statistical analysis. The Holcofire database on prestressed hollow core fire tests covers a period of 45 years from 1966 until 2010 [2.1]. In this database, 153 fire tests resulting in 162 individual analysable fire test results have been collected. Original test reports of another 22 fire tests could not be retrieved anymore. In this meta-analysis the database is mirrored against the design rules given in the European design standard EN1991-1-2:2004 and the European product standard EN1168:2005+A3:2011, and against the requirements given in the European fire testing standards EN1363-1:1999 (+ EN1363-2:1999) and EN1365-2:1999. A meta-analysis is defined as a systematic method of evaluating data statistically, is based on results on the same problem of several independent studies, and produces stronger conclusions than can be provided by any individual study. This Chapter gives an overview on the design rules and requirements, and presents the firm conclusions of the meta-analysis that states the accuracy of the design models.

2.2. Design rules for resistance to fire from design standard 1992-1-2

EN1992-1-2 considers only bending and spalling. The bending capacity of a hollow core slab exposed to fire may be calculated by using simplified calculation methods according to EN 1992-1-2, or can be assessed by tabulated data given in EN1992-1-2 [2.4]. Tabulated data give recognised design solutions for the standard fire exposure up to 240 minutes. For that, tables have been developed on an empirical basis confirmed by experience and evaluation of fire tests. In the fire resistance design of hollow core slabs, the tables give a minimum floor thickness and a minimum axis distance of the prestressing strands to the exposed surface. EN1992-1-2 Table 5.8 gives minimum floor thickness values that is needed to reach the insulation criteria I, see Figure 2.1. However, the values in the table correspond with the minimum floor thickness for solid slabs. Clause 5.2 (2) states that if calcareous aggregates are used in slabs the minimum dimensions of the cross section may be reduced by 10%.

Standard fire resistance	Minimum dimensions (mm)			
	slab thickness h_s (mm)	axis-distance a		
		one way	two way: $l_y/l_x \leq 1,5$	two way: $1,5 < l_y/l_x \leq 2$
1	2	3	4	5
REI 30	60	10*	10*	10*
REI 60	80	20	10*	15*
REI 90	100	30	15*	20
REI 120	120	40	20	25
REI 180	150	55	30	40
REI 240	175	65	40	50

l_x and l_y are the spans of a two-way slab (two directions at right angles) where l_y is the longer span.

For prestressed slabs the increase of axis distance according to 5.2(5) should be noted.

The axis distance *a* in Column 4 and 5 for two way slabs relate to slabs supported at all four edges. Otherwise, they should be treated as one-way spanning slab.

* Normally the cover required by EN 1992-1-1 will control.

Figure 2.1. Minimum dimensions and axis distances for simply supported one-way solid slabs according to EN1992-1-2 Table 5.8

To determine the axis distance of the strands or wires to the exposed surface, EN1992-1-2 Table 5.8 gives values for one-way spanning prestressed slabs. The minimum axis distance is needed for resistance R, because failure to support the load is directly related to material degradation in the soffit of the prestressed strands and concrete. Hence, the larger the axis distance, the lower the temperature in the strand, and thus the higher the fire resistance. Since the seventies national standards had prescribed a larger axis distance to obtain higher fire resistance time. For prestressed hollow core slabs Table 5.8 needs to be adjusted as Table 5.8 is based on a critical temperature of mild steel of 500 ºC, while the critical temperature for prestressing strands is 350 ºC. Consequently, an increase of 15 mm axis distance is needed according to EN1991-1-2 clause 5.2 (5) when no special check is done. Then, for hollow cores slabs the minimum axis distance is, for example, 45 mm for REI 90. On the other hand, according to clause 5.2.(7) a reduction of the axis distance is possible when more numbers of prestressing strands are applied than needed according to ULS design at ambient temperature.

In design standard EN1992-1-2 clause 4.1 it is written that spalling shall be avoided by appropriate measures, or the influence of spalling on performance requirements (R and/or EI) shall be taken into account. In clause 4.5 it is indicated that explosive spalling is unlikely to occur when the moisture content of the concrete is less than k % by weight: the recommended value of k is 3. It may be assumed that where members are designed in accordance with the requirements for exposure class X0 and XC1, the moisture content is less than k % by weight, where $2{,}5 \leq k \leq 3{,}0$. Above k % a more accurate assessment of moisture content, type of aggregate, permeability of concrete and heating rate should be considered. [Note: This statement is not valid for hollow core floors only, but also for other concrete elements in precast and cast in-situ. Parking garages where exposure class XC3 is applicable (high humidity) have a moisture content above k.] EN1992-1-2 clause 4.5 states further that for floors, if the moisture content of the concrete is more than k % by weight, the influence of explosive spalling on load-bearing function R may be assessed by assuming local loss of cover to one reinforcing bar or bundle of bars in the cross section and then checking the reduced load-bearing capacity of the section. It is noted that where the number of bars is large enough, it may be assumed that an acceptable redistribution of stress is possible without loss of the stability (R). This includes solid slabs with evenly distributed bars. Falling off of concrete in the latter stage of fire exposure shall be avoided, or taken into account when considering the performance requirements (R and/or EI).

2.3. Design rules for resistance to fire from product standard EN1168:A3

EN1168:A3 [2.3] considers bending, shear and anchorage, and spalling. It contains the informative Annex G that gives guidance to calculate the resistance to fire of hollow core slabs. The fire resistance (R) regarding bending and shear and anchorage may be calculated by the following assumptions:

- The temperature in the cross section of the hollow core slab is calculated according to G.1.1. The hollow core slab is divided into two parts A and B, separated by the so called line $a_{50\%}$ on which thwe width of the webs is equal to the width of the cores (see Figure 2.2).

$$a_{50\%} = \text{level on which } \sum_{i=1}^{n} b_{w(i)} = \sum_{i=1}^{m} b_{c(i)} \tag{1}$$

    - In area A below the $a_{50\%}$ level, it is assumed that the temperature is equal to the temperature of a solid slab (see EN1992-1-2 Figure A.2) as presented in Figure 2.3;

- In area B above the $a_{50\%}$ level, a linear interpolation is taken between the temperature calculated at the $a_{50\%}$ level and the temperature at the top of the floor. The temperature at the top of the floor is assumed to be equal to that at the $a_{50\%}$ level, but with a maximum allowed temperature for the insulation criterion of 160°C (140°C + 20°C ambient temperature);

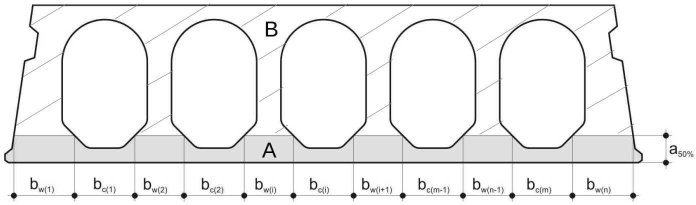


Figure 2.2. Position of line $a_{50\%}$ and area A where solid slab temperatures may be assumed (grey) and area B for linear interpolation (hatched)

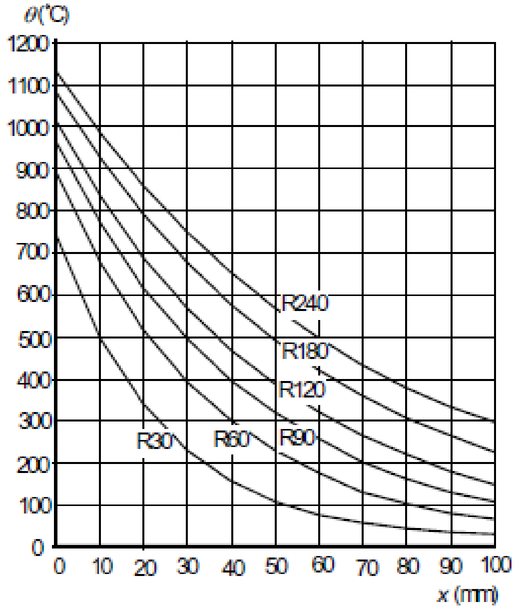


Figure 2.3. EN1992-1-2 Annex A Figure A.2: temperature θ at depth x in slab

- The resistance to bending in fire may be calculated by the using simplified calculation method according to EN 1992-1-2:2004 clause 4.2. In this simplified calculation method the bending capacity of a cross section in ultimate limit state can be determined by multiplying the prestressing force with the internal lever arm, given that the prestressing force should be in equilibrium with the force in the concrete compressions zone. The bending resistance is a force couple between the resulting force of the compression zone at the top of the member, and the tensile strength of the strands. But as the floor is exposed to fire at its soffit only, the compressive zone will remain cold. Therefore, the bending resistance of a hollow core floor is governed by the degradation of the strength of the prestressing reinforcement in function of the temperature. Failure to support the load is directly related to material degradation of the prestressed strands (and surrounding concrete) in the soffit. EN1992-1-2 Figure 4.3 shows curves 1a and 1b for the reduction $k_p(\theta)$ of the characteristic strength βf_{pk} of prestressed steel under fire, see Figure 2.4. The choice for curve 1a or 1b for use in a country may be found in its national annex. In this study curve 1b (class B) is taken, curve 1a (class A) gives a lower reduction, especially between 350 °C and 650 °C. With the temperature profiles given in EN1992-1-2 Figure A.2 (Figure 2.3) at a given axis distance the temperature in a strand is determined. Hence, the simplified expression is:

- The bending resistance under fire conditions $M_{Rd,c,fi}$ is:

$$M_{Rd,c,fi} = N_{p\theta}\, z \tag{2}$$

- In which the parameters are

$N_{p\theta}$ the force in the prestressing steel

$$= \beta f_{pk}\, k_p(\theta)\, A_p \tag{3}$$

with β = (recommended value = 0,9) (class B)

z the internal lever arm

$$\cong 0{,}9\,(h + h_{topping} - e_p) \tag{4}$$

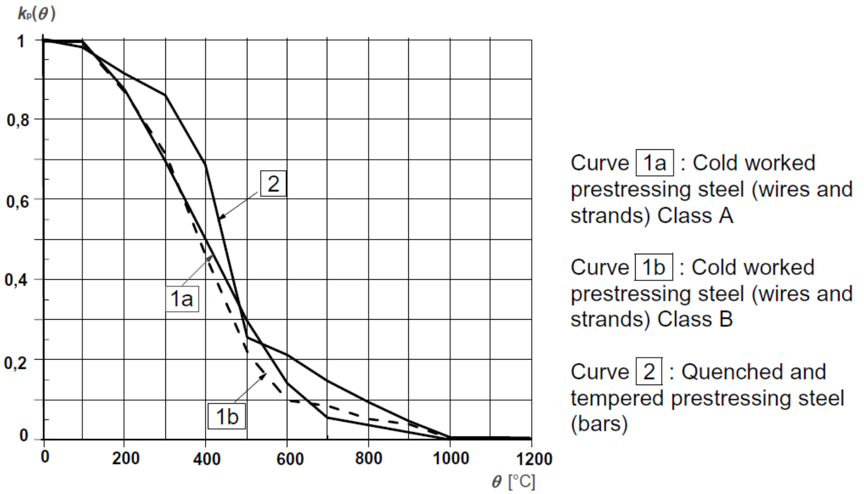


Figure 2.4. EN1992-1-2 Figure 4.3: coefficient $k_p(\theta)$ allowing for decrease of characteristic strength (βf_{pk}) of prestressing steel

- The shear and anchorage resistance in fire may be calculated according to clause G.1.2. This empirical formula is an extension of the formula for the shear capacity of prestressed structural members given in EN1992-1-1. An extension as it takes into account the reduction of the characteristic compressive strength of concrete and the characteristic strength of reinforcing and prestressing steels due to fire. This shear and anchorage verification is only needed for fire resistance classes above 30 minutes, as it has been assumed in the standard (and proofed in this database study) that for times up to 30 minutes the ambient shear tension capacity still governs.

- The shear and anchorage resistance under fire conditions $V_{Rd,c,fi}$ is:

$$V_{Rd,c,fi} = \left[C_{\theta.1} + \alpha_k \cdot C_{\theta.2}\right] \cdot b_w \cdot d \tag{5}$$

- In which the main parameters are:

$C_{\theta.1}$ coefficient accounting for concrete stress under fire conditions:

$$= 0.15 \cdot \min(k_p(\theta_p)\sigma_{cp,20°C}; \frac{F_{R,a,fi,p}}{A_c}) \tag{6}$$

$$\alpha_k = 1+\sqrt{\frac{200}{d}} \leq 2{,}0 \quad \text{where d is in mm ,} \tag{7}$$

$C_{\theta.2}$ coefficient accounting for anchored longitudinal reinforcement:

$$= \sqrt[3]{0.58 \cdot \frac{F_{R,a,fi}}{f_{yk} \cdot b_w \cdot d} \cdot f_{c.fi.m}} \tag{8}$$

b_w total web thickness of the hollow core slab

d effective depth at ambient temperature

$$= (h + h_{topping} - e_p) \tag{9}$$

EN1168:A3 section G2 presents tabulated data on the minimum floor thickness that is needed to reach the insulation criteria I. As EN1992-1-2 Table 5.8 (see Figure 2.1) correspond only with the minimum floor thickness for solid slabs, EN1168:A3-G2 gives a conversion as presented in equation (10) in order to use Table 5.8 for the effective thickness of hollow core slabs. By assuming that A_c = 0,4 bh, for REI 90 and an actual slab height h of 160 mm and a width b of 1200 mm, the effective thickness is 100 mm which is comparable to 100 mm for a solid slab.

$$t_e = h \sqrt{(A_c / b \cdot h)} \tag{10}$$

To prevent spalling, the requirements from EN1168:A3 G.3.5 are used, as a complement to EN1363-1 clause 8.1. The moisture content of the slabs should be representative for the real conditions in a structure (after a reasonable time of exploitation), usually it does not exceed 3 mass percent or 3% m/m. In general, 3 months storage of the slabs in indoor conditions (≈20°C, ≈50%RH) can be considered as acceptable.

2.4. Requirements from fire testing standards EN1363-1 and EN1365-2

The European standards for fire testing EN1363-1 [2.5] and EN1365-2 [2.7], specifically addressing load bearing floors, prescribe the requirements for fire resistance tests on load bearing floors. The standard EN1363-1 prescribes general fire test requirements and fire loading. The performance criteria of EN1363-1 are playing a crucial role in the assessment of fire tests in relation to excessive deflections or rate of deflection. Standard EN1363-1 clause 11.1 gives a clear definition for “load bearing capacity.” The load bearing capacity is the time in completed minutes for which the test specimen continues to maintain its ability to support the test load during the test. Support of the test load is determined by both the limiting deflection and the rate of deflection calculated from the measurements taken. Since relative rapid deflections can occur until stable conditions are reached, the rate of deflection criteria is not applied until a deflection of L/30 has been exceeded. For purposes of the standard, failure to support the load is deemed to have occurred when both of the following criteria have been exceeded for flexural loaded elements:

- Limiting deflection $$D = \frac{L^2}{400d} \quad \text{[mm]} \qquad (11)$$
- Limiting rate of deflection $$\frac{dD}{dt} = \frac{L^2}{9000d} \quad \text{[mm/min]} \qquad (12)$$

In which *L* is the clear span of the test specimen [mm] and *d* is the total depth, that is the distance in the cold situation from the extreme fibre of the compression zone to the extreme fibre of the tension zone of the structural section [mm]

In the standard EN1363-1 [2.5] it is further stated in clause 8 for conditioning that at the time of the test the strength and moisture content of the test specimen shall approximate to those expected in normal service. The test specimen shall preferably not be tested until it has reached an equilibrium resulting from storage in an ambient atmosphere of 50% relative humidity at 23 ºC. If the specimen is conditioned in a different way it shall be clearly stated in the test report. Concrete and masonry elements or specimens containing concrete parts shall not be tested until they have been conditioned for at least 28 days. Massive constructions may take a very long period to dry out.

Important for the floor geometry is fire testing standard EN1365-2 [2.7]. EN1365-2 indicates that the test specimen shall be full size unless the actual size is larger than can be accommodated in the furnace. Because of the limiting size of the available furnaces that were used for fire tests (normally up to 6 m length), EN1365-2 describes that when the actual size cannot be accommodated in the furnace, the dimensions of the test construction shall be such that at least the exposed length is 4 m and the exposed width larger than 2 m, provided the relevant given requirements are accommodated. These given requirements are further stipulated in EN1365-2, and prescribe that the test specimen shall simulate the conditions of the use of the floor or roof construction in practice.

2.5. Holcofire Database with 162 independent analysable results

In the Holcofire database, a total of 162 independent analysable fire test results on prestressed hollow core floor units and floor structures covering a period of 45 years have been collected. An extract of the database is tabulated in Appendix 2.A2; in rows the fire tests are given with its most important parameters tabulated in columns. Every fire test result has been given an unique Holcofire number, starting with the letter H, for example H48. This H48 is VTT-PAL 4450 [1984]; VTT-PAL is the abbreviation of the fire test laboratory (see Appendix 2.B), 4450 the test ID given by that laboratory, and 1984 the year in which the fire test was conducted. To summarise, the collected fire tests from Europe have been concentrated around certain fire test laboratories and test themes, namely:

- First market acceptance tests in Germany at TUB (Braunschweig) in 1966;
- Belgium studies by CBR (Lier) and RUG (Gent) starting in 1971 up to 1999 as pioneering studies to understand the phenomena of individual hollow core slabs and slabs in structures with connections under elevated temperatures, mainly addressing bending in the 1970s and shear phenomena in the 1990s;
- Finnish studies performed by VTT-PAL (Helsinki) between 1971 and 1991 as pioneering studies to understand the phenomena of hollow core slabs in structures under fire. These tests mainly addressed bending and they were executed as acceptance tests for the practical application of hollow cores;
- French CTICM (Mezieres-les-Metz) and Swiss ETH EMPA (Dubendorf) studies on slim floor structures between 1992 and 1996, as well as the tests conducted at SPTRI (Borås) by Peikko in 2009;
- Studies between 1983 and 1996 for market acceptance tests in Austria at IBS (Linz), in Germany at TUB (Braunschweig), in Italy at CSI (Milan) and IG (Bellaria);
- Danish studies executed by DIFT (Hvidovre) and SPTRI (Borås) between 1998 and 2005 addressing shear;
- Dutch TNO (Delft) studies addressing shear and anchorage, performed on double web elements and slabs between 1999 and 2001;
- Study of a complete building structure with hollow core floors under natural fire conditions in UK at BRE (Middlesbrough) in 2007 addressing connections between the slabs and the supports;
- Studies in the Eastern part of Europe between 2001 and 2010 for acceptance tests of hollow cores in new markets in Poland at ITB (Katowice), in Slovenia at ZAG (Ljublana), and in Belarus at RIFS (Minsk);
- Small scale tests on slab slices in The Netherlands by Efectis (Delft) conducted in 2010 focusing on horizontal cracking due to thick structural toppings as a result of the so-called Rotterdam fire case.

Figure 2.5 shows in the graph on the left-hand side the number of fire tests per period of 5 years; about 3,5 fire tests per year were executed. The graph on the right-hand side shows the fire resistance time R obtained in the 162 fire tests. It emerges that 46 tests were exposed to more than 120 minutes of fire, and in 104 tests the fire exposure was between 30 minutes and 120 minutes. Only in 12 fire tests the fire resistance time was under 30 minutes.

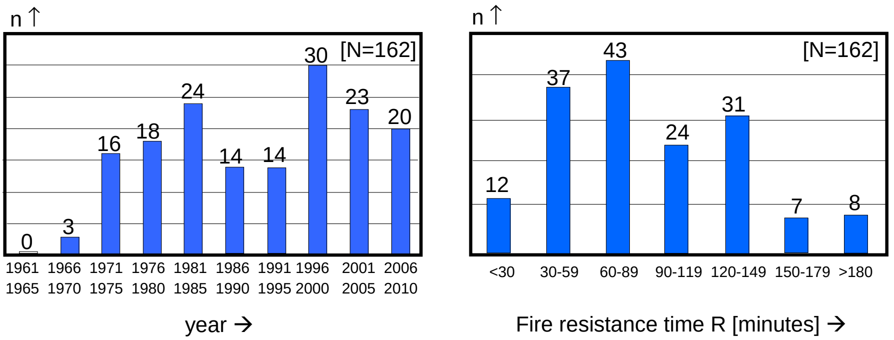


Figure 2.5. Holcofire database; division of fire test results over years, and fire resistance time reached in fire tests

Figure 2.6 overviews the hollow core slab depths used in the fire tests. The database contains 80 fire tests with depths between 241 and 280 mm; this depth is most commonly used in construction practice. Further, the database contains 64 results with lower depths and 18 with higher depths. The span-to-depth ratio used in the fire tests was on average 19,2. In 82,1% of cases it was less than 24, and in 24,1% even less than 12. In practice as a rule of thumb 35 is used. 38 test results have a structural topping on the slab.

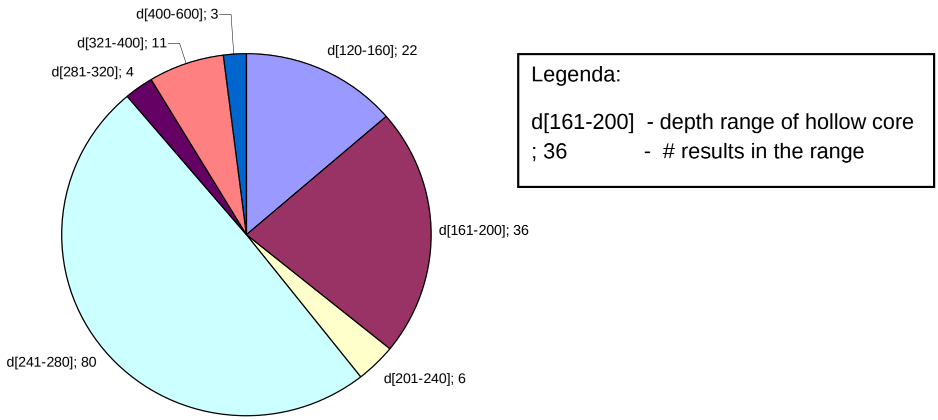


Figure 2.6. Holcofire database; depths [mm] of hollow cores used in fire tests

Figure 2.7 sketches the fire test set-up lay-outs that are present in the Holcofire Database. In 10 fire tests [SLICE] slices supported in transversal direction were used (Dutch Efectis tests). In 19 fire tests [WEBS] double-web elements were used (Dutch Fellinger tests). In 31 fire tests [HCS] only a single hollow core element was tested on the furnace, mostly without connections. In 9 fire tests [FLR] a floor was constructed consisting of 1.5 slabs with one filled joint. 93 floors [SYS] and [SYSB] were constructed as a floor system with 2 slabs or more, and connection reinforcement with the supporting beams, and some peripheral tie beam around the floor. Of that, 19 individual analysable fire test results consisted of a test set up with an intermediate beam in order to study slim floor construction, or in order to study only shear and not bending. As in these fire test configurations actually two floors are present at both sides of the intermediate beam, in 5 cases two different floor configurations were applied on the left and right side. Hence, in the Holcofire database one fire test was then split into two individual analysable fire test results. Accordingly, these 19 individual analysable fire test results the results of 14 fire test set-ups. And in some more fire tests several slabs with for example various thicknesses were tested at the same time. Then, the fire test was split into several results.

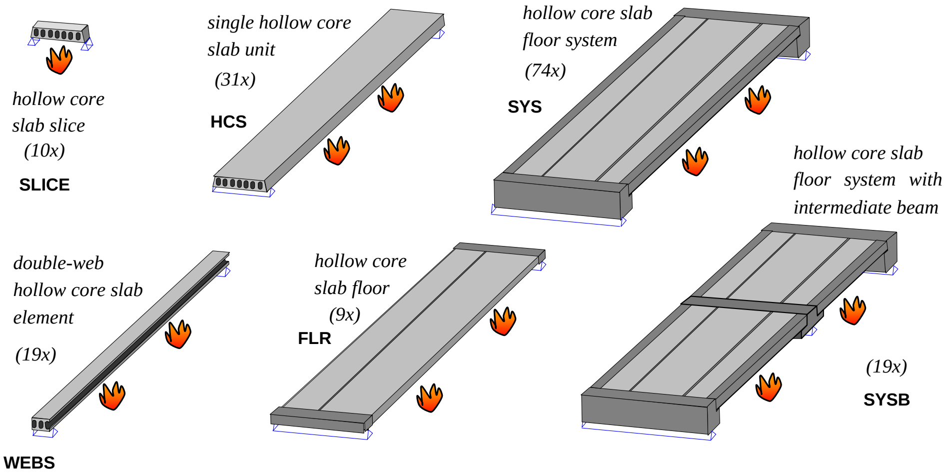


Figure 2.7. Test set-up lay-outs in database (162x); slices, double-web elements, single slabs, floors, and systems

In order to evaluate the fire tests from the database in a consequent way, every fire test has been marked whether it was in compliance with the fire testing standards as mentioned above. When assessing the database with the fire testing standards, it is clear that the requirements were not accounted for in many tests. Mainly at the start in 1960s and 1970s, too small test set-ups with single hollow core slabs were used in fire tests as European standards were not harmonized. But also later in research-oriented tests self-defined test set-up requirements were used. In tests carried out for the market acceptance the requirements from the standards were taken into account. Hence, 89 test set-ups did comply to the fire testing standards, while 73 test set-ups did not comply to the standards as the size of the floor did not comply to minimum of 4 x 2 m^2. It is evident from Figure 2.7 that only the floor systems (SYS and SYSB) did comply as in these configurations only the minimum requirements for exposed length and width could be met. However, in some of the tests on systems the exposed length was significantly smaller than 4 m (H140, H141, H142) and therefore not all system tests conform to standard EN1365-2. In Figure 2.8, which will be explained more in detail in the next section, the green field indicate the 89 fire tests that comply to the fire testing standards, the yellow field indicate the 73 fire tests that did not comply.

When looking at the database in relation to the used fire curve, it became clear that 154 out of the 162 independent analysable results were executed with the standardised ISO 834 fire. In 4 fire tests a parametric or natural fire was used (BRE fire tests in United Kingdom), while in another 4 fire tests (RIFS fire tests in Belarus) heating was performed by means of electrical heating elements according to GOST 30247.0-94.

Regarding the load on the test floor, fire tests have been executed mostly with accidental load or unloaded. In some cases sand bags were used, but according to EN1365-2 point loads shall be transferred to the test specimen through distribution plates, which should not cover more than 16% of the total surface area in order not to disturb the temperature flow through the structure. Note that in some fire tests the load was too high to be resisted.

2.6. Identification of failure mechanisms in Holcofire Database

As defined in the Eurocode “Basis of structural design” EN 1990 3.2(2)P and 6.4.3.3(4), fire is to be considered as an accidental action. The relevant design situations and the associated accidental actions of fire should be determined on the basis of fire risk assessment. In principle only the ultimate limit state has to be verified. This means that large deformations and important local damage are acceptable on condition that the following basic requirements are satisfied; the load bearing resistance of the construction or parts of it, can be assured for a specific period of time (criterion R); the generation and spread of fire and smoke within the building is limited (criterion E); and the occupants can leave the building or can be rescued by other means (criterion I).

The action effects of a severe fire on a concrete component and/or a concrete structure are very complex due to several phenomena occurring at the same time. As a consequence, internal cracking and spalling will occur. However, cracked concrete sections are still able to transfer, to a certain degree, stresses by aggregate interlock, on condition that the cracks remain closed. Tests on furnaces indeed revealed that components were still able to carry an important load despite extensive damages to the elements. Practically, it means that in the assessment of a laboratory fire test (or a real fire), one should principally look to the global behavior of the components, rather than local damage, on condition that the basic requirements are fulfilled. From this viewpoint the database was assessed.

The most interesting on the Holcofire database is that it provides independent but registered information on controlled tests on hollow core slabs and floors under fire conditions in order to verify design models. In most of the cases the objective of the fire tests was to reach a certain fire resistance time. In some cases premature failures took place. In other cases tests were intended to fail in order to study a certain failure mechanism more thoroughly from a research point of view. All the fire test reports in the Holcofire Database have been analysed and the failure mechanisms are summarised in Figure 2.8. The following groups can be distinguished:

- In 102 fire test results the fire resistance R(EI) was obtained and granted as failure did not take place. Of these, in 80 fire test results the fire test was completely stopped and reported on. In 22 fire tests, the test continued. Of that 22, in 8 fire tests the test was finally stopped without any failure and in 14 fire test a failure occurred. These tests were executed either with continuing fire under same loading, or without fire and increasing the load. This “additional testing” was also documented in the test reports.
- In 60 fire test results the fire resistance time was not granted. In 3 fire tests the test was stopped without a failure. In 57 test results the researchers observed a failure before a targeted R(EI) was reached, either unexpectedly or intended. The fire resistance time varied between a range from 10 minutes up to 135 minutes.
- In total in 91 test results a failure did not occur, and in 71 (57 + 14) test results a failure did occur;
- The observed failure mechanisms in the 71 fire test results were;
    - o 11 fire test results resulted in a bending failure of the cross section;
    - o 42 fire test results exhibited a clear shear and anchorage failure;
    - o 6 fire test results exhibited a combined shear-bending interaction failure;
    - o 5 fire test results showed extensive explosive spalling;
    - o 4 fire test results showed clearly horizontal cracking through the webs;
    - o and in 3 fire tests another failure type occurred (punching, bond, and unknown).

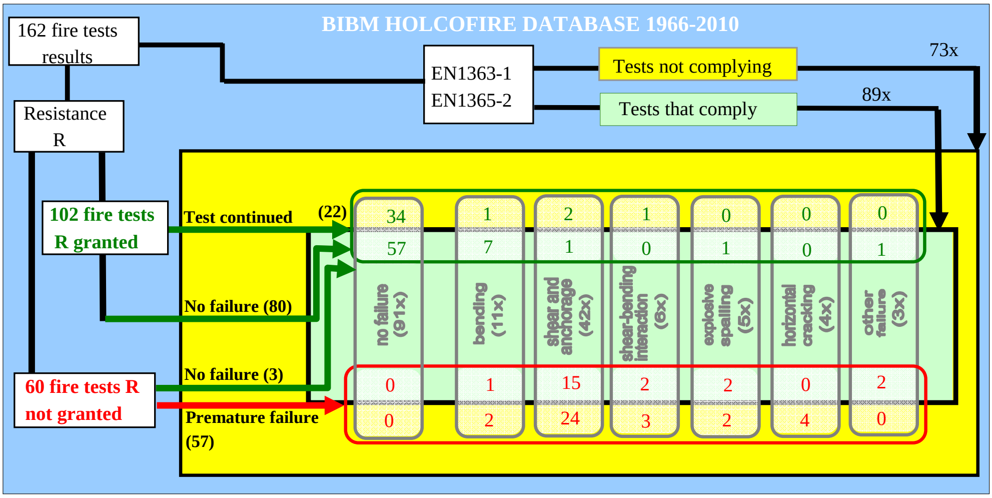


Figure 2.8. HOLCOFIRE fire test database comprising 162 fire test results on hollow cores

2.7. Bending resistance under fire conditions

To determine the bending resistance under fire, the ultimate load bearing capacity of a heated cross section was calculated using a simplified cross-section method and taking into account the reduction of the characteristic strength of prestressing steel. The bending resistance of a hollow core floor is governed by the degradation of the strength of the prestressing reinforcement in function of the temperature.

It is concluded that the calculated strand temperature with EN1992-1-2 Figure A.2 gives a very good prediction of the mean temperature in the strands. The analysis on 25 fire tests shows that the ratio of average-measured temperature over calculated temperature is 99.8% while the coefficient of variation is 14.9%. These 25 fire tests cover fires from 45 minutes to more than 2 hours and contain hollow core slabs with axis distances between 30 mm to 60 mm and with mean strand temperatures ranging from 250 ºC to 500 ºC. But although EN1992-1-2 Figure A.2 gives very accurate results for mean temperatures of the strands, it is also evident that in a fire test the scatter of the highest temperature could easily be more than 30% higher than the mean temperature of the strand.

As an example fire test H67 is recalculated. Figure 2.9 shows the bending failure mechanism of fire test H67 in which the rate of deflections was exceeded at 122 minutes. From EN1992-1-2 Figure A.2 (see Figure 2.3) one can calculate that after 122 minutes and 54 mm axis distance the (mean) temperature in the strands is 367 ºC. Then, from EN1992-1-2 Figure 4.3 (see Figure 2.4) it can then be calculated that for fire test H67 at 367 ºC the ratio $k_p(\theta)$ is 0.546 (line 1b for class B). Hence, the ultimate stress in the strands with quality FeP1860 was f_{py} = 0.546*0.9*1860 = 914 N/mm^2. With that it follows that $N_{p\theta} = A_p \times \sigma_{fire}$ = 416 mm^2 x 914 N/mm^2 = 380 kN. With an internal lever arm of 0.9 d = 0.9*((240+60)-54) = 221 mm, the cross section could resist a moment of $M_{R,fi}$ = 84.1 kNm. The moment present in the cross section under the loading point was $M_{S,fi}$ = 74.3 kNm. Hence, the calculated use of the cross sectional capacity was $M_{S,fi}$ / $M_{R,fi}$ = 74.3 / 84.1 = 88.3%. This is not equal to 100%, and this can be mainly explained by the fact that at some locations the temperature in the strands, which was not measured, was most probably higher as discussed above. Another explanation could be found in the fact that the axis distance of the strands could also differ from the theoretical position. If class A was assumed for the prestressing steel (β = 0.984 and $k_p(\theta)$ = 0,566) , the use of the cross sectional capacity should be $M_{S,fi}$ / $M_{R,fi}$ = 74.3 / 95.2 = 78.0%.

In the recalculation of the bending capacity according to EN1992-1-2, at the time the fire resistance R was granted (and not at the time the test ended), it was evident that in 99 of 102 fire test results the bending moment acting on the middle of the span during the fire testing time was lower than 100% of the calculated bending capacity of the hollow core slab cross section. When we consider the time the fire test was stopped, in 96 fire tests the bending moment was lower than 100% of the calculated bending capacity (see Figure 2.10). Of the points above 100%, 3 fire tests failed indeed in bending as the rate of deflection criterion was exceeded. By taking into account some scatter, it is concluded from the recalculation of the

102 fire test that the simplified expression shows clearly whether a bending failure should take place or not.

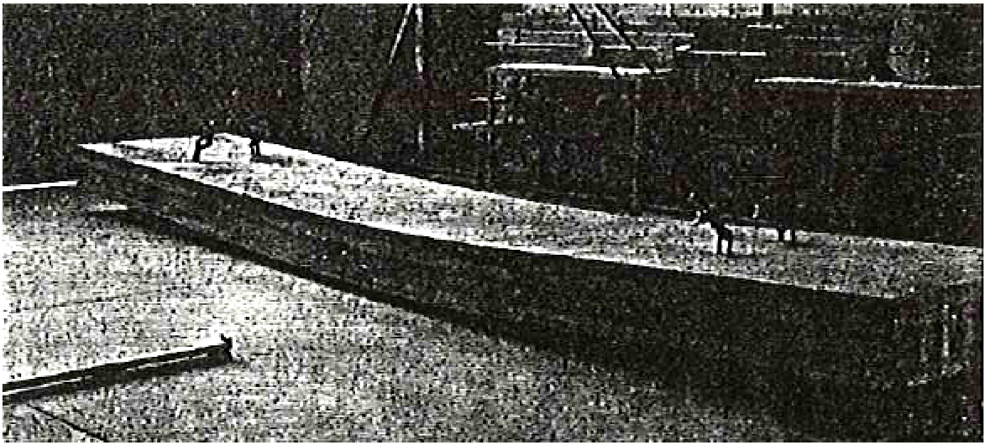


Figure 2.9. Fire test H67: Bending failure expressed by exceeding rate of deflection; large deflections visible at mid span (Note that a better photo is not available, as in many original reports either a photo is lacking, or is not always clearly prresented)

In 11 fire tests from the database the bending capacity was exceeded. These tests were stopped as according to EN1363-1 the rate of deflection was exceeded. Recalculation of these 11 fire tests shows that the increasing rate of deflection can be attributed to material degradation of the prestressing strands leading to a bending failure. It emerges that the average of the fire tests results in which the bending capacity was governing is 96.9% (ratio M_{exp} / $M_{R,fi}$), with a coefficient of variation of 24.0%. Note that in fire test H139 some local spalling occurred, so that the cover was less and thus the strands were quicker heated and the ratio M_{exp} / $M_{R,fi}$ is under 100%. The low ratio of fire test H19 is somewhat unclear, but reading the test report it emerged that vertical longitudinal cracks occurred in the slabs (what is normal in case of single slabs or 1.5 slab test set-ups), and some local spalling. Also there the temperature could be much higher than calculated with EN1992-1-2 Figure A.2. When 88 of the 102 fire tests results (the fire tests performed on slices were neglected, and tests should not be counted double) are considered that did not fail by bending the average improves. With the method of maximum likelihood it emerges that the average of ratio (M_{exp} / $M_{R,fi}$) with the 11 plus 88 results increases to 106.1% (ratio M_{exp} / $M_{R,fi}$), with a coefficient of variation of 22.8%.

The main conclusion in relation to bending capacity is that the Eurocode EN1992-1-2 to calculate the bending capacity under fire gives very good and safe predictions (106.1%) on the ultimate bending capacity for hollow core slab floors exposed to fire. The analysis confirms that in general, there is hardly any discussion on the bending capacity.

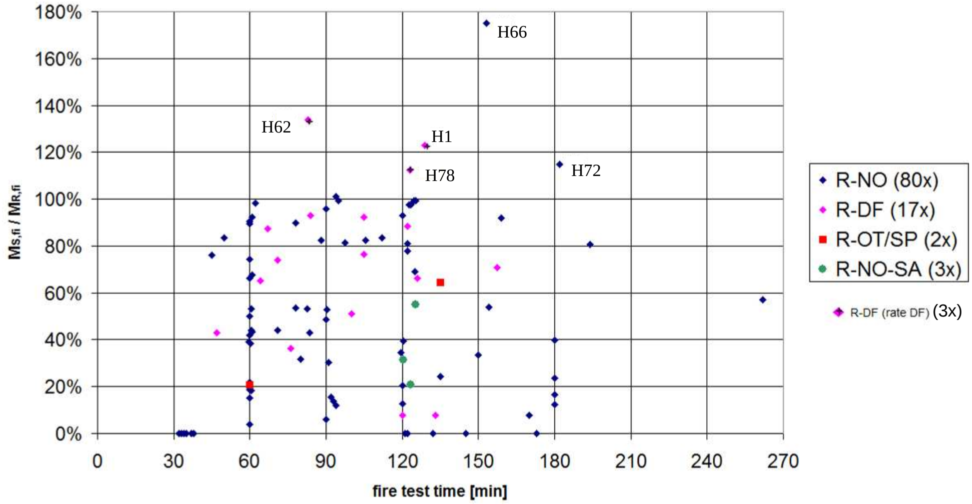


Figure 2.10. Bending moment over bending capacity versus time of 102 tests that did not fail in bending

2.8. Shear and anchorage resistance under fire conditions

In the recalculation of the shear and anchorage capacity according to EN1168 Annex G, the 42 fire tests that exhibited a shear and anchorage failure were analysed as well as the 102 fire tests that reached the required fire resistance time R (do note that 3 tests are overlapping). We consider a total of 141 (102 + 42 - 3) fire tests. In 99 fire tests a shear failure did not occur, in 3 tests the fire was stopped and the slabs were loaded to shear and anchorage failure afterwards, and in 39 fire tests shear and anchorage failure occurred during the fire test.

This study evaluated at first the formula of EN1168:A3 Annex G formula with 42 fire test results carried out in numerous laboratories throughout Europe that exhibited a shear and anchorage failure (See Appendix 2.A1 for overview of 42 fire tests and analysed results) . In 39 fire tests failure occurred during the fire test, in 3 tests the slabs were loaded to shear failure afterwards. In these 42 fire tests shear and anchorage failure played a different role. In only 20 fire tests (RUG, VTT, EMPA, DIFT, ITB) the slab(s) failed unexpectedly and prematurely, which was the main cause of discussions in Europe and deterioration of the good image of hollow core slab. The other tests were designed to fail during the fire test in order to study systematically shear. As it emerged from the analysis, the concrete strength has a significant influence on the shear capacity according to EN1168 Annex G. All the fire tests were recalculated with concrete strength at 28 days, but also the strength at the day of testing has been used in the analyses. In order to get an idea about concrete age and conducting a fire tests, it is good to state the best practice that the time between production of the hollow core slabs and the fire tests would be normally 6 months or 180 days. Hence, the concrete age of the prestressed slabs would normally be 180 days or more.

One fire test is recalculated as an example. The failure mechanism of test H122 is given in Figure 2.11; a clear shear crack is visible in the shear span as well as a crack at the level of the strands. $V_{Rd,c,fi}$ of this test is calculated according to formula EN 1168 with the following parameters (note that slab width is 445 mm): support length 100 mm, h = 258 mm, A_c = 63865 mm^2, b_w = 130 mm, $a_{50\%}$ = 61 mm, $f_{cm,28}$ = 63 N/mm^2, aggregate = silicious, f_{yk} = 500 MPa, A_s = “no reinfo”, coldworked steel, y_s = not applicable, σ_{pt} = 900 MPa (this parameter has no influence on end result), type of prestress is strand, $Ø_p$ = 12.5 mm, A_p =186 mm^2, and y_p = 58.5 mm. The result of this calculation $V_{Rd.c.fi\ (123)}$ = 42.9 kN/m. The shear load was 29.0 kN/m, so that failure was at 68% of the calculated capacity. Alike test H122, all 42 fire test have been recalculated, and results are given in Figure 2.12 where a distinction was made between double-web elements, single slab units, and floor systems.

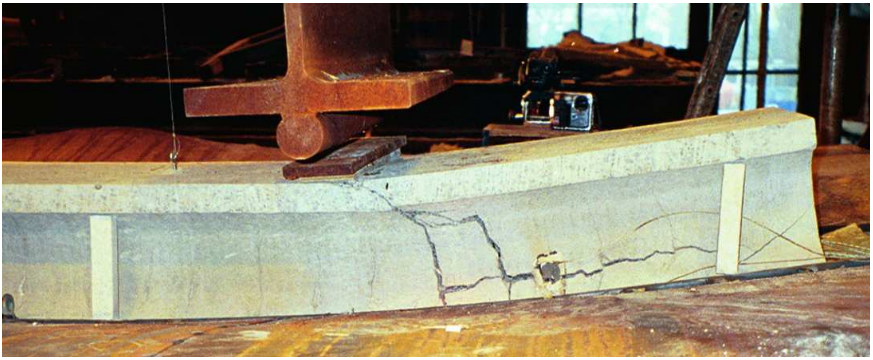


Figure 2.11. Fire test H122: Shear and anchorage failure; shear crack visible at shear span and crack at level of strands

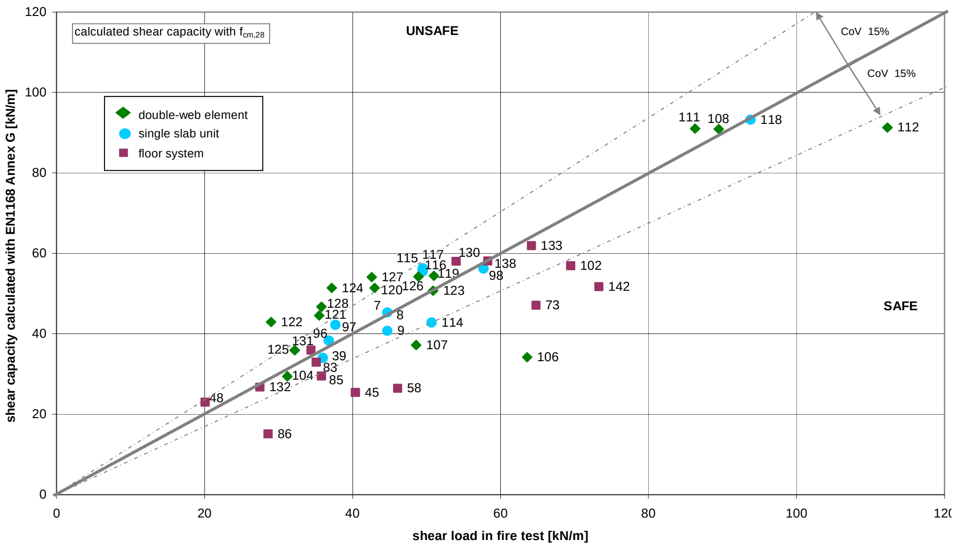


Figure 2.12. Relation shear capacity from fire test versus shear capacity calculated with EN1168 Annex G

By recalculating the shear and anchorage capacity according to EN1168 Annex G of 102 fire tests that did not fail by shear and anchorage in the real fire test, it was demonstrated that in 80 of the fire test results the actual shear load was lower than the calculated shear and anchorage capacity according to EN1168 Annex G, while in 22 fire tests the shear load was higher than the calculated capacity (Figure 2.13). However, 18 out of these 22 fire tests showed a higher capacity by means of the system effect since the fire tests were conducted on floor systems. As became evident from the study, the system effect is not accounted for in the EN1168 Annex G formula. The “system effect” is an increase in shear capacity mainly caused by the introduction of a longitudinal blocking effect that closes the vertical cracks and acts positively on the shear and anchorage capacity. Therefore, this system effect explains that the actual shear capacity in the 18 tests is higher than calculated with Annex G. Of the other 4 tests that were not floor systems, one specimen was actually restraint in longitudinal direction; one did really fail in shear, while the other 2 may fall within the scatter of the calculation.

Then, a statistical analysis was made using the results of the 42 recalculated fire tests that failed including the results of the 102 fire tests that did not fail (maximum likelihood method, see Appendix 2.C). In general, from this meta analyses that evaluates the empirical formula of EN1168 Annex G formula with 42 fire tests carried out in numerous laboratories that exhibited a shear and anchorage failure, and 102 fire tests that reached the required resistance time, it is firmly concluded that the EN1168 Annex G shear formula for hollow core slabs is safe for the application of hollow core slabs in floor systems.

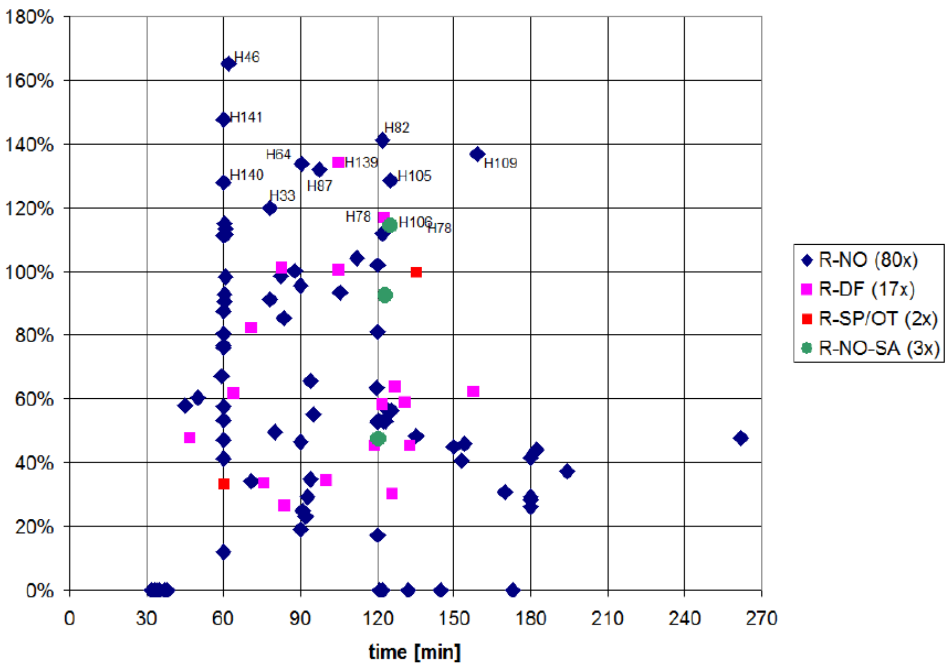


Figure 2.13. Shear load over shear capacity versus time of fire tests in which R was granted of 102 tests

The following conclusions are drawn from the meta-analysis on the fire test results:

- EN1168 Annex G does not account for the system effect (safe approach). From the analysis it became evident that the system effect has a positive influence on the shear and anchorage capacity. Therefore, the conclusion is splitted in two parts: for single slab units without system effect, and floors with system effects. Both the 42 fire tests that failed as well as the 92 fire tests that did not fail are included:
    - Single slab without system effect. When taking into account 28 of the 42 fire tests that failed in shear and anchorage, and by the maximum likelihood method 27 of the 92 fire tests that reached the required fire resistance time (and not count the 3 shear tests and the 7 fire tests on slices), the fire tests results are 98.8% of the calculated shear capacity, and coefficient of variation is 22.3%. This is a single slab which has no interaction with a surrounding structure:
        - The slab is simply supported;
        - the slab is either without or with connection reinforcement placed in the joint or core anchored to the support structure (connection reinforcement at mid height or lower);
        - a structural topping could or could be not present on the single slab;
    - Floor with a “system effect”. When taking into account 14 of the 42 fire tests that failed in shear and anchorage, and by the maximum likelihood method 65 of the 92 fire tests that reached the required fire resistance time (and not count the 3 shear tests and 7 fire tests on slices), the fire tests results are 129.0% of the calculated shear capacity, and coefficient of variation is 24.3%. This is a floor in which there is interaction with the surrounding floor field:
        - the slabs are cast against the support (either a beam or a wall);
        - the slabs are either with or without connection reinforcement placed in the joint or in the core and anchored into the support structure (connection reinforcement at mid height or lower);
        - the joints are filled between the slab to form a floor field;
        - a structural topping could be present;
        - a peripheral tie beam, or equivalent, is cast around the floor;
    - In practice, a local fire on a large floor already induces a system effect by the unheated surrounding slabs.
    - The scatter in time with coefficient of variation of 75.9% for all 42 fire tests is however very high. When taking into account only the 14 system and when the 3 “outliers” are neglected, the coefficient of variation for time decreases to 37.0%.
- The fire test database on shear and anchorage contains hollow core depths ranging from 185 mm to 400 mm. These 42 tests consist of 30 fire tests on 255-275 mm deep hollow core slabs, 8 fire tests on 185-220 mm deep slabs, and 4 tests on 400 mm deep slabs. It is concluded that the tests on 400 mm hollow core slab depth and 185-220 mm hollow core slab depth show the same dispersion around the

theoretical line as the tests with 255-275 mm depth (see Figure 2.12 in combination with Appendix 2.A2). Hence, no outliers have been identified at the lower depths or the higher depths. Therefore, it is concluded that EN1168:A3 Annex G is valid for all heights between 185 mm and 400 mm.

- The decrease in shear capacity in time due to a fire, as calculated with EN1168 Annex G, is clearly observed in independent fire tests with more or less identical cross sections. It is evident from fire tests that when a lower shear load is applied on the hollow core, a longer fire time is achieved.
- It is verified through the fire tests in the database that when the amount of strands in a hollow core is increased, the shear capacity increases.
- It is verified through the fire tests in the database that when connection reinforcement is included in the test floor, the shear capacity increases.
- It can be concluded from the fire tests and shear capacity calculations at ambient temperature (shear flexure according to EN1992) that the shear capacity calculated for fire at 0 minutes is on average higher than 100% of the flexural shear capacity at ambient temperature, and 70% at 120 minutes. In between, a linear interpolation can be applied.
- Further, the outcome of Annex G shear capacity is mainly sensible to parameters as amount of connection reinforcement, amount of prestressing reinforcement, mean concrete (cylinder) strength, and geometry of the slab.
- It is recommended that EN1168 Annex G states that for fire calculation η_1 = 0.7 (bad bond conditions) should be used, as this value (implicit parameter) has been used for validation calculations. For protruding strands and wires, for the protruding part beyond the hollow core head η_1 = 1.0 can be used.

From this meta analyses, that evaluates the empirical formula of EN1168 Annex G formula against 42 fire tests carried out in numerous laboratories where shear and anchorage was governing failure, and 102 fire tests that did not fail in shear when R was granted, it is strongly concluded that the EN1168 Annex G shear formula for hollow core slabs is safe for the application of hollow core slabs in floor systems.

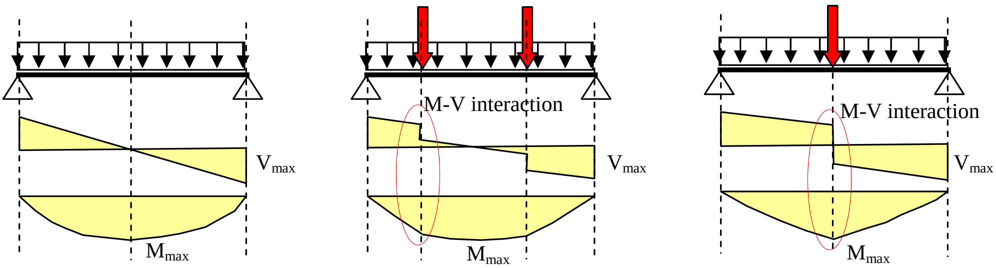


Figure 2.14. Shear-bending interaction in fire tests due to use of point loads in experiments

2.9. Shear-bending interaction resistance under fire conditions

In 6 fire tests from the database the researchers stated that a failure occurred, but the cause of the failure was not clear or misjudged. In the Holcofire analysis it emerged that shear-bending interaction was the governing failure type. In general, it is stated here that in a fire test it is not so easy to simulate a live load. Using point loads is needed to add live load in a test, but the moment and shear distribution over the floor is not comparable with practical applications where there is always a more distributed load. The standards EN1363 and EN1365 prescribe that it is not allowed to place load provisions covering more than 16% of the surface on the top of the floor, as this might influence the temperature flow through the floor during the fire test. Hence, point loads or line loads need to be applied. By consequence, this forces cause a high shear force while the bending moment is at its maximum; the so called V-M interaction. Figure 2.14 shows the problem of point loads schematically for different test set ups. This is a typical problem in laboratory tests, as in practical situations with distributed loads this does not occur. In practice, mostly, the maximum shear load is acting at the support where the bending moment is low, while the maximum bending moment is acting at midspan where the shear load is low.

Figure 2.15. Fire test H91: a shear-bending interaction failure

In the standards shear-bending interaction is not defined. Formula 12 gives a definition that has been used in this study; a combination of the formulas (6) and (3) has been used. This formula has not been officially published, but is discussed in task group TG1 of working group WG1 of technical committee CEN TC229. Hence, for shear-bending interaction in the relevant cross section the following unity check is used:

$$\frac{V_S}{V_{R,fi}} + \frac{M_S}{M_{R,fi}} \leq 1.0 \tag{13}$$

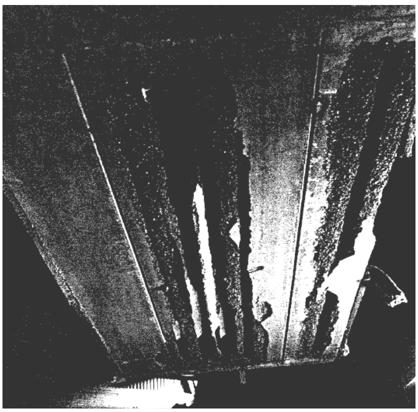
The analysis of the 6 fire test results showed that with this interaction formula the average of the ratio (V_{exp} / $V_{Rd,c,fi}$ + M_{exp} / $M_{Rd,c,fi}$) is 125%, and coefficient of variation is

17%. Hence, the interaction formula gives a safe prediction of the failure mechanism shear-bending interaction. Figure 2.15 shows fire test H91 in which a shear-bending interaction failure occurred.

2.10. Explosive spalling under fire conditions

In five fire tests extensive explosive spalling occurred leading to local damage of the slab. Figure 2.16 shows the soffit and top surface of H60. Most probably the moisture content was high; this was estimated to be between 2.5% and 3.0% but can be much higher in reality. However, the moisture content in some slabs could not be retraced, as measurements were not taken in the slab. In the database of Holcofire there are six other fire test results in which explosive spalling also occurred but it did not lead to failure in the test. There, the moisture content was estimated to be between 2.0 and 2.5%. But as the moisture content was not measured in many fire tests, the number of days the slabs were dried to get the moisture content down could be used as a proxy. Then it is concluded that the slabs were very young in fire test H103. This could also explain well the observed extensive explosive spalling.

To prevent explosive spalling it is important to get the moisture content down with enough drying days of the slabs. In general, 3 months storage in indoor conditions (≈20 °C, ≈50% RH) can be considered as acceptable. But note that the time between production of the hollow core slabs, and the fire tests, would then be normally 6 months or 180 days. In the experience of Holcofire a slab age of minimum 180 days is needed to exclude explosive spalling: 1 month normal curing for the slabs to get their 28 day strength under normal conditions, subsequently followed by 3 months drying under controlled conditions, then 1 month of test preparation (wires and so on), assembling (reinforcement, surrounding structure) and casting of joint and/or topping, and finally 1 month for curing of the joints and/or topping, and ofcourse, the preparation of the fire test. This results in about 180 days or 6 months.



Spalling during the fire test

Part of hole that occured at 39 minutes

Figure 2.16. Fire test H60 with extensive spalling stopped after 66 minutes

It is concluded from the fire tests in which explosive spalling occurred, that from the given moisture measurements it is not evident whether the slabs did contain concrete with a moisture content above the 3%. In one fire test the slabs were used for the third time, with very young slabs, while in another test lightweight aggregate was used from which it is known that it is spalling sensitive. It is remarkable that all fire tests with explosive spalling were conducted on floor systems with restrained conditions. In all 5 fire test cases the floor itself did not collapse. Hence, as the fire tests do not fully explain the influence of the moisture content and restrained conditions, explosive spalling remained subject for further study in the Holcofire project, and is reported on in Chapters 5 to 7.

2.11. Horizontal cracking of the webs under fire conditions

It is evident from the database that in case of 4 fire tests, horizontal cracking of the webs occurred and the specimen failed. But it is also clear from the database that this only occurred in the 3 Dutch tests on hollow core slices (H153, H154, H159) and double-web element loaded in shear (H110), while this was not observed in the other 158 fire test results. In the Efectis fire tests performed on slices the tested specimen consisted of extreme toppings with a thickness of 100 mm or 300 mm, or with the addition of external restraints. In Figure 2.17 the failure mechanism of fire test H153 is clearly visible; in a slice of a hollow core with a 300 mm topping a horizontal crack initiated after 10 minutes through the webs and the under flange separated from the concrete specimen. In the TNO test the double-web element was restrained longitudinally by means of a jack in such a way that a horizontal crack was initiated at the level of the strands that led to failure. By a simple hand- and computer model it was demonstrated in The Netherlands that bending moment and tensile forces in the second web led to these horizontal cracks as a result of the thick restraining topping. External restraints from the floor were not investigated in the Dutch study.

Figure 2.17. Fire test H153 with 300 mm topping and horizontal cracks through the webs

A design rule does not exist for horizontal cracking, but in The Netherlands an intermediate measure is used prescribing that a structural topping may not exceed a certain thickness without applying additional measures. As there is not yet a design formula, neither an accepted design model to analyse the results, it has to be concluded that the mechanism of horizontal cracking through the webs is not yet fully understood, although it becomes clear that in fire tests a thick topping and restraints are important parameters. This failure mechanism therefore was studied more in detail in the Holcofire project and is reported on in Chapter 5 to 7.

2.12. Fire tests with another failure type and maximum deflections

In case of 3 fire tests the failure type was different from the ones we described; one test showed bond problems with 15.2 mm diameter strands in a 2.4 m wide slab, one test showed punching failure, and the failure mode of another test could not be retrieved from the authentic test report, however, in this case failure occurred after the fire resistance time was granted. The results of these tests can therefore also be understood, but do not say much about the overall behaviour of hollow core slabs under fire conditions. They do say that doing a fire tests can lead to unexpected outcomes.

Finally, it is remarked that in 11 fire tests the deflection criterion L/30 was exceeded, and the fire test was stopped. Hence, in these tests the L/30 value was kept as maximum deflection criterion. However, according to current standards instead of L/30 the value of L^2/400d should be the limiting value, which actually gives a larger limiting value. In all cases the fire tests should have been continued as no failure occurred. The thermal gradient over the height of the cross section explains the linearly increasing deflections during a fire test causing the deflection of the floor towards the fire. But in practical applications, where structures are not statically determinate but redundant, the deflections will not govern. And even, in practice, steel structures with steel-concrete floor make use of the tensile membrane action during fires which is actually only possible for large deflections. Hence, in the analyses on the database, the fire test results in which the maximum deflection was exceeded, was considered not to have failed and is grouped in Figure 2.8 under “no failure.”

2.13. Conclusions

Under the BIBM Holcofire project 162 independent European fire test results on hollow core slabs and floors were collected out of 153 fire tests. These fire tests were carried over a period of 45 years between 1966 and 2010 in well-established fire testing laboratories. The Holcofire database has been set up to enable a more thorough meta-analysis over the test results that produced stronger conclusions than can be provided by any individual study. From the database it emerged that in 102 fire tests the fire resistance time was granted. In 91 fire test results failure did not occur, while in 71 fire tests a failure did occur.

The overall conclusion and implication of this extended meta-analysis on the 162 independent fire test results is that if the nowadays available resistance models and requirements (EN1168, EN1192-1-2, EN1363-1, EN1365-2) are strictly followed, the fire test results on hollow core slabs can be fully explained for 94.5% of the database:

- Fire tests that did not fail (91x);
- Fire tests failed in bending (11x);
- Fire tests failed in shear and anchorage (42x);
- Fire tests failed due to interaction of shear and bending (6x);
- Fire tests with another type of failure (3x).

The theoretical models for explosive spalling as well as horizontal cracking are not yet fully understood with the knowledge elaborated on in this Chapter, although it becomes clear that in fire tests moisture content, a thick topping, and floor restraints are important parameters. Therefore, explosive spalling and horizontal cracking was studied further in the Holcofire project and reported on in Chapter 5 to 7. Hence, with the knowledge and standards elaborated on in this Chapter, the fire tests results on hollow core slabs cannot be fully explained by that for 5.5% of the database:

- Fire tests in which explosive spalling led to failure or hole in the slabs (5x);
- Fire tests in which horizontal cracking occurred (4x).

Regarding the fire test results that did not fail during the fire at the moment when R was granted (102x), all tests have been recalculated for bending capacity with EN1992-1-2 and for shear and anchorage with EN1168 Annex G. Both standards give good and safe predictions on the ultimate bending capacity and shear and anchorage capacity for hollow core slabs and floors exposed to fire. In the recalculation of the bending capacity it emerged that in only 6 fire tests the load was higher than 100% of the capacity (M_{exp} / $M_{Rd,c,fi}$), while only 3 tests did fail by bending. Some scatter in the strand temperatures is the explanation for this. In the recalculation of the shear and anchorage capacity, it emerged that in 22 fire tests the shear and anchorage capacity was higher than the shear load (V_{exp} / $V_{Rd,c,fi}$), while no tests failed in shear. This can be explained by the “system effect” that increases the shear and anchorage capacity.

Regarding the bending capacity, it emerged that in 11 fire tests in which the rate of deflections was exceeded, actually failed in bending. The bending resistance of a hollow core floor is governed by the degradation of the strength of the prestressing reinforcement in function of the temperature. From 25 fire tests it was concluded that the ratio of average-measured temperature over calculated temperature is 99.8% while the coefficient of variation is 14.9%. An analysis over the 11 results and taking into account 88 no-failure fire test results by the “maximum likelihood method”, it is concluded that the average of ratio (M_{exp} / $M_{Rd,c,fi}$) is 109.7%, and the coefficient of variation is 23.5%. This confirms that the EN1192-1-2 is a safe prediction, and confirms that in general, there is hardly any discussion on the bending capacity, and the fire test results give no reason to do that.

Regarding shear and anchorage capacity according to EN1168 Annex G, a statistical analysis was made using the results of the 42 recalculated fire tests that failed by shear and anchorage, and the results of 92 of the 102 fire tests in which R was granted (not taking into account the 3 shear tests and the 7 fire tests on slices). It is concluded that EN1168 Annex G does not take into account the ”system effect” (safe approach). From the analysis it became evident that the “system effect” has a positive influence on the shear and anchorage capacity. Therefore, the conclusion is split in two parts: for single slab units without “system effect”, and for floors with “system effect”.

- Single slab without “system effect”. When taking into account 28 of the 42 fire tests that failed by shear and anchorage, and by the maximum likelihood method 27 of the 92 fire tests that reached the required fire resistance time, the ratio (V_{exp} / $V_{Rd,c,fi}$) of the fire tests results are 98.8% of the calculated shear capacity, and coefficient of variation is 22.3%. Hence, EN1168 Annex G basically calculates well the capacity of one single slab unit.
- Floor with a “system effect”. When taking into account 14 of the 42 fire tests that failed by shear and anchorage, and by the maximum likelihood method 65 of the 92 fire tests that reached the required fire resistance time, the ratio (V_{exp} / $V_{Rd,c,fi}$) of the fire tests results are 129.0% of the calculated shear capacity, and coefficient of variance is 24.3%. Hence, EN1168 Annex G neglects the excess capacity by virtue of the “system effects” that can be considered as additional safety.
- The above given conclusions are valid for the 28 day mean strength of the concrete that is used to calculate the design capacity for shear and anchorage under fire conditions according to EN1168 Annex G. Normally, a fire tests is conducted after a longer period of hardening.
- The scatter in time with a coefficient of variation of 75.9% for all 42 fire tests is however very high. When taking into account only the floor systems and when “outliers” are neglected, the coefficient of variation for time decreases to 37.0%.

Regarding the bending and shear and anchorage interaction, an interaction formula is not given by the standards. In this study the interaction formula (V_{exp} / $V_{Rd,c,fi}$ + M_{exp} / $M_{Rd,c,fi}$) has been considered. The analysis on the 6 fire test results showed that with this interaction formula the average of the ratio (V_{exp} / $V_{Rd,c,fi}$ + M_{exp} / $M_{Rd,c,fi}$) is 125%, and coefficient of variation is 17%. Hence, the interaction formula gives a safe prediction of the combined failure mechanism.

References Chapter two

[2.1] BIBM, Holcofire, Behaviour of prestressed hollowcore floors exposed to fire, database fire tests 1966-2010, main report including subreports A bis D, 2013

[2.2] Jansze, W, Arnold Van Acker, Bruno Della Bella, Ronald Klein-Holte, Gösta Lindström, Andreas Nitsch, Jean-Paul Py, Matthieu Scalliet. Fire resistance of hollow core floors regarding shear and anchorage capacity. 7th International Conference on Structures in Fire, edited by M. Fontana, A. Frangi, M. Knobloch, Zurich, Switzerland, June 6-8, 2012

[2.3] CEN, EN1168:2005+A3:2011, Precast concrete products – Hollow core slabs, October 2011.

[2.4] CEN, EN1992-1-2:2004 , Eurocode 2: design of concrete structures, part 1-2: general rules, structural fire design, December 2004

[2.5] CEN, EN1363-1:1999, fire resistance tests, part 1: general requirements, August 1999

[2.6] CEN, EN1363-2:1999, fire resistance tests, part 1: alternative and additional procedures, August 1999

[2.7] CEN, EN1365-2:1999, Fire resistance tests for loadbearing elements – Part 2: Floors and roofs, December 1999.

Appendix 2.A1 – 42 fire test results with shear and anchorage failure

Validation of 42 fire test results with EN1168 Annex G with concrete strength at 28 days. [Data from Holcofire Database Subreport A, and published also in SIF12 - 7th International Conference on Structures in Fire, M. Fontana, A. Frangi, M. Knobloch (Eds.), Zurich, Switzerland, June 6-8, 2012]

		Fire test result		EN1168:A3 Annex G	Fire test / EN1168-G
	TEST ID	Shear load [kN/m]	Time to failure [min]	Shear capacity [kN/m]	[%]
H7	RUG 943 element I [1971]	44.7	36	45.2	98.9%
H8	RUG 943 element II [1971]	44.7	29	45.3	98.7%
H9	RUG 943 element III [1971]	44.7	33	40.7	109.8%
H39	VTT PAL 2480 [1982]	36.0	63	34.0	105.9%
H45	VTT PAL 4248 [1984]	40.4	49	25.4	159.1%
H48	VTT PAL 4450 [1984]	20.1	130	23.0	87.4%
H58	VTT PAL 566d [1985]	46.1	77	26.4	174.6%
H73	VTT PAL 90228 [1990]	64.8	27	47.1	137.6%
H83	EMPA B2-2 [1995]	35.1	49	32.9	106.7%
H85	EMPA B2-4 PL [1995]	35.8	75	29.5	121.4%
H86	EMPA B3-1 [1995]	28.6	97	15.1	189.4%
H96	DIFT X52650d [1998]	36.8	21	38.3	96.1%
H97	DIFT X52650e [1998]	37.7	26	42.2	89.3%
H98	DIFT X52650f [1998]	57.7	21	56.2	102.7%
H102	RUG 9158 [1999]	(27.2) 69.5	(120) 145	56.9	122.1%
H104	TNO R-A200 [1999]	31.2	96	29.4	106.1%
H106	TNO R-XB200 [1999]	(32.9) 63.6	(120) 125	34.2	186.0%
H107	TNO R-VX265 [1999]	48.6	35	37.2	130.6%
H108	TNO R-K400 [1999]	89.5	60	90.9	98.5%
H111	TNO R-K400-R [1999]	86.3	30	91.0	94.8%
H112	TNO R-K400-F [1999]	112.3	24	91.3	123.0%
H114	TNO U-VX265 [1999]	50.7	33	42.8	118.5%
H115	TNO U-HVP260A-1 [2000]	49.5	40	56.3	87.9%
H116	TNO U-HVP260A-2 [2000]	49.5	42	55.5	89.2%
H117	TNO U-HVP260A-3 [2000]	49.5	39	55.9	88.6%
H118	TNO U-K400 [2000]	93.8	33	93.2	100.6%
H119	TNO R-HVP260A23 [1999]	51.0	55	54.4	93.8%
H120	TNO R-HVP260A20 [2001]	43.0	56	51.4	83.7%
H121	TNO R-HVP260A17 [2001]	35.5	114	44.5	79.8%
H122	TNO R-HVP260A14 [2001]	29.0	123	42.9	67.6%
H123	TNO R-HVP260S23 [2001]	50.9	48	50.7	100.4%
H124	TNO R-HVP260S17 [2001]	37.2	45	51.4	72.4%
H125	TNO R-HVP260S11 [2001]	(24.5) 32.2	(120) 123	35.9	89.7%
H126	TNO R-HVP260A23F [2001]	48.9	49	54.2	90.2%
H127	TNO R-HVP260A20F [2001]	42.6	50	54.1	78.7%
H128	TNO R-HVP260A17F [2001]	35.8	99	46.7	76.7%
H130	ITB LP 534.2 [2001]	54.0	47	58.0	93.1%
H131	ITB LP 534.3 [2002]	34.4	140	35.9	95.8%
H132	ITB test 1 (F.18.1)	27.5	35	26.7	103.0%
H133	ITB test 4 (F.19.1)	64.2	65	61.9	103.7%
H138	DIFT DCPA [2004] (F.22)	58.3	25	58.1	100.3%
H142	SPTRI P502076 SP3 [2005]	73.3	46	51.7	141.8%
				average =	107.0%
				coefficient of variation =	26.5%

With the maximum likelihood method, explained in Appendix 2.C, also the 102 fire tests that did not fail by shear and anchorage when exposed to fire can be taken into account. Then the average of all 42 fire test results increases to 109.1% and coefficient of variation decreases slightly to 25.8%.

Appendix 2.A2 – Holcofire Database consisting of 162 fire test results

Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width[mm]	Total area of strand [mm^2/slab]	distance [mm]	length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H1	TUB IBMB 66 4653-I	1966	ISO 834	140	0	497	204	174	31	4,75	2	4	SYS	1	131	R-DF-BN
H2	TUB IBMB 66 4653-II	1966	ISO 834	140	0	497	204	174	23	4,75	2	4	SYS	1	127	R-DF-BN
H3	FROSI 4904	1969	ISO 834	152	0	600	170	187	38	4	3	5	SYS	1	120	R-NO
H4	VTT PAL 1927	1971	ISO 834	265	0	1200	237	372	36	5,63	3,1	3	SYS	1	80	R-NO
H5	VTT PAL 2892	1971	ISO 834	265	0	1200	237	372	36	5,63	2,4	2	SYS	1	60	R-SP
H6	RUG 942	1971	ISO 834	200	0	1200	382	465	40	7,4	1,2	1	HCS	0	120	R-NO
H7	RUG 943 element I	1971	ISO 834	265	0	1200	287	558	25	6,1	1,2	1	HCS	0	36	SA
H8	RUG 943 element II	1971	ISO 834	265	0	1200	287	558	25	6,1	1,2	1	HCS	0	29	SA
H9	RUG 943 element IIII	1971	ISO 834	265	0	1200	287	558	25	6,1	1,2	1	HCS	0	33	SA
H10	RUG 1017	1971	ISO 834	200	50	1200	382	520	49	5,9	1,2	1	HCS	0	67	DF
H11	TNO-CB BV-72-13	1971	ISO 834	200	0	1200	382	520	54	7,25	1,2	2	HCS	0	71	R-NO
H12	RUG 944	1972	ISO 834	265	50	1200	286	1302	48	5,9	1,2	1	HCS	0	76	DF-BN
H13	RUG 1450 (protected)	1972	ISO 834	265	50	1200	287	1302	48	5,9	1,9	2	FLR	0	119	R-DF
H14	VTT PAL 2163/72	1972	ISO 834	265	0	1200	232	558	36	6	2,4	2	SYS	1	61	R-NO
H15	VTT PAL 6710/73	1973	ISO 834	265	0	1200	232	372	39	6	2,4	2	SYS	1	90	R-NO
H16	RUG 1734 (protected)	1973	ISO 834	265	50	1200	287	1302	48	5,9	1,9	2	FLR	0	170	R-NO
H17	RUG 1870 (protected)	1974	ISO 834	265	50	1200	287	1302	48	5,9	1,9	2	FLR	0	133	R-DF
H18	VTT PAL 7116-74	1974	ISO 834	265	0	1200	232	372	35	6	2,4	2	SYS	1	60	R-NO
H19	RUG 2196	1975	ISO 834	265	50	1200	287	744	40	5,9	1,9	2	FLR	0	92	DF-BN
H20	RUG 2830	1977	ISO 834	265	50	1200	287	520	64	5,45	1,2	1	HCS	0	109	SB
H21	VTT PAL 1376/77	1976	ISO 834	200	0	1200	227	372	35	6	2,4	2	SYS	1	60	R-NO
H22	CBR 78/85 SPG 20/9	1978	ISO 834	200	0	1200	377	468	44	1,8	1	1	HCS	0	145	R-NO
H23	CBR 78/85 SPG 27/6	1978	ISO 834	265	0	1200	233	312	40	1,8	1	1	HCS	0	122	R-NO
H24	CBR 78/85 SPG 32/16	1978	ISO 834	320	0	1200	247	832	48	1,8	1	1	HCS	0	122	R-NO
H25	CBR 78/85 SPK 27/10	1978	ISO 834	265	0	1200	233	520	69	1,8	1	1	HCS	0	132	R-NO

Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width[mm]	Total area of strand [mm^2/slab]	Axis distance [mm]	length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H26	CBR 78/85 SPG 27/6 (protected)	1978	ISO 834	265	0	1200	233	312	40	1,8	1	1	HCS	0	173	R-NO
H27	VTT PAL 9498	1979	ISO 834	150	0	1200	231	208	35	6	2,4	2	SYS	1	50	R-NO
H28	VTT PAL 0795	1980	ISO 834	290	0	1200	260	372	80	6	2,4	2	SYS	1	190	DF
H29	RUG 3681	1980	ISO 834	152	30	595	175	248	39	6	1,8	3	FLR	0	76	R-DF
H30	RUG 3682	1980	ISO 834	200	30	595	266	118	33	6	1,8	3	FLR	0	126	R-DF
H31	VTT PAL 1146b	1980	ISO 834	265	0	x	x	x	45	6	2,4	2	SYS	1	88	R-NO
H32	VTT PAL 1191	1980	ISO 834	265	0	x	x	x	45	6	2,4	2	SYS	1	112	R-NO
H33	VTT PAL 1350	1980	ISO 834	265	0	x	x	x	45	6	2,4	2	SYS	1	78	R-NO
H34	VTT PAL 1038a	1980	ISO 834	265	0	1200	232	279	55	6	2,4	2	SYS	1	105	R-DF
H35	VTT PAL 1038b	1980	ISO 834	150	0	1200	231	208	33	6	2,4	2	SYS	1	45	R-NO
H36	VTT PAL 1038c (protected)	1980	ISO 834	150	0	1200	231	208	33	6	2,4	2	SYS	1	262	R-NO
H37	VTT PAL 1275a	1980	ISO 834	265	0	1200	232	279	65	6	2,4	2	SYS	1	105,5	R-NO
H38	VTT PAL 2358	1982	ISO 834	150	0	1200	208	364	38	5,9	2,4	2	SYS	1	64	R-DF
H39	VTT PAL 2480	1982	ISO 834	275	0	1200	243	558	65	4	2,4	2	SYS	1	63	SA
H40	VTT PAL 2481	1982	ISO 834	215	0	1200	241	651	38	5,9	2,4	2	SYS	1	78	R-NO
H41	RUG 4514	1982	ISO 834	265	50	1200	287	520	61	5,45	1,2	1	HCS	0	150	R-NO
H42	TUB IBMB 82 1424 / I-IV	1982	ISO 834	140	0	497	204	174	35	4,75	2	4	SYS	1	47	R-DF-BN
H43	TUB IBMB 82 1424 / V-VI	1982	ISO 834	160	0	497	254	145	35	4,75	2	4	SYS	1	95	R-NO
H44	IBS 2311	1983	ISO 834	160	0	1200	462	520	35	4,1	2	2	SYS	1	90	R-NO
H45	VTT PAL 4248	1984	ISO 834	265	0	2400	465	744	64	5,185	2,4	1	SYS	1	49	SA
H46	VTT PAL 4337	1984	ISO 834	265	0	1200	233	312	33	5,185	2,4	2	SYS	1	62	R-NO
H47	VTT PAL 4448	1984	ISO 834	160	0	1200	276	208	37	5,185	2,4	2	SYS	1	36	DF
H48	VTT PAL 4450	1984	ISO 834	265	0	1200	233	312	61	5,185	2,4	2	SYS	1	130	SA
H49	VTT PAL 4451	1984	ISO 834	275	0	1200	227	372	91	5,185	2,4	2	SYS	1	30	DF-BN
H50	VTT PAL 4452	1984	ISO 834	265	0	1200	225	930	33	5,185	2,4	2	SYS	1	135	R-NO

Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width [mm]	Total area of strand [mm^2/slab]	Axis distance [mm]	length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H51	VTT PAL 4453	1985	ISO 834	265	0	1200	225	208	34	5,185	2,4	2	SYS	1	60	R-NO
H52	VTT PAL 4454	1985	ISO 834	265	0	2400	465	1395	64	5,185	2,4	1	SYS	1	43	OT
H53	VTT PAL 566a/a	1985	ISO 834	200	0	1200	227	651	34	3,165	1,2	1	HCS	0	60	R-NO
H54	VTT PAL 566a/b+c	1985	ISO 834	200	0	1200	227	651	34	3,165	2,4	3	FLR	0	59,5	R-NO
H55	VTT PAL 566a/d	1985	ISO 834	200	0	1200	227	651	37	4	1,2	1	HCS	0	60,4	R-NO
H56	VTT PAL 566b	1985	ISO 834	265	0	1200	223	930	37	5,165	2,4	3	SYS	1	60,4	R-NO
H57	VTT PAL 566c	1985	ISO 834	265	0	1200	248	558	36	5,165	2,4	3	SYS	1	39	SP
H58	VTT PAL 566d	1985	ISO 834	265	0	1200	233	558	57	5,165	2,4	3	SYS	1	77	SA
H59	VTT PAL 5308	1985	ISO 834	265	0	2400	465	468	31	5,185	3	3	SYS	1	61	R-NO
H60	VTT PAL 5327	1985	ISO 834	265	0	1200	382	602	63	5,175	2,4	3	SYS	1	66	SP
H61	IG 8973	1985	ISO 834	240	0	1200	415	638	45	4	2,4	2	SYS	1	154	R-NO
H62	VTT PAL 5377	1986	ISO 834	265	0	1200	248	558	34	5,165	2,4	3	SYS	1	83	R-DF-BN
H63	IG 11686	1986	ISO 834	160	0	1200	349	71	45	4,3	2,4	2	SYS	1	84	R-DF-BN
H64	IBS 2697/87 I	1986	ISO 834	265	0	1200	225	744	35	4,1	2,388	2	SYS	1	90,4	R-NO
H65	IBS 2697/87 II	1987	ISO 834	400	0	1200	278	930	35	4,1	2,388	2	SYS	1	90	R-NO
H66	IG 12751	1987	ISO 834	160	0	1200	349	141	51	4,3	2,4	2	SYS	1	153	R-NO
H67	IG 42093/0088	1990	ISO 834	240	60	1200	361	416	54	6,3	1,2	1	HCS	0	122	R-DF-BN
H68	CSI 055/90/CF-1	1990	ISO 834	240	40	1200	404	348	35	4	1,2	1	SYS	1	180	R-NO
H69	CSI 055/90/CF-2	1990	ISO 834	380	40	1200	376	416	35	4	1,2	1	SYS	1	180	R-NO
H70	CSI 055/90/CF-3	1990	ISO 834	600	40	1200	309	682	35	4	1,8	2	SYS	1	180	R-NO
H71	RUG 6285	1990	ISO 834	250	45	600	236	520	53	6,135	1,8	3	FLR	0	194	R-NO
H72	RUG 6286	1990	ISO 834	150	45	600	241	520	53	4,135	1,8	3	FLR	0	182	R-NO
H73	VTT PAL 90228	1990	ISO 834	265	0	1200	232	766	71	5,165	2,4	3	SYS	1	27	SA
H74	VTT PAL 00360/90a	1990	ISO 834	420	0	1200	243	465	56	5,165	2,4	3	SYS	1	120	R-NO
H75	VTT PAL 00360/90b	1990	ISO 834	420	0	1200	243	465	56	5,165	2,4	3	SYS	1	120,4	R-NO

Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width [mm]	Total area of strand [mm^2/slab]	Axis distance [mm]	length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H76	VTT PAL 1126/91	1991	ISO 834	265	0	1200	232	520	55	5,225	2,4	3	SYS	1	157,4	R-DF
H77	VTT PAL 1127/91	1991	ISO 834	400	0	1200	242	465	56	5,185	2,4	3	SYS	1	61	R-NO
H78	IBS 3391/93Z	1993	ISO 834	200	0	1200	247	651	35	4,1	2,4	2	SYS	1	123	R-DF-BN
H79	IBS 3350/93	1993	ISO 834	265	0	1200	225	930	35	4,1	2,4	2	SYS	1	135	R-OT
H80	CTICM 93-G-127	1993	ISO 834	160	0	1200	558	416	45	6,4	6	5	SYS	1	32	SB
H81	EMPA 95-1	1994	ISO 834	160	80	1200	526	624	30	4,7	2,4	3	SYSB	1	122	R-NO
H82	EMPA B2-1	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	122	R-NO
H83	EMPA B2-2	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	49	SA
H84	EMPA B2-3	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	74,6	OT
H85	EMPA B2-4 PL	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	75,4	SA
H86	EMPA B3-1	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	96,6	SA
H87	EMPA B3-1 PL	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	97,4	R-NO
H88	CTICM 95-E-467	1995	ISO 834	160	50	1197	530	624	50	4	2,4	2	SYS	1	50	SB
H89	CTICM 95-E-533	1995	ISO 834	160	50	1197	530	624	30	4	2,4	2	SYS	1	100	R-DF
H90	CTICM 96-U-349	1996	ISO 834	160	50	1197	530	624	30	4	1,2	1	HCS	0	71	R-DF-SB
H91	CTICM 96-U-350	1996	ISO 834	160	0	1197	530	624	30	4	1,2	1	HCS	0	42	DF-SB
H92	RUG 8871 plaat1	1998	ISO 834	200	0	1196	229	364	50	6	2,4	2	SYSB	1	82,6	R-NO
H93	RUG 8871 plaat2	1998	ISO 834	200	50	1196	229	364	50	6	2,4	2	SYSB	1	83,4	R-NO
H94	RUG 8872 plaat1	1998	ISO 834	200	0	597	237	155	49	6	2,4	4	SYSB	1	122,6	R-NO
H95	RUG 8872 plaat2	1998	ISO 834	200	0	597	237	155	49	6	2,4	4	SYSB	1	123,4	R-NO
H96	DIFT X52650d	1998	ISO 834	185	0	1197	336	416	30	6,2	2,4	2	SYS	1	21	SA
H97	DIFT X52650e	1998	ISO 834	220	0	1197	336	416	30	6,2	2,4	2	SYS	1	26	SA
H98	DIFT X52650f	1998	ISO 834	270	0	1197	336	930	32	6,2	2,4	2	SYS	1	21	SA
H99	RUG 9157 plaat1	1999	ISO 834	200	0	597	237	155	49	6	2,4	4	SYSB	1	124,6	R-NO
H100	RUG 9157 plaat2	1999	ISO 834	200	0	597	237	155	49	6	2,4	4	SYSB	1	125,4	R-NO

- H O L C O F I R E   D A T A B A S E -

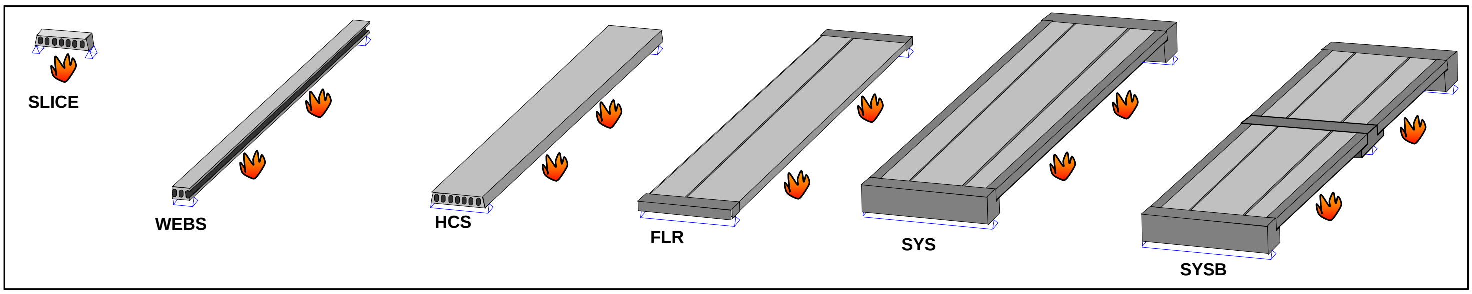
Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width [mm]	Total area of strand [mm^2/slab]	Axis distance [mm]	length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H101	RUG 9158 plaat1	1999	ISO 834	265	0	1196	322	520	49	6	2,4	2	SYSB	1	119,6	R-NO
H102	RUG 9158 plaat2	1999	ISO 834	265	30	1196	322	520	49	6	2,4	2	SYSB	1	120,4	R-NO-SA
H103	DIFT COWI PG 10724	2000	ISO 834	220	80	1197	278	651	36	6,14	2,4	2	SYS	0	25	SP
H104	TNO R-A200	1999	ISO 834	200	0	314	78	104	42	3,9	0,314	0,25	webs	0	96	SA
H105	TNO R-X200	1999	ISO 834	200	0	300	84	104	41	3,9	0,3	0,25	webs	0	125	R-NO
H106	TNO R-XB200	1999	ISO 834	200	0	316	87	104	46	3,9	0,316	0,25	webs	0	125,2	R-NO-SA
H107	TNO R-VX265	1999	ISO 834	275	0	444	73	186	39	3,9	0,444	0,37	webs	0	35	SA
H108	TNO R-K400	1999	ISO 834	403	0	561	146	580	57	3,9	0,561	0,5	webs	0	60	SA
H109	TNO R-XB200-R	1999	ISO 834	200	0	321	90	104	45	3,9	0,321	0,25	webs	0	159	R-NO
H110	TNO R-VX265-R	1999	ISO 834	270	0	440	73	186	36	3,9	0,44	0,37	webs	0	25	HC
H111	TNO R-K400-R	1999	ISO 834	399	0	582	144	580	59	3,9	0,582	0,5	webs	0	30	SA
H112	TNO R-K400-F	2000	ISO 834	402	0	570	139	580	57	3,9	0,57	0,5	webs	0	24	SA
H113	TNO U-XB200	2000	ISO 834	200	0	1197	357	416	45	3,9	1,2	1	HCS	0	117	SB
H114	TNO U-VX265	2000	ISO 834	264,6	0	1197	234	558	40	3,9	1,2	1	HCS	0	33,4	SA
H115	TNO U-HVP260A-1	2000	ISO 834	260	0	1197	375	558	53	3,9	1,2	1	HCS	0	40	SA
H116	TNO U-HVP260A-2	2000	ISO 834	260	0	1197	372	558	53	3,9	1,2	1	HCS	0	42	SA
H117	TNO U-HVP260A-3	2000	ISO 834	260	0	1197	369	558	53	3,9	1,2	1	HCS	0	38,6	SA
H118	TNO U-K400	2000	ISO 834	400	0	1197	321	1160	47	3,9	1,2	1	HCS	0	33	SA
H119	TNO R-HVP260A23	1999	ISO 834	260	0	445	31	65	1	3,9	0,445	0,37	webs	0	55	SA
H120	TNO R-HVP260A20F	2001	ISO 834	258	0	440	130	186	60	3,9	0,44	0,37	webs	0	56	SA
H121	TNO R-HVP260A17F	2001	ISO 834	257	0	448	134	186	60	3,9	0,448	0,37	webs	0	114	SA
H122	TNO R-HVP260A14	2001	ISO 834	258	0	445	130	186	59	3,9	0,445	0,37	webs	0	123	SA
H123	TNO R-HVP260S23	2001	ISO 834	260	0	440	131	0	1	3,9	0,44	0,37	webs	0	48	SA
H124	TNO R-HVP260S17	2001	ISO 834	260	0	440	131	0	1	3,9	0,44	0,37	webs	0	45	SA
H125	TNO R-HVP260S11	2001	ISO 834	260	0	440	131	0	1	3,9	0,44	0,37	webs	0	123,4	R-NO-SA

Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width [mm]	Total area of strand [mm^2/slab]	Axis distance [mm]	length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H126	TNO R-HVP260A23F	2001	ISO 834	260	0	440	134	0	1	3,9	0,44	0,37	webs	0	49	SA
H127	TNO R-HVP260A20F	2001	ISO 834	260	0	440	134	0	1	3,9	0,44	0,37	webs	0	50	SA
H128	TNO R-HVP260A17F	2001	ISO 834	260	0	440	134	0	1	3,9	0,44	0,37	webs	0	99	SA
H129	ITB NP-534.1	2002	ISO 834	199,6	50	1200	227	651	36	5,07	2,6	3	SYS	1	60	R-NO
H130	ITB NP-534.2	2002	ISO 834	265	50	1200	233	930	56	5,07	2,4	3	SYS	1	47,5	SA
H131	ITB NP-534.3	2002	ISO 834	265	50	1200	233	558	56	5,07	2,4	3	SYS	1	139,5	SA
H132	ITB 1 (F18.1)	2003	ISO 834	200	0	258	1200	0	1	5,2	1,2	1	HCS	0	35	SA
H133	ITB 4 (F19.1)	2003	ISO 834	270	50	335	1200	0	1	5,2	2,4	3	SYS	1	65	SA
H134	BRE DTLR slab-A	2003	parametric	200	50	X	X	0	31	6	6	5	SYS	1	60,1	R-NO
H135	BRE DTLR slab-B	2003	parametric	200	0	X	X	0	31	6	6	5	SYS	1	59,9	R-NO
H136	UP HPLWC	2004	ISO 834	200	0	X	X	0	1	4	2,4	2	HCS	0	76	SP
H137	IBS 07012911	2004	ISO 834	160	0	1200	259	468	48	5	3,6	3	SYS	1	94	R-NO
H138	DIFT PG 11304	2004	ISO 834	265	0	1200	238	930	40	6,065	2,4	2	SYS	1	24,6	SA
H139	ZAG 160/04-530-1	2004	ISO 834	320	0	1200	288	1209	35	5,12	2,4	2	SYS	1	105	R-DF-BN
H140	SPTRI P501342 SP-1	2005	ISO 834	265	0	1200	238	930	40	2,935	2,4	3	SYS	0	60	R-NO
H141	SPTRI P502015 SP-2	2005	ISO 834	264	0	1200	238	930	40	2,935	2,4	3	SYS	0	60	R-NO
H142	SPTRI P502076 SP-3	2005	ISO 834	265	0	1200	238	930	40	2,935	2,4	3	SYS	0	46	SA
H143	BRE test1	2007	parametric	200	0	1200	330	651	31	7	17,76	15	SYS	1	60	R-NO
H144	BRE test2	2007	parametric	200	0	1200	330	651	31	7	17,76	15	SYS	1	60	R-NO
H145	SPTRI Peikko P802216A	2009	ISO 834	270	0	1200	286	930	35	5,8	3,6	4	SYSB	1	60	R-NO
H146	SPTRI Peikko P802216B	2009	ISO 834	270	0	1200	286	930	35	5,8	3,6	4	SYSB	1	60,4	R-NO
H147	SPTRI Peikko P802216C	2009	ISO 834	270	0	1200	286	930	50	5,8	3,6	4	SYSB	1	120	R-NO
H148	SPTRI Peikko P802216D	2009	ISO 834	270	0	1200	286	930	50	5,8	3,6	4	SYSB	1	180	R-NO
H149	RIFS 04-52 1178	2009	GOST 30247.0	180	0	1200	478	520	45	5,88	3,6	3	HCS	0	91	R-NO
H150	RIFS 04-52 705	2010	GOST 30247.0	250	0	1200	476	930	45	6	3,6	3	HCS	0	92	R-NO

Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width [mm]	Total area of strand [mm^2/slab]	Axis distance [mm]	length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H151	RIFS 04-52 704	2010	GOST 30247.0	300	0	1200	342	952	45	6	3,6	3	HCS	0	93	R-NO
H152	RIFS 04-52 703	2010	GOST 30247.0	400	0	1200	342	952	45	6	3,6	3	HCS	0	94	R-NO
H153	EFNL S-A260-T300	2010	ISO 834	260	300	1200	320	n.a.	1	1,2	1,2	1	SLICE	0	10	HC
H154	EFNL S-A260-T300R	2010	ISO 834	260	300	1200	320	n.a.	1	1,2	1,2	1	SLICE	0	27	HC
H155	EFNL S-A260-T0	2010	ISO 834	260	0	1200	320	n.a.	1	1,5	1,2	1	SLICE	0	121	R-NO
H156	EFNL S-A260-T50	2010	ISO 834	260	50	1200	320	n.a.	1	0,15	1,2	1	SLICE	0	32	R-NO
H157	EFNL S-A260-T75	2010	ISO 834	260	75	1200	320	n.a.	1	0,15	1,2	1	SLICE	0	34	R-NO
H158	EFNL S-A260-T100	2010	ISO 834	260	100	1200	320	n.a.	1	0,15	1,2	1	SLICE	0	37	R-NO
H159	EFNL S-A260-T100R	2010	ISO 834	260	100	1200	320	n.a.	1	0,15	1,2	1	SLICE	0	16	HC
H160	EFNL S-A400-T50	2010	ISO 834	400	50	1200	400	n.a.	1	0,15	1,2	1	SLICE	0	38	R-NO
H161	EFNL S-A400-T75	2010	ISO 834	400	75	1200	400	n.a.	1	0,15	1,2	1	SLICE	0	35	R-NO
H162	EFNL S-A400-T100	2010	ISO 834	400	100	1200	400	n.a.	1	0,15	1,2	1	SLICE	0	33	R-NO

R = fire resistance time granted, NO = no failure, DF = deflection criteria exceeded, BN = bending failure, SA = shear and anchorage failure, SB = shear bending interaction, SP = spalling, HC = horizontal cracking, OT = other failure type

In addition to these 153 fire tests with 162 analyzable fire test results collected in the Holcofire Database, it is known by the author from references in publications that about another 22 fire tests have been carried out in Europe in the same time period. 14 of these fire tests were conducted between 1968 and 1977, and 8 tests have been conducted between 1992 and 2009. It was not possible in this study to retrieve the test reports, and therefore those studies have been neglected. These tests have also not been taken into account in this database. It is believed that the available 153 fire tests with 162 analysable results will give a broad enough perspective for the meta-analysis to make solid conclusions. Further, after 2010 fire tests have been carried out, amongst others by Holcofire, and these are not included in the database analysis.



Appendix 2.B – Listing of fire testing laboratories - abbreviations

BRE	Building Research Establishment	Middlesborough	United Kingdom
CBR	CBR Ergon laboratory	Lier	Belgium
CSI	CSI Gruppo IMQ	Milan	Italy
CTICM	Centre Technique Industriel de la Construction Metallique	Mezieres-les-Metz	France
CVUT	Technical University in Prague	Mokrsko	Czech republic
DIFT	Danish Institute for Fire Technology	Hvidovre	Denmark
EFNL	Efectis Nederland	Delft	Netherlands
FROSI	Fire Research Organisation Special Investigation	Unknown	United Kingdom
IBS	Institut für Brandschutztechnik und Sicherheitsforschung	Linz	Austria
IG	Instituto Giordano - Laboratorio di Recherche di fiscica tecnica	Bellaria	Italy
ITB	Building Research Laboratory	Katowice	Poland
EMPA	Eidg. Materialprüfungs- und Versuchsanstalt für Industrie, Bauwesen und Gewerbe (ETH Zurich)	Dubendorf	Switserland
SPTRI	SP Technical Research Institute of Sweden	Borås	Sweden
RIFS	Ministry for Emergency Situations	Minsk	Belarus
RUG	Rijksuniversiteit Gent	Gent	Belgium
TNO	Toegepast natuurwetenschappelijk onderzoek	Delft	Netherlands
TUB	Technische Universitat Braunschweig	Braunschweig	Germany
UP	University of Perugia		Italy
VTT PAL	Valtion Teknillinen Tutkimuskeskus - Palotekniikan laboratorio	Helsinki	Finland
ZAG	ZAG fire laboratory	Ljubljana	Slovenia

*) TNO and EFNL is the same laboratory, but tests were conducted under different ownership

Appendix 2.C - Method of maximum likelihood

In the evaluation talks with the reviewers it became clear that the results of the 102 fire that did not fail during a fire test can be taken into account to determine a better mean and scatter of the ratio (capacity test / capacity calculated). The principle is that a test leading not to failure contains information on the failure behaviour; namely, it indicates that for example bending is not governing under a certain load. The method of "Maximum Likelihood" can be used to take these 102 fire tests into account. The values of the average and the scatter that maximize the function L are the maximum likelihood estimators. A hypothetical example is given in this Appendix.

We start with 9 hypothetical results of ratio (capacity test / capacity calculated). The 9 results are 0.80 – 0.85 - 0.90 – 0.95 – 1.00 – 1.05 – 1.10 – 1.15 – 1.20. The average and standard deviation can be calculated with normal statistics, Then it is found in case of MS Excel that the average equals 1.00 and standard deviation equals 0.137.

Another approach is as follows. Consider the likelihood function L:

L = f(x1) f(x2) f(x3) …. f(x8) f(x9)

In which f(x) is the probability density function equal to

$$f(x_i) = \frac{1}{\sigma\sqrt{(2\pi)}} exp\left\{-\frac{(x_i-\mu)^2}{2\sigma^2}\right\}$$

It emerges that the likelihood L in this simple case is μ = 1.00 and μ= 0.137 by using the excel solver.

Example A:

Now we add 17 hypothetical results that did not fail in a test with a ratio of (capacity test / capacity calculated) equal to 0.10 (3x) – 0.20 (3x) – 0.30 (3x) – 0.50 (3x) - 1.10 – 1.20 – 1.30 -1.40 – 1.50. Of five results of ratio (capacity test / capacity calculated) are above 100% of the capacity. This implies that the test was much better, and that the calculation method is conservative. Now the maximum likelihood function should be expanded with F(x). Hence:

L = f(x1) f(x2) f(x3) …. f(x8) f(x9) · F(x1) F(x2) F(x3) …. F(x16) F(x17)

In which f(x) remains the probability density function, and in which F(x) is the probability distribution function.

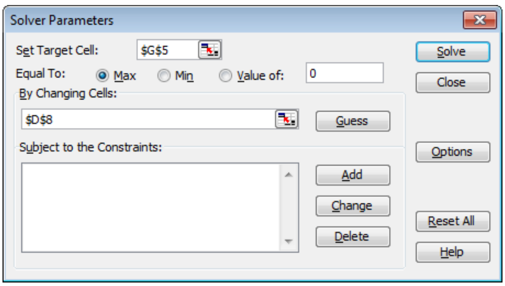
$$F(x_i) = \Phi\left\{\frac{x_i-\mu}{\sigma}\right\}$$

In this function F(x) the variable Φ(.) is the distribution function of the standard normal distribution variable (with average 0 and standard deviation 1). Calculation in Excel with function NORMSDIST and solver gives another average, namely 99.3%. In this case, the standard deviation remains 13.7%. See next page for print screen of Excel calculation.

Example B:

Now we add other 17 hypothetical results that did not fail with a ratio (capacity test / capacity calculated) equal to 0.10 (3x) – 0.20 (3x) – 0.30 (3x) – 0.50 (3x) – 0.90 (5x). The loads are now under 100% of the (capacity test / capacity calculated), which means that the calculation function is a good one. Calculation in Excel with solver with function NORMSDIST gives now a better average, namely 102.0%. In this case, the standard deviation decreases slightly to 13.6%.

	A	B	C	D	E	F	G	H	I	J	K
1											
2											
3						f(x1) f(x2) ..f(xn)=	273,405691	SUM	0,15044		
4						F(x1) F(x2) ..F(xn)=	0,72029431	SUM/8	0,0188		
5						f(xn) F(xn)=	196,932563	STDEV	0,13713		
6											
7		273,406									
8				0,993	0,13713				THEN		
9				AVERAG	STDEV	f(x)		IF AVG	(x-AVG)^2		
10		0,80		0,9930	0,1371	1,0803		0,9930	0,0373		
11		0,85		0,9930	0,1371	1,6887		0,9930	0,0205		
12		0,90		0,9930	0,1371	2,3113		0,9930	0,0087		
13		0,95		0,9930	0,1371	2,7695		0,9930	0,0019		
14		1,00		0,9930	0,1371	2,9055		0,9930	0,0000		
15		1,05		0,9930	0,1371	2,6686		0,9930	0,0032		
16		1,10		0,9930	0,1371	2,1460		0,9930	0,0114		
17		1,15		0,9930	0,1371	1,5109		0,9930	0,0246		
18		1,20		0,9930	0,1371	0,9313		0,9930	0,0428		
19							F(x)	IF AVG		ERF(.)	P(x<x)
20		0,10		0,9930	0,1371		1,0000	0,9930		-6,5122	0,0000
21		0,10		0,9930	0,1371		1,0000	0,9930		-6,5122	0,0000
22		0,10		0,9930	0,1371		1,0000	0,9930		-6,5122	0,0000
23		0,20		0,9930	0,1371		1,0000	0,9930		-5,7830	0,0000
24		0,20		0,9930	0,1371		1,0000	0,9930		-5,7830	0,0000
25		0,20		0,9930	0,1371		1,0000	0,9930		-5,7830	0,0000
26		0,30		0,9930	0,1371		1,0000	0,9930		-5,0538	0,0000
27		0,30		0,9930	0,1371		1,0000	0,9930		-5,0538	0,0000
28		0,30		0,9930	0,1371		1,0000	0,9930		-5,0538	0,0000
29		0,50		0,9930	0,1371		0,9998	0,9930		-3,5953	0,0002
30		0,50		0,9930	0,1371		0,9998	0,9930		-3,5953	0,0002
31		0,50		0,9930	0,1371		0,9998	0,9930		-3,5953	0,0002
32		1,10		0,9930	0,1371		0,7823	0,9930		0,7801	0,7823
33		1,20		0,9930	0,1371		0,9344	0,9930		1,5093	0,9344
34		1,30		0,9930	0,1371		0,9874	0,9930		2,2386	0,9874
35		1,40		0,9930	0,1371		0,9985	0,9930		2,9678	0,9985
36		1,50		0,9930	0,1371		0,9999	0,9930		3,6970	0,9999



Appendix 2.D – Review report

Review of

Holcofire Report on Behaviour of prestressed hollowcore floors exposed to fire
Evaluation of 162 fire test results

Review by Prof.ir. A.C.W.M. Vrouwenvelder and Prof.dr.ir. J.C. Walraven

FINAL

Date:

6 June 2013

Client:

BIBM HOLCOFIRE
p/a Dr.Ir. W. Jansze MBA
Bd. Du Souverain 68
B-1170 Brussels
Belgium

Authors:

Prof.ir A.C.W.M. Vrouwenvelder
Prof. dr. ir. J. C. Walraven

Consulted experts:

Prof dr.ir. Hordijk (TU-Delft, Bureau Hageman)
ir. C. Both (Promat)

1. Introduction

On request of BIBM Holcofire the authors give a review of the Holcofire Report **"Behaviour of prestressed hollow-core floors exposed to fire, the Evaluation of 162 fire test results.",** dated May 10, 2013**.**

For the record it is mentioned that a first draft of this report (main report and subreports A, B, C and D) was received in January 2013 and –subjected to a first review on 13 February 2013. The comments raised at that time have resulted in additional calculations and some adjustments, leading to the final version mentioned above.

In the BIBM Holcofire report a set of 162 tests has been studied. The total set can be subdivided into four categories:

- 42 tests ending with clearly identified shear / anchorage failures
- 24 tests stopped because of exceeding the predefined limit-deflection (generally δ = L/30).
- 16 tests with failure due to horizontal cracking, spalling, shear/bending interaction and other failure modes not corresponding to one of the previous main categories
- 80 tests stopped before failure, for various reasons.

These categories have been described and discussed in the sub-reports A, B, C and D respectively. For the mechanisms bending failure, shear/anchorage failure and shear-bending interaction a comparison has been made between the tests on the one hand and available calculation models (EN 1992-1-2 4.2 and Annex B, EN1168 4.3 and Annex G) on the other.

The conclusion in the BIBM report is that the majority of the models describing those failures give satisfactory results. In spite of the large selection of tests with a wide scope of influential parameters, some questions remain open. This refers especially to the mechanisms of horizontal crack formation and explosive spalling. In the report it is recommended to focus on the effect of restrained deformation on horizontal cracking and of explosive spalling in upcoming research.

2. Comments

The reviewers would like to emphasize that they consider the work done to be a very good initiative and a valuable contribution to the assessment of structural safety of floors assembled with hollow-core slabs subjected to fire. The large number of tests from various origins, with a large spectrum of parameter variations, have been classified with regard to their failure mode and have been analyzed appropriately. The following comments are made with regard to particular aspects of precast concrete floors, assembled of prestressed hollow-core slabs:

2.1 Safety philosophy

According to Eurocode EN 1990, Basis of Structural Design, Section 5.2, design by testing should lead to the required level of reliability, properly taking care of the effects of model uncertainties as well as statistical uncertainties. The informative Annex D offers a further elaboration on this aspect. The given procedure is in principle also applicable for fire testing. It is regarded reasonable to reduce the generally required reliability index β=3,8 for the case of fire, in order to compensate for the accidental nature of fire.

In international fire engineering practice, however, still a more traditional way of dealing with structural safety is followed. It is a widely accepted procedure to put only one single specimen of a product to a fire test and approve it if the required time of fire duration is met without failure. No safe design value in the tail of the statistical distribution is determined. Safety with respect to fire is achieved by specifying some safe value at the loading side (duration of the fire) in combination with the recognition that fire in itself has a low probability of occurrence. Also in this report this line of thinking has been adopted. The consequence is that the models of Annex G are considered as being confirmed if the mean value of the ratio between experimentally obtained and predicted results is at least equal to one. Moreover the variation of this ratio should be within certain acceptable limits.

2.2 Shear and anchorage capacity

For each test the shear/anchorage resistance V_{calc} of the hollow-core slab subjected to fire has been calculated according to the specifications given in EN1168 Annex G and compared with the shear/anchorage resistance V_{exp} observed in the test. The main conclusions in the report are based on the evaluation of the 42 tests, described in Subreport A, that resulted in clear shear/anchorage failures. However, in a considerable number of other tests the expected shear/anchorage failure was not reached, for instance because of the fact that the test was stopped since the required time had been reached. For the case of shear/anchorage capacity 92 tests of this type were selected. Also this information has been taken into consideration in the statistical evaluation using the method of Maximum Likelihood. Using this additional method resulted in a slightly better result than obtained considering the set of members failing in a clear shear/anchorage failure mode alone.

It turns out that the mean ratio V_{exp}/V_{calc} is about 1.0 for single slabs and 1.29 for slabs being a part of a floor system. These values hold for a concrete strength based on 28 days. If the actual strength of the concrete at the age of testing would have been taken, the values V_{exp}/V_{calc} would drop by about 7 %. Which is due to the circumstance that the age of the concrete of the test specimens subjected to the fire tests was mostly several months. A reasonable argument to test at a higher age is, however, that in specimens with an age lower than about 3 months the moisture content is still that high, that explosive spalling could occur, which would not be representative for the utmost part of the service life. It is therefore regarded to be acceptable to use the real age of the concrete specimens in the comparisons.

The coefficients of variation for V_{exp}/V_{calc} are 22 % and 24 % respectively, for single slabs and slabs being a part of a larger floor system (with restraint action at the boundaries) . This seems high though acceptable. The shear/anchorage capacity, as clearly pointed out in Subreport A, decreases only slowly as a function of the fire duration. In fire engineering practice, however, not the bearing resistance is the governing design criterion, but the time of fire exposure during which the structure is able to carry the load corresponding to the defined accidental loading situation. The scatter in the time of duration of fire exposure at a given loading level is expectedly higher than the scatter in bearing resistance. With regard to the duration of fire exposure, a coefficient of variation in the order of magnitude of 50 % could therefore be expected to be realistic, which is about twice the value of the coefficient of variation for the bearing resistance. According to the data given in Subreport A, indeed coefficients of variation for the ratio between experimentally found and calculated fire exposure duration of about 40% are obtained for single slabs, and 65 % for the slab systems. The mean values are well above unity. The value of 65% is remarkably high. A more detailed analysis shows, however, that the large scatter is predominantly the result of some very conservative estimates by Annex G: 30 minutes, where the experimental fire exposure durations were much higher. If those cases are removed as “outliers” good results are obtained.

The formula in Annex G, enabling the determination of the shear/anchorage resistance of a prestressed hollow-core slab, is an extension of the formula for the shear capacity of prestressed structural members given in EN1992-1-1, Cl. 6.2.2 for normal temperatures. On the one hand this formula is quite practical, since it combines the shear- and the anchorage capacities which are often hard to distinguish in experiments. On the other hand it is inevitably empirical, like the original equation (6.2.a)The real behaviour of the slabs subjected to fire, however, is very complex , with thermal stresses leading to cracks, which may act both in a favorable and in an unfavorable way. This is neglected by the formulas. Restraint effects always seem to be important, but also they are not a part of the formula. It is also interesting to observe that one needs to insert into the model some correction values (characteristic values and η = 0,7) in order to achieve a formula predicting the mean value. On the other hand it has to be estimated that a practical formula for the shear and anchorage bearing resistance is given, valid for fire conditions. This enables a verification of the structure under fire conditions which is more extended than the limited verification for the bending capacity only as used up to now.

Furthermore, when it comes to a statistical evaluation, meaningful results can only be obtained if the set of experiments is representative for the (future) population of structures. An update of Annex G could therefore be considered, specifying minimum requirements for anchor systems, support conditions (effects of restraints), coupling reinforcement and ties.

2.3 Bending

In the set of 162 tests no test were indicated as having failed by exceeding the bending capacity. However, in the report it is argued that a number of tests, which were stopped because of exceeding the specified limit rate of deflection, were at the onset of producing bending failures. The reviewers accept this argument. In addition a set of 88 no-failure tests could be added leading to a statistical acceptable result. It should be noted that the bending failure mode has never been subject of a serious dispute.

2.4 Bending and shear interaction

For this failure mode, which can be regarded to be in the transition range between flexural-shear and anchorage failure only 6 relevant tests could be found. Actually this requires the statistical uncertainty to be taken into account. An interaction formula is given, showing a mean value and standard deviation which are fair enough. It might be wondered if this is really needed, since the lowest bearing resistance obtained from the separate equations for bending and shear may be expected to give a reasonable design value as well.

3. Conclusion

The gathering and evaluation of all test data can be considered a very valuable initiative. The reviewers consider the contents of this report as a fair description and interpretation of the authentic 162 fire test results. The statistical evaluation shows that within the normal context of actual fire safety engineering bending and shear predictions by Annex G can be classified as acceptable. Here testing slabs with an age of a few months, with a moisture content below 3% of mass may be considered as reasonable since this excludes explosive spalling which is not fully representative for the utmost part of the service life. Given the weak physical background one should be careful to describe the circumstances for which the formulas may be applied in detail. In particular for non-system floors an additional partial safety factor of 1.1 could be considered as well as the addition of an appropriate deflection limit. The reviewers agree with the conclusion that the collection of fire tests regarded does not give enough information on the effect of structural toppings nor effects of restraints, and that the mechanisms spalling and horizontal cracking still require further research.

[6 January 2014 - Addition by Holcofire on last sentence "*The reviewers agree with the conclusion that the collection of fire tests regarded does not give enough information on the effect of structural toppings nor effects of restraints, and that the mechanisms spalling and horizontal cracking still require further research*." This sentence from the final review report is from 6 June 2013. The subjects of spalling and horizontal cracking are elaborately described in the Chapters 5 to 7 of this book and fully explained by the authors.]

Chapter Three

3

Shear and Anchorage

Fire tests to validate the shear and anchorage capacity according to EN1168:2005 + A3:2011 Annex G

Keywords: *fire tests, hollow core slab, floor structures, shear, parameters, product standard, validation*

Abstract. *In 2011, a new formula for the shear and anchorage capacity of hollow core floors under fire conditions was introduced in Annex G of the product standard EN1168:2005+A3:2011 [3.1]. In order to evaluate the formula, a database was created within the project HOLCOFIRE with all available data from fire tests and 42 relevant tests of this database were analysed [3.16]. Additionally, a thorough test programme was set up with fire tests on hollow core floors. This Chapter addresses the fire tests executed with 7 different configurations. The first fire test G1 on an unloaded element showed that the slab conditioned according to the selected procedure was insensitive to spalling. In the three subsequent fire tests – the so-called G2/G3, G4/G5, G6/G7 – the influence of specific parameters on the behaviour of the hollow core floor was tested. Using a 265 mm deep hollow core slab, the investigated parameters were: the type of connection reinforcement; presence of a structural topping; presence of protruding strands; presence of a longitudinal tie beam; and presence of external longitudinal bars to simulate blocking. It emerged from the fire tests that during 120 minutes of ISO fire the floor was capable to resist a shear load equal to the shear and anchorage capacity calculated with EN1168:2005 +A3:2011 Annex G. Also, it emerges that*

the experimentally obtained peak shear capacity was 1.6 to 2.7 times higher than the calculated shear and anchorage capacity. Hence, it is concluded that with the usage of the shear and anchorage capacity formula, given in the European product standard EN1168:2005+A3:2011 Annex G, the designed hollow core floor is safe for the ultimate limit state for fire design.

Review. *The background reports of G series were been reviewed by Prof. dr.-Ing. D. Hosser and Dr.-Ing E. Richter of IBMB, Technische Universität Braunschweig. The final review (25 pages) is presented in the report "Gutachtliche Stellungname" dated 10 January 2013.*

3.1. Introduction

Concrete structures possess a high fire resistance and a large resilience to fire because of their robustness and their capacity to redistribute the acting loading. This is also valid for precast concrete hollow core floors considering the past overall performance of the total estimated stock of installed hollow core floors nowadays in Europe of about 1000 million square meters. Not many cases are known to the authors where hollow core floors structurally failed within the required fire resistance time. Unfortunately, researchers did not always design a correct set-up for the fire test, especially when testing hollow core on a small scale. A few cases of premature shear failure in standard fire tests were reported [3.9, 3.11, 3.12]. As a consequence, it led to reluctant clients and authorities, although shear hardly governs in daily floor design [3.10, 3.14]. The question was raised whether this premature shear failure constitutes a real structural problem for this type of floor, or whether the reason lies in a lack of understanding of the behaviour of hollow core floors under fire conditions, resulting in poor fire test set-up designs, in particular in the mentioned small-scale laboratory tests. The discussions around these premature failures affected the good image of the hollow core floor in some European countries, although hardly any problems in the application of the product are known, even after a thorough market research.

In order to systematically study shear failure under fire conditions, laboratory tests were conducted between 1998 and 2005 in Belgium [3.8, 3.15], The Netherlands [3.10] and Denmark [3.13]. These fire tests have been reported on in literature, however, publications lacked in a good guideline to design for shear and anchorage. Only recently, in 2011, the European Standardisation Institute CEN published rules in EN1168:2005+A3:2011 Annex G [3.1], the European product standard for hollow core slabs. This amendment to the product standard contains an informative annex G that provides a design method to design for shear and anchorage for single span hollow core floor without shear reinforcement exposed to fire.

3.2. Objective of Holcofire fire test series G

Near the support, we distinguish under ambient conditions flexural-shear failure, shear-tension failure, and anchorage failure. Under fire conditions the type of failure is different than those observed at ambient conditions. Due to induced thermal strains with increasing temperature, vertical cracks are present in the webs of the hollow core slab. At the same time, the underflange (and top flange or topping) is subjected to high compression stresses. See Figure 3.1. Hence, under fire conditions the ambient failure mechanisms flexural shear and shear tension cannot occur.

The Annex G in the newly published EN1168:2005+A3:2011 provides a formula that has been validated with 9 fire tests that failed in shear as described in the background document. But these tests used for this shear resistance cover only the period less than 65 minutes. Further, a lot of test reports in the database missed information and parameters such as the concrete quality which was not always evident. Sometimes even the failure mode was not given or clearly described. And as the fire test set-ups were not standardised it is difficult to compare results of the various test laboratories from past 45 years (although this has been done by the authors in [3.16] and good conclusions could be drawn). Therefore, in the European HOLCOFIRE project new fire tests were designed in order to confirm the EN1168:2005+A3:2011 Annex G formula after its publication. Further in this document, these series of tests will be named "test series G". The objective of test series G was to check the validity of shear formula of EN1168:2005+A3:2011 Annex G with new fire tests. An additional objective was to validate the fire test arrangement as described in Annex G.

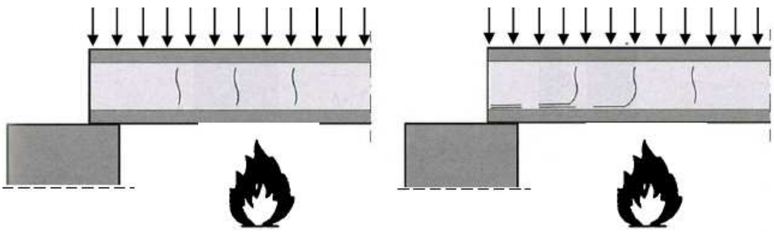


Figure 3.1. Thermal cracking due to thermal stresses (left) and cracking due to shear stresses (right)

3.3. Fire resistance according to EN1168:2005+A3:2011 Annex G

Product standard EN1168:2005+A3:2011 [3.1] Annex G provides a design method to calculate the shear and anchorage fire resistance of hollow core floors for fire conditions. According to this annex, the resistance regarding shear and anchorage failure may be determined by using simplified calculation methods (see [3.4] clause 4.2 and Annex B and Annex D), but taking into account the following assumptions:

- Firstly, it is assumed that below the level on which the total web width is equal to the total core width (level $a_{50\%}$), the temperature in the hollow core at a distance x from the exposed soffit is equal to the temperature at the same position in a corresponding solid slab (see Figure 3.2).
- Secondly, above that level a linear interpolation is taken between the temperature at that level and the temperature at the top of the floor. The maximum allowed

temperature for the insulation criterion is 160°C (140°C + 20°C ambient temperature) if no additional information is available;

- Thirdly, for a fire resistance class ≤ R60 this verification is not needed.

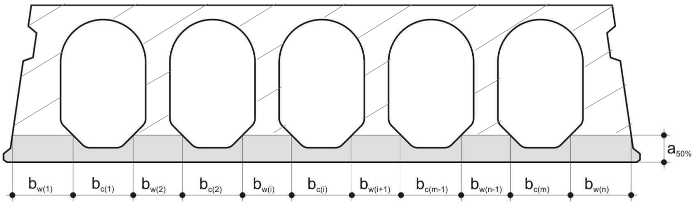


Figure 3.2. Area where solid slab temperatures may be assumed (grey area)

To determine the shear and anchorage resistance under fire conditions, the formula (1) from Annex G is used, see also Figure 3.3.

$$V_{Rd,c,fi} = \left[C_{\theta.1} + \alpha_k \cdot C_{\theta.2}\right] \cdot b_w \cdot d \tag{1}$$

$C_{\theta.1}$ coefficient accounting for concrete stress under fire conditions:

$$= 0.15 \cdot \min(k_p(\theta_p)\sigma_{cp,20°C}; \frac{F_{R,a,fi,p}}{A_c}) \tag{2}$$

$$\alpha_k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 \quad \text{where d is in mm ,} \tag{3}$$

$C_{\theta.2}$ coefficient accounting for anchored longitudinal reinforcement:

$$= \sqrt[3]{0.58 \cdot \frac{F_{R,a,fi}}{f_{yk} \cdot b_w \cdot d} \cdot f_{c.fi.m}} \tag{4}$$

b_w total web thickness of the hollow core slab
d effective depth at ambient temperature
x the anchorage length of the strand for the considered section

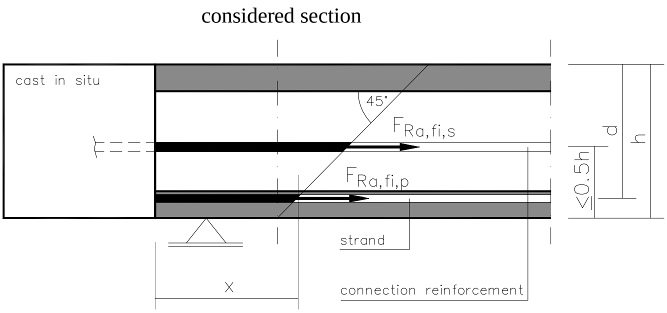


Figure 3.3. Model for calculating shear and anchorage resistance (example without protruding strands)

3.4. Experimental design of Holcofire fire test series G

In order to study the shear and anchorage capacity in the European HOLCOFIRE project, the standardised configuration of the test set-up described in EN1168:2005+A3:2011 was used as a basis for the test series G. In order to reach the objective, 7 fire tests were designed in test series G; G1, G2/G3, G4/G5, and G6/G7. The slab thickness was 265 mm for all slabs, with 6 ø12.5 mm strands at 50 mm axis distance (see Appendix 3.A). Since the main purpose was to check the shear capacity and not the flexural capacity, the floor length was limited to max 3.90 m in order to fit within the width of the furnace. The targeted fire exposure time was 120 minutes using the ISO 834 curve for all tests in the G series. Figure 3.4 sketches the floor assembly, while in Appendix 3.B technical drawings of the support details are given. Note that the support beam and longitudinal beams were not insulated during the fire test.

The following fire tests have been designed for G-series (Table 3.I overviews the fire tests, the chosen parameters and their values in order to study the shear capacity):

- Test G1: This fire test is a spalling test that was executed 4 months after casting of the slabs. The aim of the test was to study whether the moisture content present in the slab leads to spalling. The test was conducted in a small furnace, where the specimen could not be subjected to an external load. Different from G2-G7, a part of the element was tested and the size of the sample was 1.75 m x 0.75 m.

- Tests G2/G3: The aim of the fire test G2/G3 was to study tie reinforcement and boundary conditions under fire conditions. G2 contained connection tie reinforcement in open cores filled with concrete, while in G3 the connection reinforcement was placed in the joint. For boundary conditions, only in G3 two longitudinal bars ∅25 mm were used at both edges of the test floor to simulate the partial blocking of a real floor by the surrounding structure, together with longitudinal tie beams.

- Tests G4/G5: The aim of the fire test G4/G5 was to study the influence on the shear capacity under fire conditions of a 50 mm reinforced structural topping. In G4 170 mm protruding strands were used and direct slab support of 30 mm, while in G5 slab support was equal as the other G test (see Annex B). The reinforcement in the structural topping of G4 consisted of a mesh ∅4.5@200/200, while in G5 a mesh ∅7@150/150 was used. Only in floor G4 longitudinal tie beams were applied.

- Tests G6/G7: The aim of the test G6/G7 was to study masonry wall boundary conditions under fire conditions in which the connection tie reinforcement is not fully anchored in the peripheral tie beam. In the test, masonry was not used, but a deep concrete beam with the same structural principle. G6 was without vertical connection to the support beam over the width of the slabs, while in G7 a deep support L-shaped beam was used to generate friction with the connection tie reinforcement. Note that only the reinforcement area lower than mid height is taken into account, according to Annex G. For boundary conditions, only in G6 two longitudinal bars ∅25 mm were used at both edges of the test floor to simulate the partial blocking of a real floor by the surrounding structure, while in both floors longitudinal tie beams were applied.

Table 3.I presents at the last row the shear and anchorage capacity at 120 minutes calculated with EN1168:2005+A3:2011 Annex G as presented in formula (1). The calculated capacities are the result of the parameters presented in Table 3.I. As a reference, if the shear and anchorage capacity of one slab with 6 ∅12.5 mm strands without connections is calculated at 120 minutes of ISO-fire exposure, the calculated shear and anchorage capacity is 35,5 kN/m (calculated with nominal values of the cross section, and not with actual properties as the floors consisted of more than one slab). For the shear and anchorage capacities G2, G3, G5, G6 and G7 a strand temperature 390ºC is calculated after 120 minutes of fire using [3.4]. For the capacity of fire test G4 a more advanced 2D-calculation was used to determine the temperature in the strand in the anchorage zone. After 120 minutes, the calculated temperature is 193°C at the end of the hollow core and 39°C at the end of the protruding stands due to the influence of the support.

Table 3.I Fire tests and parameters (nominal values) in HOLCOFIRE test series G

fire test # **parameter**	**G1**	**G2**	**G3**	**G4**	**G5**	**G6**	**G7**
length of tested floor [m]	1.75	3.9	3.9	3.9	3.9	3.9	3.9
width of tested floor [m]	0.75	2.4	2.6	2.6	2.4	2.6	2.6
Height of slab [mm]	265	265	265	265	265	265	265
Structural topping in mm	0	0	0	50	50	0	0
Reinforcement topping	-	-	-	Ø4.5 200/200	Ø7 150 /150	-	-
Protruding strands in mm	0	0	0	170	0	0	0
Connection reinfo per slab	0	2Ø10	1Ø12	0	0	1Ø12	1Ø12
Shape connection reinfo	0	bar	bar	mesh	mesh	hairpin	hairpin
Connection area [mm^2/m]	0	131	94	0	0	94	94
Location connection reinfo	0	2 cores	joint	topping	topping	joint	joint
Vertical stirrup at support	0	2Ø8-150	2Ø8-150	2Ø8-150	2Ø8-150	0	Ø8-150
Longitudinal tie beam[1)] [mm^2]	no	no	100 x 265	100 x 315	no	100 x 265	100 x 265
External bars Ø25	no	no	2 Ø25	no	no	2 Ø25	no
Type of load on floor	none	shear	shear	shear	shear	shear	shear
Annex G $V_{Rd,c,fi,120}$ [kN/m]	35.5	52.3	48.7	63.7	40.7	48.7	48.7

1) reinforcement in longitudinal tie beam consisted of 2 bars ø12 mm and stirrups ø6-200 mm

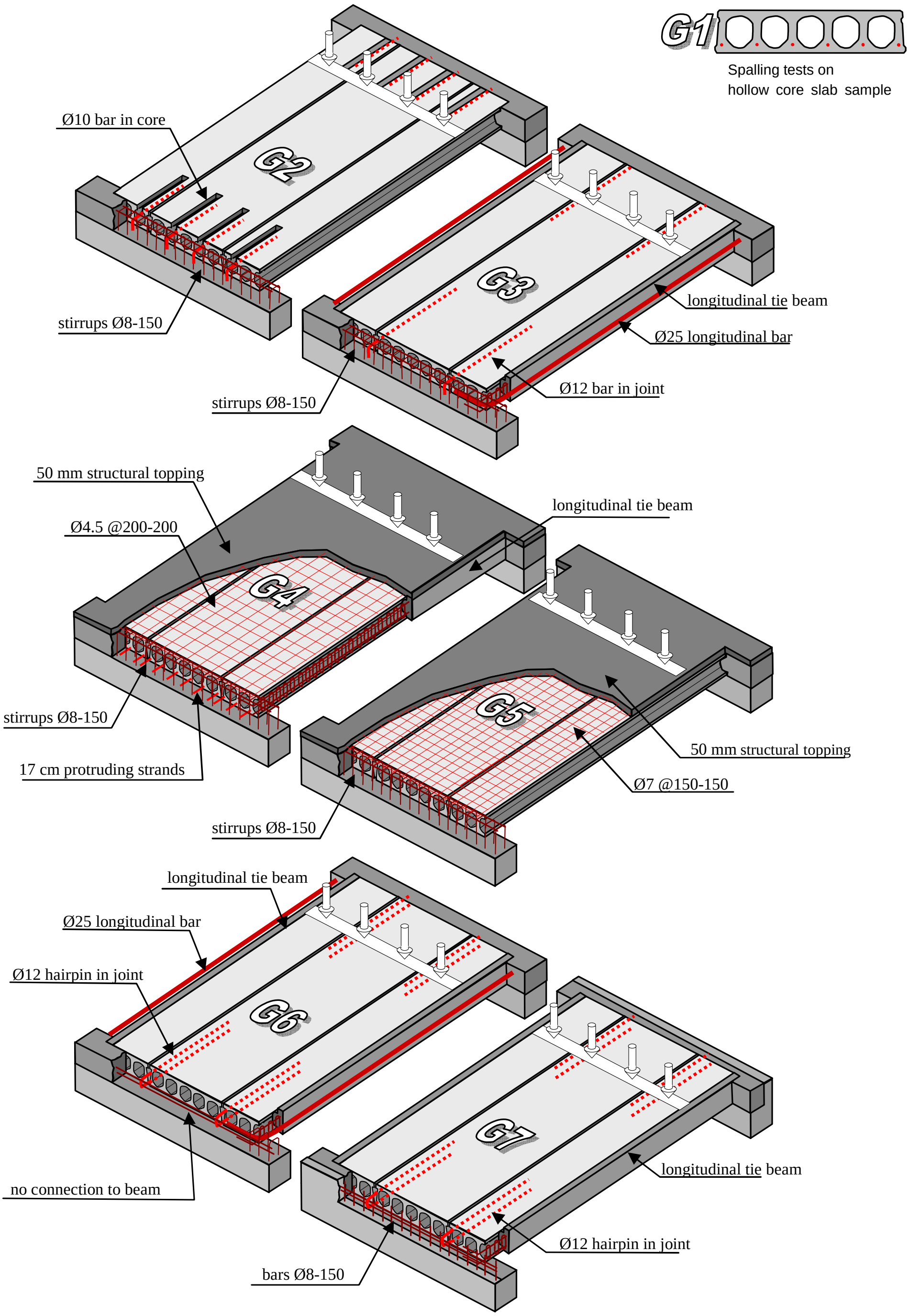


Figure 3.4. HOLCOFIRE series G – overview of floor geometries G1 to G7

3.5. Hollow core slabs and floor assembly

For all the fire tests in series G a 265 mm deep hollow core slab was used (See Figure 3.5, and Appendix 3.A for details). The slabs were cast on 18.08.2010 with concrete grade C45/55 and siliceous aggregate. The slabs were produced without and with protruding strands. The hollow core slabs were first stored inside the factory and after 7 weeks transported to the fire test laboratory. There, the slabs were further stored under controlled conditions (20ºC, 50% RH). The test floors were assembled one month before test date in order to enable the jointing material to harden. The hollow core floors G2 and G3 were assembled on 17.05.2011 (joints and tie beams). The floors G4 and G5 were assembled 08.08.2011 joints and tie beams), while the structural topping was cast on 09.08.2011. On 16.09.2011 the joints and tie beams of the floors G6 and G7 were cast. After the floor was assembled, test floors were further stored indoor under 20ºC, 50% RH in the climate room.

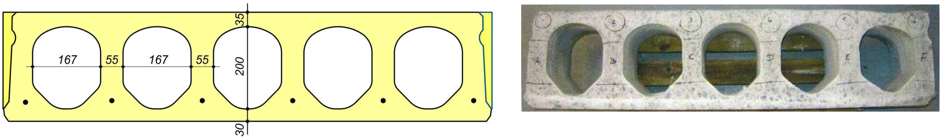


Figure 3.5. Hollow core cross section with depth 265 mm and 6 strands Ø12.5 at 50 mm axis distance

For the hollow core slabs at 28 days the recalculated mean cylinder strength (h300 mm, d150 mm) is f_{cm} = 50.0 N/mm^2 and the tensile strength f_{ctm} = 3.7 N/mm^2. The quality control tests (drilled concrete samples h53 mm, d 52 mm) at 442 days on slabs stored under the same conditions as the slabs for the fire tests, and recalculated to h300-d150 cylinder strengths resulted in $f_{cm,442}$ = 56.2 N/mm^2. The tensile strength (samples h120 mm, b100 mm, t50 mm) was $f_{ctm,442}$ = 3.8 N/mm^2. 13 weeks after production of the hollow core slabs the moisture content averaged 3.2%, while after 44 weeks the moisture content decreased to an average value of 2.5%.

The concrete grade used for the joints and topping was C25/30, the maximum diameter of aggregate 8 mm, and slump classification S5/S4. Vibration was not used. The floor topping and the peripheral tie beam were a C25/C30 concrete grade, with D_{max} = 16 mm and slump classification S3 (normal concrete). Vibration was used.

The Ø12.5 strands used for the hollow core slab production have a mean tensile strength f_{pm}= 1951 N/mm^2. Likewise, mean 0.1% strength $f_{pm,0.1\%}$ = 1735 N/mm^2 and mean Young's modulus E_m = 196650 N/mm^2. The characteristic value of the steel reinforcement bars was assumed f_{yk} = 500 N/mm^2.

3.6. Ambient shear tests according to EN1168:2005+A2:2009 Annex J

Five slabs with a length of 5300 mm were casted to test the shear capacity at ambient temperature with full scale reference tests according to EN1168:2005+A2:2009 Annex J [3.2] (at that time EN1168:2005+A3:2011 [3.1] was not yet available, however, in this Chapter further reference will be made to [3.1]). The casted length was 5300 mm (4.0 m plus 5h cantilever) as both ends were tested to give two experimental results per slab. The shear span applied in the Annex J test was 2.5 x 26.5 cm = 66.3 cm, and the support length at side A was 100 mm, see Figure 3.6. Span length L = 4.0 m, while 1.25 m was cantilevering beyond right-hand side support (accounted for in the reaction at support A).

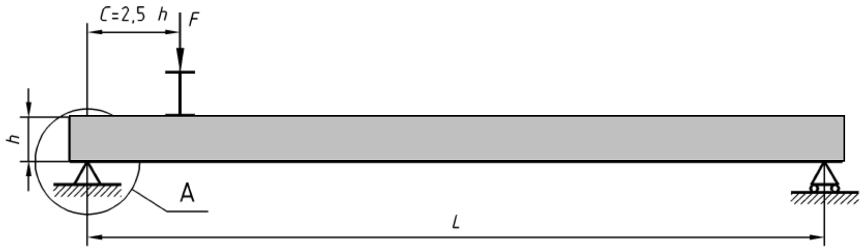


Figure 3.6. Loading scheme of slabs according to EN1168:2005+A2:2009 Annex J

According to EN1168:2005+A3:2011 clause 4.3.3.2.2.2 (shear resistance in uncracked regions, extended expression) the following shear capacities can be calculated (28 days, nominal values of the concrete cross section):

- ultimate design shear tension capacity (design values: $f_{ctd,}$ ℓ_{pt2}):
  V_{Rd} = 126.4 kN per slab;
- ultimate shear tension capacity (characteristic values: $f_{ctk;0.05}$, ℓ_{pt2}):
  V_R = 183.1 kN per slab;
- ultimate shear tension capacity (mean values: f_{ctm}, ℓ_{pt}):
  V_{Rm} = 258.1 kN per slab;

According to EN1168:2005+A3:2011 clause 4.3.3.2.2.3 the shear capacity is:

- ultimate design shear tension capacity (design values: f_{ctd}, ℓ_{pt2}):
  $V_{Rd,simplified}$ = 116.8 kN per slab (simplified expression).

The slabs have been tested at ambient temperature with 2 cycles at 70% of the ultimate design shear tension capacity. Then, the ultimate capacity was approached with steps of max 10%. See Figure 3.7 for failure pattern.

When in Table 3.II the results of 2011 are compared with the results of 2010, it can be concluded that the shear capacity did not change significantly as mainly the bond behaviour and tensile capacity influences the shear tension result. Therefore, the experimental capacity is taken as an average of 10 experimental results. Hence:

- Experimental capacity $V_{R,exp}$ = 262,0 kN per slab, standard deviation = 25.5 kN/slab, coefficient of variance = 25.5/262.0 = 9.7%;
- Ultimate design capacity V_{Rd} = 126.4 kN/slab → $V_{R,exp}$ /V_{Rd} = 262.0 / 126.4 = 2.07;
- Ultimate capacity (mean value) V_R = 258.1 kN / slab → $V_{R,exp}$ /V_R = 262.0 / 258.1 = 1.015.

Table 3.II Test results of reference EN1168 Annex J tests under ambient conditions

Slab #	Test date	Age [days]	Direct test load F [kN/slab]	Shear capacity $V_{R,exp}$ [kN/slab]
#31 – left	15.11.2010	89	351.3	301,7
#31 - right	15.11.2010	89	285.1	246.3
#35 - left	16.11.2010	90	287.9	248.6
#35 - right	16.11.2010	90	277.5	239.9
#34 - left	17.11.2010	91	306.1	263.9
#34 - right	17.11.2010	91	333.7	287.0
#32 – left	05.09.2011	383	337.7	291.1
#32 - right	05.09.2011	383	319.7	275.6
#33 – left	06.09.2011	384	270.2	232.9
#33 - right	07.09.2011	385	270.0	232.8
			Average per slab	262.0 kN/slab
			Average per m1	218.3 kN/m

Figure 3.7. Shear tension failure crack at ambient temperature in slab #31 at both sides

3.7. Fire test G1

On 15.11.2010 a fire spalling test was carried out in the small furnace of CERIB, France. In this furnace, the hollow core slab is placed up-side-down in the furnace, and is laid just on the floor without any particular boundary conditions. The slab is heated from the top side, see Figure 3.8. The test concluded that after 2 hours of ISO fire no spalling was observed under unrestrained and unloaded conditions. One day after the test vertical cracks were observed in the core at the soffit and the topside, which seems normal for these tests, see also Figure 3.8.

Figure 3.8. Specimen G1: before test (left) and after 2 hour fire test (middle). Vertical cracks in core after one day (right) (Note that slab was turned up-side-down in test so the soffit was facing upwards)

3.8. Fire tests G2 to G7 with 120 minutes of ISO fire

In 2011 the three floor fire tests were carried out in the Promethee furnace of CERIB, France. The fire tests were executed with an ISO 834 fire of 120 minutes. The exact spans and loads to reach the required shear load are depicted in Figure 3.9 and illustrated in Table 3.III. Appendix 3.B shows technical drawings of the support details of G2 to G7. One day before the fire test, all the floors were shortly preloaded at ambient temperature at 70% of the ultimate design shear tension capacity. Hence, the floor was preloaded with a load in order to obtain a shear force at the face of the support of 93.9 kN/slab (78.3 kN/m).

In the fire test the shear load in the hollow core floor at the face of the support was set equal to 100% of the shear and anchorage capacity calculated with EN1168:2005+A3:2011 Annex G at 120 minutes of ISO fire. Table 3.III gives the shear loads at the support ($V_{support}$). So, when with EN1168:2005+A3:2011 Annex G it is calculated that at 120 minutes of ISO fire the shear and anchorage capacity is 48.7 kN/m for G3, this was applied as a shear load $V_{support}$ during the 120 minutes fire test. From this shear load at the support, the magnitude of load in the jacks was derived, taking into account the self-weight of the hollow core floor, and including the self-weight of the steel beam used for the distribution of the load (see also Table 3.III).

Figure 3.10.a shows the total load in the jacks on the total floor in relation to time. In accordance with EN1363-1 [3.5] and EN1365-2 [3.7] the load was applied at least 15 minutes before the fire started. Further, in this Figure 3.10.a we see that there were different load levels during the fire test on the different floors as the shear and anchorage capacities were different. Also, we see in Figure 3.10.a that the load level remained constant during 120 minutes of ISO fire. Figure 3.10.b shows the deflection at mid span of the floor in relation to time. After preloading the linear deflection was in the order of 2 mm in the middle of the span. But we clearly observe in the Figures 3.10.a and 3.10.b that with a constant external load the deflection increased during the fire test. First, when the fire started at 0 minutes, the deflections quickly increased. After 30 minutes, deflections between 19 mm and 23 mm were measured at mid span on the different floors. Then, the rate of deflection decreased and the

deflections stabilized during the remainder of the fire test. Finally, at 120 minutes the deflections reached 35 to 41 mm.

Just after 120 minutes, the insulation (I) and integrity (E) criteria were checked and subsequently the fire was stopped. In none of the floors a shear failure had occurred. Table 3.IV shows the REI granted to the fire tests.

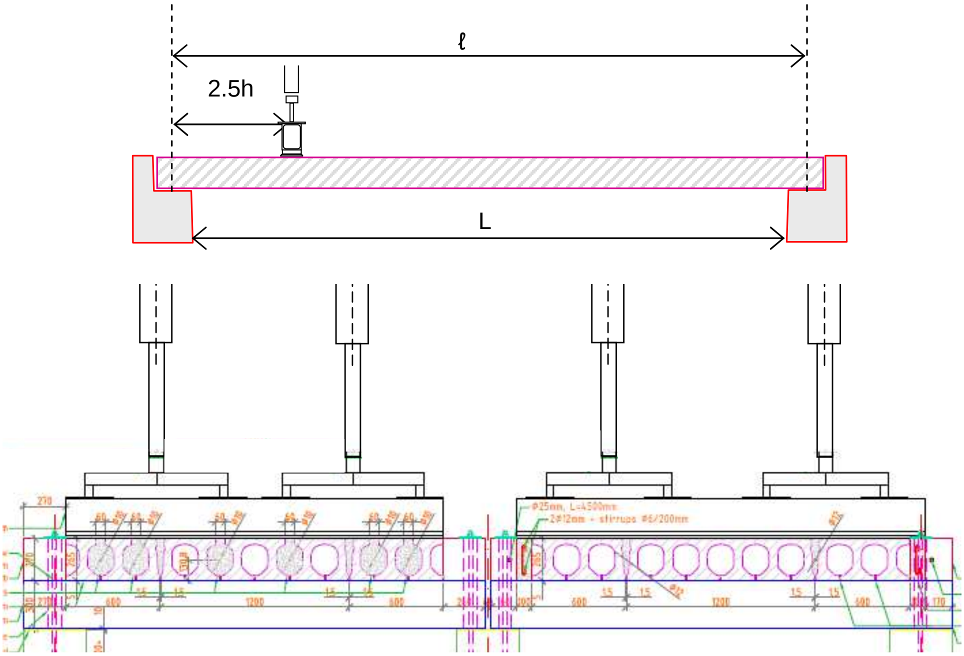


Figure 3.9. Loading scheme of slabs in fire test series G of G2/G3– longitudinal schematic view (up) and cross sectional view (down)

During the fire tests, many measurements were conducted, of which only minimum and maximum measured values are given in the Tables 3.V to 3.VII (do note the scatter in real measured values). Table 3.V gives measured values of the temperature at half height in the centre webs of the slab. After 60 minutes the temperature at mid height reaches approximately 100 ºC, while at 120 minutes the temperature varied between 134 ºC and 218 ºC with an average of 184 ºC. Table 3.V also gives the theoretical value calculated with Annex G at a depth from the soffit of 132 mm. At 120 minutes, Annex G calculates 274 ºC which implies that the temperature calculated by Annex G is higher than real measured (average) value. This is mainly because a maximum allowed temperature at the top of the slab of 160 ºC is assumed at 120 minutes, and the temperature at half height is calculated by interpolation between $a_{50\%}$ level and top of floor, see chapter 3.

Table 3.VI gives the measured temperature in the strands. After 1 hour the temperature reaches an average of 201 ºC, while at 120 minutes the temperature varied between 167 ºC and 517 ºC with an average of 382 ºC. Table 3.VI also gives the theoretical value calculated with Annex G at the axis distance of 50 mm (In Annex G strand is calculated 390 ºC after 120

minutes of fire). There is a very good agreement between the measured temperature in the floors and the temperature calculated by means of Annex G in the strands at axis distance of 50 mm.

Finally, in Table 3.VII the slip in the strands is given for G2 to G6. At 120 minutes the slip was only between 1 to 3 mm. The slippage in floor G4 with the 170 mm protruding strands is not necessarily less than the slippage in the floors without protruding strands (G2 and G3). The slip in G5 is about double the value of the slip in G4. In G7 slip measurement was not possible due to the stand-up of the L-shaped support beam.

Table 3.III Load in jacks based on calculated shear capacities ($V_{support}$)

Fire test ID	L mm]	ℓ [mm]	Structural topping thickness [mm]	Shear span (2.5 h) [mm]	Capacity 120 min. $V_{Rd,c,fi}$ [kN/m]	F_{jacks} [kN/m1]	Width of test floor [m]	F_{jacks} kN	# jacks [kN]	F_{jack} kN/jack
G2	3300	3400	-	662.5	52.3	56.9	2.4	136.6	2	68.3
G3	3300	3400	-	662.5	48.7	52.5	2.6	136.4	2	68.2
G4	3300	3500	50	787.5	63.7	70.8	2.6	183.9	2	92.0
G5	3300	3400	50	787.5	40.7	41.8	2.4	100.2	2	50.1
G6	3340	3420	-	662.5	48.7	52.3	2.6	136.1	2	68.0
G7	3340	3420	-	662.5	48.7	52.3	2.6	136.1	2	68.0

Table 3.IV Fire resistance results of fire tests G2 to G7

Fire test ID	date	Start fire [time]	Stop fire [time]	Fire loading time [minutes]	R [min]	E [min]	I [min]
G2	23-06-2011	10:50:05	12:58:04	128 minutes	128	120	120
G3	23-06-2011	10:50:05	12:58:04	128 minutes	128	120	120
G4	22 -09-2011	09:39:00	11:40:13	121 minutes	121	120	120
G5	22-09-2011	09:39:00	11:40:13	121 minutes	121	120	120
G6	21-10-2011	09:47:35	11:50:04	122 minutes	122	120	120
G7	21-10-2011	09:47:35	11:50:04	122 minutes	122	120	120

Table 3.V Temperature (min - max) measured h/2 at web, Annex G calculation at 132 mm from soffit

	G2 [ºC]	G3 [ºC]	G4 [ºC]	G5 [ºC]	G6 [ºC]	G7 [ºC]	Annex G [1)] [ºC]
30 min	36 - 70	37 - 49	35 - 45	38 - 58	51 -57	39 - 46	86
60 min	97 - 105	96 - 104	94 - 99	91 - 100	98 - 100	94 - 99	179
90 min	101 - 172	120 - 153	99 - 138	114 - 193	125 - 149	107 - 138	229
120 min	134 - 238	165 - 212	158 -197	169 - 218	167 - 210	149 - 186	274

1) calculated without a structural topping

Table 3.VI Temperature (min – max) measured in strands, Annex G calculation at 50 mm from soffit

	G2 [ºC]	G3 [ºC]	G4 [ºC]	G5 [ºC]	G6 [ºC]	G7 [ºC]	Annex G [1)] [ºC]
30 min	101 - 120	101 - 145	67 - 174	100 - 104	103 - 105	100 - 104	110
60 min	146 - 208	158 - 267	119 - 314	147 - 255	209 - 239	215 - 242	230
90 min	245 - 315	251 - 371	174 - 428	189 - 295	308 - 350	323 - 381	320
120 min	322 - 398	335 - 462	301 - 517	167 - 369	394 - 434	408 - 480	390

1) calculated without a structural topping

Table 3.VII Slip of strands in middle slab

	G2 [mm]	G3 [mm]	G4 [mm]	G5 [mm]	G6 [mm]	G7 [mm]
30 min	0.6 - 1.3	0.5 - 0.5	0.6 - 0.3	1.4 - 0.8	1.3 - 0.9	not possible
60 min	0.8 - 1.6	0.9 - 0.9	1.3 - 0.8	2.1 - 1.1	1.9 - 1.4	not possible
90 min	0.9 - 1.8	1.0 - 1.0	1.4 - 1.0	3.0	1.9 - 1.4	not possible
120 min	0.9 - 1.9	1.1 - 1.1	1.5 - 1.2	3.1	2.0 - 1.4	not possible

During the fire tests the strain in the ∅25 longitudinal bars in G3 and G6 was registered. After 120 minutes the strain in the ∅25 bars in G3 was approximately 2200 μm/m and G6 is 1500 μm/m. Also, the strain in the reinforcement bars ∅12 of the longitudinal tie-beam was registered. On average, at 120 minutes the strain in the ∅12 bars in G3, G4, G6 and G7 was approximately 1050 μm/m.

3.9. Peak shear and anchorage capacity of floors G2 to G7

After the fire was stopped, just after 120 minutes of fire, the floor was further loaded up to shear failure. This took in G4 to G7 between 30 and 50 minutes due to the loading rate of 10 kN/min for the whole floor, and due to the fact that the second floor could only be tests after the first floor. In test G2 and G3 the loading time was about 80 minutes due to the fact that a loading rate of 5 kN/min for the whole floor was initially chosen. At failure (between 150 minutes and 204 minutes), the measured temperature in the strands was on average a little bit higher than the temperature at 120 minutes, while the measured temperature at mid height of the web was significantly higher than the temperature at 120 minutes. Hence, although the fire was stopped at 120 minutes, the floor was still exposed to the accumulated heat.

Table 3.VIII presents details about the load applied by means of hydraulic jacks and the resulting peak shear load at the support, taking into account the self-weight of the slabs and the steel beam. The experimentally obtained peak shear capacity give rise to distinguish two groups; G2, G5 and G7 have a shear capacity of 82.5 kN, 76.3 kN and 87.8 kN, respectively, and G3, G4, and G6 have a shear capacity of 129.5 kN, 134.7 kN, and 125.0 kN, respectively. The lowest peak shear capacity was obtained in G5 (with 50 mm structural topping), while the highest peak shear capacity was obtained in G4 (also with 50 mm structural topping, and with protruding strands). These peak shear capacities were reached

between 150 minutes and 204 minutes. Hence, although it is evident that the calculated Annex G shear and anchorage capacity was resisted by the floor for 120 minutes, it is not evident that this peak shear load would have been resisted by the hollow core floor when it was loaded over the 120 minutes of fire with this high shear load. But we can conclude that there is a significant reserve capacity after 120 minutes of ISO fire.

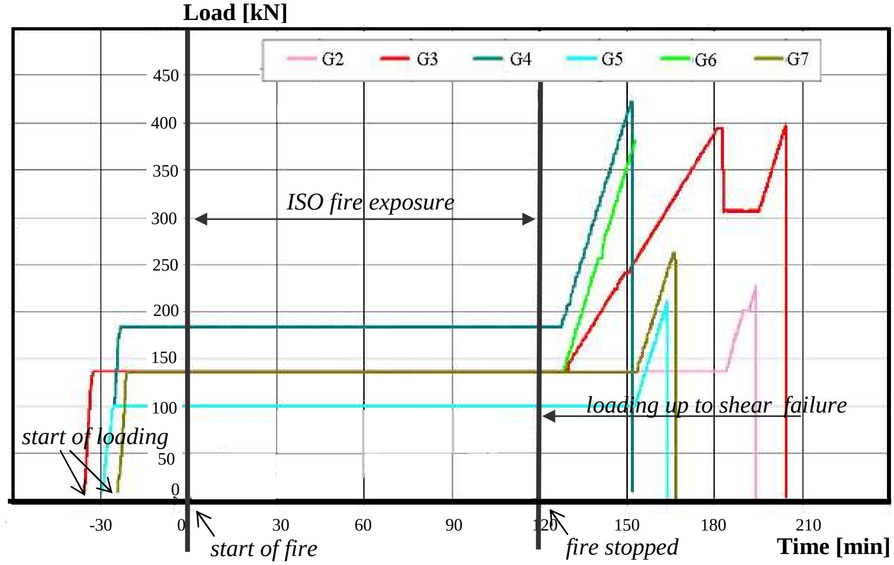


Figure 3.10.a Load – time diagram of fire tests G2 to G7

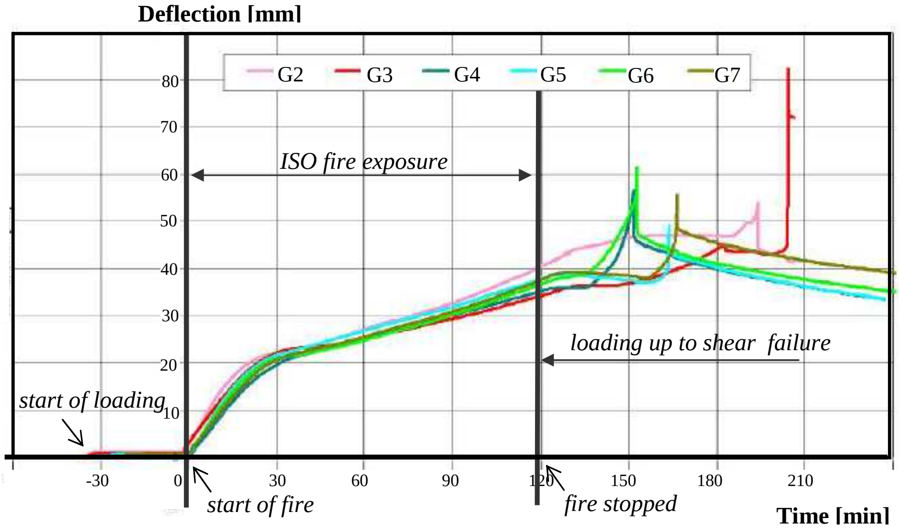


Figure 3.10.b Time – deflection diagram of fire tests G2 to G7

Figure 3.10b shows the time – deflection diagram of the six floors during the 120 minutes of fire, and during the time the floors were loaded up the failure. It shows that after the fire was stopped the deflection more or less kept increasing. At moment of failure, the magnitude of deflection rate increased significantly, and then test was stopped in order to prevent the floor to fall into the furnace. The total ultimate deflection varied between 49 mm and 83 mm for the various floors.

One day after the fire test the furnace was opened, and the floors were examined for spalling and cracks. Explosive spalling did not occur during the fire tests as no spots were observed were the concrete had spalled. When the soffit of the floors was examined, all floors clearly showed a failure mechanism indicating shear and anchorage failure. Figure 3.11 shows the failure patterns of the floor assemblies after failure. G2 and G5 do not contain a longitudinal tie beam, and these floors clearly show the inclined shear crack. These inclined shear cracks initiate from a bending crack and/or a vertical cracks due to thermal stresses, see also Figure 3.1. G3, G4, G6 and G7 do have a reinforced longitudinal tie beam which gives a bit unclear view of the failure crack as it hides the inclined shear crack.

During the fire tests the strain in the longitudinal ∅25 bars in G3 and G6 was registered. At failure the strain in the ∅25 bars in G3 was approximately 2200 μm/m and G6 was 1500 μm/m. Hence, the tensile stress equals 0,0022x210000 = 462 N/mm^2 and 0,0015x210000 = 315 N/mm^2 in the ∅25 longitudinal bar, respectively. This leads to a total force in the 2 longitudinal bars of 2 x 490 mm^2 x 462 N/mm^2 = 453 kN and 309 kN, respectively. On a total floor area with A_c = (2x171750 + 2x100x200) mm^2 = 383500 mm^2 this gives a “blocking” of 1.2 N/mm^2 in G3 and 0.8 N/mm^2 in G6. Also, the strain in the reinforcement bars ∅12 of the longitudinal tie-beam was registered. On average, at 120 minutes the strain in the ∅12 bars in G3, G4, G6 and G7 was about 1050 μm/m. Hence, this gives a tensile stress of 0,00105x210000 = 220 N/mm^2 in the bar, which leads to a total force in the 4 longitudinal bars of 4 x 113 mm^2 x 220 N/mm^2 = 100 kN. On a total floor area of 383500 mm^2 this gives a blocking of 0.2-0.3 N/mm^2 in G3, G4, G6 and G7.

Table 3.VIII Experimental peak shear capacities (V_{test}) derived from experimental load in jacks

test	Start loading [min]	Stop loading [min]	Loading rate [kN/min/ floor]	F_{jacks} [kN]	Width [m]	F_{jacks} [kN/m]	V_{jacks} [kN/m]	$V_{selfweight}$ [kN/m]	V_{test} [kN/m]
G2	184.2	194.0	10	226.7	2.4	94.5	76.0	6.5	82.5
G3	129.0	204.2	5 and 10	397.4	2.6	152.9	123.0	6.5	129.5
G4	127.5	151.3	10	422.2	2.6	162.4	125.9	8.8	134.7
G5	152.3	163.6	10	211.5	2.4	88.1	67.7	8.6	76.3
G6	128.5	152.6	10	382.2	2.6	147.0	118.5	6.5	125.0
G7	153.6	166.0	10	262.1	2.6	100.8	81.3	6.5	87.8

Figure 3.11. Photos of failure cracks at the sides of the floors after further loading up to failure after the fire was stopped

3.10. Analysis of test results and intermediate conclusions

In Table 3.IX the results of the fire tests are given in comparison with the calculated capacities according to EN1168:2005+A3:2011 Annex G. Overall observations based on the outcome of the fire tests show that:

- During 120 minutes of ISO 834 fire the floors G2 to G7 were capable of resisting a shear load $V_{support}$ equal to the calculated shear and anchorage capacity $V_{Rd,fi,c}$ according to EN1168:2005+A3:2011 Annex G;
- In all tests the peak shear capacity V_{test}, experimentally obtained after further loading after the fire was stopped at 120 minutes, is between 1.6 and 2.7 times higher than the shear capacity $V_{Rd,fi,c}$ calculated with EN1168:2005+A3:2011 Annex G at 120 minutes.
- The peak shear and anchorage capacities V_{test} of G3, G4 and G6 (125.0 to 134.8 kN/m, on average 129.7 kN/m) are significantly higher than the capacities of G2, G5, and G7 (76.3 to 87.8 kN/m, on average 82.2 kN/m).

Table 3.IX Test peak shear capacities (V_{test}) compared with EN1168:2005+A3:2011 Annex G ($V_{Rd,c,fi}$) under fire at 120 minutes

	G2	G3	G4	G5	G6	G7
$V_{Rd,c,fi,120}$ [kN/m]	52.3	48.7	63.7	40.7	48.7	48.7
$V_{support}$ [kN/m]	52.3	48.7	63.7	40.7	48.7	48.7
V_{test} [kN/m]	82.5	129.5	134.7	76.3	125.0	87.8
V_{test} / $V_{Rd,c,fi}$ [%]	158%	266%	211%	187%	257%	180%

Observations over the various floors based on the outcome of the fire tests show that:

- G3 and G6 show that the 2Ø25 mm bars in combination with the peripheral tie beam significantly increases the shear capacity;
- The difference between the shear capacities obtained in G2 and G3 cannot be explained by the difference in position of the connection reinforcement (G2 in core, G3 in joint) but by the restraining effect caused by the longitudinal tie beam and 2Ø25 bars, see also Table 3.X;
- G4 shows that when applying protruding strands (17 cm) and the peripheral tie beam without the 2Ø25 mm bars the shear capacity increases significantly;
- The difference in shear capacity between G4 and G5 is mainly attributed to the longitudinal tie beam and the protruding strands in G4;
- When comparing G6 to G3, it shows that a vertical connection between the floor system and the peripheral tie beam is not needed;
- When comparing G7 to G2, G3 and G6, it is evident that an L-shaped beam also is capable of achieving a good shear and anchorage capacity.

The results of the individual failure tests of the G series could also be compared to the total longitudinal reinforcement (converted to kN) at the support connections in cores or joints, exterior bars ∅25, topping reinforcement plus additional anchorage bars, and protruding strands. For the latter, we have calculated the tensile capacity by the anchorage of the strands in the support tie beam at the slab end. The classification of the tests according to the quantity of longitudinal reinforcement is given in Table 3.X.

Conclusions on the fire tests based on Table 3.X:

- There is good agreement between the classification of total longitudinal reinforcement and shear capacity. Classified in longitudinal reinforcement 1-3 also classifies in shear capacity 1-3. And same for 4-6;
- There is also a relation between longitudinal blocking and shear capacity measured during the test. Table 3.X gives information on the longitudinal confinement and classification of the test results. The longitudinal efforts are calculated assuming arbitrarily that the reinforcements crossing the support section are mobilized to their yield strains. This analysis does not take into account the influence of location of the steel in the depth of the cross section;
- The exterior bars ∅25 provide a longitudinal restraint of the thermal expansion, and are hence keeping the vertical cracks closed. Series G demonstrate that the longitudinal bars ∅25 are not needed for the shear capacity, but they influence the final shear capacity in an important way;
- The longitudinal connecting reinforcements are playing an important role in the shear capacity of the HC floor, but in different ways:
- The bars in the joints and filled cores are keeping the vertical thermal cracks closed, and thus preserving thus the shear capacity of the slabs;
- The protruding strands are active in the anchorage of the prestressing at the slab ends and improve in this way the shear capacity significantly;
- The exterior bars ∅25 provide a longitudinal restraint of the thermal expansion, and are hence keeping the vertical thermal cracks closed;
- The vertical connections through stirrups in the supporting beam are not needed, if good detailing is provided (for example like test G6, section of 1.88 cm2/m of connection reinforcement in each joint and a peripheral tie reinforcement of 2∅12 surrounding the whole floor structure).

Table 3.X Total longitudinal reinforcement at support compared to shear capacity

Test number	Shear capacity [kN/m]	Classification shear capacity	Total longitudinal reinforcement, converted to kN	Classification longitudinal reinforcement	Longitudinal blocking [N/mm^2]
G3	129.5	2	996	2	1.4
G4	134.7	1	784	3	0.2
G6	125.0	3	1027	1	1.0
G2	82.5	5	402	6	0
G5	76.3	6	473	5	0
G7	87.8	4	596	4	0.2

3.11. Shear capacity under fire compared to ambient temperature

In the Holcofire G-series research we have obtained the shear and anchorage capacity ($V_{Rd,fi,c}$) at 120 minutes of ISO 834 fire, the peak shear and anchorage capacity (V_{test}) after further loading up to failure, and the average shear tension capacity ($V_{R,exp}$) at ambient temperature as an average result of Annex J tests. Both Table 3.XI and Figure 3.12 present a comparison with the different capacities. Note that in case of fire tests G4 and G5 also for comparison with $V_{Rd,fi,c}$ and V_{test} the reference of the shear tension capacity $V_{R,exp}$ without a structural topping is taken. On the one hand, it emerges that in the different hollow core floors the shear load during the fire test, which was chosen to be equal to the Annex G shear and anchorage capacity, ranged from 18.6% to 29.2% of the shear tension capacity obtained from Annex J tests at ambient temperature. During 120 minutes of ISO 834 fire the hollow core floors were capable of resisting this as a shear load. The peak shear capacity, on the other hand, ranged from 35.0% to 61.7% of the shear tension capacity obtained from Annex J tests at ambient temperature.

Table 3.XI Annex G shear capacity ($V_{Rd,fi,c}$) and peak shear capacity (V_{test}) compared with experimental shear tension capacity ($V_{R,exp}$) at ambient temperature and shear capacity calculated with simplified expression ($V_{Rd,simplified}$)

	G2	G3	G4	G5	G6	G7
$V_{Rd,fi,c}$ [kN/m]	52.3	48.7	63.7	40.7	48.7	48.7
V_{test} [kN/m]	82.5	129.5	134.7	76.3	125.0	87.8
$V_{R,exp}$ [kN/m]	218.3	218.3	218.3	218.3	218.3	218.3
$V_{Rd,fi,c}$ / $V_{R,exp}$ [%]	24.0%	22.3%	29.2%	18.6%	22.3%	22.3%
V_{test} / $V_{R,exp}$ [%]	37,8%	59,3%	61,7%	35,0%	57,3%	40,2%
$V_{Rd,simplified}$ [kN/m]	97.3	97.3	97.3	97.3	97.3	97.3
$V_{Rd,fi,c}$ / $V_{Rd,simplified}$ [%]	53,8%	50,1%	65,5%	41,8%	41,8%	50,1%
V_{test} / $V_{Rd,simplified}$ [%]	84,8%	133,1%	138,4%	78,4%	128,5%	90,2%

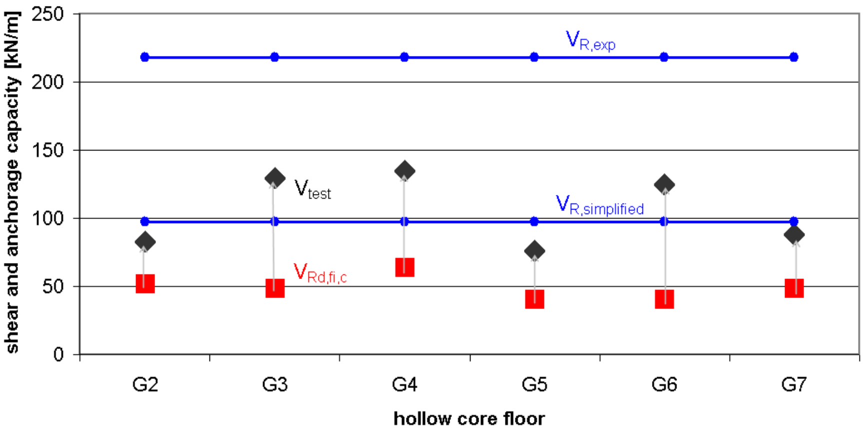


Figure 3.12. Shear capacity during fire test ($V_{Rd,fi,c}$) and peak shear capacity (V_{test}) compared to ambient shear capacity ($V_{R,exp}$)

In EN1168:2005+A3:2011 Annex G tabulated values are given in Table G.2. In Table G.2 we derive for a slab thickness 240-280 mm and REI-120 that $V_{Rd,c,fi}$ = 0.55 $V_{Rd,c,cold}$. As we have experimental results of both the shear and anchorage capacities under fire conditions and at ambient temperature, we can make a comparison with Table G.2. In this Table G.2 the shear tension capacity $V_{Rd,c,cold}$ is calculated with the simplified shear tension model expression of EN1168:2005+A3:2011. In chapter 6 it was calculated that with the simplified expression for the cross section used in G-series $V_{Rd,simplified}$ = 116.8 kN/slab = 97.3 kN/m. But in the Table G.2 the 55% tabulated value of $V_{Rd,c,fi}$ = 0.55 $V_{Rd,c,cold}$ is based on an example where $V_{Rd,fi,c}$ is calculated with 1.88 cm^2/m of longitudinal tie reinforcement and 70 mm support length. In our fire tests, G2 comprised 1.31 cm^2/m longitudinal tie reinforcement, G3, G6, G7 comprised 0.94 cm^2/m longitudinal tie reinforcement, and G4 and G5 did not have longitudinal tie reinforcement at all. Also the support lengths were different in the Holcofire fire tests. Consequently, the calculated values will be different than the tabulated value of 55%. Accordingly, from Table 3.XI emerges that the calculated value would be equal to the shear and anchorage capacity that was resisted during the fire test; this ranged for the different hollow core floors between 41.8% and 65.5% of the shear tension capacity calculated with the simplified expression. These percentages can be directly compared to 55% from Table G.2 discussed earlier for the particular example of 1.88 cm^2/m of longitudinal tie reinforcement and 70 mm support length. Because the peak shear and anchorage capacity ranged between 78.4% to 138.4% of $V_{Rd,simplified}$ (=$V_{Rd,c,cold}$) we can conclude that the tested hollow core floors had a significant reserve capacity. This additional reserve capacity proofs that EN1168:2005+A3:2011 Annex G is on the safe side and that Table G.2 gives a safe lower tabulated limit of shear and anchorage capacity.

3.12. Conclusions

It is concluded from the fire test that during 120 minutes of ISO 834 fire the hollow core floors were able to withstand a shear load equal to the shear and anchorage capacity calculated with EN1168:2005+A3:2011 Annex G. The experimentally obtained peak shear capacity is 1.6 to 2.7 times higher than the shear and anchorage capacity calculated at 120 minutes by means of EN1168:2005+A3:2011 Annex G. This implies that in the tested hollowcore floors there is a significant reserve in the shear and anchorage capacity under fire conditions. Hence, it is concluded that with the usage of the shear and anchorage capacity formula in the European product standard EN1168:2005+A3:2011 Annex G the designed hollow core floor is safe for ultimate limit state for fire design.

The peak capacities of G3, G4 and G6 (on average 129.7 kN/m = 155.7 kN/slab) are significantly higher than the peak shear capacities of G2, G5, and G7 (average 82.2 kN/m = 98.6 kN/slab). This can be explained by the total longitudinal reinforcement (converted in kN) that was present in the floors of G3, G4 and G7, because Annex G formula does not consider explicitly the “system effect”, such as the use of peripheral ties beams or external blocking. These effects influence the shear and anchorage capacity positively. From the parameters investigated in the fire tests it is concluded that the Annex G formula safely predicts the shear capacity of a single prestressed hollow core slab, including parameters such as protruding strands, connection reinforcement, and a structural topping. In addition, considering the overall beneficial contribution of applying slabs in a whole building system, the designed hollow core floor is safe for ultimate limit state for fire design with EN1168:2005+A3:2011 Annex G.

Reference shear tests at ambient temperature showed that the average shear tension capacity of the slabs is 262 kN/slab (= 218.3 kN/m). A comparison concludes that the peak shear capacity after 120 minutes ranged from 35.0% to 61.7% of the shear tension capacity obtained by Annex J shear test (without topping) at ambient temperature.

At the same time, the standardised fire test arrangement described in EN1168:2005+A3:2011 Annex G showed that there is a contribution of the longitudinal tie beam and Ø25 mm bar by virtue of blocking in longitudinal direction. The addition of a longitudinal tie beam or Ø25 mm longitudinal bar is necessary in the fire test arrangement when the general floor arrangement and connections ensure this longitudinal blocking. Moreover, the longitudinal tie beam assures the “system effect” when performing a fire test on a floor. But regarding the test set-up as described by EN1168:2005+A3:2011, it is concluded that the longitudinal bars ∅25 are not necessarily needed.

References Chapter three

[3.1] CEN, EN1168:2005+A3:2011 "Precast concrete products – Hollow core slabs" - Annex G: Resistance to fire, October 2011.

[3.2] CEN, EN1168:2005+A2:2009 "Precast concrete products – Hollow core slabs" - Annex J: Full scale test, 2009.

[3.3] CEN, EN1992-1-1 EC2: "Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings", December 2004.

[3.4] CEN, EN1992-1-2: "Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design",

[3.5] CEN, EN1363-1: "Fire resistance tests – Part 1: General requirements", August 1999.

[3.6] CEN, EN1363-2: "Fire resistance tests – Part 2: Alternative and additional procedures", August 1999.

[3.7] CEN, EN1365-2 "Fire resistance tests for load bearing elements – Part 2: Floors and roofs", December 1999.

[3.8] Van Acker, A., "Shear resistance of prestressed hollow core floors exposed to fire". Structural Concrete, Volume 4, Issue 2, pp. 65 –74, 2003.

[3.9] Hietanen, T. "Fire tests for Finnish hollow core slabs". Published by the Association of the Concrete Industry of Finland, p. 17, May 1992.

[3.10] Fellinger, J. "Shear and anchorage behaviour of fire exposed hollow core slabs", PhD thesis Delft University of Technology, ISBN 90-407-2482-2, p. 261, 2004.

[3.11] Andersen, N.E., Lauridsen, D.H. "Hollow Core Concrete Slabs, Basismiddel project X52650", Technical Report X 52650, part 2, p. 29, 1999.

[3.12] Bogogno, W., Fontana, M. "Versuche zum Tragverhalten von Betonhohlplatten met flexibler Auflagerung bei Raumtemperatur und Normbrandbedingungen". Institut fur Baustatik und Konstruktion, ETH Zurich, p. 246, 1996.

[3.13] Jensen. J.F. "Huldæk og Brand - dokumentation vedrørendeforskydningskapacitet". Udarbejdet for Betonelement-Foreningen, Birch & Krogboe A/S, p. 24, 2005.

[3.14] C.G. Bailey, Lennon "Full-scale fire tests on hollowcore floors". The Structural Engineer, 18, pp. 33-39, March 2008.

[3.15] Van Acker, A. "Fire safety of prestressed hollowcore floors, shear capacity of hollow core floors". CPI Concrete Plant International, 1, pp. 182-196, 2010.

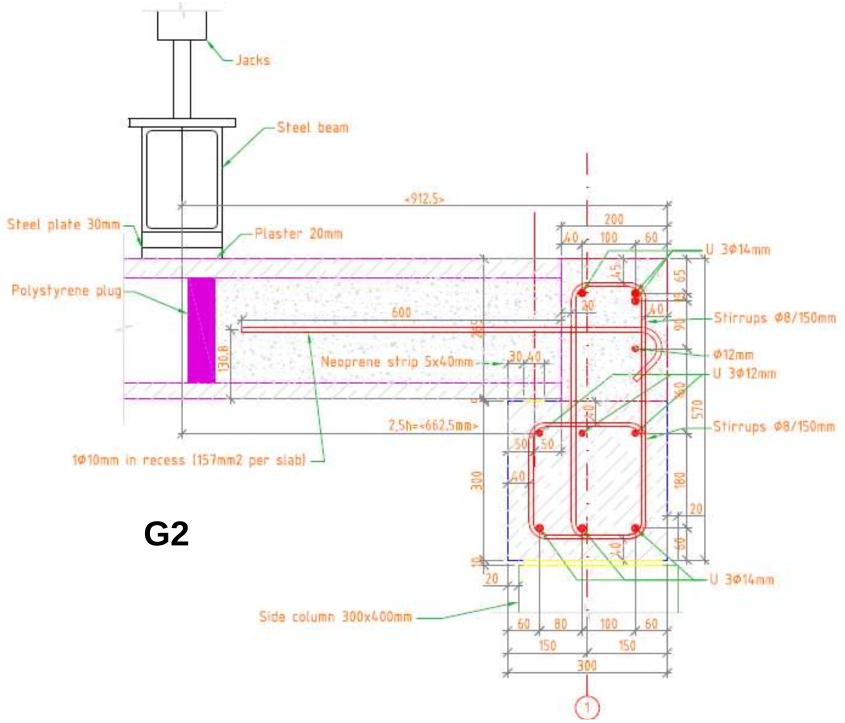
[3.16] Jansze, W, Arnold Van Acker, Bruno Della Bella, Ronald Klein-Holte, Gösta Lindström, Andreas Nitsch, Jean-Paul Py, Matthieu Scalliet. "Fire resistance of hollow core floors regarding shear and anchorage capacity. " 7th International Conference on Structures in Fire, edited by M. Fontana, A. Frangi, M. Knobloch, Zurich, Switzerland, June 6-8, 2012, pp 709-718.

Appendix 3.A - Hollow core slab cross section data

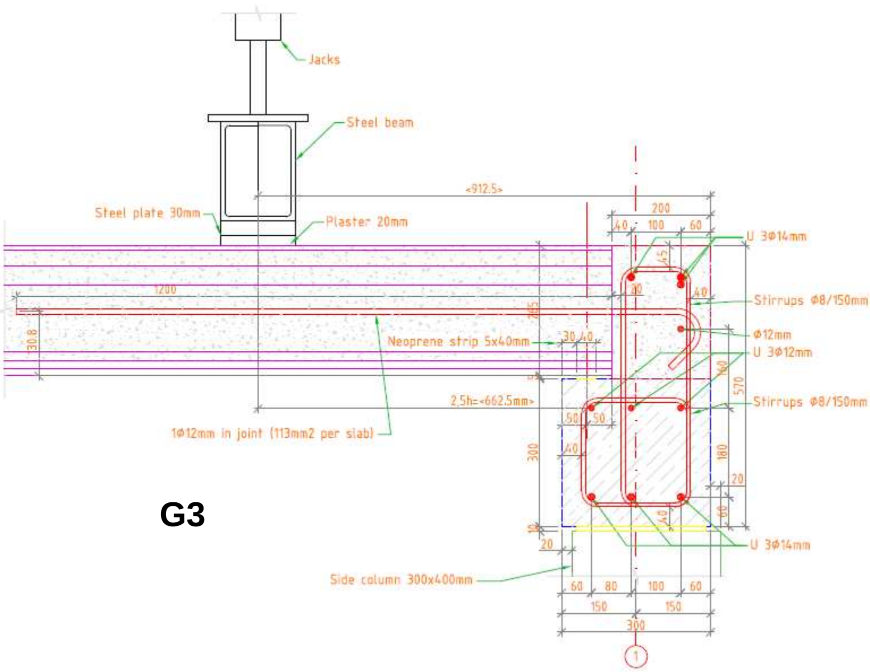
Concrete slab		
Slab depth	h	= 265 mm
Slab width	b	= 1197 mm
Concrete area	A_c	= 168467 mm^2
Centre of gravity from soffit	z_c	= 134 mm
Total web thickness	b_w	= 326 mm
Level where web thickness is 50% of total width	$a_{50\%}$	= 58 mm
Second moment of inertia	I_c	=1447377000 mm^4
Section modulus, top	$W_{c,top}$	= 10781 cm^3
Section modulus, bottom	$W_{c,bottom}$	= 11070 cm^3
Concrete slab with joint filling		
Cross section	A	= 171750 mm^2
Centre of gravity from soffit	z	= 135 mm
Second moment of inertia	I	=1474200000 mm^4
Section modulus, top	W_{top}	= 10889 cm^3
Section modulus, bottom	W_{bottom}	= 11374 cm^3
Concrete quality (target)		
Concrete quality	C	= C45/55
Mean cylinder compressive strength Eurocode concrete 28 days	f_{cm}	= 53 N/mm^2
Aggregate	= silicious	
Mean cylinder concrete compressive strength (results from quality control)		
28 days, recalculated to cylinder h300 mm, d150 mm	$f_{cm,,28}$	= 50.0 N/mm^2
121 days outdoor, recalculated to cylinder h300 mm, d150 mm	$f_{cm,121}$	= 67.3 N/mm^2
442 days conditioned, recalculated cylinder to h300 mm, d150 mm	$f_{cm,442}$	= 56.2 N/mm^2
Prestressing steel		
Mean tensile strength	f_{pm}	= 1951 N/mm^2
Mean 0.1% strength	$f_{pm,0.1\%}$	= 1735 N/mm^2
Young’s modulus	E_p	= 196650 N/mm^2
Initial prestressing	σ_{pm0}	= 1100 N/mm^2
Type of tendon	type	= “strand”
Diameter of tendon	$Ø_p$	= 12.5
Total area of tendon	A_p	= 6 * 93 = 558 mm^2
Axis distance of prestressing reinforcement	y_p	= 50 mm
Design capacities		
Design bending moment capacity	M_{Rd}	= 176 kNm/slab
Design shear capacity (shear tension, region not cracked in bending)	V_{Rd}	= 126.4 kN/slab
Design shear capacity (flexural shear, regions cracked in bending)	V_{Rd}	= 84.3 kN/slab
Shear and anchorage capacity under fire conditions		
Shear and anchorage capacity at 60 minutes of ISO 834 fire	$V_{Rd,c,fi,60}$	= 45.0 kN/m
Shear and anchorage capacity at 120 minutes of ISO 834 fire	$V_{Rd,c,fi,120}$	= 35.5 kN/m
Shear and anchorage capacity at 180 minutes of ISO 834 fire	$V_{Rd,c,fi,180}$	= 27.0 kN/m

(Shear and anchorage capacity under fire conditions based on following values: ISO 834, h = 265 mm, A_c = 168423 mm^2, b_w = 326 mm, $a_{50\%}$ = 58 mm, f_{cm} = 53 N/mm^2, aggregate = silicious, f_{yk} = 500 N/mm^2, A_s = “no reinfo”, y_s = n.a., σ_{pt} = 960 N/mm^2, strand, $Ø_p$ = 12.5 mm, A_p =558 mm^2, y_p = 50 mm, a = 100 mm, η_1 = 0,7)

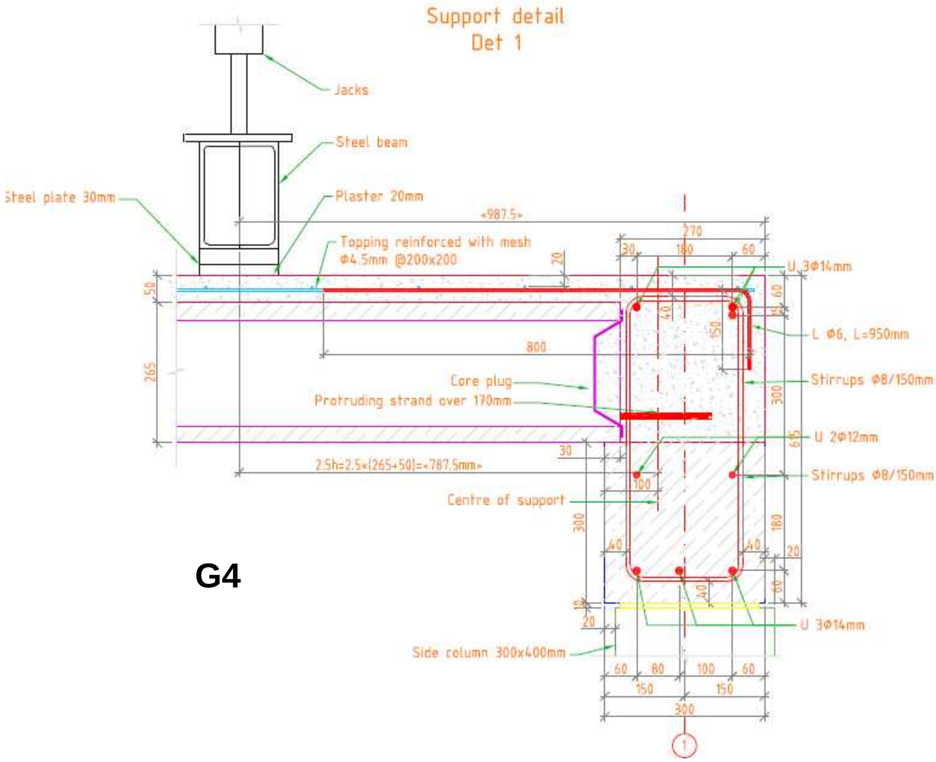
Appendix 3.B - Overview of support details of G2 to G7; technical drawings



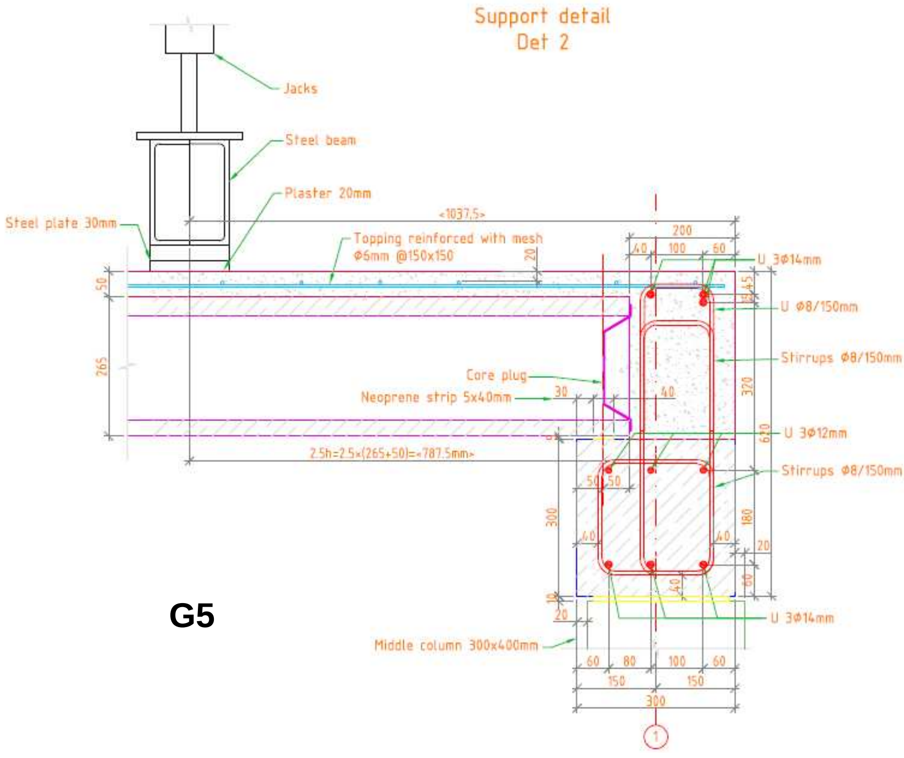
G2



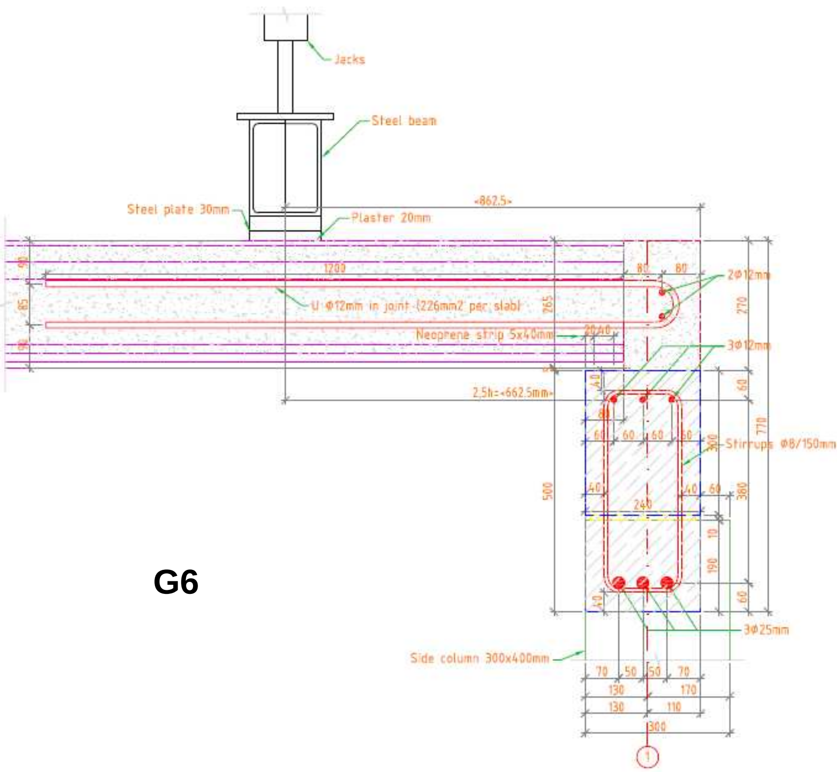
G3


Support detail
Det 1
Jacks
Steel beam
Steel plate 30mm
Plaster 20mm
Topping reinforced with mesh
Φ4.5mm @200x200
U 3Φ14mm
L Φ6, L=950mm
Stirrups Φ8/150mm
Core plug
Protruding strand over 170mm
U 2Φ12mm
Stirrups Φ8/150mm
2.5h=2.5×(265+50)=<787.5mm>
Centre of support
U 3Φ14mm
Side column 300x400mm

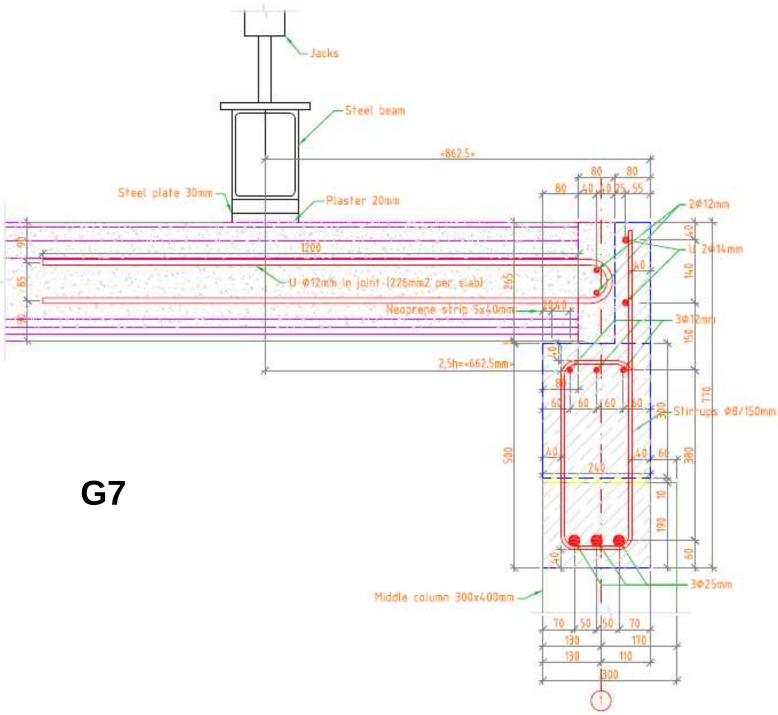
G4


Support detail
Det 2
Jacks
Steel beam
Plaster 20mm
Steel plate 30mm
Topping reinforced with mesh
Φ6mm @150x150
U 3Φ14mm
U Φ8/150mm
Stirrups Φ8/150mm
Core plug
Neoprene strip 5x40mm
U 3Φ12mm
Stirrups Φ8/150mm
2.5h=2.5×(265+50)=<787.5mm>
U 3Φ14mm
Middle column 300x400mm

G5


Jacks
Steel beam
Steel plate 30mm
Plaster 20mm
862.5
1200
U Φ12mm in joint (226mm2 per slab)
Neoprene strip 5x40mm
2Φ12mm
3Φ12mm
2,5h=662.5mm
Stirrups Φ8/150mm
3Φ25mm
Side column 300x400mm

G6


Jacks
Steel beam
Steel plate 30mm
Plaster 20mm
862.5
1200
U Φ12mm in joint (226mm2 per slab)
Neoprene strip 5x40mm
2Φ12mm
U 2Φ14mm
3Φ12mm
2,5h=662.5mm
Stirrups Φ8/150mm
3Φ25mm
Middle column 300x400mm

G7

4

Chapter Four

Flexible Supports

Shear resistance of hollow core slabs on flexible supports under fire conditions

Keywords: *Hollow core slab, flexible support beams, shear resistance, fire, EN1168 Annex G*

Abstract. *It is generally known that the shear resistance of the hollow core slabs is reduced under ambient conditions if the slabs are supported by non-rigid beams, the socalled flexible supports. Under ambient conditions it is evident that the deformation of the beam initiates composite action that alters the mode of behaviour of the structure and introduces additional bending and additional shear stresses in the transversal direction of the slab. Both phenomena lead to a reduced shear capacity under ambient conditions for slabs on flexible supports. Nowadays, there is still no design procedure for flexible supports specified in the European standard EN1168:2005 +A3:2011 for hollow core slabs. EN1168 states only that "in case of flexible supports, the reducing effect of transversal shear stresses on the shear capacity shall be taken into account." In some countries however the design recommendations set out in fib Bulletin 6 are used in construction to design hollow core with flexible supports. But most countries do not take into account the flexible support effect under ambient conditions, because for most practical applications sufficient shear resistance remains. This Chapter motivates that the decrease of shear capacity due to flexible supports is not magnified by fire conditions. On the contrary: the thermal gradient over the cross sections due to the fire compensates the negative effects of flexible supports. On the one hand,*

the underflange of the hollow core slabs expands and is under compression such that the additional bending stresses are compensated. On the other hand, vertical thermal cracks occur in the webs of the hollow core slab such that a shear tension failure cannot occur anymore. As a result, the shear resistance "falls back" to the level of flexural shear resistance. Accordingly, it is concluded in this Chapter that for hollow core slabs on rigid supports and flexible supports, EN1168 Annex G can be used for determining the shear resistance under fire conditions.

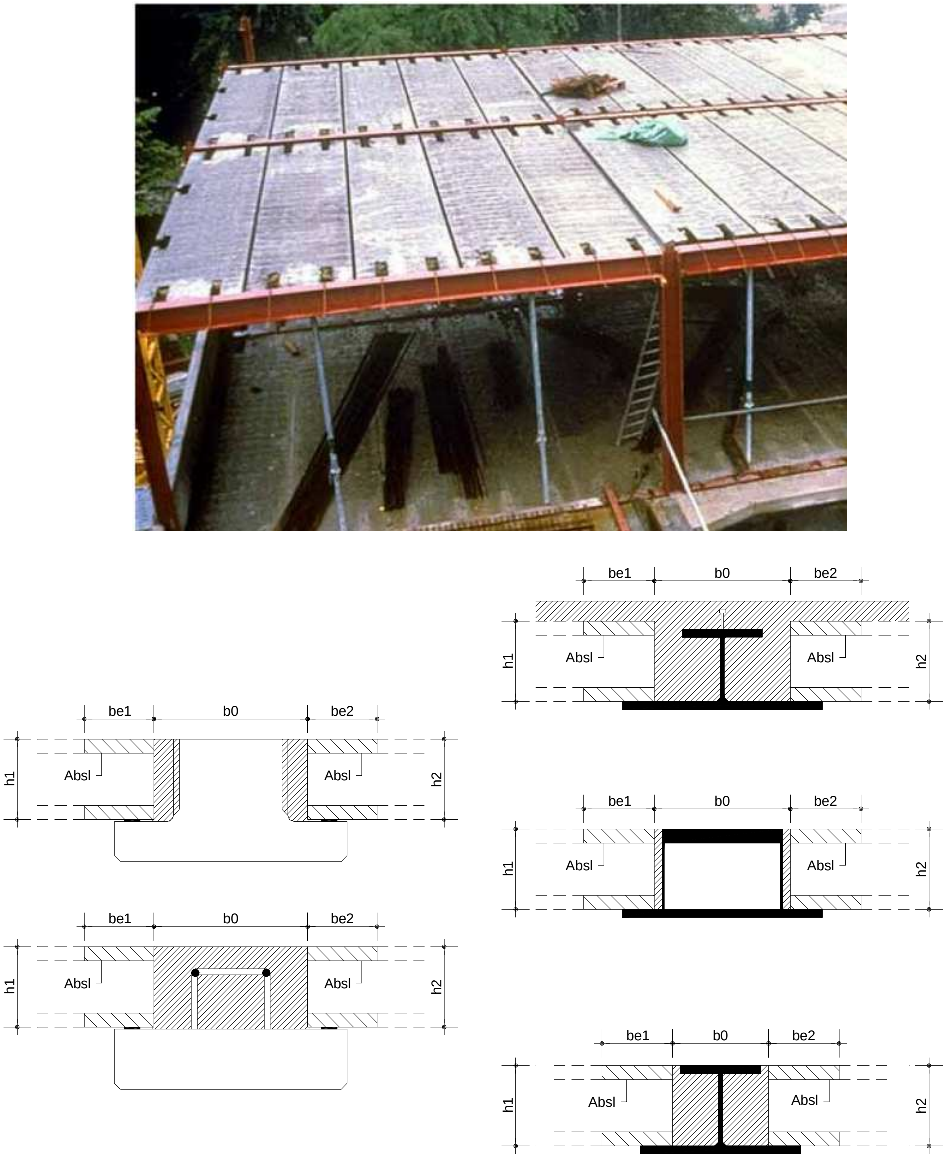


Figure 4.1. Slim floor structure with hollow core slabs (up) [4.17]; various types of slender concrete and steel beams (down) [4.4]

4.1. Introduction

During the first decades when the prefabrication of concrete structures was in full development, hollow core slabs were commonly supported on rigid concrete walls. Then, in the 1980s, in Scandinavia in slim-floor structures flexible supports were introduced by supporting the hollow core slabs on slender steel beams. These slender beams (nowadays either steel, concrete or composite) deflect under permanent and live loading and are therefore flexible supports [4.19]. Of these integrated beams the beam height is usually slightly greater than the height of precast slab element such that slim floor structures are obtained, see Figure 4.1. The prestressed hollow core slabs are currently the most popular solutions for long span floors (6 – 20 m) in slim floor structures, and are supported on the wide lower flange. Mainly, the fast erection, low self weight, and high stiffness with relatively span-to-depth ration (l/35) are decisive for their success. Also, interior walls and facades can be arranged freely, or even replaced, since these walls have no load bearing function anymore. Finally, the flat soffit gives freedom to arrange the electrical and water ductwork. A minimum slab weight is obtained by the combination of high quality concrete (C45/55 or C55/C67) and effective prestressing, leading to 30 % less concrete and 50 % less steel. The minimum weight also affects also the supporting structure underneath and the foundation [4.13].

However, it is evident that due to flexibility of the support the hollow core slabs follow the deflection of the support which reduces the shear capacity of the hollow core floor [4.13]. When hollow core floors are supported on beams, the deformation of the beam will initiate composite action between the floor slab and the beam. This action alters the mode of behaviour of the structure and introduces additional bending and shear stresses in the transversal direction of the slab. Depending on the rigidity or flexibility of the support beam, the transversal stresses will affect to a certain extent the shear tension capacity of the floor. The effect of the flexible supports on the load bearing capacity of the hollow core slabs was investigated extensively at VTT in Finland in 1990s [4.1, 4.2, 4.3]. Pajari observed in his six tests that the shear resistance was only 40-70% of the shear resistance observed in reference tests in which the slab units were supported on rigid supports. The reduction of shear capacity is due to transverse deformation in the hollow core slab units caused by the deflection of the beam. Despite, at failure, the deflection of the supporting beam was relatively small, typically l/400 to l/250 of the span of the beam.

In spite of the reduction in shear resistance of hollow core slabs due to the influence of flexible supports, sufficient shear resistance remains for most practical applications of precast construction [4.4]. Accordingly, the practical implication of this reduction is however limited for two reasons. On the one hand, the shear tension capacity of the hollow core slab generally does not govern the design. On the other hand, the load bearing capacity of the beam generally limits the allowable load on the total floor system rather than the hollow core slab itself. [4.13]. But the question is whether the practical implication of this reduction is also limited under fire conditions, or that due to the fire exposure additional stresses are introduced that limit the shear capacity of the hollow core floors on flexible supports even more. There is a belief that the thermal expansion during a fire has a positive effect on the shear capacity because of the induced compression in the lower part of the cross section as a result of thermal gradient over the height. Fellinger mentioned some words about this aspect in his PhD thesis [4.13] on page 57: “Premature shear and anchorage failure can not be attributed to the reduction in the shear tension capacity by flexible supports if there exists any negative effect of flexible supports on the fire resistance at all.” Accordingly, the objective of this Chapter is to address whether the decrease of shear capacity under flexible supports as stated at ambient temperature, is magnified by the extreme fire conditions, or that the flexible support effect needs not to be taken into account at all under fire conditions?

4.2. Flexible support and design procedure

Nowadays, despite the frequent usage of hollow core slabs on flexible supports, there is no design procedure or model specified [4.19] in the European standard for hollow core slabs EN1168:2005+A3:2011. EN1168 [4.11] states only in clause 4.3.3.2.2.1 under the general verification procedure for shear and torsion capacity that “in case of flexible supports, the reducing effect of transversal shear stresses on the shear capacity shall be taken into account.” However, this lack of design procedure in the standard has been acknowledged, and flexible supports will possibly be integrated in a new revision of EN1168 by TG1 of WG1 of TC229. Outside the European standards, a well known design procedure is available in fib Bulletin 6 published in January 2000 [4.7]. This design procedure is mostly used in the Northern part of Europe, whereas in the Southern part of Europe this method is not used and the reduction of shear tension is still ignored. But as the shear capacity of hollow core slabs is not fully used in many slim floor structures, a reduction of the shear capacity by flexible supports may not be a problem. In Italy for example, the shear tension capacity is not largely used since, due to large use of continuity design of floors , shear is limited to flexural shear capacity. German instructions [4.20] limit the shear capacity of the hollow core slab to 50% and the permissible beam deflection to L/300 (at ultimate limit state load).

As the slim floor structures were developed in Scandinavia, the first important work was the Nordic research project carried out in 1991-1993. In 1990, an experimental project at VTT in Finland was carried out in which two full-scale loading tests were performed for slim floors made of 265 mm hollow core slabs [4.1]. The results however showed that further research was needed. Accordingly, a project on hollow core slabs on flexible supports started in 1991. From that work in 1993 the first version of the well known Code Card 18 was published in Finland [4.6]. The support beams were designed in such a way that the deflection due to the test loading was equal in order to exclude torsional moments from the hollow core floor [4.5]. The work on hollow core slabs on flexible supports continued with series of tests carried out at VTT Technical Research Centre in Finland up to the year 2006 by Pajari and Leskelä. The influence of shear and torsional moments in the floor is extensively researched by Broo, Lundgren and Engström in the Holcotors project in period 2002-2004. Most recently, Roggendorf [4.17] executed at RWTH Aachen 8 full-scale tests on two-span floor systems with a shallow steel beam (IFB) as middle support. A peripheral tie beam was cast around the floor to consider a systems effect as common in practice. Hence, considering the tests by Pajari and Leskelä and Roggendorf, a database comprising 39 fullscale tests has been established that was used by Roggendorf to derive an enhanced design model to determine the shear resistance of hollow core slabs on flexible supports The design models by Pajari-Leskelä and Roggendorf are not contradictory, but rather complementary to each other, since they are based on the same experimental tests. However, the main shortcoming of the model by Pajari-Leskelä is the omission of the non-linear effects, for example cracks in the joint, which the model by Roggendorf tries to take into account [4.19, 4.20].

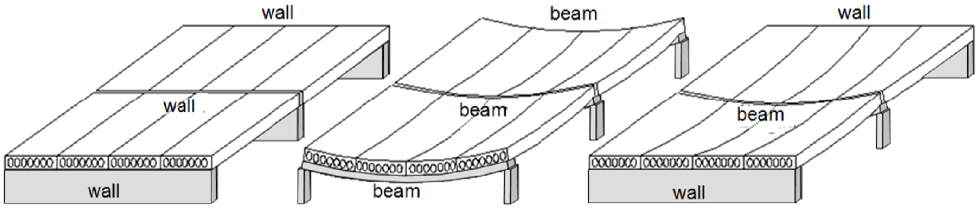


Figure 4.2. Rigid suppport (left); flexible supports without torsion (m) and with torsion (right) [4.5]

4.3. Reduction of shear capacity under flexible supports

The analyses of the hollow core slab floor are in general based on 2-dimensional stress distributions [4.7]. This is theoretically valid when the slab is subjected to symmetrical and uniformly distributed loading and is supported on a rigid support such as a wall or a deep beam, under the assumption that the supports are parallel and the angle between span direction and support direction is 90° (Figure 4.2). In 1990, further steps were taken in the application of steel beams to support hollow core floors. Two floors supported by a newly developed steel profile in Finland were tested in the laboratory of VVT in Helsinki to investigate the composite action between the integrated beam and the hollow core floor. But to the surprise of the researchers and the producers the slim floor structure failed far before the expected capacity was reached; not the integrated beam but the precast hollow core slab was decisive. This was the start of a wider research in Scandinavia to the effects of a flexible supported hollow core slab floors [4.1, 4.2]. Nowadays, in many cases the hollow core slabs are supported on steel, prestressed or reinforced beams of moderate stiffness for rigid or flexible supports, or a combination, see Figure 4.2.

The effect [4.13] of the flexible supports on the capacity of the hollow core slabs is illustrated in Figure 4.3. Due to the limited stiffness of the supporting beams, the hollow core slabs will deform not just in the spanning direction of the slab, but also in the transversal direction which is the span of the beam. Depending on the stiffness of the supporting beam relative to the transverse stiffness of the hollow core slabs, the load is more or less directly transferred from the hollow core slab to the columns. The beam and the slab will act together as a composite beam, either intended or unintended (but unavoidable due to friction). This composite action causes a lower shear capacity due to additional stresses, where the shear capacity is based on the uncracked situation respectively on shear-tension failure.

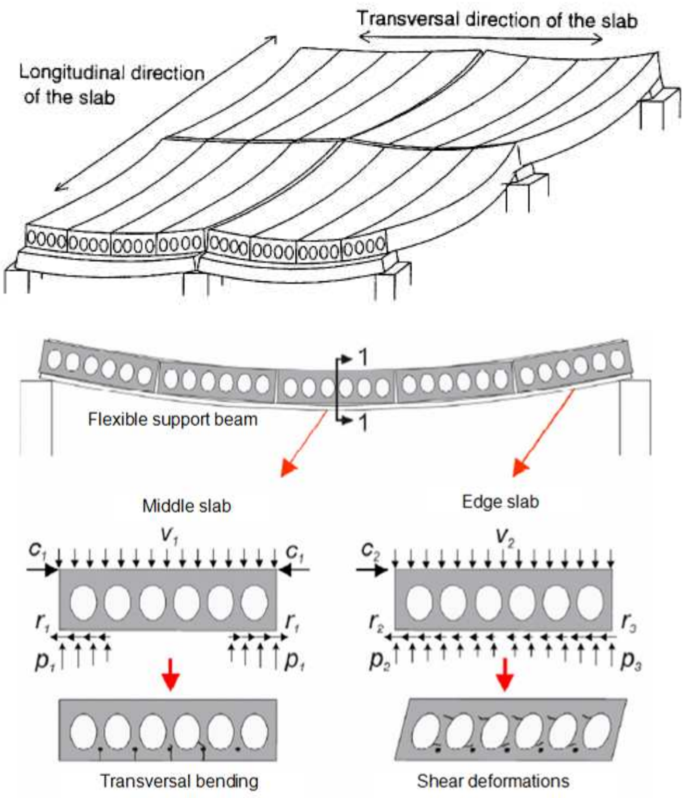


Figure 4.3. Effect of flexible supports on the hollow core slab at two positions [4.17]

The additional stresses are depending on the position along the beam:

- At mid span of the slender support beam it introduced additional transverse bending stresses in the underflanges of the hollow core slab [4.13]. If the hollow core slabs are supported so low on the beam that bending of the beam gives rise to transverse tensile stress at the bottom of the slab, the soffit of the slab tends to crack longitudinally parallel to the strands [4.19, 4.20]. These tensile stresses can initiate splitting cracks along the strands that reduces the bond between the strands and the concrete [4.9]. As a consequence, a reduced amount of anchored strands will reduce the shear capacity of the slabs affected by cracks in the underflange [4.13]. Longitudinal cracks also reduce the transverse bending stiffness of the slabs. But this transversal bending phenomenon is not determining the lower shear capacity at flexible supports.
- Near the stiff column position additional transverse shear stresses in the webs of the hollow core slabs are introduced. On the basis of series of experimentally full-scale tests it has been shown that the failure of the floor always begins from the outermost slabs. Hence, while the deflection of the beam occurs, the hollow core slabs are supported only in the area of their outermost webs [4.19]. Due to friction between

the hollow core slab and the slender beam along the concrete the support area, the deformation at the support is hindered and a horizontal force originated in the webs of the slabs in transverse direction that induces shear stresses [4.5]. Due to the additional shear stresses in the webs, shear tension failure will occur at a lower applied load than in case the hollow core slabs are supported on rigid walls [4.13]. So, the edge slabs are subjected to shear deformation, which is the main reason for reducing the shear capacity. The lack of parallelism of opposite ends of the hollow core slab may result in additional stresses due to torsion [4.19].

Pajari developed design recommendations [2 in 4.13] on the basis of a composite beam model that predicts the reduction of the shear tension and anchorage capacity. In this model, both the effective part of the hollow core slab and the supporting beam are considered as bending members, satisfying Bernoulli's hypothesis of plane cross sections remaining plane. The shear deformation of the hollow core slab and the longitudinal slip of the hollow core units along the beam are implicitly taken into account by an adjustment of the effective width b_{eff} of the hollow core slab contributing to the composite beam. By variation of b_{eff}, the calculated transverse shear flow in the webs varies accordingly. Torsional effects are not explicitly taken into account in Pajari's model. Excessive torsion is excluded by a limitation of the field of application to composite beams that remain linear elastic under the ultimate load with limited curvature.

Partial composite action with the slab increases the stiffness of the support beam [4.4]. The ultimate bending resistance of the beam should ignore composite action because cracks develop at the interface with the hollow core units. However, the design may take account of composite action where mechanical shear connectors are provided and reinforcement is placed across the beam and embedded in the hollow cores or joints. The structural resistance of the hollow core slabs on flexible supports can be improved by [4.4]:

- Providing additional shear resistance, such as by infilling the ends of the hollow core units to a distance equal to the depth of the hollow cores, or with in situ toppings over the units with a suitable amount of reinforcement.
- Increasing the stiffness of the supporting beam, such as by developing continuity by use of a continuous beam or through its connections, or by choosing a heavier or deeper beam than may be required for bending resistance.

4.4. Recent tests on flexible supports at ambient temperature

The most recent thorough study on flexible supports was conducted in Germany by Roggendorf [4.17]. The floor measures 10 m (slabs span two times 5 m) x 6 m (IFB beam), see also Figure 4.4 for summary and photo of test set-up. General conclusions from the experimental research are:

- The test show that at beam deflections from L/100 - L/200 the ratio of the shear resistance on flexible over rigid supports V_{fl} /V_r ranged from 52 - 78 %. The finite element (FE)-analyses show that an increase in shear resistance occurs when the beam deflection is limited significantly. The results indicate, however, that the range of flexible supports extends to relatively stiff structures. Further evaluation yields that the shear resistance should be reduced appropriately for any support with deflections greater than L/3000 under service loads.

- The tests show that slabs with slender webs obviously reach smaller ratios V_{fl} /V_r as expected in [4.1]. However, the ultimate load was controlled by the concrete tensile strength and the total web width.
- The tests show that premature shear failure is mainly attributable to shear deformations of the edge slabs due to composite action with the beam. The FE-analyses models confirm that the premature shear failure of hollow core slabs on flexible supports is attributable to a transverse shear flow and shear deformations. They reveal that the load transfer and the corresponding damage within the edge slabs are not uniform but concentrate towards the outermost webs of a floor, where failure is initiated.
- The FE-analyses show that the interaction properties between the slabs and the beam govern the occurring shear deformations. A low degree of composite action leads to greater beam deflections but enhances the edge slabs shear resistance. However, even in the cracked state of the floor's grouted joints a considerable transverse shear flow through the webs appears. The tests show that an in-situ concrete filling of selected hollow cores did not increase the shear resistance but the results from tests 3 and 4 indicate that the resistance against shear deformations is enhanced.
- Especially in the tests with the stiffer beam only marginal longitudinal cracking due to transverse bending of the inner slabs without effects on the overall bearing capacity of the floors occurred. The shear resistance of the edge slabs was not affected by longitudinal cracks in any test.

test	slab type	parameters	v_{fl} / v_r [-]	deflection of IFB u_{max} [mm]
1	A		0.66	56.2 (L/107)
2	B		0.61	31.4 (L/191)
3	A		0.68	31.5 (L/190)
4	B		0.60	29.1 (L/206)
5	B		0.52	26.8 (L/224)
6	A		0.78	34.9 (L/172)
7	A		0.71	32.5 (L/185)
8	B		0.57	29.8 (L/201)

Figure 4.4. Table with summary of experimental result from Roggendorf and photo of test set-up [4.17]

4.5. Previous tests on flexible supports under fire conditions

Several tests programmes on the shear resistance of hollow core slabs on flexible supports under fire conditions were carried out in the past. All available fire tests on slim floor structures are with steel beams as the steel industry was pro-active to test the slim floor construction experimentally under fire conditions. In the Holcofire database in total 18 fire test results were collected [4.18]:

- Fire tests on hollow core floor supported on a beam with steel boot carried out at the CTICM laboratory in France in June 1993 [Holcofire database H80] and November 1995 (Holcofire database H88, H89, H90, H91: [4.18])
- Fire test on a slim floor structures carried out at ETH/EMPA Zurich Fire by Borgogno and Fontana [12] in 1994 and 1995 (Holcofire database H81, H82, H83, H84, H85, H86, H87: [4.18])
- Full scale tests on hollow core floors at BRE by Prof. Bailey of University of Manchester [14] in May 2007 (Holcofire database H143 and H144: [4.18])
- Tests on Fire resistance of hollow core slabs supported on non-fire protected Deltabeams [15] conducted in Sweden in November 2009 by Peikko (Holcofire database H145, H146, H147, H148: [4.18])

A clear overview of the mentioned fire tests is presented in Figure 4.5 [4.18]. In the table under test set-up, SYS means that the test set up was a system with 2 support beams, SYSB that the slabs were supported on 3 beams, and HCS one single slab. Under failure type, R means that the required fire resistance time was reached. NO indicates that no failure occurred. SA stands for anchorage failure, SB for shear-bending interaction, DF for deflection, OT for other type of failure. Combinations of above mentioned abbreviations are possible.

In all tests at CTICM some type of premature failure occurred due to incorrect test set-up. The obtained fire resistance ranged from 32 to 100 minutes. They would also have failed with a concrete supporting beam.

Only the EMPA tests at ETH [4.12] had correct designs with a reinforced topping and stirrups connecting the hollow core slab to the support. The test that did not fail gave more than 100 minutes of fire resistance. The aim of the test was to check the fire resistance of a slim floor construction with hollow core slabs. The enlarged bottom flange of the steel girder was without fire insulation. The shear load on the hollow core was V_{max} = 31.9 kN/m. The test was stopped after 120 minutes ISO fire and after two hours of cooling down phase the slabs were loaded up to failure. The maximum shear load at failure was 79.1 kN/m. From the drawings and pictures in the test report, it could be concluded that the shear failure was mainly due to normal shear failure, which is in fact a consequence of the absence of tie connections between the hollow core floor and the supporting beam.

In order to evaluate the effect of flexible supports on the fire behaviour of hollow core slabs, the reduction of the shear tension capacity by the flexible supports was calculated by Fellinger [4.13]. He concluded that the flexible shear tension capacity varied in these tests between 68-98 % of the shear tension capacity corresponding to rigid supported hollow core slabs. The effect of flexible supports on the shear and anchorage behaviour of fire exposed hollow core slabs is overshadowed by the scatter in the results of fire tests on hollow core slabs. Fellinger concluded that premature shear-anchorage failure cannot be attributed to the reduction in the shear tension capacity by flexible supports if there exists any negative effect of flexible supports on the fire resistance at all.

Holcofire fire test #	Fire test name	test year	Fire curve	Slab depth [mm]	topping [mm]	slab width [mm]	total web width [mm]	Total area of strand [mm2/slab]	Axis distance [mm]	Expo sed length of test set up, [m]	width of test set-up [m]	no of slabs	test set-up	EN1365-2	time [min]	Failure after fire
H80	CTICM 93-G-127	1993	ISO 834	160	0	1200	558	416	45	6,4	6	5	SYS	1	32	SB
H81	EMPA 95-1	1994	ISO 834	160	80	1200	526	624	30	4,7	2,4	3	SYSB	1	122	R-NO
H82	EMPA B2-1	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	122	R-NO
H83	EMPA B2-2	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	49	SA
H84	EMPA B2-3	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	74,6	OT
H85	EMPA B2-4 PL	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	75,4	SA
H86	EMPA B3-1	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	96,6	SA
H87	EMPA B3-1 PL	1995	ISO 834	200	0	1200	472	624	30	4,7	2,4	3	SYSB	1	97,4	R-NO
H88	CTICM 95-E-467	1995	ISO 834	160	50	1197	530	624	50	4	2,4	2	SYS	1	50	SB
H89	CTICM 95-E-533	1995	ISO 834	160	50	1197	530	624	30	4	2,4	2	SYS	1	100	R-DF
H90	CTICM 96-U-349	1996	ISO 834	160	50	1197	530	624	30	4	1,2	1	HCS	0	71	R-DF-SB
H91	CTICM 96-U-350	1996	ISO 834	160	0	1197	530	624	30	4	1,2	1	HCS	0	42	DF-SB
H143	BRE test1	2007	parametric	200	0	1200	330	651	31	7	17,76	15	SYS	1	60	R-NO
H144	BRE test2	2007	parametric	200	0	1200	330	651	31	7	17,76	15	SYS	1	60	R-NO
H145	SPTRI Peikko P802216A	2009	ISO 834	270	0	1200	286	930	35	5,8	3,6	4	SYSB	1	60	R-NO
H146	SPTRI Peikko P802216B	2009	ISO 834	270	0	1200	286	930	35	5,8	3,6	4	SYSB	1	60,4	R-NO
H147	SPTRI Peikko P802216C	2009	ISO 834	270	0	1200	286	930	50	5,8	3,6	4	SYSB	1	120	R-NO
H148	SPTRI Peikko P802216D	2009	ISO 834	270	0	1200	286	930	50	5,8	3,6	4	SYSB	1	180	R-NO

Figure 4.5. Table with summary of experimental result from Holcofire [4.18]

The largest fire test is the BRE full scale fire test in Cardington. The tests are actually flexible support tests, but the flexible support was not the subject of study. Two full scale fire tests at BRE [4.14] were designed in a compartment of internal plan dimensions 7.02 x 17.76 m^2, with an internal floor to soffit height of 3.6 m. Figure 4.6 shows the floor plan. The units were supported on steel beams that were fire protected with fire board. The compartment was formed using 100 mm thick blockwork, which was protected with 15 mm thick fire board, with unprotected hollow core slabs forming the ceiling. One longitudinal block wall was masoned inside the steel frame, whereas the opposite wall was positioned outside the frame. In the latter case, the steel profiles were free to deflect. The situation corresponded to a flexible support structure.

The two fire tests were identical except for the end restraint conditions to the hollow core slabs. In the first test the slab units sat directly onto the supporting beams with the units notched around the columns. The joints between the units, and the gaps around the columns and the units, were filled with grout. In the second test, 2 T12 bars per unit were placed in the cores and around a 19 mm shear stud fixed to the steel beam. The cores with the rebars, the end of the slab, the gap between the units and the gap between the units and steel columns were filled with grout.

The natural fire concept was used, and assuming the design for an office, the fire load density was 570 MJ/m^2 (80% fractile). All hollow core slab units performed very well during the heating phase of the fire, which was more severe than the standard fire curve during about 80 minutes. The floor, as a whole, performed also well during the cooling phase of the fire, see Figure 4.7. The applied load of 4.5 kN/m^2 was achieved using 60 sandbags (each weighing 1 ton) evenly positioned over the floor plate. This gave an applied load of 4.71 kN/m^2. The self-weight of the units was 2.96 kN/m^2, creating a total load of 7.67 kN/m^2, and an applied moment at the time of the fire of 56.37 kNm per with of unit. This gave a load/capacity ratio of $M_{ed}/M_{rd,fi}$ = 0.34 for bending and $V_{ed}/V_{rd,fi}$ = 0.26 for shear.

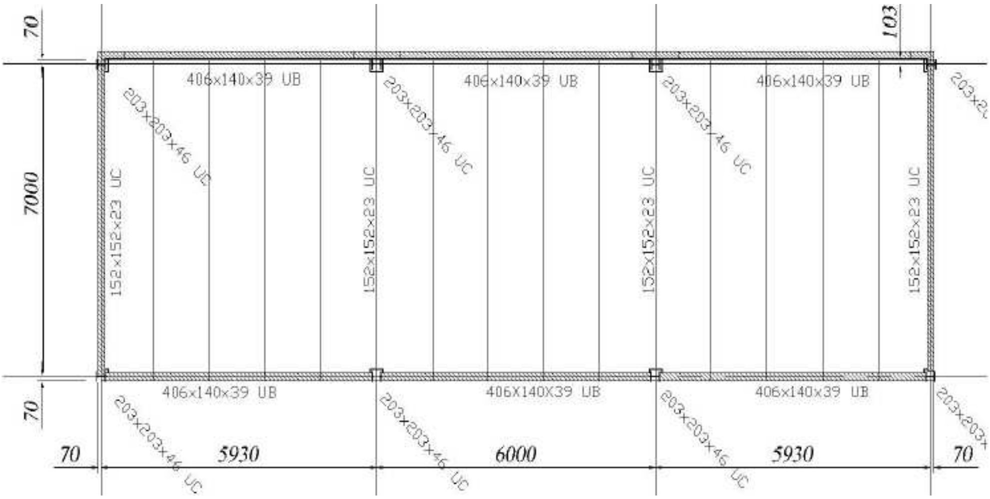


Figure 4.6. Plan of BRE full scale fire test in Cardington [4.14]

Figure 4.7. View within the compartment after the fire of the first test [4.14]

The last four tests from Figure 4.5 that were carried out on flexible supports research the ability of a Deltabeam without fire insulation to support hollow core slabs during a fire situation [4.15]. The four fire tests had respectively fire exposures of two tests with 60 minutes, one test with 120 minutes and one test of 180 minutes. The span length of the

Deltabeam was 3.915 m, and the span length of the hollow core 265 was 2.35 m (Figure 4.8). The concrete filling of the voids in the hollow core units was 50 mm. The hollow core units were connected to the Deltabeam with 1∅12 in each longitudinal joint. The load was 48 kN/m at a distance of 675 mm (2.5xH_{slab}) from the Deltabeam in the first and third test, 57.6 kN/m in the second test, and 30 kN/m in the last test. All four tests were successful, because the floors maintained their load bearing capacity during the entire test periods.

Figure 4.8. Top view of flexible support test by Peikko with Deltabeam [4.15]

The capacity of the shear load transferred from slab to Deltabeam during 60 minutes of standard fire plus 120 minutes of standard cooling phase was found to be 46 kN/m inclusive dead load of the slab. This value corresponds to 35.0% of the characteristic cold shear value of the tested slab. The capacity of the shear load transferred from slab to Deltabeam during 120 minutes of standard fire plus 248 minutes of standard cooling phase was found to be 39 kN/m inclusive dead load of the slab. This value corresponds to 29.4% of the characteristic cold shear value of the tested slab. The slab has been prepared to resist 120 minutes of fire by increasing the bottom cover on the strands by 15 mm. The capacity of the shear load transferred from slab to Deltabeam during 180 minutes of standard fire without a cooling phase was found to be 26 kN/m inclusive dead load of the slab. This value corresponds to 19.8% of the characteristic cold shear value of the tested slab. The slabs and the Deltabeams were in this test designed to resist 120 minutes of standard fire. Due to the choice of typical hollow core slab the test results can be assumed to be valid for all normal hollow core slabs supported on Deltabeams. The bearing capacity in the fire situations is given as a fraction of the characteristic bearing capacity in a cold design situation.

Report 15] concludes that the Deltabeam was able to carry the load from the hollow core slab during the four fire tests. [4.15] states that the transfer of load from the hollow core slab to the Deltabeam did not happen through the support of the slab on the bottom flange of the Deltabeam, as the bending capacity of the bottom flange in all the tests was practically zero due to the high temperatures of the bottom flange ~ furnace temperature. The load transfer must therefore rely on the compression of the slab to the inclined web of the Deltabeam – a bow action – plus friction along the web surface. The compression arises from tension in the joint reinforcement between the hollow core slabs and possibly also from the hindrance of the expansion of the slab structure.

4.6. Thoughts on hollow core slabs on flexible supports at fire

We can conclude that to date, and in comparison with the flexible support tests under ambient conditions, no comprehensive studies or tests are available on the fire behaviour of hollow core floors with flexible supports to make thorough conclusions. This could on the one hand be attributed to the size of the testing furnaces. In order to study the flexible support effect, longer spans of both the hollow core floor and the slender support beam are needed to study the effect of flexible support. This is not possible in furnaces that measure 4 m x 6 m. On the other hand, to conduct a unique real fire test like BRE is too costly to execute, but knowing that most likely it will not yield to different information and different insights. Therefore, in the following, the Holcofire authors therefore analyse by simple reasoning, the positive and negative effects of a fire on the behaviour of a hollow core floor with flexible supports. It is argued that the shear capacity of a hollow core floor with flexible supports exposed to fire is influenced by several parameters which are explained hereafter:

a. Induced thermal stresses and vertical web cracking;
b. Thermal expansion of underflange;
c. Deflection of the supporting beam;
d. Continuous supporting beam;
e. Imposed loading;
f. Web width;
g. Tensile strength of the concrete;
h. Type of connection with the supporting beam;
i. Structural topping.

a. Induced thermal stresses and vertical web cracking

Source [4.13] explains that vertical thermal cracks develop in the webs at regular distance over the entire length of the slab within 14-16 minutes of standard fire exposure, irrespective to the geometry, the prestress level, or the loading configuration, see Figure 4.9. The induced thermal stresses, acting in the longitudinal direction of the floor, are due to the temperature gradient over the hollow core cross-section during a fire. The phenomenon of the occurrence of vertical cracks at regular distances will not be influenced by a flexible supporting beam. Hence, as is the case at rigid supports, shear tension failure by definition cannot occur anymore because the webs are vertically cracked. Consequently, the situation of vertical cracking will not be different between rigid and flexible supports. The negative effect on the shear resistance at fire can be annihilated through measures at the support keeping the thermal cracks closed. In this way the shear capacity will be restored through the interlocking effect within the cracked webs.

Figure 4.9. Vertical thermal cracks in hollow core slabs due to temperature gradient over cross section [4.14]

b. Thermal expansion of underflange

The temperature profile exists also in the transversal direction of the slab. Due to the expansion of the bottom flange with respect to the top flange, the hollow core units will deflect in the transversal direction (Figure 4.10). However, the temperature gradient over the cross-section will not be the same as in the longitudinal direction, because of the presence of continuous voids. There will be compression in the top and bottom flanges, no tension in the webs in the transversal direction, but shear stresses in the webs due to the differential deformation of the top and bottom flanges (Figure 4.10). At the support zone of the slabs, the transversal thermal deformation will probably not cumulate with the transversal deformation due to the deflection of the supporting beam, since both phenomena are independent from each other. It can be concluded that the fire situation will not aggravate the transversal shear stresses at the support. On the contrary, due to the introduction of compressive forces in the bottom flange in the transversal direction, there might even be a decrease of those shear stresses.

In the same way as for rigid supporting beams, the longitudinal restraint caused by the blocking of the floor will restore the shear capacity through the interlocking effect in the thermal cracks. So far there is no difference between a rigid and a flexible supporting beam. As far as the transversal blocking concerns, the question is whether the deflection of the supporting beam affects the restraint of the floor in the transverse direction. At the support zone, there will be no negative influence, as argued above under § a. Outside the supporting zone of the floor span, there will probably be a partial accumulation of the deflection of the supporting beam and the transversal thermal deflection of the floor. This phenomenon could reduce the total blocking of the underflange by the surrounding structure. Would this be positive or not, depends on the situation of the slabs in the floor area.

In the middle of a large floor, the biaxial compression stresses are dangerous for buckling spalling of the underflange. Any reduction of those stresses will have a positive effect on this phenomenon. At the edge of a floor, there will be much less accumulation of the afore mentioned transversal deflections. In addition, the restraint by the surrounding structure will be small. Also here, it could be concluded that a flexible support will not be more unfavourable in the fire situation than a rigid support. Opposite, a flexible support introduces more flexibility that as a positive effect could release the compression stresses in the soffit of the floor.

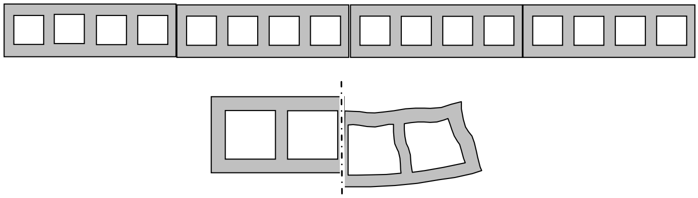


Figure 4.10 Idealized cross-section a of hollow core floor subjected to fire; initial state (left) and deformed state due to thermal expansion of the bottom flange (right)

c. Deflection of the supporting beam

The deflection of the supporting beam will probably increase during a fire, dependent whether the support beam is protected against the fire or not. In a flexible floor structure with a concrete beam, the latter is directly exposed to fire, and will deflect. The behaviour of the supporting beam in a slim floor structure with steel beams is not directly affected by the fire if the beam is insulated. In case of unprotected steel beams, the soffit is directly exposed to fire and will deflect thermally. Due to this deflection, and dependent on the composite action, an additional transversal shear flow will occur in the hollow core floor. But due to expansion of the soffit, there is also a shear flow in underflange that works in the other direction and compensates the shear flow due to flexible supports. Hence, here also a positive effect is found under fire of flexible supports.

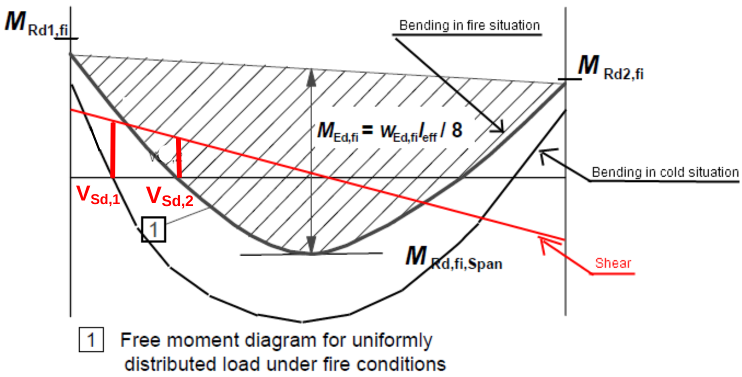


Figure 4.11. EN1992-1-2 Figure E.1 indicating positioning of free bending moment under fire, and additionally showing the reduction of shear force at point of zero moment ($V_{Sd,1} \rightarrow V_{Sd,2}$)

d. Continuous beam

One point is favourable when considering the influence of flexible support when this support is a continuous beam. The bending capacity on the support is less affected by the fire than the capacity at mid span. In consequence, the point of zero moment moves toward the axis of the span as shown in figure 4.11. This Figure clearly shows that the moments at the supports of a beam contribute to significantly reduce the effects of shear due to the flexibility of the beam during a fire. As the influence of flexible support is related to the value of the shear at the point of the zero moment in the Pajari model, this, combined with the reduction of the applied load (see e.), induces a significant decrease of the shear stresses.

e. Live loading

The design value of the live loading is much smaller (frequent load value ψ_1 or ψ_2) at the fire situation than at ambient conditions. Consequently, the transversal shear stresses in the hollow core floor with a flexible support will be smaller at fire than in the ambient situation. The load level in shear defines the proportion of the design shear resistance that may be resisted in ambient conditions, and is defined by V_{Sd} / V_{Rd}. Where the effect on flexible supports is included in normal design at ambient temperature, the design shear resistance of the hollow core units is further reduced to $V_{Rd,flex}$ which also reduces the load level of V_{Sd}. Hence, this reduces also the load level under fire conditions [4.4].

f. Width of the webs

The web width of hollow core slab is directly proportional to the shear capacity of the floor. Slabs with large web widths will get a larger shear capacity, also in case of flexible supports. Hence, the width of the webs has no different influence in case of flexible supports compared to rigid supports.

g. Tensile strength of the concrete

The tensile strength of the concrete in the webs directly depends on the temperature of the concrete. In general the tensile strength will be slightly affected during a fire, despite that the webs are protected by the underflange. The temperature increase can be calculated with the help of 1992-1-2 Table A.2, and thus the reduction in tensile strength can be calculated. Also here, in principle, there will be no significant difference in the tensile strength reduction between hollow core slabs on the rigid and flexible supports.

h. Type of connection with the supporting beam

The shear capacity of a hollow core floor with flexible supports can be enhanced by the type of connection between the floor and the beam: filling of cores in the vicinity of the support and continuity of the floor over the support. This remains valid in the fire situation, and no difference occurs between rigid and flexible supports.

i. Structural topping

A structural reinforced concrete topping will increase the fire resistance of a hollow core floor on rigid supports [4.4]. There is no reason to assume that this would not be the case with flexible supports. The topping increases the section modulus of the floor and limits the temperature rise at the top. But there also seem to be some negative influences, so regarding the structural topping one could conclude “you win some, and you loose some”, but at the end it will probably be the same.

In this section it is concluded that the vertical web cracks and the expansion of the underflange are the factors that could influence the performance at fire of hollow core slabs on flexible supports.

4.7. Shear capacity for hollow core slabs at fire

EN1168 clause 4.3.3.2.2.1 states the general verification procedure for the shear capacity at ambient temperature. Shear failure of hollow core slabs may occur in regions cracked in bending or in regions uncracked by bending. If a flexural crack arises within the anchorage length of the reinforcement, an anchorage failure can also occur. All the three failure modes have to be considered at ambient temperature:

1) Shear resistance in cracked regions shall be calculated using EN1992-1-1:2004 expression (6.2.a) and (6.2.b);
2) Shear resistance in uncracked regions shall be calculated using EN1992-1-1:2004 expression 6.4, taking into account the additional shear stresses due to the transmission of the prestressing force. A procedure to apply this calculation is given in 4.3.3.2.2.2.;
3) Resistance against anchorage failure shall be calculated following EN1992-1-1:2004 9.2.1.4.

The formula in Annex G, enabling the determination of the shear/anchorage resistance of a prestressed hollow-core slab, is an extension of the formula for the shear capacity of prestressed structural members given in EN1992-1-1, Cl. 6.2.2 for normal temperatures [4.18]. The review by Walraven and Vrouwenvelder in [4.18] states that on the one hand this formula is quite practical, since it combines the shear- and the anchorage capacities which are often hard to distinguish in experiments. On the other hand it is inevitably empirical, like the original equation (6.2.a) the real behaviour of the slabs subjected to fire, however, is very complex, with thermal stresses leading to cracks, which may act both in a favourable and in an unfavourable way. This is neglected by the formulas. Restrained effects always seem to be important, but also they are not a part of the formula. It is also interesting to observe that one needs to insert into the model some correction values (characteristic values and η = 0.7) in order to achieve a formula predicting the mean value. On the other hand it has to be estimated that a practical formula for the shear and anchorage bearing resistance is given, valid for fire conditions. This enables a verification of the structure under fire conditions which is more extended than the limited verification for the bending capacity only as used up to now.

The effect of the flexible supports under ambient and fire conditions on the capacity of the hollow core slabs is illustrated in Figure 4.12. In ambient conditions, due to the limited stiffness of the supporting beams, the hollow core slabs will deform not just in the spanning direction of the slab, but also in the transversal direction which is the spanning direction of the beam. The shear capacity at ambient temperature is based on the uncracked situation respectively on shear-tension failure. This means that there is a plate effect, and depending on the stiffness of the supporting beam relative to the transverse stiffness of the hollow core slabs, the load is directly transferred from the hollow core slab to the columns. In the calculation rules, this is not dealt with by increasing the shear load near the columns. No, this is taken into account by introducing τ_2 and by reducing the tensile stress at the support.

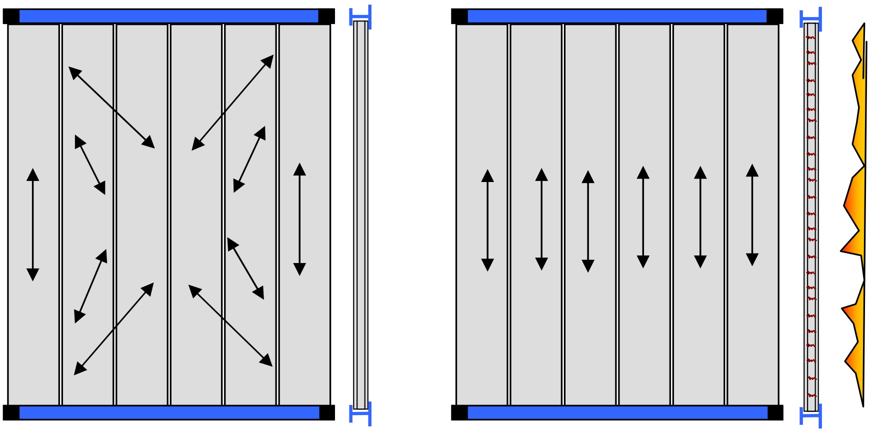


Figure 4.12. Under ambient conditions in stiff floor field the load is distributed towards stiff columns (left), while under fire conditions when webs are cracked the floor is less stiff and load is distributed in longitudinal direction (right)

Under fire conditions the situation is different. Due to the temperature gradient over the hollow core cross section, vertical cracks occur in the webs of the slabs, so now there is a cracked situation. This means there is less "plate behaviour" and the bearing model is more an orthogonal beam model. In Figure 4.12 we can clearly see the difference of load distribution towards the support under ambient conditions (left) and fire (right).

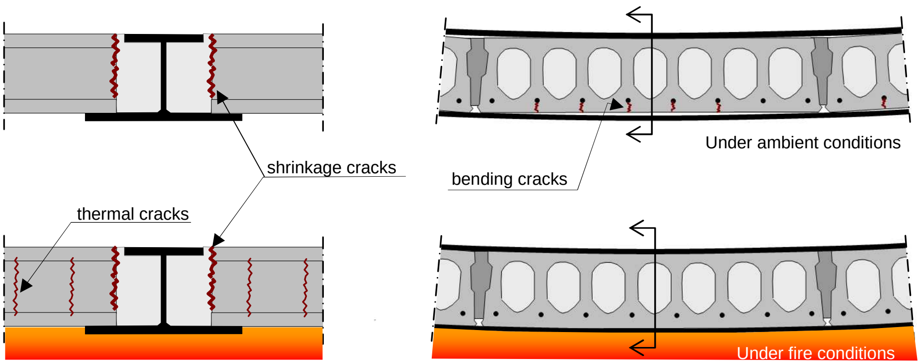


Figure 4.13. Flexible support effect at midspan under ambient conditions (above) and under fire conditions (under)

At mid span (see Figure 4.13) of the slender support beam it introduced additional transverse bending stresses in the underflanges of the hollow core slab [4.13]. If the hollow core slabs are supported so low on the beam that bending of the beam gives rise to transverse tensile stress at the bottom of the slab, the soffit of the slab tends to crack longitudinally parallel to the strands [4.19, 4.20]. These tensile stresses could initiate splitting cracks along the strands, but in a fire situation the underflange expands, and the cracks are closed. As a consequence, the strands will remain well anchored in the support which has positive effect on shear capacity of the slab.

In Figure 4.14 the effect near the stiff column position is explained. The partial composite action between the hollow core slab and the slender beam causes a compression force in the upper parts of the hollow core slab [4.7]. Since the webs in the hollow core slabs serve as connectors between the compression flange and the rest of the section, a transverse shear force will act in the webs in the direction parallel to the beam axis. This causes a shear flow in the hollow core slab transverse to the webs. This shear flow causing transversal stresses can be calculated assuming only contact between the slab and the beam in the supporting area. Thus the vertical joint between the beam and the slab in span direction of the slender beam is considered as completely cracked and frictionless [4.7]. The horizontal shear stresses have to be combined with the vertical shear stress, which will reduce the shear capacity of the slab compared with rigid supported hollow core slabs. Due to the additional shear stresses in the webs, shear tension failure will occur at a lower applied load than in case the hollow core slabs are supported on rigid walls [4.13]. This is not the case under fire conditions. Due to the thermal gradient over the cross section, the underflange expands and a large compression force is introduced in the transversal direction. This compression force withholds the shear flow in the cross section such that additional shear stresses τ_2 are not introduced. Also, the thermal gradient introduces large tensile stresses in the webs in the longitudinal direction such that vertical cracks occur at regular distances. Consequently, shear tension cannot occur anymore in the webs: the shear capacity falls back at the level of flexural shear capacity under fire conditions.

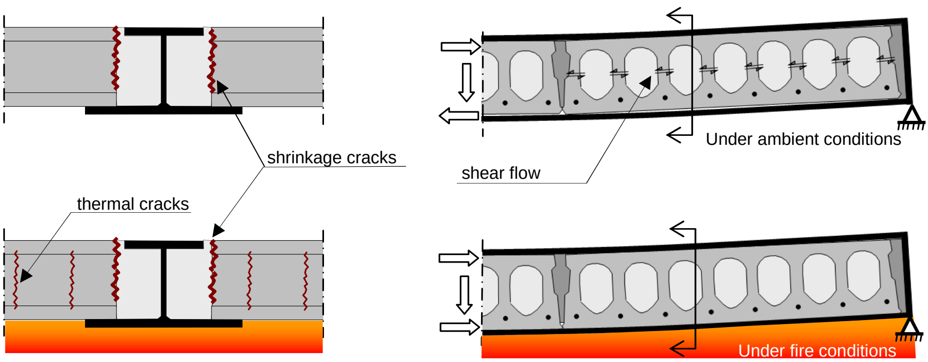


Figure 4.14. Flexible support effect at support under ambient conditions (above) and under fire conditions (under)

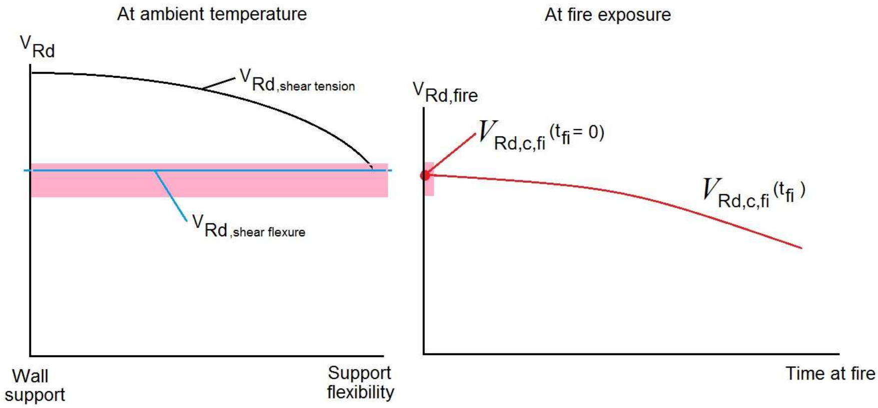


Figure 4.15. Ambient shear resistance (left) and shear resistance under fire (right)

With this clear situation we can now conclude that as there is no shear tension due to horizontal cracking, EN1168 Annex G can be applied also on hollow cores on flexible support at fire. Finally this is explained more in Figure 4.15 depicting the shear resistance, on wall and beam support (left) and when exposed to fire (right). At ambient conditions, depending of the selected detailing of the support (beam or wall), the shear resistance point is somewhere on the shear tension line. But for sure it is higher when compared to shear flexure capacity. (In some countries for flexible supports always the lower limit of shear flexure resistance is used). Further, the Figure indicates on the right-hand side that the shear resistance at fire has a starting point at t_{fi} = 0 that is equal to shear flexure resistance in ambient temperature. During the fire, the shear capacity is reduced as is anticipated in the EN1168 Annex G formula.

4.8. Annex G shear capacity for hollow core slabs on flexible supports

Product standard EN1168:2005+A3:2011 [4.1] Annex G provides a design method to calculate the shear and anchorage fire resistance of hollow core floors for fire conditions. This design method is now also recommended for hollow core slabs on flexible supports under fire. According to this annex, the resistance regarding shear and anchorage failure may be determined by using simplified calculation methods (see [4.4] clause 4.2 and Annex B and Annex D), but taking into account the following assumptions:

- Firstly, it is assumed that below the level on which the total web width is equal to the total core width (level $a_{50\%}$), the temperature in the hollow core at a distance x from the exposed soffit is equal to the temperature at the same position in a corresponding solid slab (see Figure 4.16).
- Secondly, above that level a linear interpolation is taken between the temperature at that level and the temperature at the top of the floor. The maximum allowed temperature for the insulation criterion is 160°C (140°C + 20°C ambient temperature) if no additional information is available;

- Thirdly, for a fire resistance class ≤ R60 this verification is not needed.

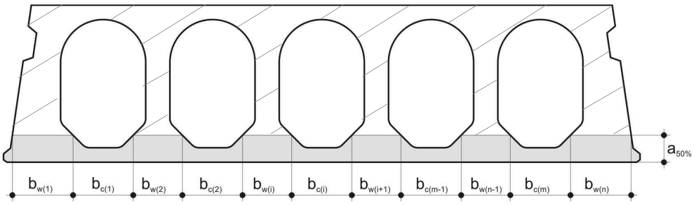


Figure 4.16. Area where solid slab temperatures may be assumed (grey area).

To determine the shear and anchorage resistance under fire conditions for rigid supports and flexible supports, the formula (1) from Annex G is used, see also Figure 4.17.

$$V_{Rd,c,fi} = \left[C_{\theta.1} + \alpha_k \cdot C_{\theta.2}\right] \cdot b_w \cdot d \tag{1}$$

$C_{\theta.1}$ coefficient accounting for concrete stress under fire conditions:

$$= 0.15 \cdot \min(k_p(\theta_p)\sigma_{cp,20°C}; \frac{F_{R,a,fi,p}}{A_c}) \tag{2}$$

$$\alpha_k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 \quad \text{where d is in mm ,} \tag{3}$$

$C_{\theta.2}$ coefficient accounting for anchored longitudinal reinforcement:

$$= \sqrt[3]{0.58 \cdot \frac{F_{R,a,fi}}{f_{yk} \cdot b_w \cdot d} \cdot f_{c.fi.m}} \tag{4}$$

b_w total web thickness of the hollow core slab

d effective depth at ambient temperature

x the anchorage length of the strand for the considered section (see Figure 4.17)

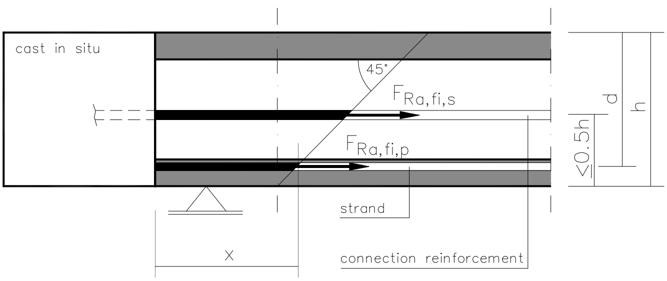


Figure 4.17. Model for calculating shear and anchorage resistance (example without protruding strands)

In the hereunder given Table three fire tests are given that were recalculated in the Database study [4.18]. The fire tests are from the EMPA study and the observed failure type was shear failure. It emerges when EN1168 Annex G is used that the calculated capacity is in all tests smaller than the real shear load. Hence, this implies that EN1168 Annex G is safe to use. However, we could be doubtful about the flexibility of the tests. The deflection of the beam was between 1.8 mm and 7.5 mm on a 2.4 m span, so L/330, L/320 and L/1200, respectively, for H83, H85 and H86. This is mainly because of the short span to be able to conduct a fire test in a small furnace.

		Fire test		EN1168 Annex G		
TEST ID		Shear load [kN/m^1]	Time to failure [minutes]	Shear capacity [kN/m^1]	Time to failure [minutes]	Test/ Annex G [%]
H83	EMPA B2-2 [1995]	35.1	49	32,9	36	106.7%
H85	EMPA B2-4 PL [1995]	35.8	75	29.6	37	121.4%
H86	EMPA B3-1 [1995]	28.6	97	15.1	30	189.4%

4.9. Calculation example

Finally, a calculation example is added in this Chapter in order to :

- Hollow cores with depth 315 mm with 5 cores cast in C45/55 and no structural topping;
- 12 strands 12,5 mm at 46 mm and 87 mm axis distance and 4 upper strands at 277 mm (X8X4-D4);
- The theoretical span of the slabs is 12180 mm (80 mm support length) and cores are filled for 50 mm;
- Thickness of upperflange 40 mm and underflange 40 mm, total web width 316 mm;
- Support: beam THQ 320 with 7200 mm support length, or rigid wall;
- Connection reinforcement 2∅16 per slab in 2 cores;
- Loads: self weight = 4.15 kN/m^2, topping = 0 kN/m^2, dead load = 1.0 kN/m^2, live load = 5.0 kN/m^2.

Other data used is: $\gamma_g = 1.2$, $\gamma_q = 1.5$, $\gamma_c = 1.5$, $b_{eff} = 144$ mm, good bond ($\eta = 1.0$).

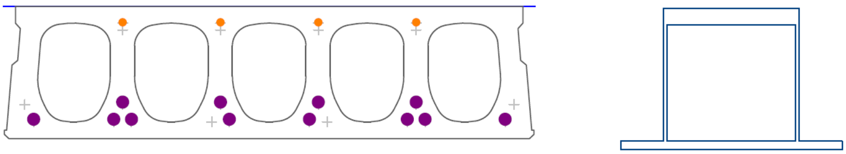


Figure 4.18 Cross sections of hollow core 315 mm and THQ320 beam

[*Shear per slab width*]	Shear capacity **rigid support**	Shear capacity **flexible support**
Shear load **ambient conditions** 100.1 kN	Uncracked situation = shear tension $V_{Rd,c}$ = 185.0 kN (EN1168) Cracked situation = shear flexure $V_{Rd,c}$ = 147.5 kN (EN1992) $V_{Rd,c,min}$ = 124.2 kN (EN1992)	Uncracked situation = shear tension $V_{Rd,c}$ = 170.9 kN (fib 6) Cracked situation = shear flexure $V_{Rd,c}$ = 147.5 kN (EN1992) $V_{Rd,c,min}$ = 124.2 kN (EN1992)
Shear load **fire conditions** 59.9 kN	Cracked situation (thermal vertical web cracks) = shear flexure 30 minutes: $V_{Rd,c,fi,30}$ = 106.8 kN (EN1168 Annex G) 120 minutes: $V_{Rd,c,fi,120}$ = 92.9 kN/slab (EN1168 Annex G) *In capacity under fire 2∅16 connection reinforcement is taken into account*	

In the results presented above both the shear loads and the shear capacities are given. At ambient it is given which formulas were used to determine the shear capacity. From the above given results can be concluded that the capacity at fire of a hollow core slab on flexible supports is always lower than the minimum shear flexure capacity in a cracked situation at ambient temperature. The two load combinations for ULS and accidents are:

- Fundamental combination: q_d = 1.2 G+1.5Q = 1.2 (5.15)+1.5 (5.0) = 13.7 kN/m^2
- Accidental combination: q_d = 1.0 G+1.0ψ_2Q = 1.0 (5.15)+1.0*0.6 (5.0) = 8.2 kN/m^2

Live load (recommended values from EN1990:2002E / EN1991-1-1:2002E, Category D) = 5.0 kN/m^2, ψ_2 = 0.6). Then we calculate that V_{sd} = 1.2*0.5q_dL is 100.1 kN/slab for ambient conditions, and 59.9 kN/slab for fire. (Remark: prestress P is not considered as favourable load, but as capacity).

4.10. Design and execution recommendations

It is assumed that the design at ambient conditions of flexible supporting beams and hollow core floor components is done in a correct way to cope with the additional shear flow in the webs. In addition, for fire for longer resistance periods, detailing measures [4.4] are increasingly important in order that the shear resistance of the hollow core slabs do not decrease more rapidly than the bending resistance. The fire resistance of structures with hollow core slabs is improved by:

- The use of tying reinforcement, in the form of suspension or other tie reinforcement, to provide alternative load paths;
- Infilling of the hollow cores to strengthen the slab locally, and to permit placement of tie reinforcement. Figure 4.19 shows a possible solution with filled cores;

- A reinforced concrete topping to control the effect of cracking and to provide additional tying action for integrity reasons;
- The effect of protection of the beam support to the hollow core slabs.

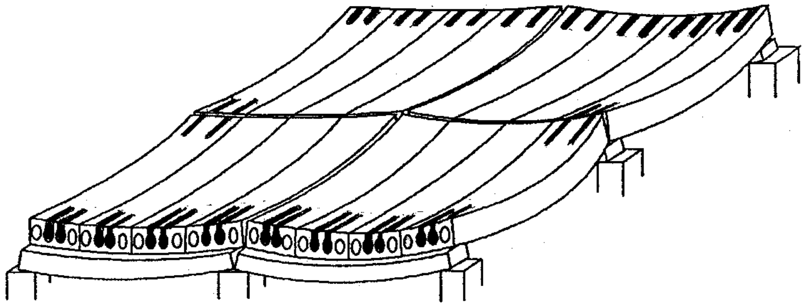


Fig. 4.19 Recommended solution for hollow core floors with flexible supports, exposed to fire

The safety of building structures with respect to fire is achieved by specifying some safe value at the loading side (duration of the fire) in combination with the recognition that fire in itself has a low probability of occurrence [4.18]. The global performance of building structures in real fires is better than that of the structural elements considered in isolation [4.4]. This is because:

- In-plane compressive forces are generated due to restraint to thermal expansion, which may increase the effective shear resistance of the floor plate;
- Real fires are often localized, and the surrounding structure may offer restraint to the localised part of the slab affected by the fire;
- At large deformations tensile membrane action occurs which provide an alternative method of load transfer in fire.

4.11. Conclusions

It is generally known that the shear resistance of the hollow core slabs is reduced under ambient conditions if the slabs are on flexible supports. Under ambient conditions it is evident that the deformation of the beam initiates composite action that alters the mode of behaviour of the structure and introduces additional bending and additional shear stresses in the transversal direction of the slab. Both phenomena lead to a reduced shear capacity under ambient conditions for flexible supports. But most countries do not take into account the flexible support effect, although for most practical applications sufficient shear resistance remains. In some countries however the design recommendation set out in fib Bulletin 6 are used in construction to design hollow core with flexible supports.

Under fire conditions the decrease of shear capacity on flexible supports is not magnified by the fire. On the contrary, as the thermal gradient over the cross sections due to the fire compensates the negative effects of flexible supports. On the one hand, the underflange of the hollow core slabs expands and is under compression such that the additional bending stresses are compensated. On the other hand, vertical thermal cracks occur in the webs of the hollow core slab such that a shear tension failure cannot occur anymore. As a result, the shear resistance “falls back” to the level of flexural shear resistance. Accordingly, it is concluded in this Chapter that for hollow core slabs on rigid supports and flexible supports, EN1168 Annex G can be used for determining the shear resistance under fire conditions.

Nowadays, there is still no design procedure for flexible supports specified in the European standard EN1168:2005 +A3:2011 for hollow core slabs. EN1168 states only that “in case of flexible supports, the reducing effect of transversal shear stresses on the shear capacity shall be taken into account.” It is recommended to include in the next revision of EN1168:

- A calculation method to determine the reduced shear resistance at ambient conditions of hollow core slabs supported on flexible supports due to the reduction of shear tension capacity as a result of the additional shear flow in the webs;
- The conclusion of this Chapter that for flexible supports under fire conditions EN1168 Annex G can be used to determine the shear and anchorage capacity.
- As advised in the research for G series, EN1168 Annex G does not explicitly state what value to describe the bond conditions should be used: η_1 = 1.0 (good bond) or η_1 = 0.7 (bad bond). In the Holcofire research in G-series calculations η_1 = 0.7 was used, in the above given example η_1 = 1.0. It is recommended to explicitly state in EN1168 Annex G what (implicit) value to use for calculations, also in consideration of rigid supports and flexible supports.

References Chapter four

[4.1] Pajari M., Yang L. Shear capacity of hollow core slabs on flexible supports. VTT Research Notes 1587, Espoo,1994, p. 111 (+26).

[4.2] Pajari M. Shear resistance of prestressed hollow core slabs on flexible supports. VTT Publications 228, PhD thesis Helsinki University of Technology, Espoo, April 1995, ISBN 951-38-4766-7, p. 132 (+28)

[4.3] Leskelä M., Pajari M.: Reduction of the vertical shear resistance on hollow-core slabs when supported on beams. Proceedings of Concrete `9 Conference, vol.1, Brisbane, 1995, p. 559-568.

[4.4] ECCS/IPHA, Guideliness for the application of prestressed hollow core slabs supported on build-in beams, No 103, April 1998, ISBN 92-9147-000-39, pp. 18.

[4.5] Van Alphen E., Bennenk H.W. Het LEVO-systeem en de draagcapaciteit van kanaalplaatvloeren in geïntegreerde constructies. MSc thesis Eindhoven University of Technology, Juni 1999. pp.

[4.6] Code Card 18: Design of hollow core slabs supported on beams, 2007.

[4.7] fib Bulletin 6, Special design considerations for precast prestressed hollow core floors. Lousanne, January 2000, ISBN 2-88394-046-0, pp. 89-116.

[4.8] CEN, EN1992-1-1 EC2: "Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings", December 2004.

[4.9] CEN, EN1992-1-2: "Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design",

[4.10] EN 1168:2005+A3:2011 Precast concrete products ‐ Hollow core slabs

[4.11] CEN, EN1168:2005+A3:2011 "Precast concrete products – Hollow core slabs" , October 2011.

[4.12] Bogogno, W., Fontana, M. "Versuche zum Tragverhalten von Betonhohlplatten met flexibler Auflagerung bei Raumtemperatur und Normbrandbedingungen". Institut fur Baustatik und Konstruktion, ETH Zurich, p. 246, 1996.

[4.13] Fellinger, J. "Shear and anchorage behaviour of fire exposed hollow core slabs", PhD thesis Delft University of Technology, ISBN 90-407-2482-2, p. 261, 2004.

[4.14] C.G. Bailey, Lennon "Full-scale fire tests on hollowcore floors". The Structural Engineer, 18, pp. 33-39, March 2008.

[4.15] Peikko group, Fire resistance of hollow core slabs supported on non-fire protected Deltabeams, Report on fire testsP802216A, P802216B, P802216C, P802216D, January 2010, pp. 18.

[4.16] Hegger, J., Roggendorf, T., Kerkeni, N. Shear capacity of prestressed hollow core slabs in slim floor constructions. Eng. Struct., vol. 31, 2009, p. 551 ‐ 559.

[4.17] Roggendorf, T. Zum Tragverhalten von Spannbeton-Fertigdecken bei biegweicher Lagerung. Dissertation, Institutsberich 250/2010, RWTH Aachen, June 2010, pp. 210 (+194)

[4.18] Jansze, W, Arnold Van Acker, Bruno Della Bella, Ronald Klein-Holte, Gösta Lindström, Andreas Nitsch, Jean-Paul Py, Matthieu Scalliet. "Fire resistance of hollow core floors - Holcofire database – Meta-analysis on 162 fire test results executed between 1966 -2010, Chapter 2, this book, 24 January 2014.

[4.19] Surma, M. Prestressed hollow core slabs on flexible supports – design model and computational examples, Juniorstav 2013, p. 6.

[4.20] W. Derkowski,W., Surma, M. Design analysis of prestressed hollow core slabs on flexible supports, Central European Congress on Concrete Engineering, CCC2013, September 4-6 2013, Wroclaw, Poland, p. 8

5

Chapter Five

Fire Case Rotterdam

Fire case parking Lloydstraat, Rotterdam
Retrospective view, new insights and outlook

Keywords*: real fire, facts, open parking garage, hollow core slab, floor structure, horizontal cracks, restraints, explosive spalling, delamination*

Abstract*. On the 1st October 2007, in the early morning a fire broke out in the 2,100 m^2 parking garage under the Harbour Edge apartment building in Lloydstraat, Rotterdam, The Netherlands. No people were injured. The hollow core floor of the parking garage did not collapse and did comply with the integrity and insulation criteria. However, significant local explosive spalling was visible on the precast façade and ceiling near the fire, while about 70 m2 of the underflanges of hollow core slabs had fallen down by delamination. In the six years after the fire many investigations were conducted in The Netherlands, leading to the publication by the Dutch precast Industry of intermediate measures in 2009 and new measures in 2011. This Chapter addresses in retrospective view this fire case and related research as well as shed a light on the playing field in which this happened. Mainly in order to inform the international reader about the facts on the Rotterdam fire, as it is felt by Holcofire that these facts are not commonly known and thus not fully understood in Europe. From new insights it emerges from FDS5 simulations that the fire was more severe than 2 hours of standard ISO 834 fire, and was dramatically different than calculated by Efectis/TNO. The FDS5 simulations on the Rotterdam car park fire conclude that at 20*

minutes into the Rotterdam fire (at 04:23h) when the maximum temperature above car 1 was reached, 33% higher temperature (900°C compared to 678°C) and 3 times higher temperature increase rate (44.7°C/min compared to 15.6°C/min) were calculated compared to the standard temperature–time curve obtained according to ISO834 of EN 1991-1-2:2002. On the basis of the regulations and guidelines a fire with this size was not to be expected. This Chapter further presents in successive steps the delamination initiated by spalling and horizontal cracking due to (internal and external) restraints. It is illustrated that the underflanges felt down only when the anchorage failed or strands were ruptured, but the floor did not collapse by virtue of redundancy in the floor structure. In the outlook Holcofire addresses the good experiences with past performance of hollow core slabs under fire, and expresses their concerns on the international impact of the Rotterdam fire. Also, it expresses that the measures taken after the Rotterdam fire are disproportionate compared to the local damage and size of the problem.

***Review**. The valuable contribution of the CaPaFi and FDS5 simulations in background report by Dr. Andreea Muntean of Consolis Technology is highly appreciated. This background report [5.31] entitled "Further analyses of Efectis fire scenario 1 with softwares CaPaFi 2.0 and FDS5" was reviewed by Dr. G. Rein of Imperial College London. The integral review text is published in Appendix 5.C of this chapter.*

Figure 5.1. Building Harbour Edge in the Lloydstraat after fire (left). Level 3: intact floor above the fire where cars were removed the day after (right, top). Level 2: locally damaged ceiling above burned car on the level where the fire took place (right, bottom) [5.4]

5.1. Introduction

On the 1st October 2007, in the early morning a fire broke out in the parking garage under the Harbour Edge apartment building in Lloydstraat, Rotterdam, The Netherlands (see Appendix 5.A for a short structural description of the building). Six cars were burned and both the surface of the precast concrete facade and the soffit of the hollow core concrete floor were heavily but locally damaged (see Figure 5.1 left-hand side). During 5 years many investigations have been conducted on this so called “Rotterdam fire” case with particular focus on the hollow core floor structure. As a result of these investigations, additional measures for hollow core floors were published in 2009 and 2011 in The Netherlands. These new measures basically prevent a floor design such as designed in the Harbour Edge building. Nevertheless, after 5 years, the Dutch “Rotterdam fire” is still affecting the good image of the hollow core slab, also internationally. The reason for this is twofold. Firstly, the facts and details about the fire case are actually not well known to many people. As an example, most people think that the floor collapsed after the fire, based on lack of information, or even misleading paper context such as “structural integrity of the floor and the entire building was jeopardized” [5.6]. On the contrary, the floor did not collapse (see Figure 5.1 right-hand side). There were even four cars parked on the floor above that were removed the next day! Secondly, because the damage to the hollow core floor has never been explained in a satisfying manner and a clear understanding how to deal with such local failures is still lacking.

Holcofire decided to write this Chapter on the Rotterdam fire case. It summarizes clear facts on the Rotterdam fire case in order to inform the international reader. And it looks back on the research activities that were conducted and decisions that were taken in order to give the international reader an understanding about the progress in this area. It also addresses how it was handled by the legislative and advising bodies in the Dutch administration. In addition, the Chapter gives Holcofire’s viewpoint on the local damage that occurred during the fire by sketching the delamination process in successive steps.

\- - - - - - - - - - - - - - - - - - PART I – RETROSPECTIVE VIEW - - - - - - - - - - - - - - - - - - -

5.2. Fires prior to Rotterdam that affected image and administration

In the years advancing the Rotterdam fire, hollow core slabs were already under discussion due to premature shear failures in laboratory fire tests on hollow cores. The discussions started in France and Switzerland with the study on slim structures in which a few cases of premature failure in standard fire tests were reported by CTICM in 1995 [5.14], Borgogno in 1996 [5.15] and Andersen in 1999 [5.16]. As a consequence, it led to international discussions, although in practical applications shear hardly governs floor design [5.17]. As well as there are, even after a thorough study, no known cases in practise where the shear mechanism occurred. The question was raised if this premature failure constitutes a real structural problem for this type of floor, or whether the reason lies in a lack of understanding of the behaviour of hollow core floors during fire, resulting in poor design, particularly for shear and for small-scale laboratory test set-ups.

It was concluded by industry and academic world that more knowledge was needed on the shear capacity of hollow core slabs under fire conditions. Accordingly, in order to systematically study shear failure under fire, many laboratory fire tests were conducted in Belgium by Van Acker in 2003 [5.18], The Netherlands by Fellinger in 2004 [5.17], Denmark by Jensen in 2005[5.19]. And even in UK by Bailey in 2008 [5.20] two fire tests were conducted on real building structures exposed to a natural fire. These fires have been reported on, however, their publications lacked a good guideline to design for shear and anchorage. Only recently, in 2011, the European Standardisation Institute CEN published rules in Annex of EN1168:A3 [5.21] that provided a formula to design for shear and anchorage for single span hollow core slabs without shear reinforcement exposed to fire. Despite, the laboratory fire tests on hollow cores and the related discussions on the market and in the academic world affected the good image of the hollow core slab among clients in some European countries.

The Rotterdam fire shows a similar history; after a local damage the (semi-)academic world initiates research in the area of hollow cores in order to clarify problems, but at the same time enlarges those. But in order to also understand the circumstances in which the related discussions were held with the Dutch authorities following the fire of Rotterdam, it is relevant to sketch the administrative playing field in The Netherlands regarding responsibilities towards fire losses. For that, three real fire cases [5.14] are relevant, all without the use of hollow cores. But these fire cases twisted the fire discussions into a political and administrative discussion that heavily affected the sentiment in the administration and thus the measures taken after the Rotterdam fire.

The change of the general administrative mood started actually in 2001 with the “Volendam-New-Years-fire” on New Years Day 2001. The fire was in a (wood-constructed) café in the Dutch town of Volendam and caused the death of 14 young people. There were in all 241 people admitted to hospital, 200 of which suffered serious burns. The fire was questioned intensively by media and politicians. New administrative regulations were introduced for decorations in cafes, nightclubs and other venues. The owner and managers of the building were indicted for culpability. As a result of the inquiry, the Mayor of Edam-Volendam resigned his position.

Then, in 2004 in the official Prime Ministers historic residence the “Catshuis-fire” caused the death of a painter carrying out renovation work while the fire destroyed much of the ground floor. Initially, the fire was blamed on the use of illegal thinners used by the painter, but later the public prosecutor’s office did find that the civil servant had failed to obtain the necessary permit for the work in the building, which would have stipulated that materials used were fire resistant. Hence, administration was blamed again.

Finally, the biggest impact on administration came from the “Schiphol-fire” that took place on 27 October 2005. A fire erupted in the K-wing of the steel-constructed immigration detention complex at Schiphol airport, resulting in the death of 11 detainees and 15 injured from foreign countries. From the start, doubts were shed on the organisation of the involved government agencies. On 21 September 2006, the Counsel for Public Safety presented the final report on the problem in the Schiphol prison. The report explicitly stated that “fewer or even no casualties” would have occurred if the government had upheld the legal safety standards. Consequently, the Judicial Authority and Building Authority (Rijks-gebouwendienst) were found co-responsible for the fire. Hence, the Justice Minister and the Environment and Construction Minister resigned immediately, as well as the Mayor of the municipality Haarlemmermeer who issued permits and is responsible for the fire service at Schiphol airport.

These above described three real fires cases in a period of 7 years advancing the Rotterdam fire lifted the fire discussion up to a political and administrative level. Although no people were found death or were injured during the Rotterdam fire, the administrative and political reaction in The Netherlands on this fire was already set: governmental authorities were looking for governmental security to be backed up by building permits and building regulations. This clearly affected the application of hollow core slabs that were already under discussion for the last 15 years.

5.3. The fire at Lloydstraat, Rotterdam, on 1st October 2007

On the 1st October 2007 in the just newly-constructed apartment building Harbour Edge in the Lloydstraat Rotterdam, a fire in level 2 of the parking garage was reported at 04:16 h by the occupants. The presence of an alarm installation would have contributed to a faster arrival of the fire brigade. Figure 5.2 visually supports the following facts taken from [5.1] and [5.3]. At the arrival of the fire brigade at 04:22 h the door from the elevator shaft to the garage was already broken. At first, the fire brigade took care of the safety of the people that were still in the building. The fire brigade and occupants reported in the garage bangs like in fire works. In the following 15 minutes 60 people were evacuated without any injuries. At 04:25 h the fire was reported as a big fire. At 4:28 h from 3 window openings flames of about 2 to 2.5 m without smoke were going outside the building. At 4:29 h smoke also developed that became dark black. From that time the evacuation was hindered by the smoke coming from the broken door from the garage. At 04:32 h the fire brigade tried to enter via the garage door, but due to the black smoke it was too dark. The fire brigade aimed with water for the red glow which was visible through the dark smoke. Then, at 4:46 h the fire brigade had to withdraw because of the loud bangs which were heard inside and outside the building. This withdrawal resulted in the use of a fire boat at 04:48 h. The fireboat splashed with all 3 guns 35,000 litres water per minute on the first, second and third window on the East side of the building. At the start of the deployment the fireboat spouted right through the building resulting in fire fighters becoming wet at the front of the building. Then, the direction of the radius of the fire boat was adjusted; it is standard to splash against the soffit of the ceiling to cool the structure and the fire. At 05.01 h the fire was under control. The fire brigade evaluated that no people were killed during the fire, and no people were injured.

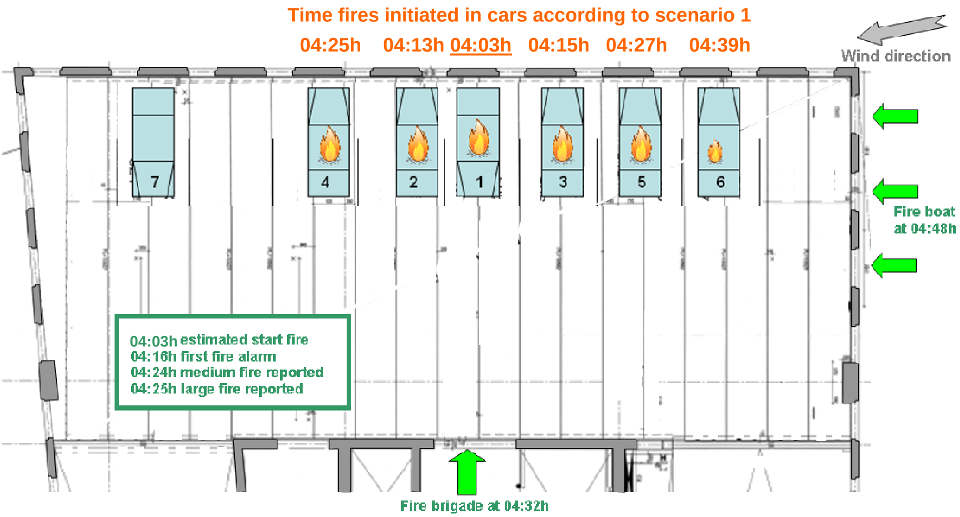


Figure 5.2. Plan overview of level 2 of parking garage with location of cars on floor and fire sequence

Figure 5.3-left. Extensive spalling on external facade surface
Figure 5.3-right. Hollow core floor ceiling with local damage, visible are cars 2, 1 and 3

Figure 5.4-left. Underflanges of hollow cores felt down, and strands were exposed,
Figure 5.4-right. Support of the hollow core slab was intact

5.4. Observation after the fire was extinguished

It was observed that during the fire in level 2 of the parking garage, a total of 5 cars were completely burned, while a sixth car was burned for 75% and a seventh car had some scorching and melting damage. At first, it is important to conclude that the floor structure of level 3 had not collapsed. It was even orally reported that in the morning after the fire the 4 cars that were parked on level 3 were removed. At 06:46 h the first photos were taken in the burned parking garage level. Around 08:55 h more photos were taken and reported in [5.1]. It appeared that at the ceiling of level 2 five and a half prestressed hollow core slabs had cracked horizontally through the webs, separating the slabs in an upper and lower half (comprising the prestressed strands) [5.6]. When the photos of 06:46 h are compared with the photos of 08.55 h, it appeared that the underflanges of three and a half slabs had almost completely separated as a consequence of these horizontal cracks before 06:46 h, while from two slabs underflanges separated in the hours after the fire. Further, on the ceiling just above the cars the hollow core slabs showed extensive spalling. Also the inside surface of the external façade showed extensive spalling (see Figure 5.3).

It was observed that of the total parking area of 2,100 m^2 for 60 cars (designed as one fire compartment), 110 m^2 of the floor and about 25 m^2 of the facade was affected by the heavy car fires. Report [5.3] mentions that the most notable damage was the excessive cracking in the hollow core slabs. This includes both the horizontal crack formation from core to core as well as the vertical crack formation of the cores to the soffit of the slab. Of about 70 m^2 of the 110 m^2 (5 ½ slabs) the lower parts of the floor has fallen down due to horizontal cracking (see Figure 5.4). On about 40 m^2 explosive spalling was clearly visible (see Figure 5.3). Also, prestressing strands were detached from the structure and had fallen down. Heavy explosive spalling had also occurred on the façade near the fire (see Figure 5.3). The mild steel reinforcement of the façade has been exposed over large areas and even spalling occurred behind the mild reinforcement. At a larger distance from the fire, explosive spalling on the hollow core slab had occurred to a spalling depth of several centimetres, in many cases up to the cores (see Figure 5.3). Much further away from the fire the hollow core slabs showed some superficial surface spalling. The hollow core slabs are supported at the facades on steel L-shaped angles that are connected to the outer wall. The steel angle was intact after the fire [5.1], and the fire protection was only partially present after cooling, see Figure 5.4.

5.5. Initial studies in 2007 and 2008

After the fire of Rotterdam, some short articles in the newspapers about the fire were published. The first report on the fire was commissioned by Veiligheidsregio Rotterdam and published by Efectis Nederland in December 2007 [5.1]. This report shortly described the parking garage and structure, and gave an extensive analysis of the fire development that took place in the parking garage. It concluded that the initiation of the fire most probably was 04:03 h, and 13 minutes later at 04:16 h it was observed and reported. The fire load was due to the sequential burning of the cars. Based on fire development models for cars, two fire development scenarios have been assumed and calculated for Rotterdam. Most probably fire scenario 1 took place, in which the total maximum fire load was 22.5 MW (Figure 5.6). It seems that the temperature in Rotterdam at the start of the fire was higher than the normalized ISO 834 curve. The duration of the fire corresponding to the fire load on the structure was shorter than the duration of the fire load according to ISO834, but on the basis of the regulations and guidelines a fire with this size was not to be expected.

The Efectis report [5.1] also states that according to the actual regulations in 2010 for the calculation of the fire resistance of concrete structures, the floor is calculated only for the bending moment. It is then assumed that the cross section heats up (including prestressing steel) and the material strength decreases. In the calculation of the bending moment capacity the concrete cover on the pre-stressing strands is assessed, because this cover forms the insulating layer between the strands and the fire. The greater the concrete cover, the less heating of the prestressed strands, and the larger the calculated bending resistance to fire. Such an approach is in principle valid when the cover remains intact (no spalling) and the bending moment is the governing failure mechanism. The concrete cover present in Rotterdam was 40 mm, and a total of 10 strands ∅12.5 mm per slab were used at an axis distance of 46 mm in the slabs consisting of 5 cores.

TNO Bouw and Ondergrond published the second report on the fire in January 2008 [5.2]. This report contained a 2-dimensional finite element (FEM) analysis conducted with DIANA with a main focus on the transversal direction. Two variants were examined (Figure 5.5): a hollow core slab (without a structural topping) that can deform freely in transversal direction, and a hollow core slab with structural topping by which the horizontal deformations in transversal direction are fully hindered. The first variant matches a situation on a standard fire test on hollow core slabs, while the second variant is a simulation of an extreme situation in which the slabs are horizontally fully restrained in transversal direction and contain a structural topping. TNO Bouw en Ondergrond concludes in [5.2] that the FEM calculations clearly explain the premature failure; in variant 1 vertical cracks occur, while in variant 2 within 30 minutes horizontal cracks occur in the webs resulting in separation of the underflange from the upper floor structure. But note that up till now FEM calculation has shown that it remains difficult to successfully simulate buildings or building parts, in which edge effects or influencing parameters are correctly parameterized.

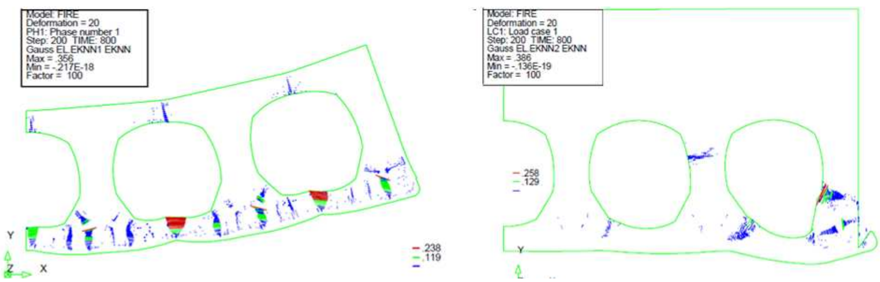


Figure 5.5. FEM simulation of hollow core with topping; unrestrained (variant 1, left) and fully restrained with thick topping (variant 2, right) [5.2]

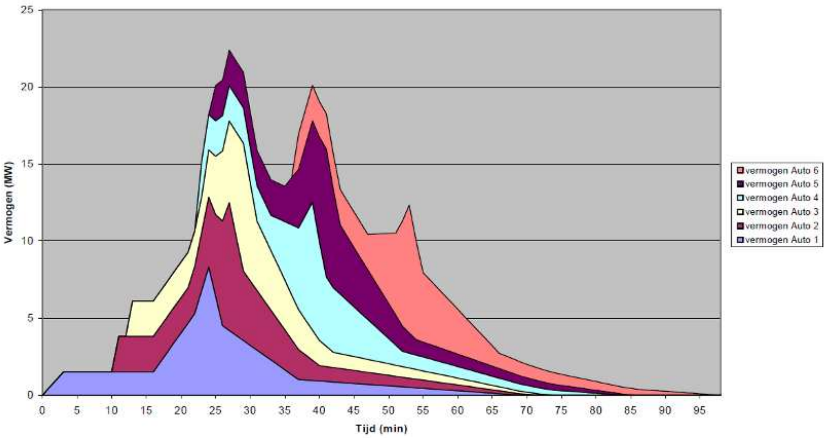


Figure 5.6. Total calculated fire power range in fire scenario 1 with 6 cars subsequently on fire [5.1]

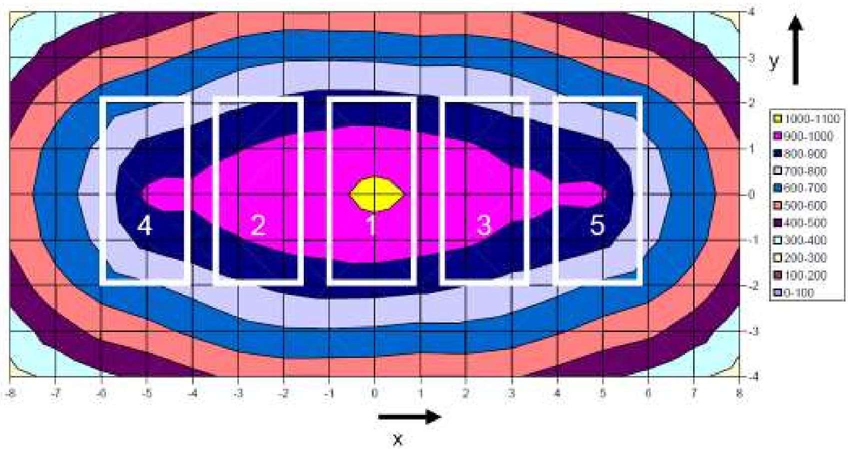


Figure 5.7. Maximum occurring temperatures as a function of location based on fire scenario 1, cars are visualized [5.3]

The publication of the two investigation reports [5.1] and [5.2], and some news articles did not lead to spectacular discussions. This completely changed in March 2008, when the Dutch precast flooring association BFBN received a letter from the Minister of Environment and Construction. This letter notified them that upon various signals regarding the fire resistance of precast concrete flooring products the Ministry would conduct an orienting inspection. These signals were mainly based on fire tests in Italy on filligran slabs with polystyrene blocks. The result of the test showed that it is possible to use polystyrene for void formers as long as the precast concrete plates are perforated or provided with vents to prevent fire-induced pressure build-ups by the trapped gas in the volume originally occupied by the polystyrene. However, one floor in the fire test exploded after 20 minutes, and a Dutch competitor of another precast system sent photographs of the exploding floor to the Ministry. As a consequence, the Ministry urged for cooperation with the Dutch industry and BFBN to investigate the fire safety of filligran slabs with polystyrene voids.

In the same cooperation, the Minister of Environment and Construction talked with the Dutch Industry about the fire resistance of hollow core slabs due to bad performance in the Rotterdam fire. This interest was mainly because the Building Authority under the Ministry of Environment and Construction was developing a new Palace of Justice in Amsterdam named “IJdock.” This Palace of Justice also contained prisons like Schiphol, and the Minister wanted to have proof that buildings with hollow core slabs are safe with respect to fire. In the discussions that followed with the Dutch industry not any answers to the questions of the Ministry was good enough. Then the Ministry conducted their own fire test in order to investigate the behaviour of hollow cores under fire; the outcome of the fire test investigation was not published, nor was it communicated to the industry. Nevertheless, it is a fact that hollow core slabs were applied substantially in the newly build “IJdock” Palace of Justice.

5.6. BFBN study in 2008 and 2009

In order to further clarify whether the horizontal cracking observed in Rotterdam was an isolated incident or whether the applicable regulations were lacking in this respect, BFBN commissioned in 2008 a study by a consortium of TNO, Efectis NL and Expertise Centre for Building Regulations, and reported Part A of the study in July 2009 [5.3].

With regard to the temperature development during the fire the conclusion in [5.3] is that this heat development with respect to time and space has not been exceptional. The report states that at 30 minutes the maximum temperatures were reached, and that the total fire took about 45 minutes (from 04:03 h to 04:48 h) after which the (heavily forced) cooling down phase started as a result of the way of extinguishing. The first 30 minutes the real fire was not significantly different than the ISO 834 fire. Figure 5.7 shows that based on fire scenario 1 it is seen that the maximum temperatures acting directly above the burning cars ranges roughly between 800 ºC and 1020 ºC. Most probably local temperature differences due to uneven heating have influenced the stresses in the hollow core slabs. The Figure also shows that the temperature at some distance from the fire quickly decreased, and that at a few metres distance the temperature on the ceiling already was several hundred degrees lower.

With regard to the development of the damage the report [5.3] concludes that horizontal cracking of the webs of the hollow core slabs were initiated during heating, so between 30 and 45 minutes. However, this is not confirmed by observations from the fire brigade; it is not clear at all from the talks with the fire brigade whether underflanges of the hollow core slabs felt down during the fire. Nevertheless, with certainty it is stated that directly after the (heavily forced) extinguishing with the fire boat some under flanges of several slabs had fallen down. It is also certain that for slabs #7 and #8, despite that the webs were horizontally cracked over a large part of the slab, the under flanges had fallen down a long time after the fire was extinguished (see photos C1 and C2 of [5.3], and also Figure 5.16).

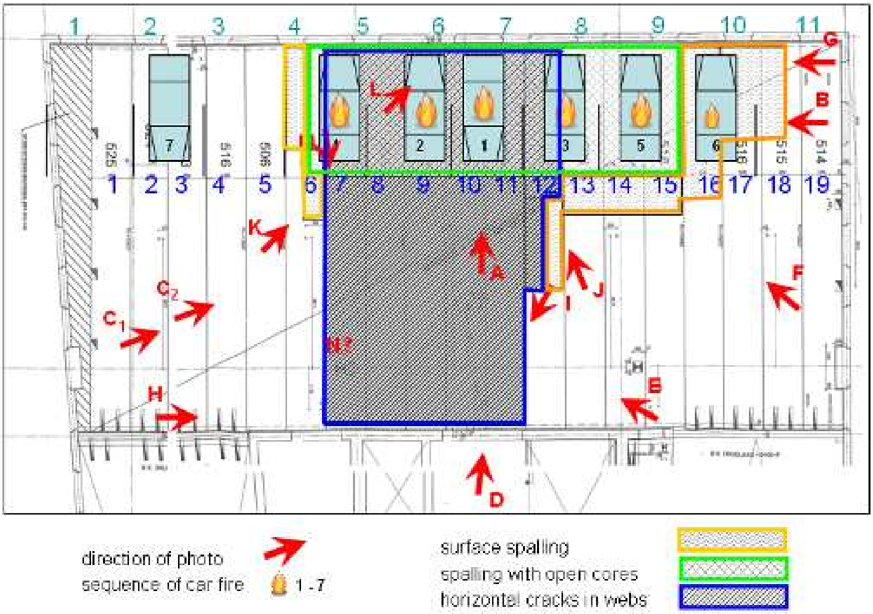


Figure 5.8. Plan overview of level 2 of parking garage with location of cars on floor and damage on the ceiling

Figure 5.8 shows the plan of the parking level with the position of the burned cars and it gives by arrows the viewpoint of photos taken after the fire. Further, it has numbered the hollow core slabs in the ceiling, and it has numbered the openings in the external façade in order to have a better understanding of the photos published in [5.3].

On the basis of these photos [5.3] it could be derived with the soot deposed in the cores and on the crack surfaces whether the horizontal cracks were already present at the time of smoke development, so before the fire was extinguished. Indeed, black cracked surfaces indicate that the cracks were already present during the fire, whereas not black cracked surfaces indicate that they appeared after extinguishment of the fire. Sooty crack surfaces are more or less present in the cores of all hollow core slabs that were horizontally cracked. At the slabs #8, #9 and #11 the soot deposits in the cores are mainly in the zone directly above the fire, while in the slabs #7, #10 and #12 this is almost over the full length in the cores. In addition, in the slabs #7, #8, #9 and #10 soot deposits are more or less present on the horizontal crack surfaces, usually near the fire, but at slab #7 also over a large part of the length. Note that the water drainage holes are applied at some distance from the hollow core support and thus this can not be the cause of the fact that the soot deposits are generally the worst near the end of the slab [5.3].

The damage and soot patterns show that the horizontal cracks initiated and opened at a time when yet a significant smoke development existed, so when the fire was still in progress. However, the initiation of these horizontal cracks did not in all cases led directly to failure of the underflanges. This is for example shown at slab #7, whereby on photos it is observed that the lower flange has fallen just a few hours after the fire, but nonetheless both the cores and parts of the horizontal crack surfaces have soot deposits [5.3]. Also [5.1] concluded that it seems that during the cooling process the cracking in slabs still continued, as some parts of the floor slabs had fallen indeed only a few hours after the fire was extinguished. When the damage that has occurred is compared with the calculated temperature distribution (Figure 5.7), it emerges that the degree of spalling damage to the hollow-core slabs generally corresponds to the degree of heating; the majority of spalling occurs in the most heated area. Also it emerged that horizontally cracked slabs were separated over the entire length, even though heating was mainly local. It can be seen that the horizontal crack surfaces have slightly undulating shape in many places, but at other locations straighter [5.5].

It is unknown to what extent extinguishing with water can cause damage to concrete structures. The report commissioned by the Veiligheidsregio Rotterdam [5.1] described that the fire brigade withdrew from the (second) commitment because they heard loud bangs and cracking from the structure. Although some experts believe that during extinguishing mostly spalling damage occurs, this has never been researched experimentally under controlled conditions [5.1]. And it is not excluded in the TNO report [5.3] that the pressure from the 35.000 l-per-minute water spray coming from the fire boat contributed to the final local damage to the soffits of the hollow core slabs.

A few other conclusions were drawn in the report of TNO [5.3]. At first, regarding the horizontal cracks in the webs of the hollow core slabs, it is concluded by TNO that these cracks are more frequently observed, both in real fire as well as fire tests. The horizontal cracks as found in the Lloydstraat in Rotterdam, are thus not exceptional, and are more common. It is however noted that the presence of these horizontal cracks does not directly need to lead to the collapse of the floor structure. In Rotterdam, while large parts of hollow core slabs had fallen down, the floor structure as such however did not collapse; it had only local damage. Secondly, it is concluded that the floor structure as applied in the parking garage is not in itself an exceptional floor structure, although the thickness of the applied structural topping is somewhat on the thick side. The hollow core floor consisted of a 7 cm to 9 cm structural reinforced topping. In addition, an asphalt layer was applied in order to drain the water. This asphalt layer is 12 cm thick at the external wall, and reduced till about 7 cm at a distance of 1 to 2 m from the elevator. A plastic foil was designed between the structural topping and the asphalt layer, but cores drilled from the floor structure (29.11.2012) disappeared so the presence of a plastic sheet was never confirmed. Thirdly, it is concluded that the load on the hollow core floor was relatively low during the fire; only 7 cars were present. In the design of the floor the total dead load was 6.70 kN/m^2, while the extreme live load was 2.0 kN/m^2 with a frequent or quasi-permanent value of 0.7. And finally, it is concluded that in the concrete mix of the applied hollow core slabs limestone was used as coarse aggregate. It is known that concrete with limestone has a lower fracture energy than for example gravel concrete. But it is known that concrete with limestone is less spalling sensitive.

5.7. Temporary measures published by BFBN in 2009

To summarize, all the investigation reports [5.1], [5.2], and [5.3], and the discussions with the Ministry of Environment and Construction lead to a focus on the behaviour of hollow core slabs and fire. BFBN felt that they had to address this by an intermediate communication advising temporary measures that would limit the application in order to show that the industry took their responsibility. Hence, based on the final report [5.3] BFBN published a letter with temporary measures on 19 November 2009. The measures were also published in Dutch Cement [5.4]. Basically, the target of the temporary design rules was “that the Rotterdam floor could not be designed again”. The temporary measures dictated extra requirements for REI > 60 minutes, namely:

1. The hollow cores need to be supported on rubber bearing strips (3 mm x 40 mm);
2. The connection reinforcement at the support shall not be placed above half the thickness of the hollow core slab (connection reinforcement may be placed in the joints or in recesses);
3. The thickness of the structural topping can not be more than 50 mm in the middle of the span, due to camber the thickness will be larger at the support;
4. The reinforcement in the structural topping close to the support may not be more than ∅6-150 mm in length direction.

Further, it was advised to already take into account the design rules for shear and anchorage according to the in 2009 published draft document EN1168:2009 Annex G [5.22].

5.8. Subsequent studies in 2009, 2010 and 2011

After the publication of BFBN’s temporary measures in 2009, the average structural engineers did not really understand the design rules as they felt that they needed more reinforcement for stability (diaphragm action) and for durability (crack widths). As an example the limitation of the amount of connection reinforcement in the structural topping is given. Although the limitation of reinforcement to ∅6-150 mm increases locally the safety under fire (less blocking for hollow core under fire) the structural engineers indicated that it reduces the overall safety of building. Accordingly, to come to better conclusions for the structural engineer, a new research group was set up in 2010. This so called “Korte termijn Actie groep” (Short Term Action Group, abbreviated with KTA) had a short term focus. The main issue of the KTA-group was to re-evaluate these special design rules, as it was felt that these measurements were not the solution to preventing horizontal cracking such as observed in Rotterdam. This KTA group consisted of TNO, Efectis Nederland, Adviesbureau J.G. Hageman, ERB and DGMR, with representatives from the Dutch hollow core industry.

The short-term study focused on the force transfer in hollow core slab floors as a result of elevated temperatures and the possible initiation of horizontal cracks in the webs of the hollow core slabs. The group focussed on the behaviour in the transversal cross section. The longitudinal direction was explicitly excluded from the research as it was argued that there is hardly any effect on horizontal cracking in this direction. The study (see Figure 5.9) encompassed collecting information from available fire tests performed on hollow core slabs; numerical analyses on the behaviour of hollow core slabs, a few fire tests, and a detailed look at the regulations and Eurocodes. The few fire tests were conducted on slices of hollow core slabs with and without structural toppings. The socalled “orienting fire tests” did contribute to the understanding of horizontal cracking on element level; but the translation to floor structures at building level was not part of the research. The sophisticated finite element analyses were carried out with mechanical-transient non-linear models in order to model the phenomenon, while a simple truss model with linear elastic elements had been developed in order to approach the problem simpler. Although both modelling types do give direction to finding causes, due to the limitations of the models they cannot provide the answers.

Figure 5.9. Experimental studies of part of hollow core slab with and without topping, restrained conditions and FEM

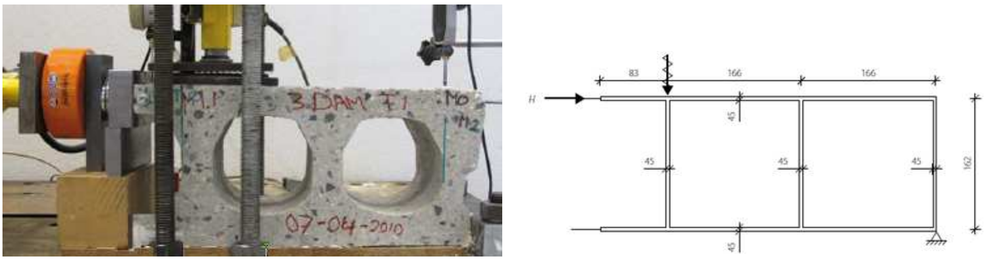


Figure 5.10. Numerical studies with simple truss analysis, shear stiffness validated with experiment

At the same time of the work of the KTA-group, Prof. Kleinman of Eindhoven University of Technology [5.7] carried out individual research on a simple truss model consisting of only linear elastic elements (Figure 5.10). For that, he conducted two shear tests on parts of hollow core in order to derive the shear strength at ambient temperature. For fire analysis, he very much simplified a linear fire on the soffit. He concluded from a parameters study that for expansion of the bottom flange during a fire, preventing the deformation of the upper flange of the hollow-core slab has a very negative effect on the fire resistance of the hollow-core slab due to (shear) cracking through the webs. When the thickness of the structural topping increases cracks through the webs initiated earlier in the fire. On the other hand, increasing the height of the hollow core slab improved the fire resistance of the hollow core slab. In the article Prof. Kleinman proposed solutions such as omitting some cores in the current cross sections, or adding pre-cuts in the soffit in order limit the expansion.

5.9. New measures published by BFBN in 2011

In June 2011 the BFBN published a letter [5.8] with these new measures for hollow core slabs under fire conditions. The conclusions formulated by the KTA-group on the basis of the study were published in Dutch journal Cement [5.9, 5.10]. The main conclusions of the study and outcome are given hereafter:

If due to a thick topping or finished screed layer (both referred to as the “topping”) at elevated temperature the topping is not able to deform sufficiently, horizontal cracks can initiate in the webs of the hollow core slab as a result of which the under flange may separate prematurely. This is because the behaviour in the transverse direction of the hollow core slab is more dominant than the behaviour in the longitudinal direction. For this reason, measures in the longitudinal direction no longer make sense and therefore limiting the reinforcement in the structural topping in the longitudinal direction is not necessary.

See Table 5.I. If the thickness of the topping is less than 50 mm, problems with the under flange are expected to be negligible. In this case, it may be assumed that the under flange will not collapse. For toppings thicker than 70 mm, there are indications that horizontal cracks in the webs may initiate, which may subsequently result in the separation of the under flange. If the thickness of the topping is between 50 mm and 70 mm, horizontal cracking of the webs and possible separation of the bottom flange could also be possible. In both cases, the additional measure “A” is needed as clarified in Figure 5.11.

If the under flange of a hollow core slab floor becomes separated due to fire, the (remaining) hollow core slab with topping may still meet the requirements of the building regulations in some cases. For example, this is the case if it still can be proven that with the reinforcement in the topping the floor will not collapse when exposed to an accidental load combination in combination with fire (a socalled alternative load path).

Compliance with the building regulations does not, therefore, mean that the occurrence of damage is excluded. Besides, this applies to every load bearing structure in fire conditions

With regard to hollow core slab floors, a distinction must be made between separation of the under flange only, the local collapse of (a part of) the floor field and situations in which disproportionate damage occurs to the entire structure in terms of the Eurocode. Only the latter situation is not permitted according to the Dutch Building Regulations. Exceptions are hollow core floors that form part of the fire compartment, and fire- and smoke-free escape routes for which local damage may not occur within the fire-resistance requirement of thirty minutes.

Table 5.I. Advice on when to apply measure "A" as a function of topping thickness and consequence classes

	$t \leq 50$ mm	50 mm $< t \leq 70$ mm	$t > 70$ mm
1	-	-	-
2a	-	-	A
2b	-	A	A
3	-	A	A

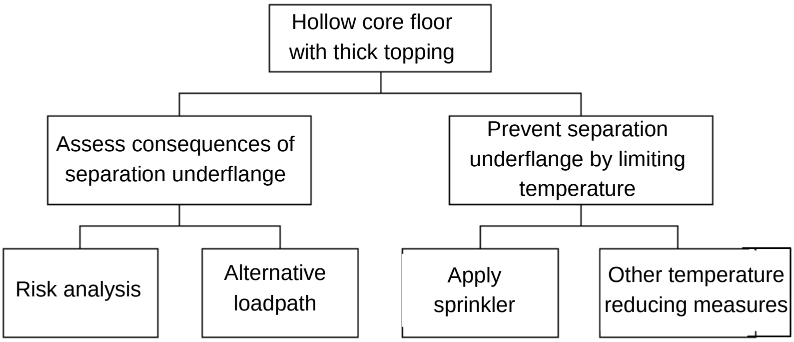


Figure 5.11. Measure "A": proposed approach for hollow core floors with thick toppings

The new BFBN measures were received now much better by the structural engineers. Despite this, two engineers openly started lobbying against the new measures in a contra-article in the Dutch journal Cement [5.11] in December 2011 saying that the new measures were not satisfactory as the authors had two problems. Firstly, their opinion was that the measures of November 2009 and June 2011 were opposing each other as the temporary measures of 2009 addressed the longitudinal direction, while the new measures of 2011 addressed the transversal direction. Secondly, in their opinion the new measures did not give an answer on how to deal with secondary load paths as proposed by the measures as an option when the structural topping is thicker than 7 cm. In January and February 2012 the BFBN in co-operation with Verenigde Nederlandse Constructeurs (United Dutch Structural Engineers) organized four evening-discussion meetings with structural engineers to explain on and discuss the measures proposed by BFBN. The structural engineers expressed that they had enough confidence in designing hollow cores according to the new measures. The 50 mm topping limitation was accepted by authorities until mid 2014 as an "intermediate solution". Until then the Dutch industry has time to come up with more answers and solutions.

5.10. Compartimentation and openings in Lloydstraat building

Compartmentation [5.24] has traditionally been defined according to the concept of fire resistance, with reference to collapse (R criterion), fire penetration (E criterion), and excessive heat transfer (I criterion). But in the case of the Rotterdam fire, judging on the criteria R, E, and I that were all met, one should conclude that the compartmentation requirement was met. The purpose of subdividing spaces into separate fire compartments is twofold. Firstly, compartmentation prevents any rapid fire spread that would trap occupants of the building. Secondly, compartmentation restricts the overall size of the fire. The compartment area of the Rotterdam parking garage was equal to the size of the garage, namely 2,100 m^2. Investigation report [5.1] concludes that this is not in accordance with the Dutch Building decree which demands a maximum compartment area of 1,000 m^2. However, the Building decree says that sizes of fire compartments may be enlarged when the space is not private, so that it can be assumed that during a longer time frame rescue and extinguishing activities can take place. And [5.1] states that together with the generally accepted principle that in a parking garage no more than 4 cars can burn at the same time, this assumption is a way to enlarge a fire compartment. The accepted principle of four cars is in line with the statistics from some cities of Europe drawn up by CTICM in 2001 [5.1], that ± 97% of the fires in underground garages remain limited to a maximum of 4 cars. There are two fires registered in underground garages where 7 cars were involved in the fire. In aboveground parking garages no cases are registered involving more than 3 cars [5.1]. The presence of an alarm installation would have contributed to a faster arrival of the fire brigade. This would have resulted in a less severe fire, and also less damage to the concrete.

In the Dutch building decree two sets of guidelines are addressed for designing car parks with a proper fire safety level. These guidelines can either focus on closed (mechanically ventilated) car parks, or on semi-open (naturally ventilated) car parks [5.28]. In the latter case, there is a correlation between the amount of open façade area and the fire safety level. Most car parks have fire compartments with an area larger than 1000m^2, and thus do not meet the prescriptive requirement for maximum fire compartment size in the Dutch building decree. The decree allows for this deviation as long as it can be shown that an equal level of fire safety is obtained in terms of the decree. This is where the guidelines for naturally ventilated car parks are applicable, for which the car park standard NEN2443 [5.25] is mostly used in practice. The openings in the Rotterdam parking garage have been designed with this Dutch standard NEN 2443 for parking of cars in garages. Efectis report [5.1] stated that Rotterdam parking garage had fulfilled these requirements. This guideline basically consists of three requirements. However, if these requirements generally result in a sufficient fire safety level for safe deployment of the fire brigade, has never been systematically investigated [5.28]. First, at least two opposing façades must have an opening. Second, in order to avoid heat development, the openings in the outer facades need to be at least 1/3 of the total wall area that form the envelope of the fire compartment, or, in one facade of two opposing facades the openings need to be at least 2.5% of the gross floor area of the compartment. Third, the openings should not be located more than 54 meters from each other. Depending of the layout of a parking, each of these requirements might result in the governing requirement on openings in the outer wall. The first and third requirements are met in Rotterdam, however, not the second requirement. Moreover, the wall part above the open windows to the ceilings prevented the heat and smoke evacuation of the fire (see Appendix 5.A level 2).

A recalculation on the second requirements by Holcofire shows that of the approximate 1,300 m^2 external wall area (height of 12.05 m, and circumference of 108.5 m) of the fire compartment of the parking garage levels, the area of openings is about 300 m^2. This is 23% and thus not more than 1/4 of the total wall area of the fire compartment envelope. TNO report [5.3] also mentions this openness of 23%. But the other second requirement shows that the openings in one facade of the opposing facades are 3.5% of the gross floor area which actually fulfils the second requirement of at least 2.5%. Regarding opening and practices in other countries, a praxis used in Sweden based on American full scale tests stipulates that the walls must have at least 30 % permanent openings to be able to consider the building as an open structure with respect to fire development. A praxis in Germany states that at least 5% of the gross floor area should be kept as open in the wall area at one side of opposing walls.

5.11. Smoke and heat development and temperatures of fire scenario 1

The fire scenario 1 simulations performed by Efectis for the Rotterdam Rijnmond Safety Region [5.1] give results on the heat development and magnitude of temperatures at the ceiling above the fire. Efectis used for the simulations the software CaPaFi 2.0, an MS Excel program developed at European level dedicated to analyse car park fires. Although the CaPaFi calculations in the Efectis [5.1] and TNO [5.2] reports seem advanced, many questions can be raised on the fire calculations performed. Ofcourse, a model is a simplification of reality, but it is believed that conclusions have been drawn based on a modelling that has been approached too simple. At first, in the Efectis CaPaFi analysis the outside wall is not accounted for, as the program does not foresee in that feature, while the wall is in reality for 78% closed. And, more in general, the configuration of the car park is not taken into account in the simulation. Secondly, naturally ventilated (semi-open) car parks are different from mechanically ventilated (closed) car parks, since they are affected by the influences of wind. The wind is not taking into account in the CaPaFi calculations, as the program does not foresee in that feature. From weather reports on Rotterdam is was found that the wind velocity was 3.5 m/s. Thirdly, report [5.1] and [5.3] make, on the basis of the CaPaFi simulation of scenario 1, a comparison between the heat development and temperatures of scenario 1 and the standard ISO 834 curve. In the conclusions of both reports contradictory statements are present about the severity of the fire compared to the standard ISO curve.

More advanced than CaPaFi is the program Fire Dynamics Simulator 5.5 (FDS5) based on computational fluid dynamics to model the Rotterdam fire case (Figure 5.12). This FDS5 simulation is as accurate as possible based on the input data given of fire scenario 1 of Efectis [5.1] and TNO [5.2]. In the background report on FDS5 [5.31] at first under question 1 and question 2 scenario 1 by Efectis/TNO (see Figure 5.7) in an open space was recalculated. In principle, the assumptions in the FDS5 simulation were identical to those in the CaPaFi simulation, like the heat release rate given in Figure 5.6. It was concluded that the results of both programs are comparable.

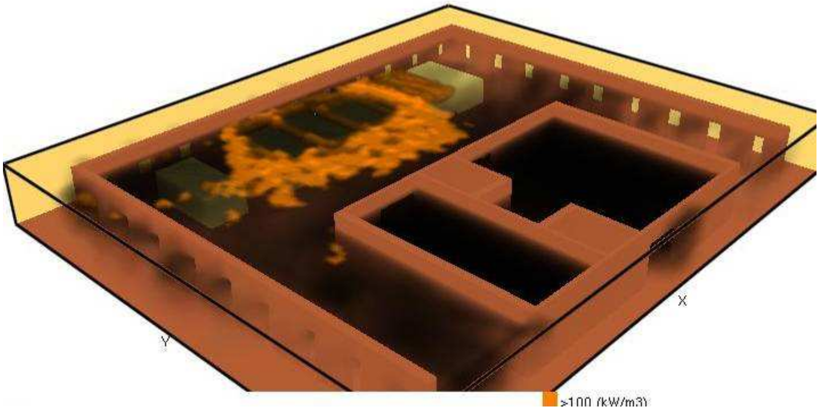


Figure 5.12. Geometry Parking Rotterdam and HRRPUV plus smoke snapshots at 30 minutes simulation in fire scenario 1 [5.31]

In order to have a more sophisticated spatial analysis of the fire development, and to include wind in the fire scenario development, fire scenario 1 of the Rotterdam fire case is recalculated with FDS5 and reported in [5.31] under question 3. See Appendix 5.B for FDS5 data [5.31]. Figure 5.12 illustrates a visualization of heat release rate per unit volume and realistic smoke movement from the inside of the compartment to the exterior at 30 minutes of fire (at 04:33 h). Fire is coloured in a dark shade of orange wherever the computed heat release rate per unit volume (HRRPUV) exceeds 100 kW/m^3. The visual characteristics of fire are not automatically accounted for. At this point in time, the cars 1, 2 and 3 have burned and the first maximum HRR peak is finished. In the figure we can clearly see the three cars burning and flames and smoke developing. After 30 minutes from the fire ignition, the layer of smoke covers the whole compartment and the openings are insufficient for the smoke evacuation. This corresponds well with the visual observations of the fire brigade that indicated that at 4:29 h smoke developed that became dark black. Clearly we see flames and dark smoke coming out of the windows. The fire brigade reported that from that time the evacuation was hindered by the smoke coming from the broken door from the garage. And at 04:32 h the fire brigade tried to enter via the garage door, but due to the black smoke it was too dark.

Faster and darker smoke is closer to the seat of the fire, and thick, dark grey smoke “pushing” out of a structure suggests a larger, more intense fire [5.29, 5.30]. “Black fire” is a good phrase to describe smoke that is high-volume, turbulent velocity, ultra-dense, and black. Black fire is a sure sign of impending auto ignition and flashover. In reality, the phrase “black fire” is accurate: it acts as a vehicle for spreading fire and the smoke itself is doing all the destruction as that flames would cause like charring, heat damage to structures, and content destruction. Black fire can reach temperatures of more than 1,000°F and is likely to be toxic.

According to NFPA921, Paragraph 3.6: “Smoke colour is not necessarily an indicator of what is burning. While wood smoke from a well ventilated or fuel controlled wood fire is light coloured or gray, the same fuel under low-oxygen or ventilation-controlled conditions in a post-flashover fire can be quite dark or black. Black smoke can also be produced by the burning of other materials including most plastics or ignitable liquids.” Hence, petroleum and petroleum-based products produce black smoke, but black smoke might also indicate under-ventilated conditions. Incomplete burning causes smoke density or smoke thickness. In essence, the thicker the smoke, the more spectacular the flashover or fire spread.

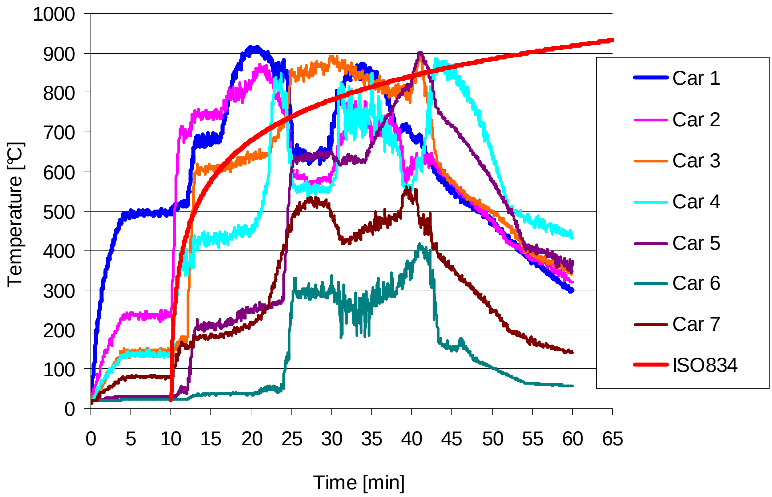


Figure 5.13. Evolution in time of the gas temperature above car 1 to car 7 and ISO 834 curve [5.31]

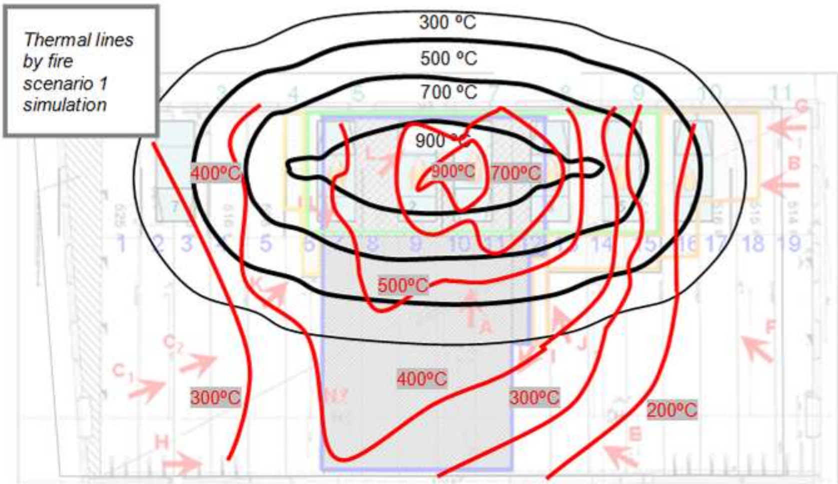
In background report [5.31] under question 4 the results of the simulation with FDS5 are compared with standard ISO 834 curve. For that, the basic principles of fire development of a natural fire will be used. A natural fire shows a growth phase after ignition. Fuel, compartment geometry and ventilation are the main factors that determine the growth phase and the shape of the fire curve. Then there is a rapid transition stage called flashover between the growth phase and the fully developed fire. Flashover is defined as the relatively rapid transition between the primary fire which is essentially localized around the first item ignited, and the general conflagration when all surfaces within the compartment are burning. Mostly, flashover takes place when the upper smoke layer reaches temperatures of about 500-600°C. In one car fire study the start of the burn period is defined as the time when the heat release rate reaches 10% of the peak, because there is a large variation among tests of the fire growth delay from the start of the tests, i.e. time of application of the ignition source, to the time when the heat release starts to rise beyond that from the ignition source. The standard fire curve does not consider the pre-flashover growth phase. It starts at the moment of flashover and the increase in temperature over a couple of minutes is huge. Figure 5.6 gives the heat release rate of fire scenario 1. It is evident from the graph that just after 10 minutes the 10% value of the peak value of 22.5 MW is exceeded. From Figure 5.13 it is evident that after 10 minutes the temperature above car 1 is in the range of 500-600 °C. Accordingly, the standard

ISO 834 curve as sketched in Figure 5.13 is assumed to start at 10 minutes. The observations in Figure 5.13 are (temperatures are thermocouple based):

- The fire up to 10 minutes can be considered as the growth phase of the fire. Temperature above car 1 is around 500°C and above car 2 is above 200°C, while at other location the temperature is lower.
- At 10 minutes flashover is assumed, after which in the Rotterdam fire case the fully developed fire acts. The standard ISO 834 curve considers only a fully developed fire and is assumed to start at 10 minutes. After 2.5 minutes the ISO temperature is 500 °C. The temperature above car 1 and car 2 calculated with FDS5 takes 10 minutes to heat up to 500°C and 250°C, respectively.
- After 500°C, the temperature calculated with FDS5 is more severe that the standard ISO curves. While the standard ISO curve indicates a temperature of about 678°C at 20 minutes (at 10 minutes of ISO fire), the FDS5 calculation gives temperatures of about 900°C. Hence, more than 200°C higher temperatures are calculated and the heat with FDS5 is 33% higher than that of the standard ISO curve.
- After 500°C, the temperature increase rate calculated with FDS5 is more severe that the standard ISO curve. See Figure 5.13. While at 20 minutes the standard ISO curve indicates a temperature increase rate of 15,6 °C/min, the FDS5 calculation gives average temperature increase rate of 44.7°C/min. Hence, at 20 minutes the temperature increase rate calculated with FDS5 is 3 times higher than the standard ISO curve. Above car 2 and 3 the temperature increase rate is even higher.
- The temperature above car 3 follows the standard ISO curve more or less up to 25 minutes, but then the temperature calculated with FDS5 is approximately 150°C higher than the standard ISO curve. For car 3 the maximum temperature is about 893°C at 30 minutes, while the ISO curve is about 782°C at 30 minutes. For car 4 the maximum temperature is about 567°C at 30 minutes, while the ISO curve is about 782°C at 30 minutes (20 minutes of ISO fire). The temperature measured above the 5th car remains below the standard curve up to 41 minutes when the maximum temperature of 903°C is reached and exceeds the standard fire curve.
- The temperature above the cars 6 and 7 remains under the standard fire curve. For car 7 a maximum temperature above the car of about 562°C is obtained due to wind influences (car 7 did not burn), while above car 6 the maximum temperature is only about 418°C.

Hence, the FDS5 – ISO comparison in the Rotterdam car park fire analysis mainly concludes that at 20 minutes more than 200°C higher temperatures and 3 times higher temperature increase rates can be observed with FDS5 compared to the standard temperature – time curve ISO834 obtained according to EN 1991-1-2:2002. Also, the ceiling area influenced with FDS5 is much larger than the ceiling area influenced by a CaPaFi calculations due to wall effects and wind influences. This explains well the locations of severe damage of the fire case Rotterdam.

Figure 5.14 shows the floor plan of the Lloydstraat parking garage. On the plan, thermal isolines of fire scenario 1 are plotted as calculated by [5.2] with CaPaFi (black lines), and as calculated with FDS5 and illustrated by Smokeview (red lines). There is a dramatic difference in results between the CaPaFi calculation and the FDS5 calculation. The influence of the outer wall can be clearly seen; while the CaPaFi model gives the same high temperatures outside the building as inside, the FDS5 simulation gives higher values of the temperature only inside the building. It is generally known that the accuracy of the CaPaFi model decreases with increasing distance to the fire. And wind was also not accounted for in the CaPaFi model, but has a significant influence on the temperature distribution at the ceiling.



↑ 20 minutes (04:23h) ↓ 30 minutes (04:33h)

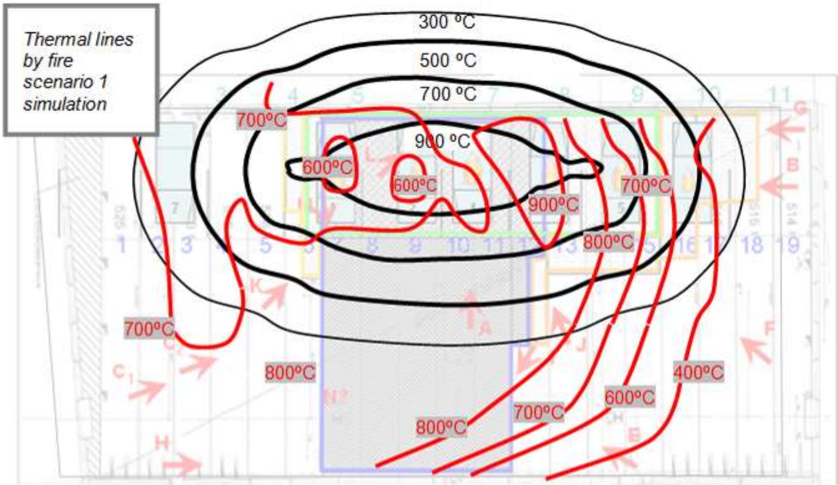


Figure 5.14. Fire scenario 1 as calculated by CaPaFi [5.1] (black lines – thermocouple temperature) and as calculated by FDS5 after 20 and 30 minutes as shown in Smokeview (red lines – gas temperature) plotted on floor plan [5.31]

In the FDS5 simulation it can be noted that maximum thermal isolines corresponding to approximately 900°C move from above car 1 and car 2 to above car 3. Also, the thermal 800°C isoline from FDS5 reaches much further to the support at the other side. This is not the case in the CaPaFi model that is very local. Hence, large temperature variations can be recorded throughout the entire compartment at the same time. This behaviour cannot be recorder with CaPaFi since the temperature is considered to be uniform in the whole compartment at any given time. The background report [5.31] was reviewed by Dr. G. Rein of Imperial College London, see Appendix 5.C for integral review text.

5.12. Explosive spalling

Although in the analyses of TNO [5.3] explosive spalling is mentioned a few times, it is not part of the main conclusion for the cause of failure; horizontal cracking is explained by thick topping or thick finished screed layer. Report [5.3] states that spalling could be of influence on the stresses in the concrete and possibly also on the tensile stresses that are present in the webs. But from the photos (see Figures 5.1, 5.3, 5.4 and 5.8) it is evident that explosive spalling did play a role in the occurrence of horizontal cracks in the fire of Rotterdam. Due to the high moisture content in the slabs explosive spalling was inevitable. Above cars #1, #2, #3 and #4 where the heat development was higher than 900ºC, the cores opened due to explosive spalling. The parking garage was an open parking, meaning that no indoor climate was present but a sheltered outdoor climate. Also, a plastic foil was present on the topping: the slabs and the topping could only dry out from the hollow core soffit (see Appendix 5.A). As a result, the moisture content of the concrete is likely to have been relatively high, which is particularly unfavourable for the spalling behaviour and indirectly for the occurrence of horizontal cracks in the webs [5.3]. According to Eurocode EN 1992-1-2:2004 [5.27] explosive spalling is unlikely to occur when the moisture content is less then 3 % by weight (recommended value). In the product standard EN1168:2005+A3:2011 a moisture level of maximum 3 % by weight is recommended when making fire tests.

From the Rotterdam building no concrete samples from unaffected parts of the floor structure were taken to determine the moisture content. The moisture level at the time of the fire was most likely significantly more than 3 %. The slabs were produced in October 2006. In October 2007 the fire broke out, so the floor structure at the time of the fire was 12 months of age, which is relatively young. The structural topping (70 – 90 mm) casted on site was according to the drawings covered by a plastic foil. This results in a moisture transport through the hollow core to reduce the initial distribution of moisture in to a distribution in equilibrium with the yearly variation of relative humidity in Rotterdam, which has an average of about 83% (statistics from Rotterdam the Hague airport). Most likely it would take 5 - 10 years for the slab to reach a yearly average on a depth of 30 mm into the underflange. In the concrete mix of the hollow core slabs that were applied limestone was used as coarse aggregate material. It is known that calcareous concrete has lower fracture energy than for example siliceous concrete. This could have a negative affect on the growth of cracks in the sense that the crack growth was faster. But it is also known that calcareous concrete is less spalling sensitive, whereas horizontal crack initiation could be influenced [5.3].

Generally speaking, explosive spalling is not explicitly taken into account in the design of concrete structures. This is evidenced by a comment from Dutch fire design standard [5.26], that the equilibrium moisture content in buildings is expected to be low in such a way that the risk of spalling is limited [5.1]. Efectis reported in [5.1] that they have the practical experience from fire tests and from damages of real fires that spalling of concrete occurs in many cases in situations where according to the Dutch fire standard [5.26] this would not occur. In the opinion of Efectis, the probability of spalling must therefore be greater than expected on the basis of the standards. For an unheated structure (and in particular a structure exposed to the outside air as the present structure is), it is also doubtful whether the mentioned conditions are met. In buildings the equilibrium moisture content is normally in the range 2 – 3 % by weight when the concrete structure has reached a state of moisture equilibrium with the inside environment with a yearly average relative humidity 40 - 50 %. For a concrete structure exposed to outside conditions (relative humidity 83 %) higher moisture contents can be expected. The concrete structure is in fact throughout its lifetime exposed to an average humidity, so that the pores of the concrete will contain more water.

5.13. The successive phases of delamination of underflanges in Rotterdam

Based on the new insights gained in the Holcofire project, Figure 5.15 illustrates five successive phases of delamination. These five phases illustrate how the fire and accompanying local damage progressed in the fire:

a) The parking garage is a socalled open parking garage with natural ventilation resulting in a sheltered outdoor climate. As a result, the moisture content of the concrete is likely to be relatively high. For unknown reasons a fire ignites in the garage. Due to high moisture content of the slabs minor explosive spalling starts on the ceiling in the soffit of the hollow core slabs just above the growing car fire.
b) At 10 minutes flashover takes place and car 2 is sequentially ignited. Then, the fire abruptly transforms into a fully developed fire with increasing release heat rate. Due to the highly intense fire with high temperature and high temperature increase rate, and restraints (thick topping and floor field), horizontal cracks initiate in the webs just above the cars. Due to high temperature gradient over cross section, spalling continues over a larger area and more deeply into the soffit of slabs such that open cores are visible just above the seat of the fire. At 20 minutes black smoke start to develop.
c) The fire is now travelling to car 3. As more cars are burning, the maximum heat release rate is reached at 30 minutes. Due to E-N-E wind the temperature at the ceiling even reached 800°C at the other support. As a result, the horizontal crack initiates further through the webs of the slab and reaches the other support. But the growth of the horizontal crack does not lead to failure; the strands remain covered by (a part of) the concrete cover (as is reported in [5.1] and [5.3]) and the strands

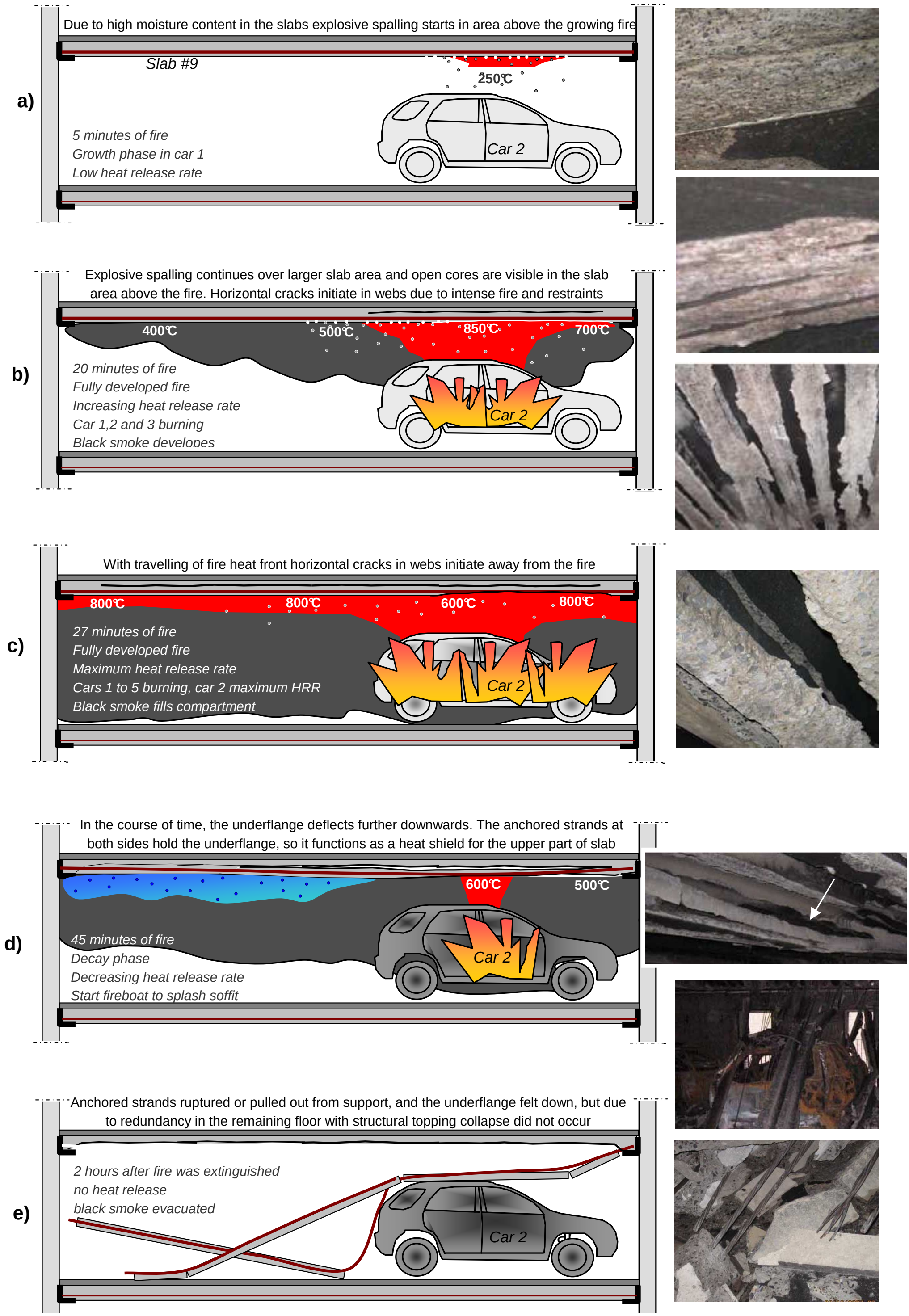


Figure 5.15 Successive phases during the Rotterdam fire of slab #9 above car 2 (KIA Sportage) (note that photos on right-hand side are meant to be illustrative and are selected from other locations)

keep functioning as a tie for the cross section of the hollow core slab. Also the redundancy in remaining floor contributes to an equilibrium state during the fire.

d) In time into the fire, the prestress in the underflange decreases and the underflange deflects further. But the well anchored strands at both sides of the supports hold the underflange in place. The underflange actually functions as a heat shield that protects the upper part of slab against the heat release of the fire. This state is kept as long as the anchorage of the strands functions, or as long as the strands themselves do not rupture. At 45 minutes of fire fireboat starts extinguishing the fire by splashing the soffit.

e) Finally, in the heat of the fire the strands rupture, or the anchorage of the strands fails and the strands are pulled out from one support. Consequently, the underflange falls down on the car and the ground. As a result of redundancy of the floor with a thick reinforced topping, in the accidental situation the floor with the thick structural topping did not collapse. Situation e) was found directly after the fire.

It should be emphasized that extinguishing the fire with the fire boat most probably has influenced the final phase of delamination of the underflanges so that they fell down on the floor. The fire boat was deployed at 04.48 h. The fireboat splashed with all 3 guns 35.000 l water per minute through the open windows; it is standard procedure to splash against the soffit of the ceiling to cool the structure and extinguish the fire. The force generated by 35.000 l of water per minute is huge, and the abrupt cooling down of concrete parts should also not be neglected. A simple consideration shows that if the fire brigade is delivering 35000 liters/minute using 3 guns together, this would result into a force of about 11 kN as an average value. This estimate is based on the fact that the fire brigade spouted through the building resulting in fire fighters becoming wet at the front of the building. Hence, using a velocity large enough to have the waterspout parabola from one side to the other, roughly 20 m long with maximum height 1.5 m, results into a total velocity of the water of 19.06 m/s (vertical velocity 5.48 m/s and horizontal velocity 18.25 m/s). Accordingly, if all guns points at the same position, the force would be 35000*19.05/60 is 11.1 kN. Theoretically, this is the force developed if one is hitting a target and the water velocity change to zero. This is quite large force, acting horizontally on the bottom flange of hollow core slabs. On top of this force, a dynamic influence of force variation depending on the strategy used by the fire brigade has to be considered.

Hence, the exact moment and cause of falling down of the soffits of slabs #9, #10, #11, and partly #12 remains unclear. But do note on Figure 5.3 that a part of slab #11 is still hanging on the ceiling, which is also seen in the small picture at d) in Figure 5.15. It is also evident that the underflanges of the slabs #7 and #8 only felt down several hours after the fire, as proved by the photos C1 (taken at 06:46 h) and C2 (taken at 09:01 h) in report [5.3]. This is for clarity sketched in Figure 5.16. The photo at 09:01 h shows that the slabs #7 and slab #8 came down somewhere between 06:46 h and 09:01 h due to the failure of the anchorage of the strand at the side of the support near the fire. The small photo in Figure 5.16 shows the end of the strands that were pulled out of the support. It is estimated that after the fire about 200 mm of anchorage length remained at the support, which was after some hours not enough to secure the anchorage of the strands of the slabs that were delaminated.

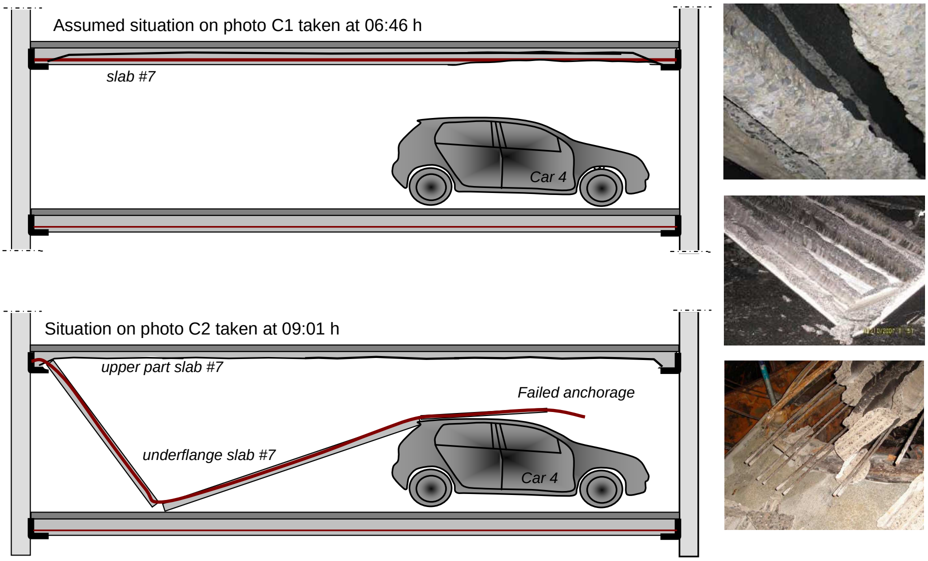


Figure 5.16. Slab #7 felt down several hours later due to anchorage failure, situations at 06:46 h and 09:01 h

- PART III – OUTLOOK -

5.14. Accidental actions by the Eurocode

Fire is an accidental action. Eurocode EN1991-1-7 [5.23] deals with accidental actions, amongst others with accidental actions due to localised failure from an unspecified cause. Localised failure is defined by Eurocode as that part of a structure that is assumed to have collapsed, or been severely disabled, by an accidental event. Annex A gives rules and methods for designing buildings to sustain an extent of localised failure from an unspecified cause without disproportionate collapse. It declares that it is an acceptable strategy if a building is designed such that neither the whole building nor a significant part of it will collapse if localised failure is sustained. The minimum period that a building needs to survive following an accident should be that period needed to facilitate the safe evacuation and rescue of personnel from the building and its surroundings. Consequence classes have been introduced to categorise building types/occupancies. Based on these consequence classes, strategies are recommended to provide a building that will have an acceptable level of robustness to sustain localised failure without disproportionate level of collapse.

- For buildings in consequence class 1 no specific consideration with regard to accidental actions is necessary provided that it has been designed according to the rules given in the Eurocodes.
- For buildings in consequence class 2 effective horizontal and vertical ties should be provided, while in class 2b the building should be checked to ensure that upon the notional removal of each structural element the building remains stable, and that any local damage does not exceed a certain limit.
- For building in consequence class 3 a systematic risk assessment of the building should be undertaken.

Eurocode EN1991-1-7 Annex A is informative, and has not been used in the design of the Rotterdam parking garage. Nevertheless, horizontal and vertical ties have been clearly considered and applied in the building for robustness. Based on Eurocode EN1991-1-7 Annex A, BFBN elaborated more in detail in the letter of June 2011 in the Appendix on their conclusion #5. They concluded that with regard to hollow core slab floors under fire a distinction must be made between the following three cases:

1. Local collapse of only the under flange of hollow core slabs during a fire that does not lead to the overall failure of the floor. This is in principle not an issue of building regulations, but plays a role in the framework of Health & Safety issues for the fire brigade.
2. Local collapse of one or more floors during a fire that does not lead to disproportional damage in relation to the cause within the legal admissible fire time. In case the hollow core floor form part of the fire compartment or part of the fire- and smoke-free escape routes this local damage may also not occur within the fire resistance requirement time of 30 minutes. Eurocode EN1991-1-7 Annex A states that the indicative acceptable limit of localised failure for building structures is 100 m^2 or 15% of the floor area, whichever is less, on two adjacent floors caused by the removal of any supporting column or wall. This is likely to provide the structure with sufficient robustness regardless of whether an identified accidental action has been taken into account.
3. Disproportional damage to one or more floor fields and or superstructure in relation to the cause within the legal admissible fire time

Case 3 and case 2 are not applicable to Rotterdam. There was only local damage by separation of the under flange. Floor level 2 was not part of the compartment envelope, nor was the floor part of the escape route. Even the underflanges of slabs #7 and #8 separated after the fire was extinguished. Finally, the separated floor area of about 70 m^2 was less than 15% of 700 m^2, or 100 m^2, whichever is less, so less than the limit of admissible failure according to Eurocode EN1991-1-7. The FDS5 – ISO comparison concludes that 33% higher temperatures and 3 times as high temperature increase rate can be observed with FDS5 in the case of Rotterdam car park fire analysis compared to the standard temperature – time curve ISO834 obtained according to EN 1991-1-2:2002. Also, the ceiling area influenced with FDS5 is much larger than the ceiling area influenced by a CaPaFi calculations due to wall effects and wind influences. This explains well the locations of severe damage of the fire case Rotterdam.

In Rotterdam case 1 was applicable, as only local collapse of the under flange occurred that did not lead to the overall failure of the floor. This implies that in the Rotterdam fire the building regulations are not an issue, but the Health & Safety of the fire fighters. This is actually confirmed by two recommendations in report [5.1]. One recommendation is to investigate in what way the safety during the deployment of the fire brigade can be improved in open parking garages with natural ventilation, the second is that the fire brigade has to consider the possibility that for several hours after the fire in a concrete building during the cooling down phase stresses can be build up and as a result deformations or displacements (read local failure) can occur. Safety of fire fighters was indeed the main issue.

5.15. Conclusion

In precast concrete floor construction the hollow core slab has been a very successful product for residential and non-residential building structures, both in concrete structures and steel frames. This success is largely the result of the highly efficient design and production methods, flexibility in use, surface finishing and structural efficiency. Yearly in Europe about 20 to 25 million square metres hollow core floors are erected. The estimated total stock of installed hollowcore floors nowadays in Europe amounts to 1,000 million square meters. Experiences with past performance of hollow core floors confirm that hollow core floors under fire conditions have excellent fire resistance [5.12].

On the 1st October 2007 in the just newly-constructed apartment building “Harbour Edge” in the Lloydstraat Rotterdam, a fire at level 2 of the parking garage was reported. After a thorough research no cases are known where the preliminary collapse of the hollow core floor led to disproportional damage, or even collapse of a building structure. Despite, the Rotterdam fire case has affected the image of the hollow core slab on international level. Also, unjustified, in floor designs where no topping or a limited topping are applied. In most cases, Holcofire experiences that people are talking about the Rotterdam fire, but do not know the real facts. Holcofire has given facts in this Chapter in order to inform readers about the facts on the Rotterdam fire, but also to explain the atmosphere in which the discussions were held. Holcofire has also given their own view with new insights about the successive phases of delamination of the hollow cores in Rotterdam fire case, and recognizes the role of restraints and explosive spalling. Despite, it is believed that the measures taken and the international impact are disproportionate compared to the local damage and size of the problem.

Fire scenario simulations performed by Efectis [5.1] and TNO [5.2] have resulted in reports on the heat development and magnitude of temperatures to the ceiling just above the cars. Although the CaPaFi calculations in the Efectis [5.1] and TNO [5.1] reports seem advanced, many questions can be raised on the fire calculations performed. Ofcourse, a model is a simplification of reality, but it is believed that conclusions have been drawn out of the two studies with great impact based on a modelling that has been approached too simple by using CaPaFi calculation software.

Computer–based models are in widespread use today as part of fire safety design. The program Fire Dynamics Simulator version 5.5 (FDS5) based on computational fluid dynamics was used by Holcofire to simulate the Rotterdam fire case. FDS5 gives very good insight in the fire heat development and temperatures development during the Rotterdam fire. The rate of heat release (RHR) and ignition time were assumed equal to fire scenario 1 of Efectis to model the fire of the Rotterdam fire case. Also the geometry and compartment openings were considered, as well as exterior conditions as temperature, relative humidity and wind. Naturally ventilated (semi-open) car parks are different from mechanically ventilated (closed) car parks, since they are affected by the influences of wind. It emerges from the more sophisticated FDS5 simulations that the fire was more severe than 2 hours of standard ISO 834 fire. The simulations conclude that at 20 minutes into the Rotterdam fire (at 04:23h) when the maximum temperature above car 1 was reached, 33% higher temperature (900°C compared to 678°C) and 3 time higher temperature increase rate (44.7°C/min compared to 15.6°C/min) were calculated with FDS5 compared to the standard temperature–time curve obtained according to ISO834 of EN 1991-1-2:2002. Also temperatures higher than 800 °C were calculated at the other side of the support of the ceiling, which were not at all calculated by CaPaFi.

In fire design regulations the standard fire has to be considered in the whole compartment, even if the compartment is huge. But the standard fire curve is a normative curve; it takes into consideration only the fully-developed fire and does not have a descending branch. All differences recorded between the analyses made with FDS5 can be justified by the fact that standard ISO curve does not take into account the parameters like compartment geometry; boundary properties; environment conditions; number and position of the burning cars; heat release rate; and fire surface. The authors agree therefore with the conclusion drawn by Efectis [5.1] on page 47 that “the fire development does not match the base of the regulations and guidelines. In particular, the base with regard to the fire load of a car and the applicability of the standard fire curve are doubtful. On the basis of the regulations and guidelines a fire with this size was not to be expected”.

Finally, a risk analysis in the design phase of the building should have led to the use of an alarm installation as in Europe car park fires occur more often. The presence of an alarm installation would have contributed to a faster arrival of the fire brigade. This could have resulted in a less severe fire, and also less damage to the concrete structure.

References Chapter five

[5.1] Efectis Nederland, Report 2007-Efectis-R0894. Onderzoek brand parkeergagarge Lloydstraat Rotterdam, in opdracht voor Veiligheidsregio Rotterdam-Rijnmond, December 2007, p. 51.

[5.2] TNO Bouw en Ondergrond, Report TNO-2007-D-R1236/C. Onderzoek naar het constructieve gedrag tijdens brand van een kanaalplaat zoals toegepast aan de Lloydstraat te Rotterdam, in opdracht voor Veiligheidsregio Rotterdam-Rijnmond, January 2008, pp.12.

[5.3] TNO Bouw en Ondergrond, Report TNO-034-DTM-2009-00651. Brandwerendheid kanaaplaten, Deelplan A inclusief bijlagen, in opdracht voor BFBN. 24 juli 2009, p. 60 (main report) + p. 122 (Appendix).

[5.4] Welling, W., Van Paassen, A. Brand in parkeergarage, Schade aan betonconstructie nader onderzocht. Cement, October 2009, Nr. 7, p. 4-8.

[5.5] Van den Bos, A., Joostensz, O. Brandsimulatie op beton, Cement, October 2009, Nr. 7, pp. 34-37.

[5.6] Overbeek, T., Breunesse, A., Gijsberts, J., Both, K., Maljaars, J. and Noordijk, L. New regulations for hollow core slabs after premature partial collapse. Structures in Fire 2010 – Proceedings of the 6th International Conference, 2010, pp. 141-148.

[5.7] Kleinman, C. Kanaalplaat in dwarsrichting bij brand. Cement, juli 2011, Nr. 5, pp. 6-13.

[5.8] BFBN, Beoordeling van kanaalplaatvloeren bij brand, full copy of letter published by BFBN in June 2011, Cement, juli 2011, Nr. 5, pp. 14-19.

[5.9] Hordijk, D., Both, K. Nieuwe maatregelen kanaalplaatvloeren (1), Cement, juli 2011, Nr. 5, pp. 20-31.

[5.10] Klösters, G., Klein-Holte, R., Van Dongen, A. Nieuwe maatregelen kanaalplaatvloeren (2), Cement, juli 2011, Nr. 5, pp. 32-39.

[5.11] Lagendijk, P., Mulder, S. Kanaalplaten vanuit de praktijk bekeken, Cement, december 2011, Nr. 8, pp. 71-76.

[5.12] Klösters, G., Klein-Holte, R., van Donden, A. Kanaalplaatvloeren: nooit bezweken bij brand. Cement, december 2011, Nr. 8, pp. 77-81.

[5.13] Schaumann, P., Kleibömer, I. Sichtung und Aufbereitung jüngerer Forschungsarbeiten zum Brandverhalten von Spannbetonhohlplatten. Forschungsbericht im Auftrag von Deutsches Institut für Bautechnik, März 2012, p. 45.

[5.14] CTICM, rapport d’essai n° 95-E-467, 2 November 1995, p. 57.

[5.15] Bogogno, W., Fontana, M. “Versuche zum Tragverhalten von Betonhohlplatten met flexibler Auflagerung bei Raumtemperatur und Normbrandbedingungen“. Institut fur Baustatik und Konstruktion, ETH Zurich, p. 246, 1996.

[5.16] Andersen, N.E., Lauridsen, D.H. “Hollow Core Concrete Slabs, Basismiddel project X52650”, Technical Report X 52650, part 2, p. 29, 1999.

[5.17] Fellinger, J. “Shear and anchorage behaviour of fire exposed hollow core slabs”, PhD thesis Delft University of Technology, ISBN 90-407-2482-2, p. 261, 2004.

[5.18] Van Acker, A., “Shear resistance of prestressed hollow core floors exposed to fire”. Structural Concrete, Volume 4, Issue 2, pp. 65 –74, 2003.

[5.19] Jensen. J.F. “Huldæk og Brand - dokumentation vedrørendeforskydningskapacitet”. Udarbejdet for Betonelement-Foreningen, Birch & Krogboe A/S, p. 24, 2005.

[5.20] C.G. Bailey, Lennon “Full-scale fire tests on hollowcore floors”. The Structural Engineer, 18, pp. 33-39, March 2008.

[5.21] CEN, EN1168:2005+A3:2011 “Precast concrete products – Hollow core slabs” - Annex G: Resistance to fire, October 2011.

[5.22] CEN/TC229/WG1/TG1, “Background report on Annex G of EN1168, February 2010

[5.23] CEN, EN1991-1-7 EC1: “Eurocode 1: Actions on structures. Part 1-7: General actions Accidental actions”, 2006.

[5.24] fib, Model Code 2010, final draft, Vol 1 and Vol 2, Bulletin 65, March/April 2012

[5.25] NEN 2443, Parkeren en stallen van personenauto’s op terreinen en in garages, March 2013, p. 136.

[5.26] NEN 6071, Rekenkundige bepaling van de brandwerendheid van bouwdelen – Betonconstructies, December 2001, p. 36.

[5.27] CEN, EN1992-1-2:2004 , Eurocode 2: design of concrete structures, part 1-2: general rules, structural fire design, December 2004

[5.28] Heijden, M.G.M., Loomans, M.G.L.C., Lemaire, A.D. and Hensen, J.L.M., Fire safety assessment of semi-open car parks, Proceedings of the 12th Int. IPBSA conference, Sydney, 14-16 November, Int. Building performance Association, pp.2843-2850

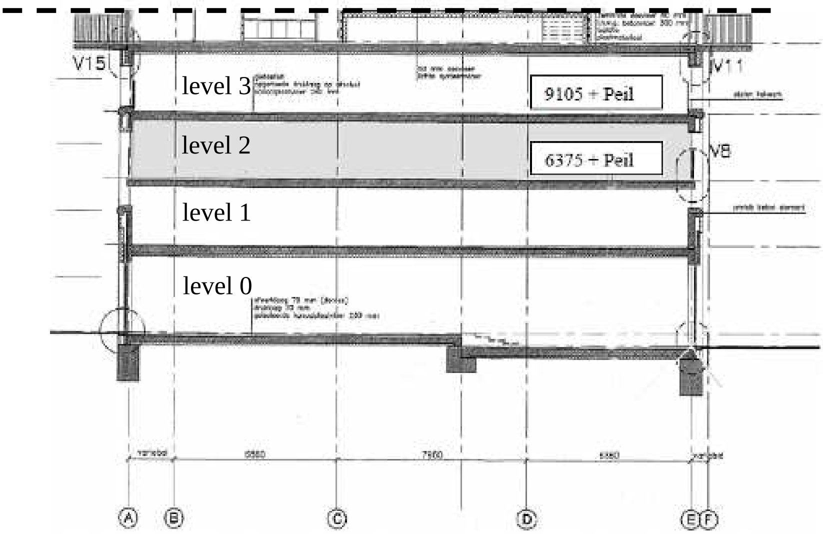
[5.29] Dodson, D.W.. The art of reading smoke. Fire Engineering, 09/01/2005, available at http://www.fireengineering.com/articles/print/volume-158/issue-9/features/the-art-of-reading-smoke.html accessed on 8 August 2013

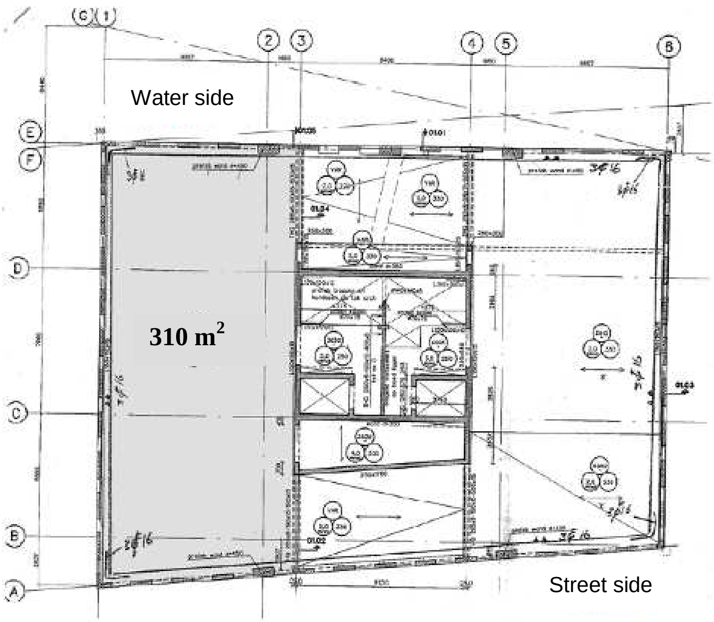
[5.30] Brouwer, E. Trainers corner; the science of reading smoke. Fire Fighting in Canada. Website http://www.firefightingincanada.com/content/view/2139/132/ accessed on 8 August 2013

[5.31] Jansze, W., Muntean, A. et al. Fire case Rotterdam lloydstraat – Further analyses on Efectis fire scenario 1 with softwares CaPaFi 2.0 and FDS 5, 25 September 2013 (revised 3 December 2013), #12817620, p. 64

Appendix 5.A – Description of structure of the building

"Harbour Edge" was in October 2007 a moderately tall new building in the street Lloydstraat, Rotterdam. The twelve storey building is used for housing on the levels 4 bis 11, while the lower part of the building (levels 0 bis 3) contain a open car park for 60 cars; level 2 had 10 parking places. The parking garage consists of 7 floors that with a height difference of about 1.5 m are situated on the left-hand and right-hand side of the lift shafts. The height of the ceiling in the car park is about 2.4 m. The floors 0 bis 3 consisted of hollow core slabs, while the floors of 4 bis 11 consisted of filligrans. The parking is a socalled open parking garage with natural ventilation. The full 2100 m^2 of the garage was one fire compartment.



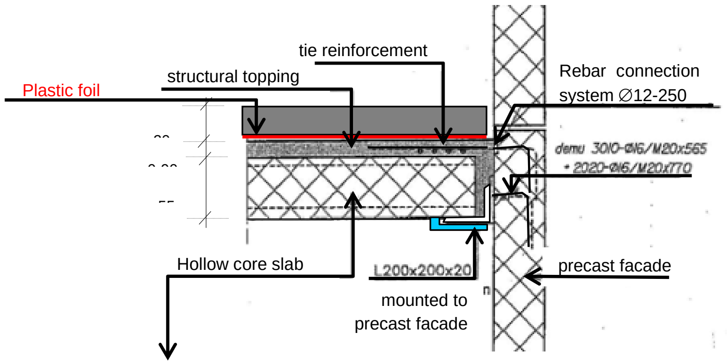


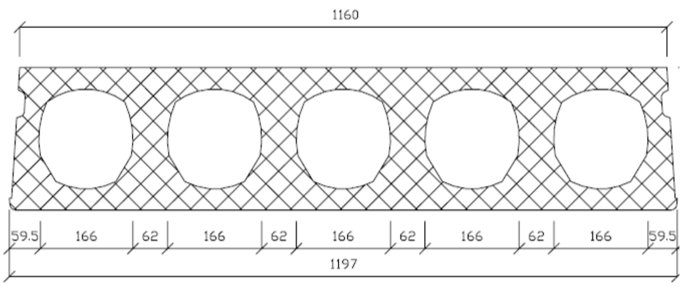
The hollow core floor consisted of 5-core hollow core slabs with 70 mm tot 90 mm structural reinforced topping and a finishing asphalt layer of 120 mm to 70 mm. The hollow core slab was prestressed with 10 strand of 12.5 mm (A_p = 930 mm^2) with an axis distance of 46 mm. Two top strands were applied in the slab. In the building permit of the building it was required that the hollow core floor had a fire resistance time of 120 minutes [3]. In addition, the floors are part of the overall structural system as it assures the cooperation between the load bearing external walls and the core of the building [3]. The fire took place on level 2, and the ceiling of level 2 (floor of level 3) was damaged.

The supplier of the hollow core slabs made the structural calculations of the hollow core slab. The span was about 10.5 m. A structural topping of 75 mm was accounted for. The following loads were taken into account:

- structural topping: 1.80 kN/m^2
- Hollow core slabs 260-5: 3.70 kN/m^2
- Finishing: 1.20 kN/m^2
- Total dead load: 6.70 kN/m^2
- Extreme live load: 2.00 kN/m^2
- frequent/quasi-permanent value: 0.7

At the location of the fire, the hollow core slabs were supported by a steel L-section. This steel L-section was protected against fire with fire-resistant plates of 12-15 mm [1]. The structural topping ranged between 70 mm to 90 mm [6]. On the structural topping an asphalt layer was applied in order to drain the water. This asphalt layer is 120 mm thick at the external wall, and reduced till about 70 mm at a distance of 1 to 2 m from the elevator. A plastic foil was desinged between the structural topping and the asphalt layer [3]. It is unknown whether this was applied.



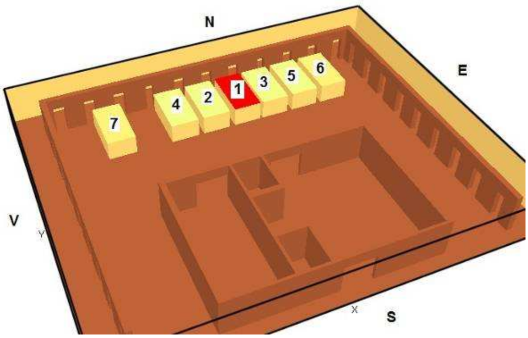


Appendix 5.B – FDS5 simulation

The Fire Dynamics Simulator version 5.5 (FDS5) is used to simulate the fire development for the scenario considered. FDS is a computational fluid dynamics (CFD) model of fire-driven flow developed and maintained by the American National Institute of Standards and Technology (NIST). The code solves numerically a form of Navier - Stokes equations appropriate for low-speed, thermally-driven flow with an emphasis on smoke and heat transport from fires. The formulation of the equations and the numerical algorithm are contained in Fire Dynamics Simulator (Version 5) – Technical Reference Guide.

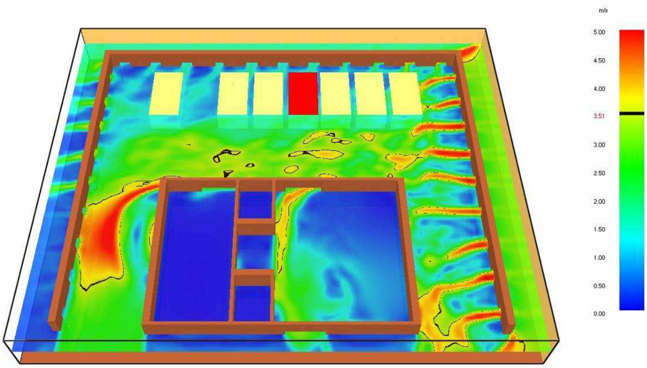
Compartment geometry and computational domain

The fire compartment was considered the area in the north part of the parking garage, from axis 1 to 4, dimensions of 24.84m × 20.70m – with two large openings (ramps) of 4.70m wide × 2.73m height and one door of 2.15m wide × 1.95m height in the south part of the parking garage. Ceiling was also modelled but for visualisation purpose it will not be shown in the snapshots presented. Also, all the openings (0.74m wide × 1.94m height) from the exterior walls were taken into account. Another assumption made in the input file was that to consider the doors from the lift shafts opened from the beginning, based on the observations that these had broken immediately after the fire had started. Hence three additional opened doors with the dimensions of 2.15m×1.95m were modelled. The computational domain was modelled as one grid mesh of 20×20×20cm3 cell sizes for the entire fire compartment.



Environment conditions

Another important parameter considered in the input file for the scenario 1 simulation is the wind. During the Rotterdam fire, an intensive wind blew from the ENE direction at an angle of 60 degrees, with 3.5 m/s mean velocity. From the beginning until the end of the simulation, the same wind velocity (3.50m/s) and exterior conditions (night temperature T=8°C and relative humidity RH=90% [7]) were considered. The next Figure presents snapshots of the wind movement inside the compartment.



Slice wind velocity [m/s] snapshot at the beginning of simulation

Fire / burner

The same burning conditions as in the case of the Efectis and TNO CaPaFi calculations for fire scenario 1 were considered in the FDS5 simulation. The following Figure present the input heat release rate considered in the model and the output HRR given.

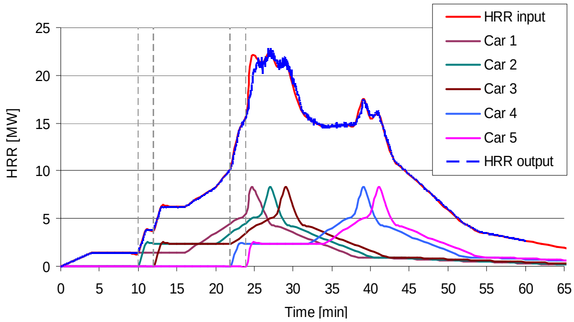
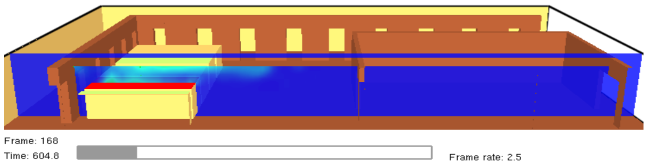


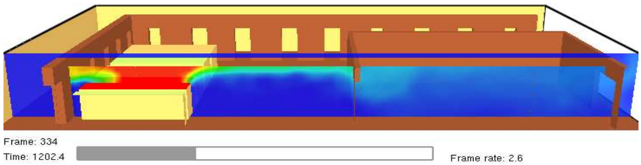
Table. Properties of the materials used in the FDS5 model

Property	Value		
	concrete	Commnon brick	Air
Density (kg/m3)	2500	1600	1.264
Heat conductivity (W/mK)	EN1992 – eq. (X)	0.69	0.0249
Specific heat capacity (J/kgK)	1.00	0.84	1.005
Emissivity	0.70	0.90	
Thickness (mm)	220 (for concrete wall) 250 (for precast wall) 330 (for slab)	100	50

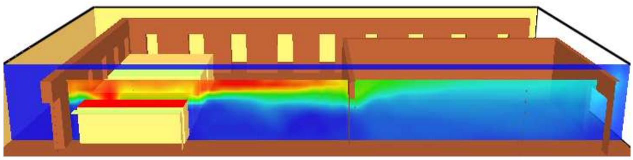
Temperature development above car 1



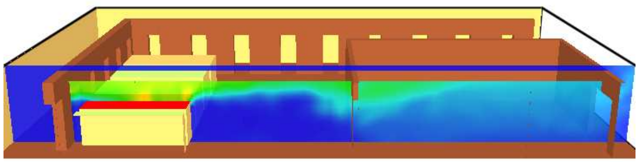
After 10 minutes



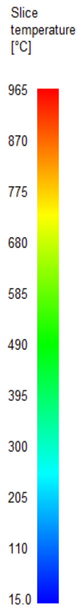
After 20 minutes



After 30 minutes



After 45 minutes



Appendix 5.C – Integral text of review by Dr. G. Rein of Imperial College

Review on background report [31] "Fire case Rotterdam Lloydstraat: Further analyses of Efectis fire scenario 1 with softwares CaPaFi 2.0 and FDS5"

Imperial College London

Department of Mechanical Engineering
Imperial College London

Mechanical Engineering Bldg, Room 711
South Kensington, Exhibition Road
London, SW7 2AZ
Tel: +44 (0) 207 594 7036

g.rein@imperial.ac.uk
www.imperial.ac.uk/people/g.rein

2 Dec 2013

Dr Guillermo Rein
Senior Lecturer

I have been asked by BIBM HOLCOFIRE through CERIB to provide an independent review of the following report:

Fire case Rotterdam Lloydstraat
Authored by Wim Jansze, Andreea Muntean, Arnold van Acker, Bruno Della Bella, Ronald Klein Holte, Gosta Lindstrom, Hermann Benhoefer, Jean-Paul Py and Matthieu Scalliet, from BIBM Product Group Hollow Core, 25 September 2013, Document: CNSDM #12817620.

This report studies the accidental fire that took place in *Harbour Edge* building at Lloydstraat, Rotterdam, on 1st October 2007. It happened overnight and involved five vehicles stationed inside a semi-open, concrete, car park. The report studies the severity of the fire conditions generated on the concrete elements of the ceiling.

This letter states my views and recommendations.

Independent views on the report:

1. The report poses four questions that it aims to answer. It is my opinion that the four questions were answered reasonably, transparently and with a justified approach.
2. The first question revisits the previous modelling studies of this fire conducted by Efectis and TNO, and estimates the input data used in their model. It is my opinion that BIBM found reasonable input data that reproduces correctly the previous modelling work. This should further be confirmed with Efectis and TNO since they have published their modelling results in a scientific conference, and scientific work must be reproducible. Hence, Efectis and TNO should be open to provide the full input data to their simulations and double check on the findings of BIBM.
3. The second question compares the modelling results of a simple analytical model that assumes a local fire (CaPaFi) to the results of a comprehensive model based on computational fluid dynamics (FDS). The comparison of both models shows that the differences are not dramatic but fall within comparable bounds for the peak temperatures. FDS provides more details of the thermal environment and is in general more accurate and reliable than CaPaFi.
4. The third question investigates the effect of a more realistic and correct scenario for the fire modelling by introducing the actual wall layout of the car park and wind conditions. The results show a dramatic effect of these scenario improvements, which led to a much larger area of the concrete structure being exposed to fire severity. The importance of introducing these scenario improvements is shown to be vital to understand the fire at Lloydstraat. The lack of these improvements in the earlier work of Efectis and TNO severely dismissing the reliability of their previous results.
5. The fourth and last question attempts to compare the thermal conditions during the Lloydstraat fire to those during a standard ISO 834 furnace test. This comparison is much sought after since it would allow translating furnace test results to the real scale of a building, but unfortunately, the current state of the art of fire safety engineering does not allow reliably to make such translation. Time resistance in furnace testing cannot be translated to time resistance to real fires because the conditions in a furnace are very

Imperial College of Science, Technology and Medicine different from the conditions in a fire (in time and space). Moreover, the Lloydstraat fire affected a whole structural system, whereas furnace testing only affects an isolated structural member. It is well known in fire safety engineering that the response of one single isolated member differs significantly from the response of a whole structure. This is a shortcoming faced by all fire safety communities. The BIBM report does a good job attempting to compare the fire conditions to test conditions but it is my opinion that this comparison cannot be made reliably at the present moment by only looking at the fire temperatures.

6. However, having said this last point, it is my opinion that based on the very high and rapid temperature increase predicted by the models (peak temperature ~900˚C in 20 min), the duration of the fire (>45 min) and the very large area of the concrete structure affected (seen when wall and wind effects are considered) that it is likely that the fire was more severe, extensive and mechanically more onerous to the ceiling structure than a 120 min of ISO 834 furnace testing on an individual element.

Recommendations for future revision of the work:

1. Calculate the resulting temperature of the concrete elements, not just the fire temperatures.
2. Calculate the structural response of the concrete structure to this heating. This will allow understanding the reasons for the failure, and differentiate whole-structure response from individual-element response.
3. Use only the gas-phase temperature to discuss the fire conditions. Do not use the adiabatic surface temperature, and do not compare this to the gas-phase temperature so to avoid misleading comparisons.
4. Extend the number of scenarios under consideration to five or so, including different resulting fire power, car fire sizes, and spread rate of the fire to different cars.
5. Conduct a sensitivity analysis of the wind conditions (velocity and direction).
6. Investigate the role of the water jets used for suppression by the Fire Service. In particular, the very rapid cooling (flash chill) of the hot concrete elements might have played a significant role in the local structural failure of concrete members.

Sincerely yours,

Dr Guillermo Rein
Imperial College London

Imperial College of Science, Technology and Medicine

6

Chapter Six

Restrained Conditions

Fire tests to validate the load bearing capacity under restrained conditions

Keywords: *fire tests, floors, restraints, horizontal web cracking, buckling spalling*

Abstract. *In the 2007 Rotterdam fire case local damage and delamination of the underflange of a part of the hollow core slab floor took place. A research conducted by Holcofire concluded that both horizontal cracking and buckling spalling are attributed to high restraints in the floor. In order to understand whether the fire resistance of the floor is assured, four fire tests were conducted with high internal floor restraints in order to provoke buckling spalling and horizontal cracking to understand the phenomena better. In addition, the fire tests should also give more information on the fire resistance time after these phenomena occurred, and additional information on the capacity of the floor after a fire time of 90 minutes. Four fire tests were conducted; R1 to R3 spanned in the length direction of the furnace (5,9 m), while R4 was spanning in the short direction of the furnace (3,9 m). R1 and R2 were conducted on 255 mm and 260 mm deep slabs, respectively, with 100 mm topping. In R2 horizontal jacks were used to simulate the continuity of the floor. R3 was conducted with 200 mm deep slabs and a 50-70 mm topping. R4 was executed with 265 mm slabs without a structural topping. Due to the high restraints at the supports in all test set ups, buckling spalling occurred in all fire tests. Horizontal cracks were initiated in R1 to R3. Nevertheless, the fire test results showed that with a design live load of 1.4 kN/m^2 a fire resistance time of*

90 minutes can be achieved. Even the bending capacity still equalled the theoretical bending capacity at 90 minutes as a result of structural redundancy. In fire test R1 the live loading with 13.3 kN/m^2 was high in order to obtain the same bending moment in the test as in the Rotterdam fire. This however led to a shear-bending interaction failure at 37 minutes. In fire test R4 at 56 minutes in one slab an open hole occurred in the top flange, and the fire test was stopped as EI was not fullfilled. Buckling spalling took place in one slab due to high restraints in the R4 floor, but the other slabs in the floor were hardly affected by the fire. Overall it was concluded that high floor restraints due to internal restraints (structural topping and support beam) can lead to buckling spalling and horizontal cracking, but these are concluded not to be failure mechanisms, as under accidental design loads the fire resistance time is still met by virtue of structural redundancy in the hollow core slab floor.

6.1. Introduction

Considering the past overall performance of the total estimated stock of installed hollow core floors nowadays in Europe of about 1000 million square meters, precast concrete hollow core floors possess a high fire resistance and a large passive redundancy to fire because of their robustness and their capacity to redistribute the acting loading. No cases are known to the authors where hollow core floors structurally failed within the required fire resistance time with losses of life. One fire case heavily discussed is the fire that broke out on 1st October 2007 in the parking garage under the Harbour Edge Apartment Building in Lloydstraat Rotterdam, The Netherlands. Although the fire did not lead to collapse of the floor, the inner surface of the precast concrete facade and the soffit of the hollow core concrete floor were locally damaged. This local damage of concrete was felt as a problem in the society, as could be noticed by articles and discussions. Concrete is perceived fire resistant and damages should not occur as that could hazard the health and safety of the fire men during a fire fight. In design practices, safety with respect to fire is achieved by specifying some safe value at the loading side (duration of the fire) in combination with the recognition that fire in itself has a low probability of occurrence.

Fire cases with the local damage like Rotterdam are rare, and the phenomena of local damage are rarely observed in fire tests. The Holcofire database on prestressed hollow core fire tests that covers a period of 45 years from 1966 until 2010 [6.1] contains only 5 fire test in which explosive spalling led to failure or a hole in the slabs, and 4 fire tests on hollow core slices in which horizontal cracking occurred. The review of the Holcofire database by Prof. Walraven and Prof. Vrouwenvelder states that “the conclusion in the BIBM report is that the majority of the models describing common failures give satisfactory results. In spite of the large selection of tests with a wide scope of influential parameters, some questions remain open. This refers especially to the mechanisms of horizontal crack formation and explosive spalling. In the report it is recommended to focus on the effect of restrained deformation on horizontal cracking and of explosive spalling in upcoming research.” This Chapter addresses four fire tests R1 to R4 with restrained deformations conducted under the R series, and should be read together with Chapter 7 [6.8].

6.2. Objective of Holcofire test series R

The local damage in the Rotterdam fire case is due to a combination of negative influences: local severe travelling fire; restraint from thick topping; restraint from blocking of longitudinal and lateral thermal deformations (see Figure 6.1); restraint from important hogging moment due to heavy connecting reinforcement between the wall and the floor at the level of the topping; young concrete with high moisture content; and decrease of the mechanical characteristics of the concrete at the fire exposed bottom flange. Moreover, after 45 minutes a fire boat with high water capacity was used to extinguish the fire from outside and forced the structure to cool down quickly.

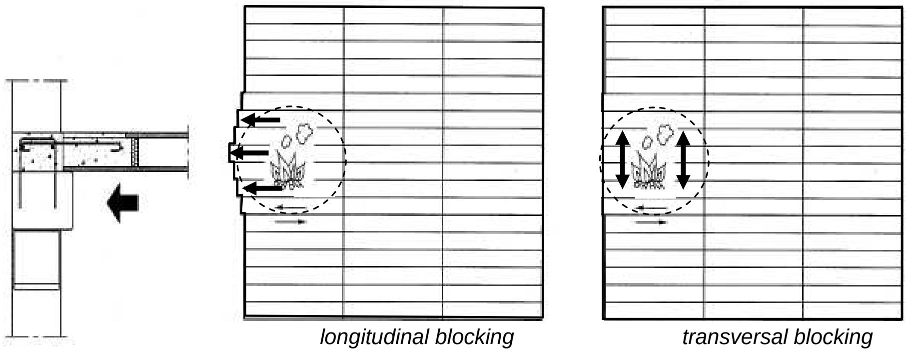


Figure 6.1 Blocking of the thermal expansion of a floor by structural topping and surrounding structure

But when looking at the damages after the real fire case of Rotterdam, it is a fact that local damage is visible by open cores and partly delamination. It is believed that these open cores and delamination are a combination of explosive spalling, buckling spalling and horizontal cracking through the webs induced by restraints under fire conditions. Blocking in span direction will have a positive effect on the shear behaviour (conclusion Holcofire G series), but a certain high level of restraints in transversal direction could cause a negative effect on the compressive stresses in the bottom flange of the hollow core. All these phenomena and influencing parameters need to be studied more in details in order to make firm conclusions. Hence, like an elk test crash in car industry, the R series has to provoke in a fire test local damage in the slabs to enable research on horizontal cracking and spalling. Hence, the objective of fire test series R is to investigate the influence of restrained conditions in hollow core floors under fire conditions and to provoke spalling on the soffit and horizontal cracking through the webs. The restraint is simulated by horizontal transversal blocking in function of some design situations, i.e. floor layout, stiffness of support beam, structural topping thickness, type of edge structure, age of slabs, shrinkage of concrete, etc.

6.3. Fire resistance according to EN1992-1-2:2004 and EN1168:2005+A3:2011

EN1992-1-2 considers only bending and spalling, and EN1168:A3 [6.3] considers bending, shear and anchorage, and spalling. The bending capacity of a hollow core slab exposed to fire may be calculated by using simplified calculation methods according to EN 1992-1-2 clause 4.2, or can be assessed by tabulated data given in EN1992-1-2 [6.4]. EN1168 contains the informative Annex G that gives guidance to calculate the resistance to fire of hollow core slabs. Regarding spalling, mainly EN1992-1-2 can be used as a reference. In this design standard in clause 4.1 it is written that spalling shall be avoided by appropriate measures, or the influence of spalling on performance requirements (R and/or EI) shall be taken into account. In clause 4.5 it is indicated that explosive spalling is unlikely to occur when the moisture content of the concrete is less than k % by weight: the recommended value of k is 3. It may be assumed that where members are designed in accordance with the requirements for exposure class X0 and XC1, the moisture content is less than k % by weight, where $2,5 \leq k \leq 3,0$. Above k % a more accurate assessment of moisture content, type of aggregate, permeability of concrete and heating rate should be considered. [Note: This statement is not valid for hollow core floors only, but also for other concrete elements in precast and cast in-situ. Parking garages where exposure class XC3 is applicable (high humidity) have a moisture content above k.] EN1992-1-2 clause 4.5 states further that for floors, if the moisture content of the concrete is more than k % by weight, the influence of explosive spalling on load-bearing function R may be assessed by assuming local loss of cover to one reinforcing bar or bundle of bars in the cross section and then checking the reduced load-bearing capacity of the section. It is noted that where the number of bars is large enough, it may be assumed that an acceptable redistribution of stress is possible without loss of the stability (R). This includes solid slabs with evenly distributed bars. Falling off of concrete in the latter stage of fire exposure shall be avoided, or taken into account when considering the performance requirements (R and/or EI).

6.4. Experimental design of Holcofire fire test series R

The Cerib Promethee furnace measures 4 m by 6 m on the internal dimensions. Four floor assemblies R1 to R4 with specific boundary conditions are tested. The standardized configuration of the test set-up described in EN1168:A3 is used as a basis for the test series R. For fire time, 90 minutes is targeted in R1 to R3, while 120 minutes is the target in R4. The load on the floor is normal (R2, R3, R4) to high (R1). The load is applied by 1-point, 2-point or 3-point loading scheme. The transversal support beam was insulated during the fire in R1 and R2 in order to have higher restraints in transversal direction, but unprotected in fire R3 and R4. All four tested floors are connected to the supporting beams with 1 tie bar ø 12 mm anchored in each longitudinal joint.

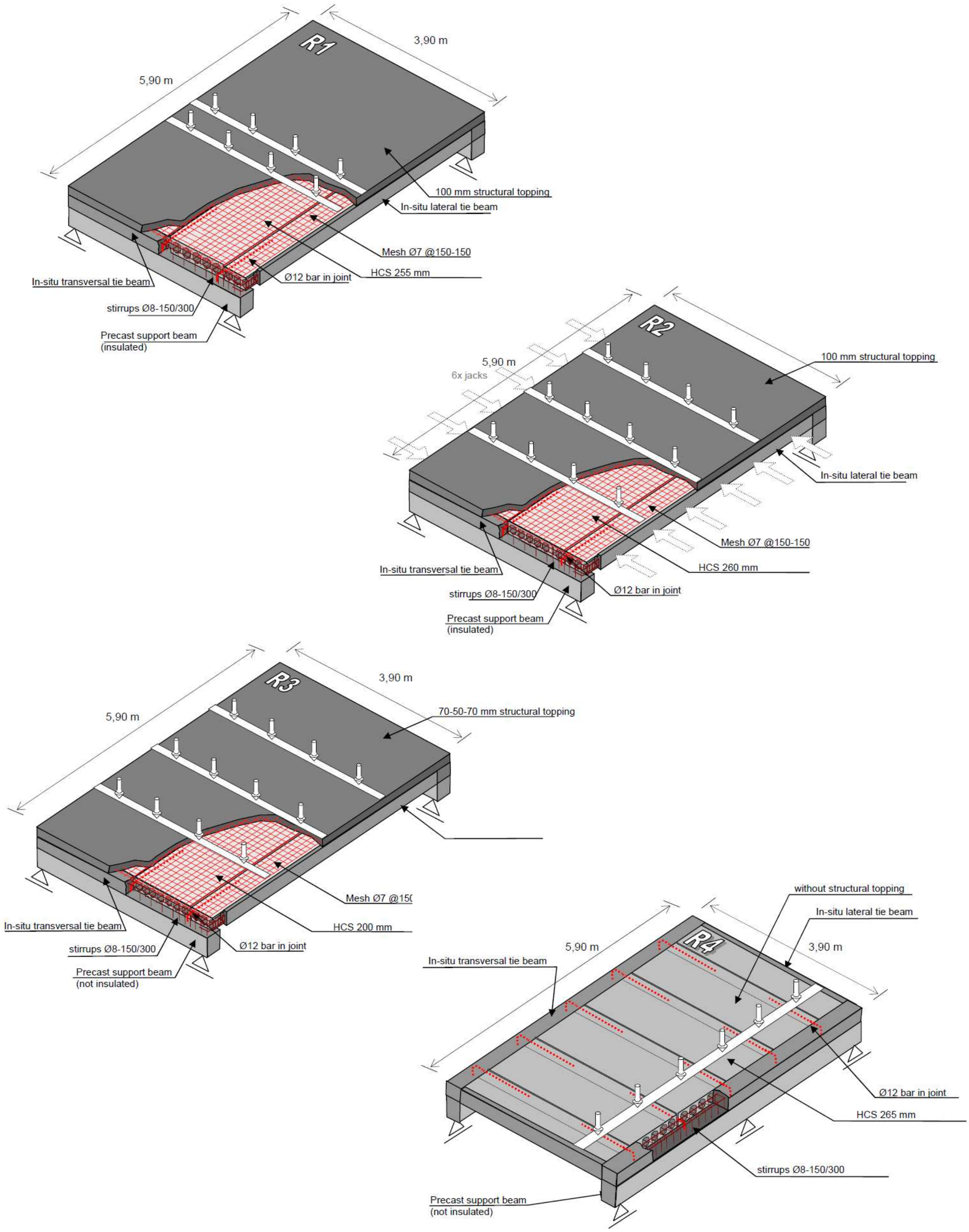


Figure 6.2 HOLCOFIRE series R – Overview of floor geometries R1 to R4

The following fire tests have been designed for R-series. More details are given in Figure 6.2 and Figure 6.3. Table 6.I overviews the fire tests, the chosen parameters and their values in order to study the restraints:

- Fire test R1: This fire test consists of a floor assembly with 255 mm deep slab (10 x 12.5 mm strands) with 5 cores and finished with a 100 mm structural topping. A peripheral tie beam is cast around the floor with 2∅10 (support) and 2∅12 (lateral tie beam) mild bar reinforcement. In order to simulate some blocking in transversal direction, the support beam is insulated. The floor assembly is freely supported by 4 columns in each corners. The precast support beam measures 300x400 mm^2. The load on the floor is such that an equal bending moment (but 30% higher shear load) as in Rotterdam was used; for that the live load by jacks equals 13.3 kN/m^2.

- Fire test R2: This fire test consists of a floor assembly with 260 mm deep slab (8 x 9.3 mm strands) with 7 cores and finished with a 100 mm structural topping. A peripheral tie beam is cast around the floor with 2∅10 (support) and 2∅12 (lateral tie beam) mild bar reinforcement. In order to simulate full blocking in transversal direction, the support beam is insulated and hydraulic jacks are applied on the lateral tie beam simulating a continuous floor field in cold situation. For reasons of execution, the longitudinal tie beam is increased in height with 50 mm, see Appendix 6.B. The floor assembly is freely supported by 4 columns in each corners. The precast support beam measures 300x400 mm^2. The live load on the floor by jacks equals 1.4 kN/m^2.

- Fire test R3: This fire test consists of a floor assembly with 200 mm deep slab (8 x 9.3 mm strands) with 7 cores and finished with a 50 mm [due to camber 50 mm at midspan but 70 mm at support] structural topping. A peripheral tie beam is cast around the floor with 2∅10 (support) and 2∅12 (lateral tie beam) mild bar reinforcement. In this fire test, the support beam is not insulated in order to reduce blocking of the support beam. The floor assembly is freely supported by 4 columns in each corners. The precast support beam measures 300x400 mm^2. The live load on the floor by jacks equals 1.4 kN/m^2.

- Fire test R4: The fire test consists of a floor assembly with 265 mm slabs, spanning in the width direction of the furnace. The 265 mm hollow core has 6 x 12.5 mm strands with 5 cores and is without structural topping. The test floor is surrounded by a peripheral beam reinforced with 4 bars ∅10 mm. The whole floor assembly is supported by 6 columns, of which 4 in each corners, and 2 in the middle of the support beam. The precast support beam measures 300x300 mm^2 and is not insulated. In R4 the shear load equals the calculated shear resistance at 120 minutes. R4 was performed as the first fire test and was earlier numbered as G0 but in final publications renamed to R4.

Table 6.I Fire tests and parameters (nominal values) in HOLCOFIRE test series R

Fire test # Parameter	R1	R2	R3	R4
length of tested floor [m]	5.9	5.9	5.9	3.9
width of tested floor [m]	3.9	3.9	3.9	5.9
Height of slab [mm]	255	260	200	265
Strands [mm] and axis distance	10 x 12.5 / 50	8 x 9.3 /44	8 x 9.3 mm	6 x 12.5 mm
Upper strands [mm] axis distance	2 x 5 / 205	5 x 5 / 222	2 x 5 / 165	-
Structural topping in mm	100	100	50 (50-70)	0
Reinforcement topping	Ø7.0-150/150	Ø7.0-150/150	Ø7.0-150/150	-
Protruding strands in mm	0	0	0	0
Connection reinfo per slab	0	2Ø10	1Ø12	0
Shape connection reinfo	Ø12 bar in joint	Ø12 bar in joint	Ø12 bar in joint	Ø12 bar in joint
Support beam [mm^2]	300x400	300x400	300x400	300x300
Vertical stirrup at support	Ø8-150-300	Ø8-150-300	Ø8-150-300	Ø8-150-300
Transversal tie beam [mm^2]	200x355	200x410	200x300(320)	200x265
Transversal tie beam bar [mm^2] 1)	2Ø10 + 1Ø12	2Ø10 + 1Ø12	2Ø10 + 1Ø12	3Ø14
Lateral tie beam [mm^2]	150x355	150x410	150x300(320)	200x265
Lateral tie beam bar [mm^2]	2Ø12	2Ø12	2Ø12	4Ø10
Type of load on floor	2-point bending	3-point bending	3-point bending	1-point shear
Moment $M_{Rd,c,fi,90}$ [kNm/slab]	300	119	77	118
Annex G $V_{Rd,c,fi,90}$ [kN/slab]	94.4	77.9	66.3	52.0

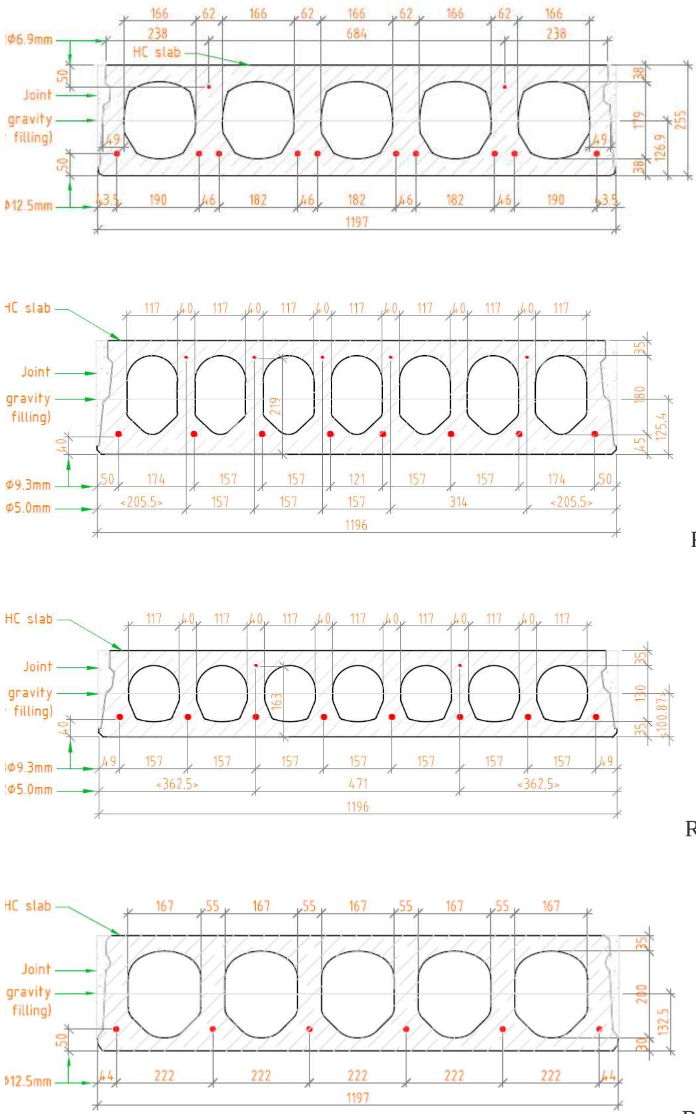
1) shear reinforcement in lateral tie beam consisted of stirrups ø6-200 mm

6.5. Hollow core slabs and floor assembly

The hollow core slab cross sections used in the fire test are depicted in Figure 6.3 (see Appendix 6.A for more details). The slabs were cast with concrete grade C55/67 and C45/55 and siliceous and calcareous aggregates. The following mean cylinder strengths are calculated with 0.833 transformation factor: R1: 57.6 N/mm^2 after 27 months; R2: unknown; R3: 42.4 N/mm^2 after 11 months; and R4: 67.5 N/mm^2 after 4 months. The hollow core slabs were first stored inside the factory and then transported to the fire test laboratory. There, the slabs were further stored under controlled conditions (20ºC, 50% RH). The test floors were assembled one month before test date in order to enable the jointing material to harden. After the floor was assembled, test floors were further stored indoor under 20ºC, 50% RH in the climate room. The following moisture contents have been determined on identical slabs:

- R1: 5 July 2012: 1.7%, 1.5%, and 2.4% at center of underflange;
- R2: 22 November 2012: 2.2%, 1.5%, and 1.7% at center underflange;
- R3: 14 December 2012: 1.8%, 3.4%, and 2.8% at center of underflange;
- R4: 16 December 2010: 3.0% and 2.9% in underflange, 2.5% at web, and 3.2% at upperflange.

The concrete grade used for the joints and topping was C25/30, the maximum diameter of aggregate 8 mm, and slump classification S5/S4. Vibration was not used. The floor topping and the peripheral tie beam were a C25/C30 concrete grade, with D_{max} = 16 mm and slump classification S3 (normal concrete). Vibration was used. The Ø12.5 and Ø9.3 strands used for the hollow core slab are of FeP1860 quality. The characteristic value of the steel reinforcement bars was assumed f_{yk} = 500 N/mm^2.



R1: 255/5-X10-D2

R2: 260/7-S8-D5

R3: 200/7—S8-D2

R4: 265/5-X6

Figure 6.3 Hollow core cross sections used in the fire tests R1 to R4

6.6. Fire tests R1 to R4 with ISO 834 fire

The fire tests were executed in Cerib on the following dates: R1 on 5 July 2012, R2 on 22 November 2012, R3 on 12 December 2012, and R4 on 16 December 2010. One day before the fire test the floor assembly is preloaded in order to settle the specimen, and to initiate cracks where the tensile strength is exceeded to simulate usage. For preloading the same load as in the fire test is used.

Fire test R1: In fire test R1 a load of 280 kN was used on a floor of 3.9 x 5.4 m^2, which induced a live load of 13.3 kN/m^2. This high load was choosen in order to have the same order of magnitude bending moment and (30% lower) shear load as in Rotterdam fire case. The fire in the fire test was stopped at 37 minutes in common deliberation. At 37 minutes the floor was not able to withstand the 280 kN load, and the maximum deflection as defined by EN1363-1 was exceeded. At about 14 minutes a horizontal crack grew through the web at the location of core camera 2. Between 23 and 37 minutes this crack opened further, and led to a shear-bending interaction failure at 37 minutes. In Figure 6.4 one can see that the soffit was delaminated from the floor. But after a thorough visual analysis it emerged that the failure type was by shear-bending interaction. This was initiated by a horizontal crack in the second web, but led subsequently into a combined shear-bending crack at the level of the strands leading to a separation of the lower part of the floor from the top part with topping. It can be concluded that shear-bending interaction was the failure mechanism.

Fire test R2: Opposite to fire test R1, in fire test R2 on the floor the real load of Rotterdam will be applied: 0.7 x 2.0 = 1.4 kN/m^2. In addition, to simulate external restraints, horizontal jacks were used. The test set-up has on each longitudinal side 6 jacks with spacing of 1 m. The capacity per jack used is 250 kN and loading was at half height of the 45 mm underflange. In the execution of the load in the horizontal jacks, a different scheme was used anticipating on the results during the test. 30 minutes before the fire tests started 5 kN per jack was applied (30 kN on the floor). Then, at 10 minutes the horizontal loading was increased. But due to horizontal cracking in the test, it was decided at 21 minutes to decrease the horizontal load from 300 kN (50 kN/jack) to 0 kN. The horizontal displacements were also registered. The fire in the fire test was stopped at 91 minutes in common deliberation. The floor was still able to withstand the live load. Figure 6.4 shows the soffit of the test floors after the test. After the initiation of horizontal cracks at about 12 minutes, the floor delaminated further into the fire test. A part of the underflange of the hollow core slab had fallen down at the end of the test. The floor did not collapse and the fire criteria R, E and I were met at 91 minutes. In one slab not all the strands were anchored anymore in the support. This slab is seen clearly in Figure 6.4; the soffit is open, and the strands have been exposed to the fire. At all other locations the strands are still more or less covered by concrete and are still anchored into the support area. The anchorage of the strands at the support is important in order to sustain the load during the fire test. After the end of the fire test, the vertical load only in the centreline was built up between 91 and 132 minutes from 30 to 291 kN. The floor reached the ultimate capacity at 291 kN at 132 minutes with bending moment at midspan 157.3 kNm/slab. The calculated capacity from lower strands and top strands is 118.7 kNm/slab and 21.8 kNm/slab, respectively. Hence, the capacity at 132 minutes was 157.3/140.5 = 112% larger than the theoretical bending capacity at 90 minutes. It can be concluded that buckling spalling and horizontal cracking are not failure mechanisms.

Fire test R3: Like R2, in fire test R3 on the floor a load of 1.4 kN/m^2 is applied. In the fire test R3 there were no horizontal jacks placed. But horizontal displacements were registered. In contrast to R2, also the displacements at the support beam were registered. The fire in the fire test was stopped at 91 minutes in common deliberation. The floor was still able to withstand the live load. Figure 6.4 shows a photo of the soffit after the test. After the initiation of horizontal cracks at about 13 minutes, the floor delaminated further into the fire test. A part of the underflange of the hollow core slab had fallen down at the end of the test. The floor did not collapse. The fire criteria R, E and I were met at 91 minutes. Note that all the strands were still fully anchored in the support. After the end of the fire test, the vertical load in the centreline was built up between 91 and 112 minutes from 30 to 119 kN. The floor reached the ultimate capacity at 119 kN at 112 minutes with bending moment at midspan is 78.3 kNm/slab. The calculated capacity from lower strands and top strands is 76.6 kNm/slab and 5.4 kNm/slab, respectively. Hence, the capacity at 112 minutes was 78.3/82.0 = 95% , which is slightly lower than the theoretical bending capacity at 90 minutes. It can be concluded that buckling spalling and horizontal cracking are not failure mechanisms.

Fire test R4: Fire test R4 had a shear load, like the G-series. One line load is applied at 2.5h distance from the theoretical support. At 21 minutes after the start of the fire test a loud bang was heard, but the test continued. At about 45 minutes cracks were observed in the second slab on the top side in transversal direction, i.e. perpendicular to the span. At 56 minutes again a loud bang was heard and an open hole occurred in the floor. A shear failure did not occur. The fire in the fire test was stopped at 56 minutes in common deliberation: as the flames passed through the floor, due to safety reasons the fire test was stopped. The floor was still able to withstand the 52.3 kN/m lineload, but in one slab a hole was present through the slab (Figure 6.4). Some edges at the soffit of other slabs also showed some edge spalling, but nothing more. It was decided not to load the floor to failure, but to investigate for horizontal cracking. For that, after one day the floor was dismantled, and all slabs were sawn in order to investigate whether there were horizontal cracks. It was however concluded that the other slabs did not have any other horizontal cracking. Accordingly, it is believed that the other slabs would have easily succeeded in 120 minutes fire resistance time, but as said, due to safety reasons the fire test was stopped. But it can be concluded that buckling spalling is not a failure mechanism.

Figure 6.4 shows the local damages at the soffit of the four tested floors. The maximum deflection at midspan of the floor for the respective fire tests is given here. But as the fire tests consisted of various parameters, it is difficult to make a conclusion on the deflections of the floors.

The deflection during the fire tests at certain time is:

- R1: 15 min = 30 mm, 30 min = 78 mm;
- R2: 15 min = 20 mm, 30 min = 32 mm, 60 min = 41 mm, 90 min = 51 mm;
- R3: 15 min = 46 mm, 30 min = 68 mm, 60 min = 93 mm, 90 min = 125 mm;
- R4: 15 min = 15 mm, 30 min 21 mm, 56 min = 25 mm.

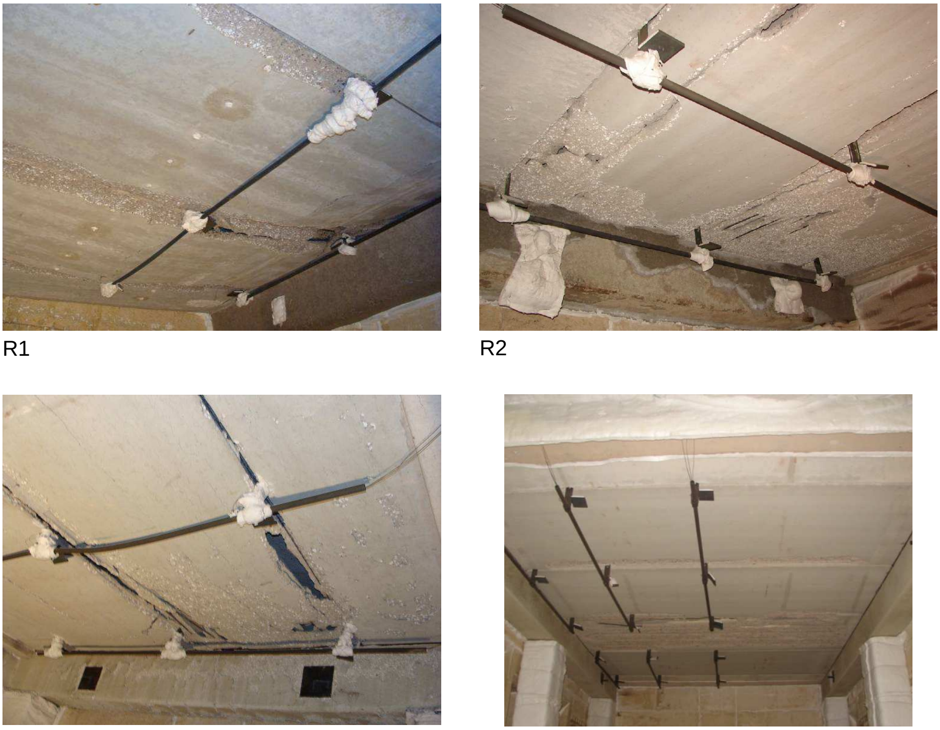


Figure 6.4. Photos of local damage at the soffit of the floors one day after the fire test was executed

Table 6.II Temperatures in the strands in time

	R1	R2	R3	R4	Theory	
	50 mm axis distance	44 mm axis distance	44 mm axis distance	50 mm axis distance	50 mm	44 mm
Time [min]	Temperature [°C]	Temperature [°C]	Temperature [°C]	Temperature [°C]	T [°C]	T [°C]
15	50, 50, 63, 69	73, 104, 40, 64	38, 86, 97, 54	50, 52, 55	65	79
30	92, 98, 100, 122	74, 308, 166, 272	224, 283, 98, 101	105, 120, 155	110	137
(37)	93, 98, 97, 124					
(56)	x			200, 205, 400		
60	x	117, 407, 463, 552	320, 500, 255, 313	x	230	272
90	x	678, 652, 591, 653	647, 577, 400, 485	x	320	365

Table 6.III Temperature at h/2 in the web of the slab

	R1:	R2:	R3:	R4:	Theory	
	127 mm from soffit	130 mm from soffit	100 mm from soffit	132 mm from soffit	125 mm	100 mm
Time [min]	Temperature [°C]	Temperature [°C]	Temperature [°C]	Temperature [°C]	T [°C]	T [°C]
15	25, 26, 27, 28	21, 21, 21, 20	26, 30, 27, 28	25	45	45
30	57, 65, 70, 71	59, 83, 68, 63	74, 78, 90, 87	35, 44, 48,56,78, 95	69	69
(37)	76, 79, 82, 85					
(56)				97,97,99,98, 182,221		
60		151, 140, 202, 150	128, 175, 205, 185		159	159
90	x	415, 369, 485, 819	236, 381, 404, 403	x	210	210

During the fire test the temperatures in the strands were monitored, see Table 6.II. From this Table can be concluded that at 15 minutes the temperature measured is in line with theory. For R1 this changed after the moment the slab failed, up to 37 minutes the temperatures are in line with theory. Due to shear-bending failure at 60 minutes there are differences compared to theory. In R2 and R3 the temperatures in the strands with 552 °C and 500 °C were much higher than theory at 60 minutes due to horizontal cracking. The average temperature at 90 minutes of R2 is 643 °C, while in R3 this is 527 °C. A general conclusion is that due to delamination in R1 to R3 the strand temperatures are higher than theory. In R4 only at one location the temperature was with 400 °C much higher.

During the fire test the temperatures at half height of the webs were monitored, see Table 6.III. From this Table can be concluded that at 30 minutes the temperature measured is in line with theory. Only from 60 minutes on at some locations the temperature is about 35% higher. At 90 minutes in R2 and R3 the temperatures at mid height of the slabs are significant higher, about double the theoretical value. The average temperature at 90 minutes of R2 is 522 °C, while in R3 this is 356 °C. It can be concluded that due to delamination the temperatures at half height of the webs are higher than expected to theory.

In all slabs the temperature at the top of the slabs and top of structural topping remained under 160 °C. Only in fire tests R4, although not measured there locally, the temperature must have been higher after the occurrence of the hole. Due to the open hole flames came out of it, and for that reason the fire test was stopped.

During the fire tests also the temperature in the longitudinal bar of the connection reinforcements between the slab and the support beam were measured. In all tests the connection reinforcement temperatures are below 350°C reinforcement bar temperature such that the tensile strength does not decrease. In R1 in the longitudinal bar at 37 minutes a temperature of 95 °C was measured. In R2 at 90 minutes temperatures of 21, 22, 297, 245° C were measured. In R3 47, 50, 377, 330 °C was measured at 90 minutes. Finally, in R4 the measurements indicated 58, 60, and 93°C. The temperatures in the meshes applied in the structural toppings of R1, R2 and R3 were measured. In R1 in the mesh at midspan at 37 min a temperature of 27 °C was measured. In R2 at 90 minutes the measured temperatures were 49, 41, 39, and 36 °C, while in R3 these were 105, 92, 103, and 94 °C. In R4 a structural topping and thus a mesh were not present.

In all fire tests in some slabs at some specific locations the slip of the strands was measured. The locations were randomly selected, and do not give the real reflection of all strands, but just give an indication. All slip measurements seem acceptable for fire tests:

- In R1 at 20 minutes the slip measured at 4 locations was 4.5 mm, 5.5 mm, 5.5 mm, and 7 mm.
- In R2 at 4 locations the slip was at 30 minutes 4 mm, 2 mm, 1 mm, and 1 mm. At 90 minutes these hardly increased to 4 mm, 3 mm, 1 mm, and 1 mm. But at 130 minutes (after the fire test was stopped and the test floor further loaded up to failure) sensor 3 and 4 increases from 1 mm to 12 mm and 15 mm respectively.
- In R3 the slip measurements at 3 locations indicated at 30 minutes a slip of 6 mm, 7 mm, and 12 mm. At 60 minutes this was 8 mm, 10 mm, and 18 mm. At 90 minutes the 18 mm slip increased to 21 mm.
- In R4 the slip measured at 56 minutes was 0 mm, -2.2 mm, 2,3 mm, and 4.5 mm measured at 4 locations.

6.7. A closer look at horizontal web cracking and buckling spalling

During the fire tests cameras were installed at specific locations in the cores in order to study the initiation of horizontal cracks more thoroughly. From outside the furnace cameras were installed at several positions to monitor the top side of the floor. And at the furnace cameras were installed to study spalling at the soffit. This section gives some photos of R2 and R3 and observations of horizontal cracking and spalling during the fire test.

From fire test R1 it emerged that for camera 1 located in the 3rd core (at ½ span) at 6 minutes a crack in the underflange of the core was initiated. But then at 10 minutes this crack in underflange closed, and no further cracking was visible in this core. In camera 2 in the 1st core next to joint (at ¼ span) at 12 minutes the initiation of a vertical crack was observed. But then horizontal cracks developed between 14 minutes and 18 minutes. This crack fully opened at 37 minutes at failure due to shear-bending interaction. In fire test R2 in the 2nd core with camera 1 (at ½ span) at 18 minutes the start of vertical cracking was observed. But at 20 minutes horizontal cracks were initiated, that developed to a large horizontal crack at 22 minutes. In the core with camera 2 located in 1st core next to joint at ¼ span at 12 minutes a crack initiated in the underflange, see Figure 6.5. At 16 minutes a horizontal crack initiated, that grew at 18 minutes to a large horizontal crack, see Figure 6.5. Nevertheless, the floor was able to resist 90 minutes of fire. In fire test R3 in the 2nd core where camera 1 was located at midspan, at 13 minutes the start of horizontal cracking was observed, see Figure 6.6. At 15 minutes this was a large horizontal crack, see Figure 6.6. In the 1st core next to joint where camera 2 was located at ¼ span at 15 minutes a large horizontal crack had initiated.

Nevertheless, the floor was able to resist 90 minutes of fire. In floor R4 only 1 camera was located in the first full slab in the first core about 90 cm from the end of the hollow core head. No horizontal cracks were observed in this slab. At 15 minutes vertical cracking as in G series initiate in lower part of core at support, and developed to 30 minutes further in the span. At 56 minutes there was an open hole in the adjacent slab, and the fire test was stopped.

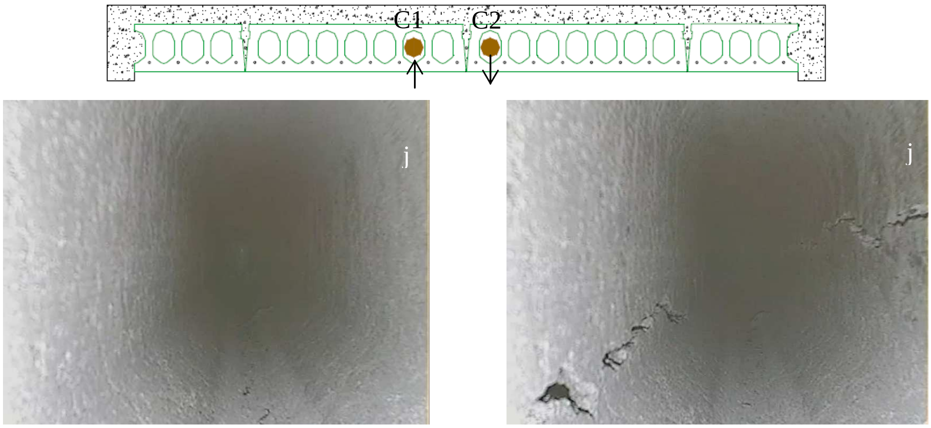


Figure 6.5 Core camera 2 in R2 at 12 minutes and core camera 2 in R2 at 18 minutes

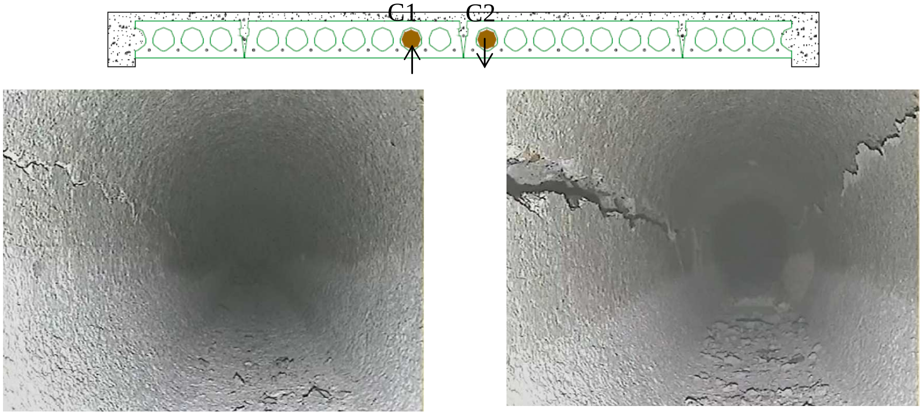
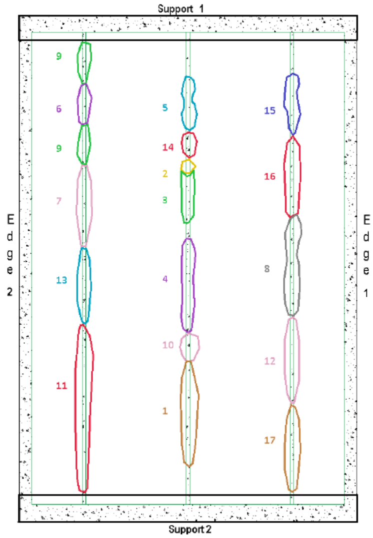
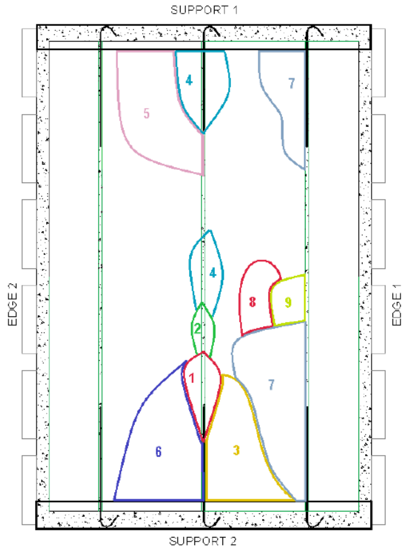


Figure 6.6 Core camera 1 in R3 at 13 minutes and core camera 1 in R3 at 15 minutes

The soffit of the floors was filmed with cameras from outside the furnace. Sketches of the soffit of the floors are given in Figure 6.7. Actually, only thermal spalling occurred at the edges of the slabs in R1. In the sketches of the spalling sequence of floor R2 and R3, we observe that in R2 part #1 fell off at 12 minutes. At 13 minutes #2 fell off, subsequently followed by #3 to #8 up to 22 minutes. In R3 parts #1 and #2 fell off at 13 minutes. At 14 minutes #3 fell off, subsequently followed by #4 to #7 up to 20 minutes. After 60 minutes parts #10 to #15 fell off. Figure 6.7 finally shows the soffit of R4. In slab 3 severe spalling took place, but in the slabs #2 and #1 only spalling at the edges can be observed.

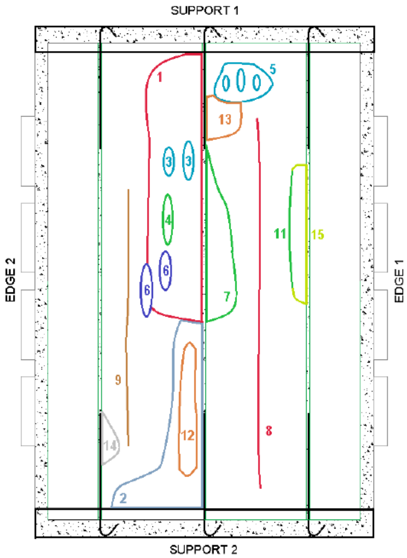


Spalling exposed side R1 ↑



Spalling exposed side R2 ↑

Spalling exposed side R3 ↓



Spalling exposed side R4 ↓

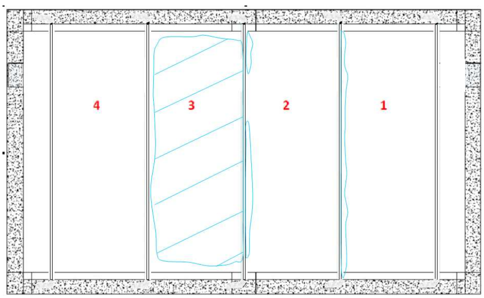


Figure 6.7. Sketches of spalling at the soffit of the tested floors registered the day after the fire test was executed

Earlier it was described that in R2 at 16 minutes a horizontal crack occurred and described that in R2 at 12 minutes buckling spalling occurred at the soffit – big pieces were pushed away from the soffit. Also, it was described that in R3 at 13 minutes a horizontal crack occurred, and described in R3 at 13 minutes buckling spalling occurred at the soffit. From these observations in time it must be concluded that both horizontal cracking and buckling spalling are the result of the same phenomenon, and thus that horizontal cracking and buckling spalling are somehow linked phenomena. In R1 this relation is not so clear: spalling occurred between 8 and 14 minutes, while horizontal cracks were initiated between 14 and 18 minutes. But there were only 2 core cameras, therefore in another core horizontal cracking could have started earlier. Spalling in R1 was also not buckling spalling, but more explosive spalling (small pieces). Also it is clear that in R1 spalling was only at the edges of the slabs near the joints (see also Figure 6.4), and did not take place at large parts of soffit as in R2 and R3. In test R4 the camera was not located in the specific slab with local damage and in which the hole in slab occurred, so no conclusions can be drawn. But it is evident that spalling started in R4 at 16 minutes in the specific slab as visible in Figure 6.7, while the others do not show spalling.

6.8. Restraint quantified in tested floors

In both floors R2 and R3 horizontal displacements and rotations are measured to observe the expansion of the floor, see Figure 6.8 and 6.10. Note that in R2 the support beam was insulated, but not in R3. In these graphs, the measurements at the locations “left” side and “right” are reworked in order to get the expansion of the floor at centroid of underflange. For example, the measurements at location #1 and #6 are reworked and combined and represented in Figure 6.8 and Figure 6.9 as (1+6). At first, in both graphs one sees that there are differences between measurements left and right, which indicates that the floor as a whole displaces.

It can be seen in Figure 6.8 in R2 that at the start of the fire the expansion near the (insulated) support beam (1+6) and (5+10) is less than the expansion near the middle field. After 15 minutes we see a clear influence of the jacks on the floor as the expansion decreases for (1+6) and (5+10). After 45 minutes the expansion of the middle field (2+7), (4+9) and (3+8) remains more or less constant. The expansion near the support beam still increases up to 90 minutes. For the middle span the total expansion derived from the displacement between measurement #3 and #8 at 30 minutes is 8.06 mm on a width of 3.90 m. At 0.95 m this is for (1+6) = 6.44 mm, and (5+10) = 5.81 mm. At 1.95 m from support this is (2+7)=7.56 mm, and (4+9)=7.44 mm at 30 minutes of ISO fire. These expansion values are visualized in Figure 6.9. Note that the expansion at the support beam location was unfortunately not measured in fire test R2.

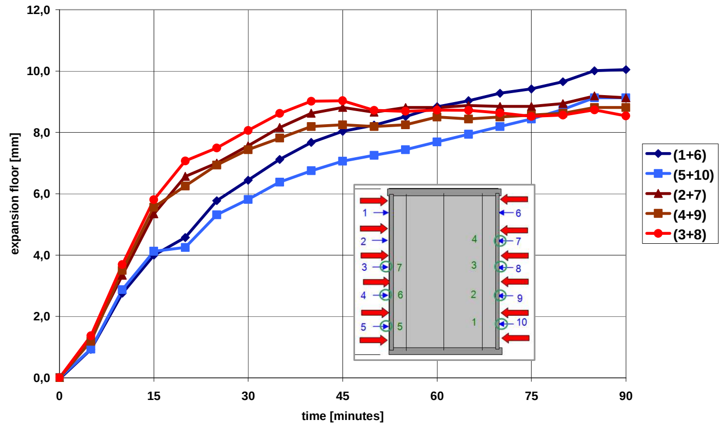


Figure 6.8. Relative displacements of floor R2 (hcs 260 mm – 100 mm topping) at centroid of underflange

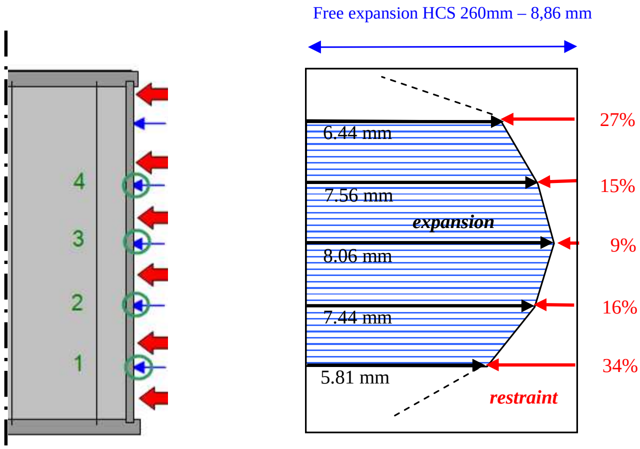


Figure 6.9. Expansion of the floor R2 at 30 minutes, and calculated restraints at centroid of underflange

From the graph R3 it is clearly seen that the expansion of the floor at the (not insulated) support beam is significantly less than the expansion of the floor beyond the support beam. We observe that in approximately the first 15 minutes the displacements grow rapidly to 6 mm and 8 mm expansion at mid span, but only about 1.6-1.7 mm at the support. For the middle span the total expansion deduced from the relative displacement between measurement #4 and # 11 at 30 minutes is 11.69 mm on a width of 3.90 m. At 1.95 m from the support the expansion is slightly less, namely 9.59 mm and 8.81 mm. At the support the expansion is significantly limited to 2.25 mm and 1.63 mm at both sides. These expansion values are visualized in Figure 6.11.

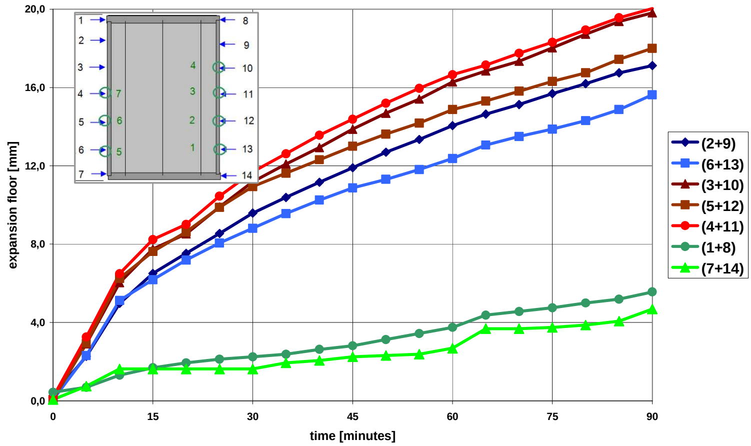


Figure 6.10. Relative displacements of floor R3 (hcs 200 mm – 50/70 mm topping) at centroid of underflange

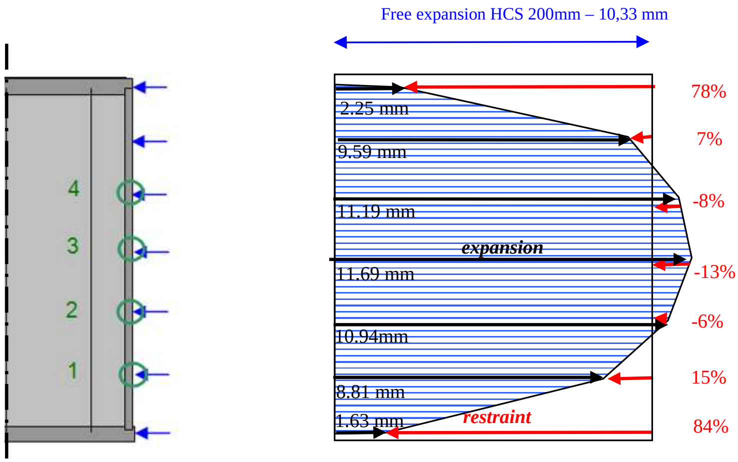


Figure 6.11. Expansion of the floor R3 at 30 minutes and calculated restraints at centroid of underflange

In order to quantify the restraint in the tested floors R2 and R3, imagine a free hollow core that is not connected and thus unrestrained. Hence, as a result of a fire at the soffit, the whole cross section of the hollow core will act together. As a result of the fire, the hollow core will expand and curve due to temperature induced stresses build up in the hollow core in order to deal with the increase of temperature. Let us take the cross section of fire test R2. A calculation with the Holcofire Frame Model [6.8] shows that without a topping and without restraint the centroid of the underflange displaces 2.725 mm at 30 minutes of ISO fire. On the way, some cracks occur in the underflange and top flange. Now, we apply a only topping on the slab, and no adjacent hollow cores are present. Hence, a calculation with the Holcofire

Frame Model shows that R2 with a 100 mm topping and without restraint the centroid of underflange displaces 2.683 mm at 30 minutes of ISO fire. When we compare this with a free hollow core without topping, the restraint of the topping on the centroid of the underflange of the cross section is only (2.725-2.683)/2.725 = 1.5% at 30 minutes. According, we can conclude that only on the cross section of one slab, the horizontal internal restraint at the level of the underflange of a structural topping is negligible. But the topping decreases the curvature of the cross section and could provoke horizontal cracks. For R3 the expansion at centroid of underflange is 3,178 mm at 30 minutes without topping, and 3.125 at 30 minutes with topping of 70 mm at support without external blocking.

Earlier it was calculated that for one free slab in R2 centroid of underflange displaces 2.725 mm at 30 minutes; this is 8.86 mm for a 3.9 wide floor without restraint. For R3 this is 10.33 mm for the 3.9 m unrestrained wide floor. These values are also visualized in the graphs in Figure 6.9 and Figure 6.11. Then, we can calculate back by subtraction what the restraints at the measured locations were. We can conclude the following about the restraints in R2 and R3. At midspan the floor is hardly restrained, while at the support this increases significantly to above 80% of an unrestrained cross section. About 1 m from the support it ranges between 7% and 34%. The 200 mm slab with 50/700 mm topping and support beam 300x665 mm^2 is less restrained than 260 mm slab with 100 mm topping and support beam 300x765 mm^2. Note again that in R2 the support beam was insulated, but not in R3.

With the Holcofire frame model we can calculate what the restraint spring is in order to get the restraints measured in the test. When we assess the floor 1 m from the support, on average we have an expansion per element of 1.88 mm (0.5x(5.81+6.44)x1.2/3.9). Hence, in R2 we have then a restraint of 200 N/mm. At midspan the derived magnitude of blocking is about 40 N/mm. But horizontal cracks already start to initiate. In R3 we get no restraint at midspan while 1 m from the support the restraint is estimated at 100 N/mm. With these restraints near the support we get initiation of horizontal cracks. This approximates the expansion in the test at the supports. Hence, the restraint is estimated to be 500-750 N/mm at the support region, about 100-200 N/mm at 1 m from support, and 0-50 N/mm at midspan in R2 and R3. For recalculation of the restraint fire tests R4, Chapter 7 [6.8] concludes an overall restraint of 400 N/mm. One can then ask whether there is a critical level of restraint. Chapter 7 [6.8] concludes that in practice under XC1 environmental conditions shrinkage cracks occur, and thus horizontal cracking and buckling spalling cannot occur under ISO fires. In case of these fire tests, the test has been done one month after assembling, while the slabs were 7 to 20 months old. And as drying shrinkage is a function of migration of the water through the hardened concrete, one can assume that no drying shrinkage cracks were present in the tested floors.

6.9. Retrospective view on some fire test results in Holcofire database

Chapter 2 on the Holcofire database [6.9] concluded that 5.5% of the fire test results in the database could not be fully explained. This 5.5% consisted of 9 fire test results related to explosive spalling and horizontal cracking. These tests as a whole are commented in this Chapter in retrospective view.

The fire tests with explosive spalling were H5, H57, H60, H103 and H136. In fire test H5 and H57 at about 40 minutes a hole occurred in one slab that very much resembles the R4 tests where at 56 minutes a hole occurred. The occurrence of a hole was preceded by significant spalling at the soffit and some open cores. In fire tests H93/H94, H101/H102 and H138 also open cores occurred due to spalling, but in these tests the fire resistance time was granted.

The fire tests with horizontal cracking were H110, H153, H154 and H159. These four tests were either on double web element or cross sectional slices, and have therefore no practical relevance. H153 had a 300 mm thick topping, and H154 and H159 not only had a 300 mm respectively 100 mm structural topping, but were also cast in from the sides creating unclear boundary conditions. The restraints are unrealistic and unknown in absolute value, but seem high enough to provoke horizontal cracking. These types of restraints cannot be found in practical application.

Fire test H103 is the most typical one in this series of the mentioned 9 fire test; these slabs were used three times in fire tests. The third fire test was stopped at 23 minutes due to extreme spalling and longitudinal cracks that resembles much the fire tests R2 and R3 (despite that the test set up had its 3rd fire test). From the technical report of the fire test emerges that the floor consisting of two slabs was highly restrained. Therefore, the mentioned spalling must have been buckling spalling, and the longitudinal cracks must have been horizontal cracks, and the phenomena come close to the phenomena observed in the R-series and Rotterdam. It is therefore believed that despite the local damage of the floor in H103, the load bearing capacity was not exceeded yet at 23 minutes when the test was stopped.

Regarding explosive spalling in practical applications in relation to standards, reference is made to standard EN1992-1-2. In section 4.5.1 explosive spalling is addressed and it states that explosive spalling is unlikely to occur when the moisture content is less than 2.5% by weight in exposure classes X0 and XC1, while otherwise 3% by weight should be taken as critical moisture content. If the moisture content is more than 3% by weight, R may be assessed by assuming local loss of cover of some reinforcement. This is mostly the case in parking garages where high humidity is present; but should then be accounted for in for all types of concrete floors. Regarding buckling spalling and horizontal cracking in practical applications in relation to standards, the same reference could be made to the principle stated in EN1992-1-2 section 4.5.1: The limit for moisture content has no meaning in this respect.

6.10. Conclusions

To investigate the influence of restrained conditions on hollow core floors, four tests were conducted within the R series with the goal to provoke buckling spalling and horizontal cracks in order to investigate the phenomena. Fire test R1 was conducted with depth of slabs of 255 mm to research the influence of a 100 mm thick topping and internal blocking. Fire test R2 was conducted with depth of slabs of 260 mm to research the influence of a 100 mm thick topping and internal/external blocking. Fire tests R3 was conducted with depth of slabs of 200 mm to research the influence of a 50 to 70 mm thick topping and internal blocking. Fire test R4 was conducted with depth of slabs of 265 mm to research the influence an untopped but stiff floor field with internal blocking. The fire tests have been executed one month after assembling, and the floors were assembled when the slabs hardened 7 to 20 months. And as drying shrinkage is a function of migration of the water through the hardened concrete, one can assume that no drying shrinkage cracks were present in the tested floors during the fire test which increase the internal restraint. In addition, in R1 and R2 the support beams were protected against the fire with insulation with the purpose to increase the restraint at the support.

The main objective was to study buckling spalling and horizontal cracking. For that, cameras were installed at the cores to study the development of horizontal cracks. Cameras at the soffit followed the spalling sequence at the soffit of the floor. In the fire test R1, R2, and R3, horizontal cracks initiated between 13 and 20 minutes. In all cases this was accompanied with spalling of the soffit: in tests R2 and R3 there was a clear visual relationship between spalling and horizontal cracking. Fire tests R1 and R4 failed prematurely at 37 minutes and 56 minutes, respectively. The fire tests R2 and R3 were continued until 91 minutes. Then the fire test was stopped and the floors were loaded further up to bending failure.

In R1 the cause of ultimate failure was shear-bending interaction due to the high live load of 13.3 kN/m^2 and selected load configuration (2-point load). This high live load was applied in order to simulate the same order of magnitude of the bending moment in the test floor as in Rotterdam. In R2 and R3 the loads were normal and load configuration was changed to 3-point load. After the fire tests R2 and R3 were stopped at 91 minutes on request of sponsors, and further loaded to failure, it emerged that the bending capacity at failure was at the level of the theoretical capacity of the cross section as a result of structural redundancy. It can be concluded from R2 and R3 that buckling spalling and horizontal cracking are not failure mechanisms. In R4 in one slab severe spalling took place due to the transversal restraints, and consequently a large hole occurred in the slab. The other slabs were however not affected, which can be explained by the fact that the restraint in the floor was released immediately when in the mentioned slab buckling spalling occurred. But due to safety fire test R4 was stopped on request of the sponsor at 56 minutes. The floor was not loaded up to failure, but the floor was thoroughly investigated for horizontal cracks in the slabs, but these horizontal web cracks were not found. Accordingly, it is believed that the other slabs would have easily succeeded in 120 minutes fire resistance time. It can be concluded that buckling spalling is not a failure mechanism.

The restraint in transversal direction was determined with the Holcofire Frame Model [6.8]. From the analyses emerged that in the fire tests the restrained effect at the support in R-series was 100-200 N/mm at about 1 m from the support, and below 50 N/mm at midspan. The restraint at the support is about 500-750 N/mm due to the size of the support beam. As the slabs were already 7 to 20 months hardened when the floor was assembled, and as the fire test was only one month after assembling, there were no shrinkage cracks present in the floor during the fire tests. Simulations with the Holcofire Frame Model supported that shrinkage cracks have a positive effect of restraints and can prevent horizontal cracking and buckling spalling in practical applications in XC0 or XC1 environments. In XC3 environment the shrinkage is less, but then the moisture content of the slabs will exceed the 3% as mentioned in EN1992-1-2 clause 4.5 and measures should be taken for explosive spalling.

The main conclusion from the R series is that high floor restraints due to structural topping and (insulated or not insulated) support beam can provoke buckling spalling in the underflange and horizontal web cracking, but these are concluded not to be failure mechanisms, as under normal design loads the fire resistance time is still met by virtue of structural redundancy in the hollow core slab floor. Buckling spalling will immediately release the restraint in the floor field, and more local damage cannot occur anymore. In order to deal with horizontal web cracking, one should follow the recommendations of EN1168 Annex G on connection reinforcement in order to improve the anchorage capacity of the strands at the support.

References Chapter six

[6.1] CEN, EN1168:2005+A3:2011 “Precast concrete products – Hollow core slabs”, October 2011.

[6.2] CEN, EN1990:2002+A1:2005 “Eurocode - Basis of structural design,” April 2002.

[6.3] CEN, EN1992-1-1:2004 EC2: “Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings”, December 2004.

[6.4] CEN, EN1992-1-2:2004: “Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design”,

[6.5] CEN, EN1363-1: “Fire resistance tests – Part 1: General requirements”, August 1999.

[6.6] CEN, EN1363-2: “Fire resistance tests – Part 2: Alternative and additional procedures”, August 1999.

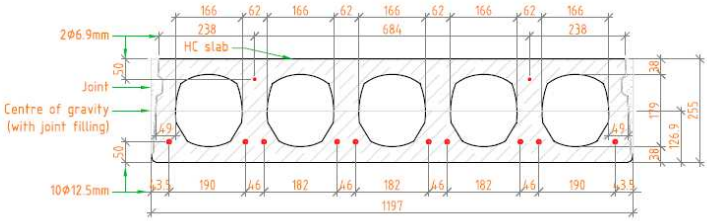
[6.7] CEN, EN1365-2 “Fire resistance tests for load bearing elements – Part 2: Floors and roofs”, December 1999.

[6.8] Jansze, W, Arnold Van Acker, Bruno Della Bella, Ronald Klein-Holte, Gösta Lindström, Andreas Nitsch, Jean-Paul Py, Matthieu Scalliet. “Holcofire Database – meta-analysis on 162 fire test results executed between 1966 and 2010”, Chapter 2, book published by BIBM, 24 January 2014.

[6.9] Jansze, W, Arnold Van Acker, Bruno Della Bella, Ronald Klein-Holte, Gösta Lindström, Andreas Nitsch, Jean-Paul Py, Matthieu Scalliet. “The Holcofire Frame Model to simulate buckling spalling and horizontal web cracking due to transversal blocking.” Chapter 7, book published by BIBM, 24 January 2014.

Appendix 6.A – Hollow core slab cross section data

R1: 255/5



Extruded concrete slab

Slab depth	h	= 255 mm
Slab width	b	= 1197 mm
Concrete area (without joints)	A_c	= 175014 mm^2
Centre of gravity from soffit	z_c	= 125,3 mm
Total web thickness	b_w	= 346 mm
Level where web thickness is 50% of total width	$a_{50\%}$	= 62 mm

Concrete slab with joint filling

Cross section	A	= 182078 mm^2
Centre of gravity from soffit	z	= 126,9 mm

Concrete

Concrete quality	C	= C55/67
Mean cubic compressive cylinder strength concrete	f_{cm}	= 70 N/mm^2
Aggregate	= silicious	
Production date slabs	15-11-2010	

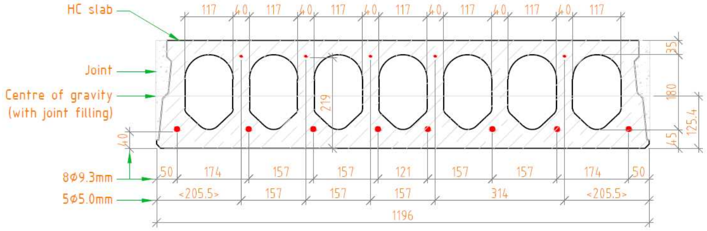
Prestressing steel X10-D2

Prestressing steel quality	= FeP1860	
Characteristic 0.1% strength	$f_{pk,0.1\%}$	= 1600 N/mm^2
Prestress at the bed before casting		
Initial prestressing	σ_{p0}	= 1100 N/mm^2
Working prestressing	σ_{pt}	= 1000 N/mm^2
Type of tendon	type	= “strand”
Diameter of tendon	$Ø_p$	= 12.5
Total area of tendon	A_p	= 10*93 = 930 mm^2
Axis distance of prestressing reinforcement	y_p	= 50 mm

Concrete joints and 100 mm topping

Concrete quality	C	= C25/30

R2: 260/7



Slipformed concrete slab		
Slab depth	h	= 260 mm
Slab width	b	= 1196 mm
Concrete area (without joints)	A_c	= 177828 mm^2
Centre of gravity from soffit	z_c	= 122,9 mm
Total web thickness	b_w	= 342 mm
Level where web thickness is 50% of total width	$a_{50\%}$	= 72,1 mm

Concrete slab with joint filling		
Cross section	A	= 188895 mm^2
Centre of gravity from soffit	z	= 125,4 mm

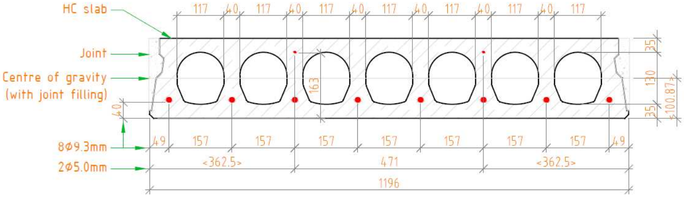
Concrete		
Concrete quality	C	= C45/55
Mean compressive cylinder strength concrete	f_{cm}	= 53 N/mm^2
Aggregate	= calcareous	
Production date slabs	22-03-2012	

Prestressing steel S8		
Prestressing steel quality	= FeP1860	
Mean 0.1% strength	$f_{pm,0.1\%}$	= 1717 N/mm^2
Prestress at the bed before casting		
Initial prestressing	σ_{p0}	= 1100 N/mm^2
Working prestressing	σ_{pt}	= 1000 N/mm^2
Type of tendon	type	= "strand"
Diameter of tendon	$Ø_p$	= 9.3
Total area of tendon	A_p	=8*51,8=414,7 mm^2
Axis distance of prestressing reinforcement	y_p	= 44 mm

D5 = 5 wires 5 mm (A_p = 5 x 19,4 mm^2, y_p = 222 mm, $f_{pm,0.1\%}$ = 1676 N/mm^2

Concrete joints and 100 mm topping		
Concrete quality	C	= C25/30

R3: 200/7



Slipformed concrete slab

Slab depth	h	= 200 mm
Slab width	b	= 1196 mm
Concrete area (without joints)	A_c	= 143823mm^2
Centre of gravity from soffit	z_c	= 99,35 mm
Total web thickness	b_w	= 344 mm
Level where web thickness is 50% of total width	$a_{50\%}$	= 43,7 mm

Concrete slab with joint filling

Cross section	A	= 151883 mm^2
Centre of gravity from soffit	z	= 100,87 mm

Concrete

Concrete quality	C	= C45/55
Mean compressive cylinder strength concrete	f_{cm}	= 63 N/mm^2
Aggregate	= calcareous	
Production date slabs	22-03-2012	

Prestressing steel S8

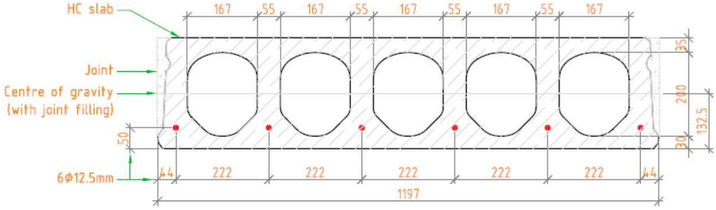
Prestressing steel quality	= FeP1860	
Mean 0.1% strength	$f_{pm,0.1\%}$	= 1717 N/mm^2
Prestress at the bed before casting		
Initial prestressing	σ_{p0}	= 1100 N/mm^2
Working prestressing	σ_{pt}	= 1000 N/mm^2
Type of tendon	type	= "strand"
Diameter of tendon	$Ø_p$	= 9.3
Total area of tendon	A_p	=8*51,8=414,7 mm^2
Axis distance of prestressing reinforcement	y_p	= 44 mm

D2 = 2 wires 5 mm (A_p = 2 x 19,4 mm^2, y_p = 165 mm, $f_{pm,0.1\%}$ = 1676 N/mm^2

Concrete joints and 50-70 mm topping

Concrete quality	C	= C25/30

R4: 265/5



Slipformed concrete slab

Slab depth	h	= 265 mm
Slab width	b	= 1197 mm
Concrete area	A_c	= 168467 mm^2
Centre of gravity from soffit	z_c	= 134 mm
Total web thickness	b_w	= 326 mm
Level where web thickness is 50% of total width	$a_{50\%}$	= 58 mm
Second moment of inertia	I_c	= 1447377000 mm^4
First moment of area, top	$W_{c,top}$	= 10780,9 cm^3
First moment of area, bottom	$W_{c,bottom}$	= 11070,1 cm^3

Concrete slab with joint filling

Cross section	A	= 171750 mm^2
Centre of gravity from soffit	z	= 135 mm
Second moment of inertia	I	= 1474200000 mm^4
First moment of area, top	W_{top}	= 10888,6 cm^3
First moment of area, bottem	W_{bottom}	= 11374,1 cm^3

Concrete

Concrete quality	C	= C45/55
Characteristic cylinder compressive strength concrete 28 days	f_{ck}	= 45 N/mm^2
Mean cylinder compressive strength Eurocode concrete 28 days	f_{cm}	= 53 N/mm^2
Mean cylinder compressive strength (50x50 mm^2) concrete 28 days	f_{cm}	= 60.0 N/mm^2
Aggregate	= silicious	
Production date slabs	18-08-2010	

Prestressing steel

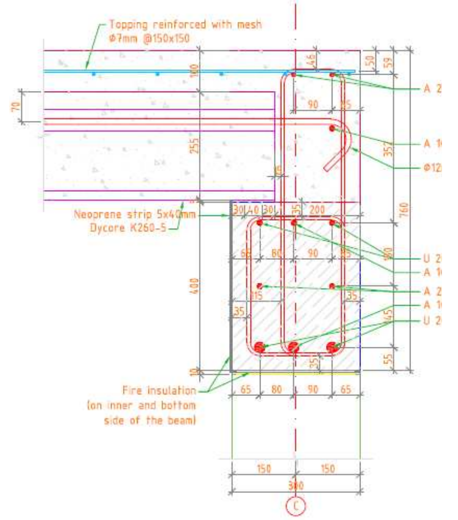
Mean tensile strength	f_{pm}	= 1951 N/mm^2
Mean 0.1% strength	$f_{pm,0.1\%}$	= 1735 N/mm^2
Youngs modulus	E_p	= 196.650 N/mm^2
Initial prestressing	σ_{pm0}	= 1100 N/mm^2
Type of tendon	type	= “strand”
Diameter of tendon	$Ø_p$	= 12.5
Total area of tendon	A_p	= 6 * 93 = 558 mm^2
Axis distance of prestressing reinforcement	y_p	= 50 mm

Capacities

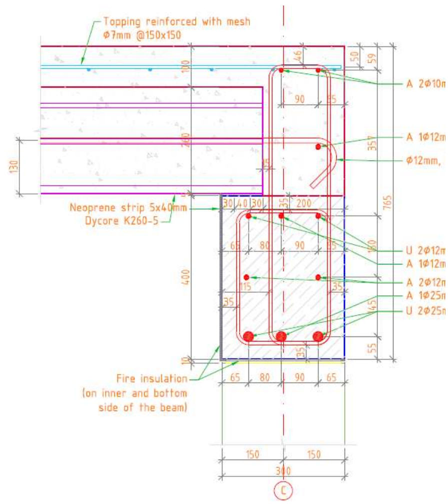
Bending moment design capacity	M_{Rd}	= 176 kNm
Shear design capacity	V_{Rd}	= 134 kN

See also “Fire test report G series” for more information of hollow core slabs

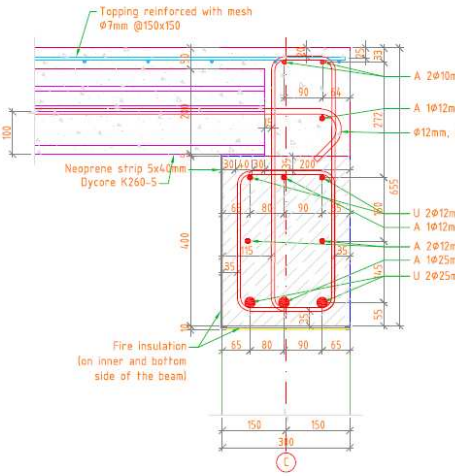
Appendix 6.B – Overview of support details of R1-R4: technical drawings



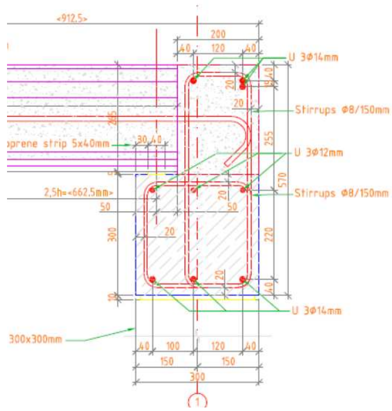
Support detail R1



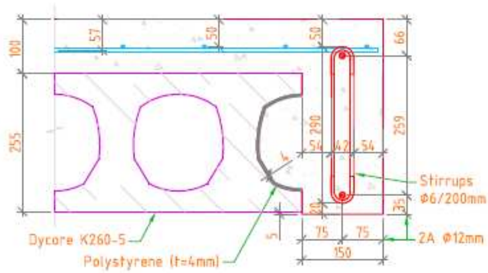
Support detail R2



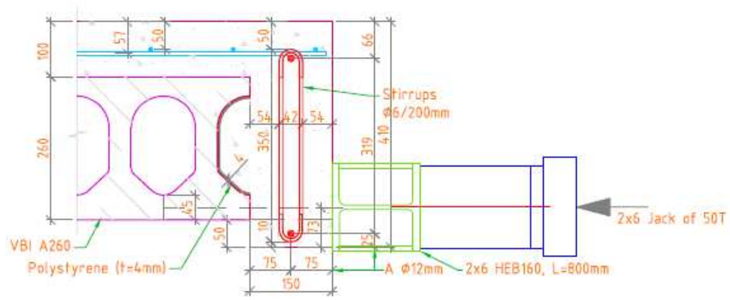
Support detail R3



Support detail R4



Lateral tie beam R1



Lateral tie beam R2 with jack

7

Chapter Seven

Holcofire Frame Model

The Holcofire Frame Model to simulate buckling spalling and horizontal web cracking due to transversal blocking

Keywords: *blocking, fire tests, hollow core slab, floor structures, frame model, horizontal cracks, parameters, spalling, validation*

Abstract. *Concrete is a highly fire resistant building material. And like all materials, concrete building components expand in a fire due to the increase of temperature over the cross section. But if concrete building components are hindered in this expansion, large additional forces can be build up that can lead to local damage respectively buckling spalling of the concrete cover and can ultimately lead to the exposure of the reinforcement. In addition, in prestressed concrete hollow core floors these large additional forces can lead to deformations of the cross section and ultimately to horizontal web cracking. This Chapter describes the features of a simple frame model that has been developed in the Holcofire project to simulate buckling spalling and horizontal web cracking followed by delamination. This simple Holcofire Frame Model consists of three sophisticated features; for every time step of 1 minute the exact average temperature and temperature gradient over the under flange based on fire characteristics; the so-called "cracking rod" that changes into a hinge for flexural cracks under compression, and into an opened crack for flexural cracks under tension; and the blocking spring to model horizontal restraint, but with so-called "free-space" to account for shrinkage cracks that are always present in concrete structures. With the Holcofire frame*

Model the exploratory fire tests on free hollow core slab slices can be recalculated. Then, as a next step, the model is used to make a comparison on hollow core cross sections under fire with blocking effects. It is shown that the initiation of horizontal web cracks and buckling spalling in the under flange can be simulated with the available parameters in the frame model. It is concluded that not the thickness of the structural topping, but the magnitude of transversal restraint is the main influencing parameter for both mechanisms. Moreover, it turns out that the available free space due to shrinkage cracks and similar dilatations in practice are enough to hold these transversal blocking effects at such a low level that horizontal web cracks respectively buckling spalling of the under flange cannot occur. This explains why these local damages are only seldom observed in practice. At the same time, the Chapter indicates that concrete building component applications in non-heated spaces can be critical. Because the humid environments with high moisture content in the concrete gives on the one hand smaller shrinkage crack widths, and on the other hand higher chances on explosive spalling of the concrete under fire. On a global level it can be stated that all observed local damages to concrete structures caused by fires look the same, regardless of the type of concrete building component. Hence, from this viewpoint, the behaviour of concrete hollow core slab components exposed to fire is not really different.

7.1. Introduction

Building materials can be classified in terms of their reaction to fire and their resistance to fire. This classification will determine respectively whether a material can be used and when additional fire protection is needed. EN 13501-1 [7.3] classifies materials into seven grades (A1, A2, B, C, D, E and F). The highest possible designation is A1 (non-combustible materials) and in 1996 the European Commission compiled a binding list of approved materials for this classification, which includes concrete and its mineral constituents. Concrete fulfils the requirements of class A1 because it is effectively non-combustible, in other words, it does not ignite at the temperatures which normally occur in fire. Concrete does not burn and is highly fire resistant. That is common knowledge, but we are not always aware of that. Concrete structures offer in the event of a fire protection to persons, property and the environment. If properly designed and constructed, concrete structures can withstand even the most extreme fire conditions. A concrete structure makes it easier to extinguish a fire because the concrete structure withstands the fire a longer period than for example steel structures. A concrete structure is an effective fire shield. Hence, a concrete wall and floor will stop the fire spreading through the compartments and separates the fire and thus reduces the risk of environmental pollution. All these features are the result of natural concrete properties [7.12]:

- Concrete does not burn and does not increase the fire load;
- Concrete has a high fire resistance;
- Concrete leaves no dripping molten material that will spread the fire further;
- Concrete does not produce smoke or toxic gases;
- Concrete is a (heat) insulating material;

- Concrete protects the cast-in structural materials against fire.

After the Rotterdam fire in 2007 and the observed local damage to wall and ceiling, elaborate technical discussion started between academics and structural engineers in The Netherlands about a possible additional failure type for floors consisting of hollow core slabs [7.1]. In 45 year of fire research on hollow cores, this is the first time that the cross section is under discussion. As this discussion took place in The Netherlands, a brief overview is presented in this Chapter. However, a clear fact is that in the Rotterdam fire case the load-bearing resistance (R) was not exceeded, and the integrity and the insulation (EI) criteria were fulfilled. Exploratory research pointed to a specific phenomenon in hollow core slices and was extrapolated to floors, and all follow-up research was based on the same assumptions. A hollow core slice is not a hollow core floor field constructed with tying systems to account for accidental loading. Hence, modelling a slice of hollow core can show fundamental behaviour, but it can never address a hollow core floor system under a severe accidental fire. Accordingly, the conclusion of an additional failure type is not shared by the Holcofire team as in Rotterdam both R and EI were achieved. By definition of EN 1365 [7.2] it was not a failure type as the load bearing capacity remained during the fire. But in order to cope with the cross sectional models developed in The Netherlands and their flaws, Holcofire developed the more sophisticated Holcofire Frame Model to study the cross sectional behaviour under fire of a hollow core slice. It should however be stated that by definition a model is a simplification of the reality. Consequently, the Holcofire Frame model has been developed to study horizontal cracking and spalling more fundamentally, but it cannot show redundancy effects as the real fire case in Rotterdam showed. And it must always be kept in mind that the cause of this type of local damage may be found in the generic behaviour of concrete or concrete structures exposed to fire instead of specific behaviour of hollow core slabs.

7.2. Objective

The external structure of a building or the floor boundary conditions itself e.g. the chosen tying system or the structural topping can cause restraining actions on the floor. These restraining actions are present in cast in-situ concrete floors, in precast concrete floors, in wooden floors, and in steel decks. Mostly these restraining actions are used to secure the stability of a building by virtue of diaphragm action. However, in the event of a fire materials expand, and as a consequence these restraining actions can cause spalling damage on the soffit of a floor or exposed side of a wall. In addition to this spalling, in Rotterdam it was clearly observed that also horizontal cracks in the webs of the hollow core slabs were initiated caused by restraints. The aforementioned studies in The Netherlands conclude that mainly due to the thick topping these horizontal cracks occur. On the one hand spalling was not commented in their studies; on the other hand the fire tests performed in the Netherlands on slices only had restraint from a thick structural topping. A conclusion that the thick topping is the cause of horizontal cracking is therefore rather obvious. Holcofire agrees that a frame model can be used to identify the principle behaviour of a hollow core cross section in order to deepen knowledge on a cross section by simple means that everybody can understand. Finite element calculations seem more advanced, and although it presents interesting and colourful pictures, it remains a black box and gives absolute values as output that cannot be validated in real fires or experiments. In this document a description is made of the characteristics of a further evolved simple but sophisticated frame model for the analysis of the behaviour of a hollow core slab cross section represented as a frame and exposed to fire. The results of this model can be compared or validated with the results of simple laboratory tests on slices of hollow core slabs and tests on a floor in a furnace. The main objective is to explain the occurrence of horizontal web cracks and buckling spalling. A further objective is finding a relation with regard to the magnitude of blocking in the model and executed tests.

7.3. Spalling of concrete exposed to fire

It is widely known that structures build with concrete have a good response to fire. The good thermal insulating properties of the concrete keep the temperatures in the reinforcement low for a certain time depending on the distance from the exposed surface. Only spalling can have a negative effect on this good thermal insulating property of concrete. The following definitions are intended to clarify the confusion which exists on the term “spalling” of concrete surfaces exposed to fire. Three types of spalling are distinguished in this research, namely thermal spalling, buckling spalling and explosive spalling. All are related to the increase of temperature of the concrete surface, but the influencing phenomena are different.

The temperature differences between the surface and the inner concrete mass are important during the first minutes of a severe fire, and may lead to splitting-off of small concrete particles at the soffit of the slab. This splitting-off of small concrete particles is defined in this research as thermal spalling. The surface is characterised by small surficial pits. It starts rather early after fire exposure, about 10 – 15 minutes after flash over. The spalling is also characterised by small but numerous bangs. It is most clearly visible at sharp corners of structural components and irregular surfaces.

Like other materials, concrete expands due to increase of temperature. Depending on the characteristics and freedom of the material this will lead to additional forces in the material respectively structural component, or the whole structure. Overload due to forces resulting from hindered deformations can cause local damage, called buckling spalling in this research. But as a positive consequence of buckling spalling, this leads to release of these additional forces such that the damage remains only locally where the temperature is increased. Buckling spalling is characterised in this research as the splitting-off of large concrete pieces at the exposed side of structural components due to large internal compressive stresses introduced by the fire. The phenomenon takes place at about 15-25 minutes after flash-over. For this study the material properties of concrete exposed to fire are derived from ”Eurocode 2: Design of concrete structures - Part 1-2: General rules - Structural fire design” [7.10]. Also the temperature profiles from EN1992-1-2 Annex A for solid floors are used.

Besides, in structures with outside environmental conditions (XC3) next to the high compressive stresses in the concrete due to hindered deformations, under fire moisture can turn into steam resulting in explosive spalling due to high internal pressure. This type of spalling is also characterised by the splitting-off of larger concrete particles at the fire exposed side of the components. It is caused by internal high vapour pressure inside a very tight concrete mass. The pressure is steadily built up by rising temperature inside the concrete mass. When the pressure exceeds the tensile capacity of the concrete, it causes local explosive splitting-off of concrete particles. The phenomenon starts after about 30 minutes, and continues as long as free water is present in the concrete mass. The surface is characterised by a rough surface of chipped concrete particles in the mortar matrix. It is well known that the compressive stress ratio influences the behaviour with regard to explosive spalling.

Figure 7.1. Examples of buckling spalling combined with explosive spalling on walls and floors [7.1, 7.11]

7.4. Previous research on horizontal cracking

In October 2007 a fire in Lloydstraat building led to delamination of under flanges of a hollow core slab floor, while it did not lead to collapse of the floor. It was concluded in the studies [7.1] that:

- “The occurrence of horizontal cracks is not an incident”;
- By virtue of restraints due to a thick topping horizontal cracks occur prematurely in the outer webs which will lead to delamination of the under layer;
- This horizontal cracking is primarily a result of hindered temperature deformations in the transversal (and vertical) direction of hollow core slabs.

Since the 1st of October 2007, the fire resistance of the hollow core floor is, on the level of the Dutch Ministry, still under discussion. For that, different studies and (2D) computer models and even advanced FEM 3D have been developed. The models developed by the various authors are shortly addressed in the next section. In 2013 a modified Dutch approval for hollow core floors was published, where the thickness of the so called top layer is limited in order to prevent horizontal web cracking.

TNO Bouw and Ondergrond commissioned by the Dutch authority “Veiligheidsregio Rijnmond” performed a short analysis [7.4] in January 2008 with DIANA 2D where the focus was on the behaviour of the joint due to blocking effects. This report contained a 2-dimensional finite element (FEM) analysis conducted with DIANA with a main focus on the transversal direction. TNO Bouw en Ondergrond concluded that horizontal cracks in the webs next to the outermost core could occur; in the variant with a thick structural topping within 30 minutes horizontal cracks occur in the webs resulting in separation of the under flange from the upper floor structure. The magnitude of the blocking was however assumed to infinite rigidity.

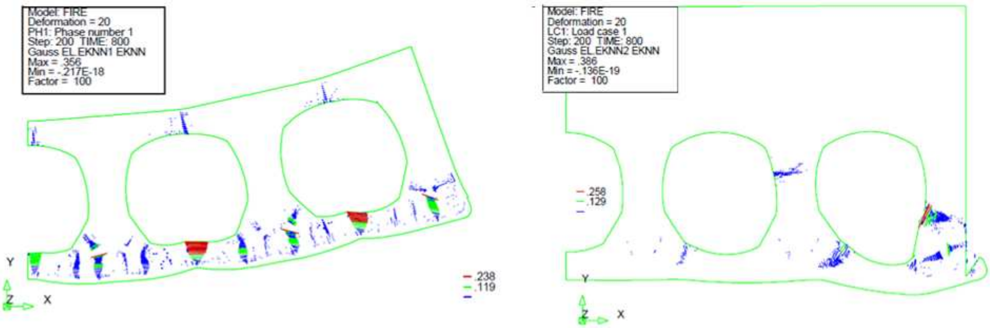


Figure 7.2. FEM simulation of hollow core with topping; unrestraint (left) and fully restraint with thick topping (right) [7.4]

Kleinman [7.5] started his short research project with shear tests on the webs of a cold cross-section, because his failure criterion focuses on the most outer web when the transversal shear capacity is exceeded. The stiffness of his shear model is validated through a laboratory test in which a part of a hollow core cross section is loaded up to failure under ambient temperature. A simple elastic frame model is constructed that matches the dimensions of a hollowcore cross section. A structural topping is modelled simply by increasing the thickness of the upper flange from 45 mm to 135 mm in case of a 90 mm structural topping.

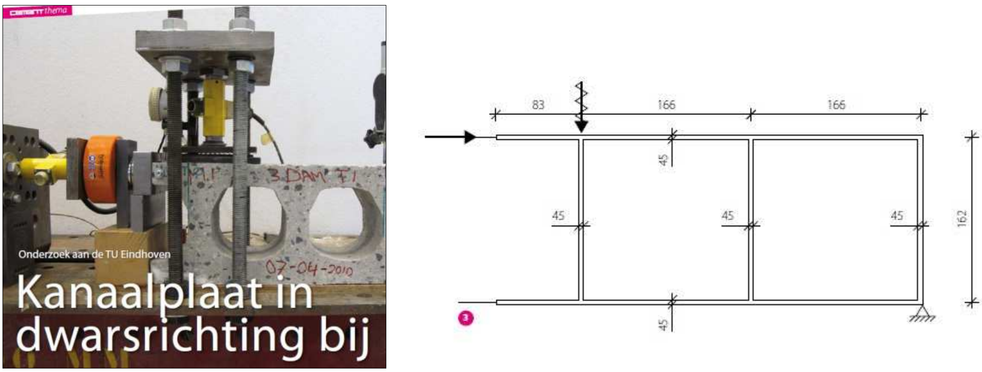


Figure 7.3. Simple framework validated on shear test on cold cross-section

For fire Kleinman only considers a linear fire load of 300 ºC acting over the height of the under flange, while the temperature gradient is neglected. On the basis of the fact that any horizontal crack initiates quickly (within 20 minutes after start of fire) means that the temperature in the concrete of the webs is not or hardly increased and thus close to the ambient temperature. He concluded from a parameters study that for expansion of the bottom flange during a fire, preventing the deformation of the upper flange of the hollow-core slab has a very negative effect on the fire resistance of the hollow-core slab due to (shear) cracking through the webs. When the thickness of the structural topping increases cracks through the webs initiated earlier in the time of fire exposure.

Figure 7.4. Fire tests on small samples of hollow core in The Netherlands. Up: Slices with a structural topping. Down: free part of hollow core 1.20x1.20 m^2 with extreme thick topping of 300 mm

Hordijk [7.6], commissioned by the Dutch precast organization BFBN, did exploratory fire tests on parts of hollow core slabs with a length of 1.2 meter. Moreover, to reduce possible three-dimensional effects also fire test were done on 0.15 meter long slices of hollow core elements with different thicknesses of the topping ranging from 50 mm to 300 mm, see Figure 7.4. The fire tests greatly contributed to more in-depth insights on the fundamental behaviour of a hollowcore cross section exposed to fire.

Van den Bos [7.7, 7.8] reported about the possibilities in using the 3D FEM package DIANA. The FEM model is a very sophisticated model. It contains non-linear material parameters, while the material properties are temperature dependent. During the last 3 years 2D and 3D models have been constructed, indicating very well the origin of cracking due to fire but not spalling. However, the model could not simulate ultimate failure in 2D and 3D, nor could it give exact indications about influences of parameters on this cracking.

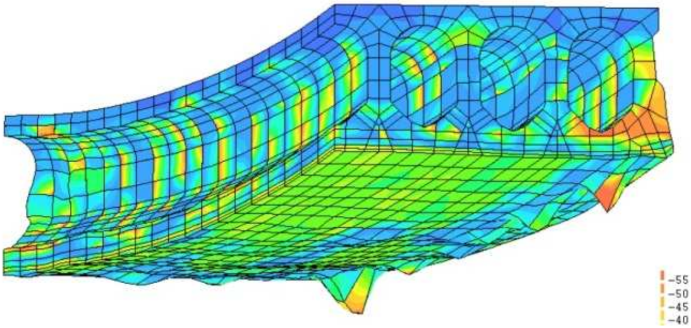


Figure 7.5. Graphical result of 3D FEM analyis with DIANA

Finally, Breunesse, commissioned by the Dutch precast association BFBN, reported in [7.9] also about the intermediate results of a frame analysis representing a hollow core cross-section. The modelling of Breunesse is quite the same as Kleinmans frame model; however, Breunesses frame model focuses on the second outer web where the principal stresses due to normal force and bending moment are exceeded. Also, the fire load contains of two parts; a linear fire load of 100 ºC acting on the under flange and a temperature gradient over the under flange of 100 ºC. But Breunesse concluded from a free model, that 90% of the deformations (curvature) is due to difference in temperature between upper- and under flange; only 10% of these deformations are attributed to temperature gradient over the under flange. He analysed which hollow core part could be expected to be cracked first. He concluded that when vertical cracks occurred as first in upper flange, horizontal cracks in webs should not occur. But with a stiff upper flange due to a topping, vertical cracks cannot occur and then horizontal web cracks can be expected at certain topping thicknesses.

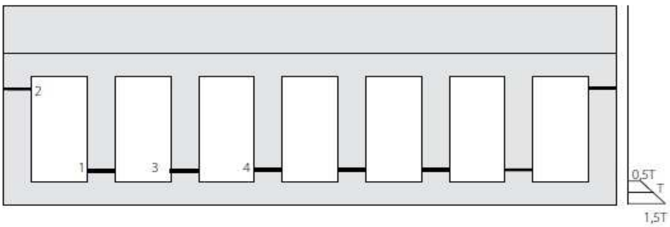


Figure 7.6. Breunesses frame model with simplified temperature load

7.5. Holcofire Frame model – a semi non-elastic model with blocking

In past research it was found that generation, performance and the interpretation of results of advanced FEM software in 2D or 3D solids was time consuming while the results were also difficult to understand. Also the definition of the boundary conditions was not clear and raised questions. Hence, in the Holcofire project also a simple frame model consisting of nodes and rods is developed to enable analyses as performed already in the years 2007 to

2010 by different researchers in The Netherlands. Also, the simple Holcofire Frame Model is actually quite sophisticated, but the aim of this model is still to gain an easy understanding of the behaviour of a cross-section under fire. Also a simple frame program is fast in calculation time and therefore it is possible to make a large number of analyses. For this project a commercial frame program was not used, but Klein-Holte programmed the Holcofire Frame Model in special software of less than 0.5 MB.

Two main mechanisms are included in the Holcofire Frame Model; buckling spalling in the under flange due to too high compressive stresses, and horizontal cracking of the webs when the bending tensile strength of the concrete is exceeded. Other mechanisms are not included in the simple model; by definition a model is a simplification of reality. It is therefore important to note that the Holcofire Frame Model cannot model the redundancy of a hollow core floor in a building which normally is present during and after a fire. One of the important sophisticated features in the Holcofire Frame Model is the temperature load in the bottom flange induced by the fire exposed soffit. In previous performed studies with frame models by Breunesse [7.9] and Kleinman [7.5] the temperature load was assumed with a constant temperature and with a constant gradient, or no gradient [7.5] at all. As in the Holcofire project this is assessed as a critical input parameter to obtain correct output of the calculations, this is correctly modelled in the Holcofire frame model. The temperature load is derived from a (curved) temperature profile at a specific time and changes during the time of fire exposure.

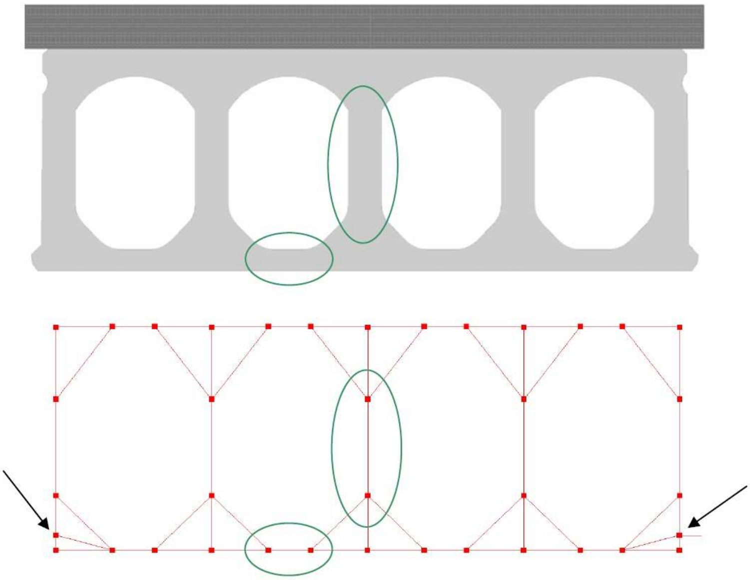


Figure 7.7. Node and rod model of hollow core cross-section

The model is generated on the basis of the general dimensional properties of the hollow core cross section and the number of cores. The way of modelling the nodes and rods is different for the Holcofire Frame Model, see Figure 7.7. The top flange is represented as a rod with the properties of the composite section of the upper flange of the hollow core slab and the structural topping. In the model always rods exist which represent the part of the webs and flanges with constant thickness, see the green circled rods in Figure 7.7. The more solid parts of the cross section next to the green circles are modelled with rod triangles in order to add local stiff concrete part, like in the real cross section. The material properties for concrete are derived from the stress strain relation as noted in Eurocode 2 part fire, as shown in Figure 7.8.

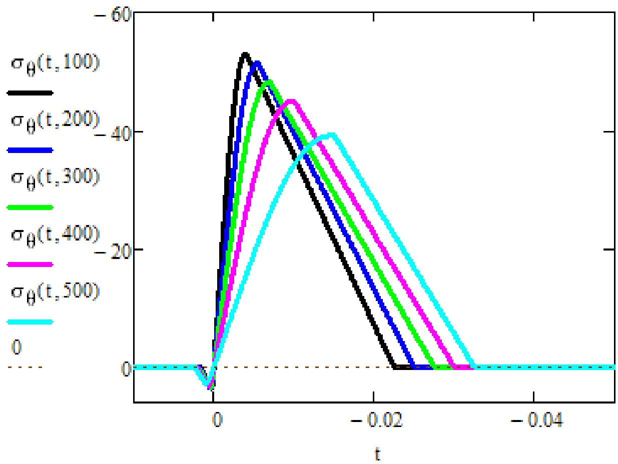


Figure 7.8. Stress-strain relation for concrete at elevated temperatures

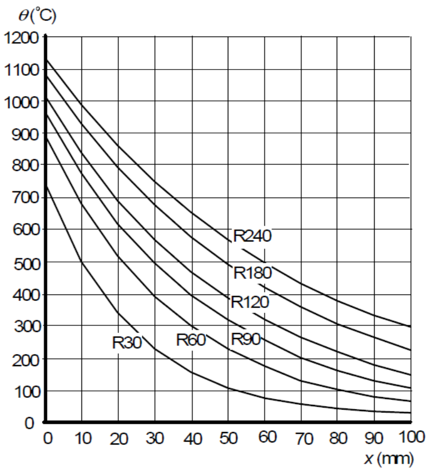


Figure 7.9. Temperature profiles for slabs (Annex A of EN 1992-1-2)

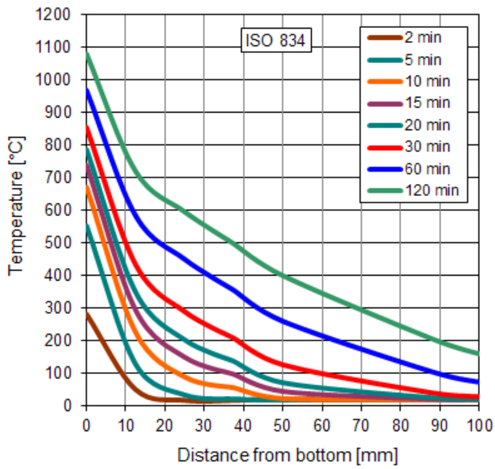


Figure 7.10. Temperature profile calculated with ISO 834 fire curve

The temperature load in the model consists of the expansion and curvature in the under flange which are the result of the average increase of the temperature and the temperature gradient in the lower flange. The temperature in the concrete is a function of time and distance, as one can conclude from EN1992-1-2 Annex A Figure A.2, see Figure 7.9. This time and distance dependency is fully incorporated in the Holcofire Frame Model. For intermediate values of the time and distance, the temperature increase is simply interpolated. As Figure A.2 does not give information of the temperature profile during the first 30 minutes of a fire, the temperatures are interpolated between 0 and 30 minutes.

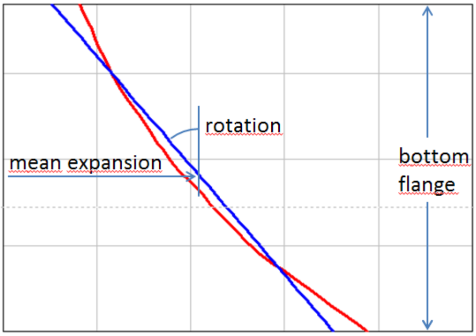


Figure 7.11. Average expansion and rotation of the under flange due to temperature

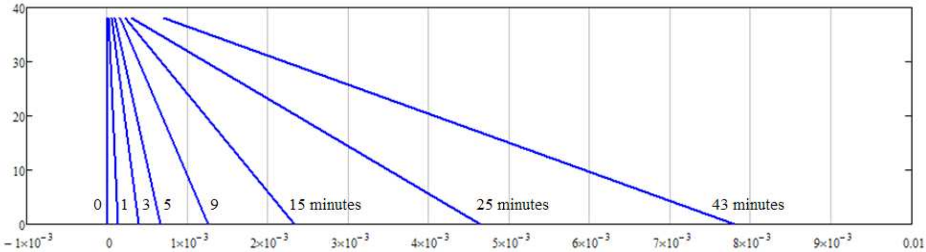


Figure 7.12. Strain of under flange at different times of fire expose with interpolated values from Annex A of EN 1992-1-2

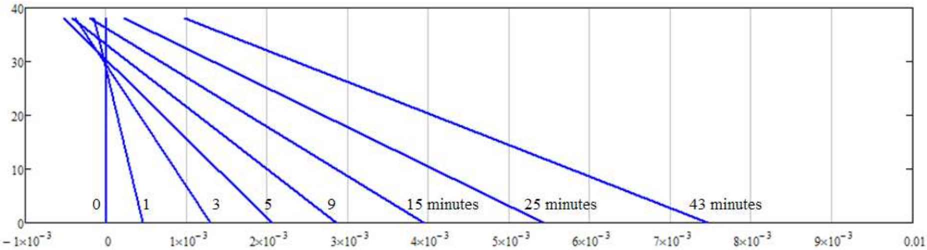


Figure 7.13. Strain of under flange following calculated values of ISO 834 fire curve

Because the temperature gradient over the depth of the bottom flange is non-linear, temperature induced stresses will be developed resulting in a distribution in equilibrium with the linear strain response of the bottom flange. The average expansion and rotation of the cross-section are determined on the basis of this equilibrium; see Figure 7.11 as an explanation. Only the temperature load is added to the rods representing the bottom flange. The rods representing the under flange are assumed to be exposed to fire and introduce temperature force in the model. The resulting free expansion derived from the temperature profile in Figure 7.9 is shown in Figure 7.12; this is in fact the applied strain due to the temperature load. Instead of using Annex A of EN 1992-1-2 for the temperatures in the concrete, temperature information can also be derived with FEM, as shown in Figure 7.10. The calculated temperature profile according to ISO 834 gives more information for the time interval 0 to 30 minutes. For exposure times of 30 minutes and higher this profile give roughly the same information as the figure of annex A of EN 1992-1-2. The graphs in Figure 7.9 and Figure 7.10 are not exactly the same because the assumptions for the determination of these graphs were not equal. In Figure 7.13 the resulting strains at different exposure times due to calculated ISO 834 fire curve are presented. When the information of Figure 7.12 and Figure 7.13 is compared it can be noticed that the interpolated values of Figure 7.9 result, for times until 25 minutes, is a lower thermal gradient than the gradient as results from the calculated ISO 834 fire curve.

The stiffness of the bottom flange is therefore also a function of the temperature profile in this rod. When compressive forces in the under flange occur the compressive strength is considered as an average stress over 20 mm in the soffit of the bottom flange. The compressive strength is a function of the temperature in the considered part.

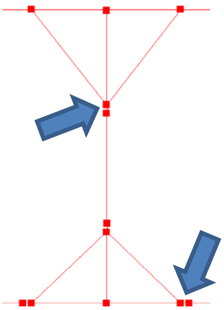


Figure 7.14. Cracking rods at the end of flange and web mid rods

A relative simple but also sophisticated feature of the model is the so called “cracking rods” at predefined locations, as shown in Figure 7.14. When the bending stress in the outermost fibre of this cracking rod exceeds the flexural tensile strength of the concrete, the properties of this rod are set to cracked properties. The stress level of cracking is assumed as the mean flexural strength at ambient temperature. The factor between the mean tensile strength and the flexural tensile strength of the concrete is taken from the fib Model Code 2010 [7.15], formula 5.1-8b:

$$\alpha_{fl} = 0.06 \cdot h^{0.7} / (1 + 0.06 \cdot h^{0.7}) \qquad (1)$$

If the rod is under compression a hinge will be introduced in the cracking rod. Otherwise when the rod is under tension an open crack will be simulated by reducing the bending and axial stiffness, the two nodes of the cracked rod have then the possibility to displace from each other thus simulating a horizontal crack. With this features included the model can be characterized as a semi non-elastic model. Dynamic or cracking energy effects are not taken into account in the model.

The boundary conditions of the model consist of a hinge (node 1) on the left side of the model and a roller in horizontal direction (node 2) on the right side of the model. This roller is connected to a rod representing the blocking spring, see Figure 7.15, and finally this rod is connected to a hinge (node 3). For free cross-sections the value of the blocking spring is zero. The height position and the magnitude of the blocking spring are two parameters in the model, the third parameter for blocking effects in the model is the free-space. This parameter can simulate the shrinkage cracks and dilatations in real structures. The model can then expand freely until the value of the free-space is reached, and before the blocking spring starts functioning to block the under flange. This advanced blocking spring is the third sophisticated feature in the Holcofire Frame Model (HFM). The expansion given in the output of the Frame Model is the displacement between node 1 and 2 as shown in Figure 7.15.

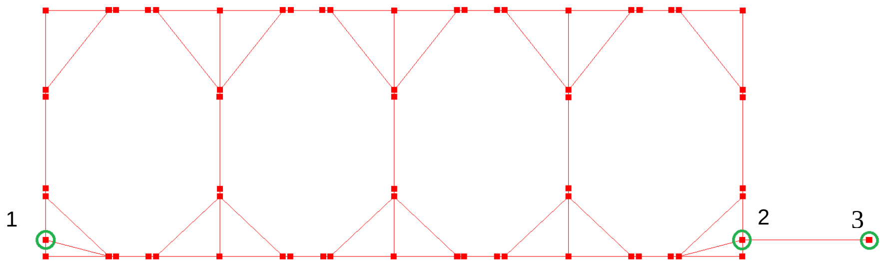


Figure 7.15. Additional rod (between node 2 and 3) representing the blocking spring

The output of the analysis is a graphical display of the deformed frame with at the cracked rod positions (if rod is cracked) the time of cracking; see as an example the output in Figure 7.16. This example shows (thermal) cracks at the topside of the bottom flange after 4 minutes and a bending crack at the topside of the outermost web after 22 minutes. In the second web the model finds a crack where the rod is under tension. Here the model shows with the cracking rod feature a crack which opens at 22 minutes.

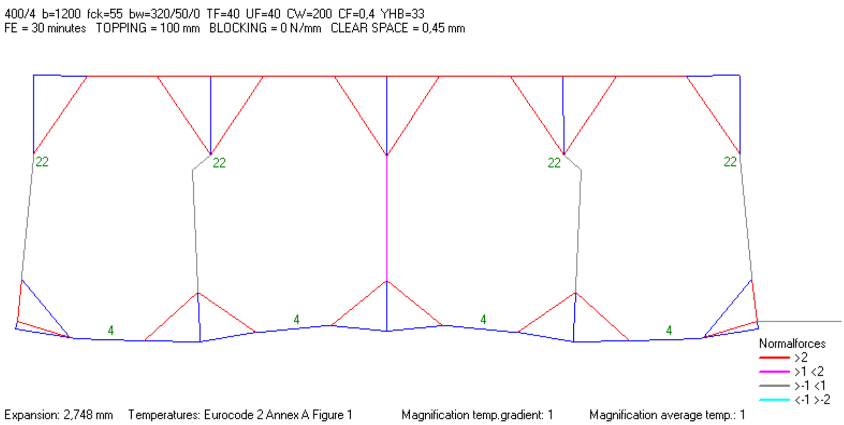


Figure 7.16. Example output of frame analysis with web cracking; in the outermost web a bending crack occurs at 22'; in the 2nd web horizontal web cracking initiates at 22'

The example shown in Figure 7.17 is a frame representing a hollow core cross-section with 4 cores and is calculated with extreme horizontal blocking. Compression failure (buckling spalling) in the under flange occurs after 23 minutes. Note that horizontal cracks did not initiate yet.

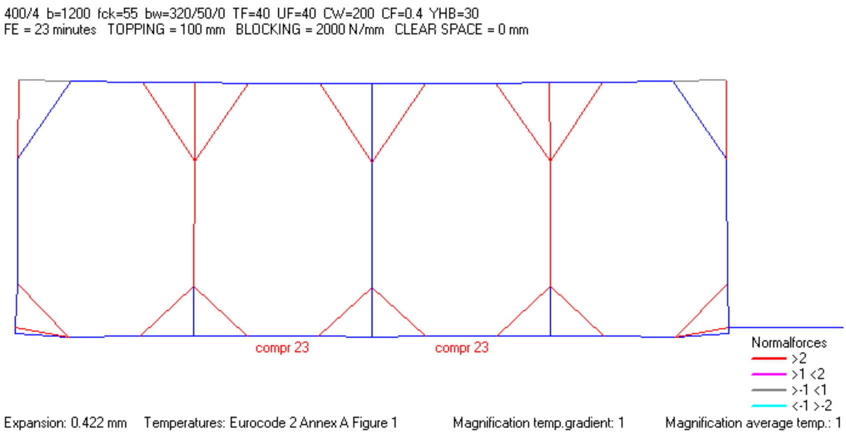


Figure 7.17. Example of frame analysis with compression failure at the soffit

7.6. Comparison with test results

To analyse the behaviour of hollow core cross-sections some exploratory tests were conducted at Efectis in The Netherlands commissioned by the Dutch Precast Association (BFBN) [7.6]. The tested sections were 150 mm long slices of hollow core with different toppings varying from no topping to 50 mm, 75 mm and 100 mm. The slices of hollow core slab had a depth of 260 mm and 400 mm. The slices were exposed to fire and the different cracks and the time they occurred were recorded and indicated with a pencil on the specimen.

A comparison of the results of the test on the slices and the frame model shows a similar behaviour. In Figure 7.18 the photo and crack registration of the tested specimen H158 with hollow core depth 260 mm and 100 mm topping is given. Between 6 to 8 minutes cracks in the topside of the under flange were visible, while at 9 and 10 minutes the cracks in the upper flange were registered. In the Holcofire Frame Model cracking of the topside of the under flange took place between 5 to 8 minutes, while the upper flange cracked at 14 minutes. Only one example is given here in the Chapter, but the other fire tests have also been recalculated, see Appendix 7.B. Overall, it is concluded that the Holcofire Frame Model is able to show two mechanisms, horizontal cracking and buckling spalling, and that it gives outcomes that are also observed in real fire tests on slices of hollowcore cross sections with toppings. Then, as a next step, many simulations have been conducted with the Holcofire Frame Model, and overall it became evident that the restraining effects highly influence the outcome of a calculation, but there are also some cross sectional geometry effects. Both topics will be addressed in the next two sections of this Chapter.

Various researchers [7.4] to [7.9] concluded that in case of a free cross-section the stiffness of the top flange due to e.g. a structural topping is strongly influencing the occurrence of horizontal cracks in the webs. With the Holcofire Frame model the same principle behaviour for free cross-sections is determined respectively confirmed.

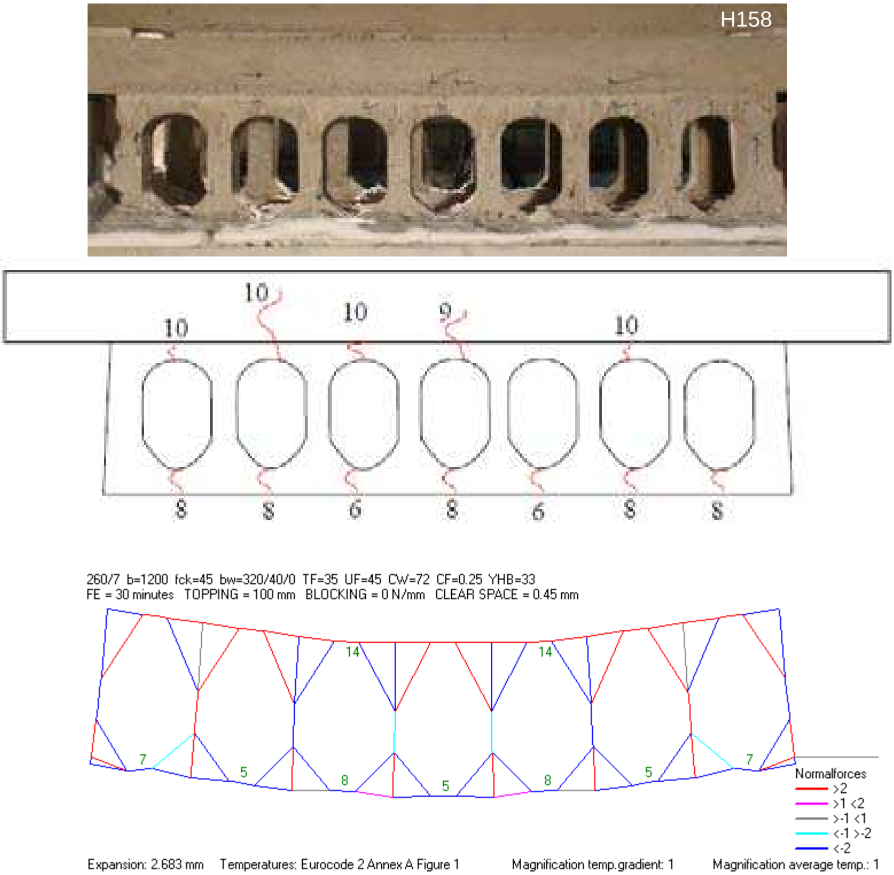


Figure 7.18. Comparison of fire test result of a slice of hollow core and topping with the frame model

7.7. Restraining effects

When restrained, the effects in the modelled cross-section due to restraints is strongly influenced by the free space. As stated earlier, this parameter can simulate the shrinkage cracks and dilatations in real structures. It is obvious that if there is enough free space the cross-section will act like a free section. If no free space is defined the predicted deformation of the model is totally different as the example in Figure 7.19 shows. In the example in Figure 7.19 above the model has no free space (= 0 mm) and web cracks are opening in the 2nd web at 19 minutes. In Figure 7.19 under an identical model, but with a free space of 0.5 mm shows a different behaviour as horizontal cracks do not occur. In normal use dry environment (XC1) shrinkage cracks in the joint with a size of 0.3 to 0.5 mm are often observed.

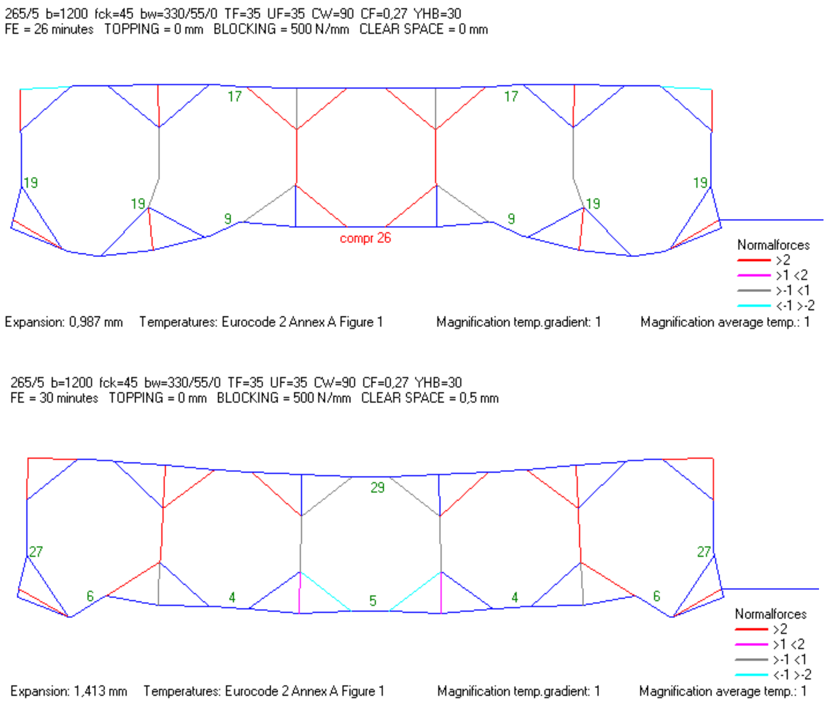


Figure 7.19. Example influence of free space = 0 mm (above) and free space = 0.5 mm (under)

Restraining effects on a floor introducing additional forces can be caused either by the surrounding structure, or by internal effects within the floor. Restraints by the surrounding structure are for example in case of a fire compartment that is surrounded by cold zones. There is only limited knowledge about the magnitude of these additional forces, knowing that it is an accidental situation, and an approximation depends highly on the lay-out of the surrounding structure.

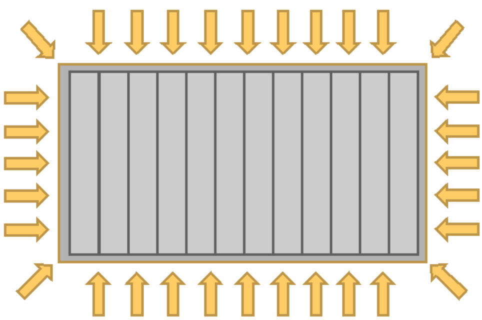


Figure 7.20. The surrounding structure induces blocking effects

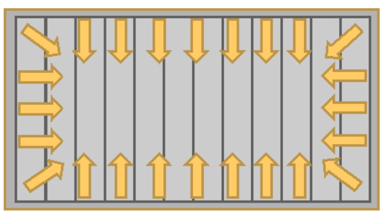


Figure 7.21. Internal blocking effects

Internal blocking forces are generated by the floor itself due to for example the tying systems such as the surrounding peripheral beams and the reinforced structural topping as shown in Figure 7.22. An order of magnitude of the blocking spring for this case can be approximated: suppose a structural topping with a mesh #6-150 and two supporting concrete beam 300 x 400 with reinforcement grade 0.5%. The amount of active blocking reinforcement is $A_s = 2 \cdot 0.5 / 100\% \cdot 300 \cdot 400 + 5 / 0.15 \cdot 28 = 1200 + 930 = 2130$ mm^2. It can be expected that the concrete due to expansion cracks and the amount of reinforcement steel determines the magnitude of the blocking spring. For an assumed span of 5 meters the average k-value can be derived. The average k-value for this case is $A_s \cdot E_s / (\text{Span} \cdot L_{system}) = 2130 \cdot 200000 / (5000 \cdot 1200) = 71$ N/mm per mm[1]. The value of the spring stiffness can be expected to be 2 to 3-times higher near the support area so the order of magnitude of the internal blocking spring can be assumed to be 150 to 200 N/mm.

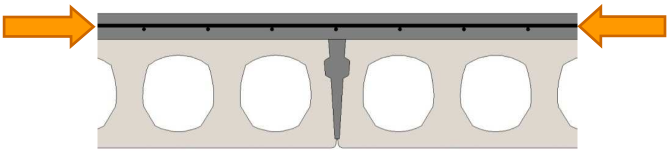


Figure 7.22. Direct blocking effect of a structural topping

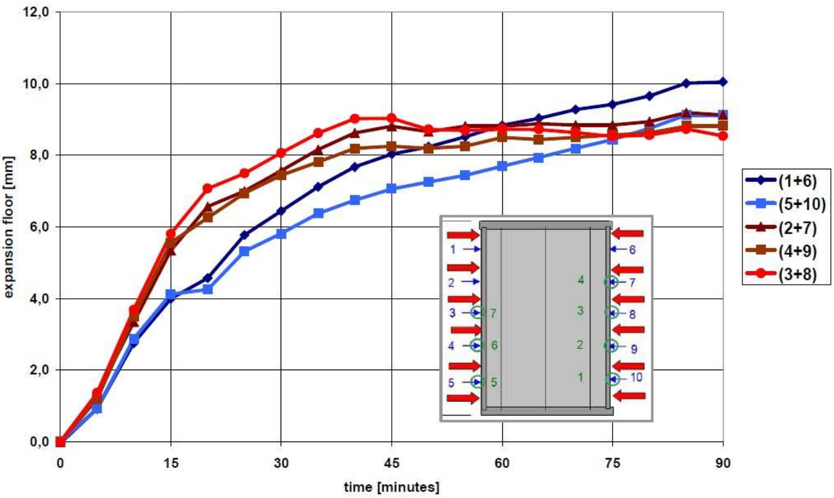


Figure 7.23. Measured horizontal transverse expansion of test R2

In the conducted fire test R2 [7.16] the transverse horizontal displacement was recorded, so a consideration of the blocking effect is possible. The measured expansion of the bottom flange is shown in Figure 7.23. Frame analysis of the R2 configuration (HC260/7 with 100 mm topping) with the Holcofire frame model gives as result for the free expansion of the bottom flange: 2.7 mm for an element.

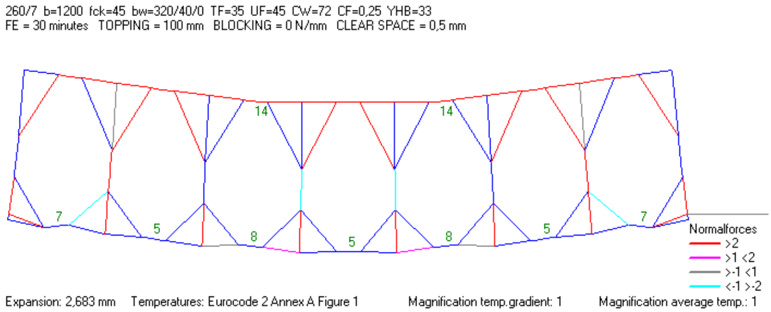


Figure 7.24. Result of analysis of free cross-section HC260/7

The width of the floor in the R2 test was 3.9 meter, so the equivalent expansion of the bottom flange is 2.7 · 3.9 / 1.2 = 8.8 mm. With this figure a comparison with measured expansion of R2 can be made:

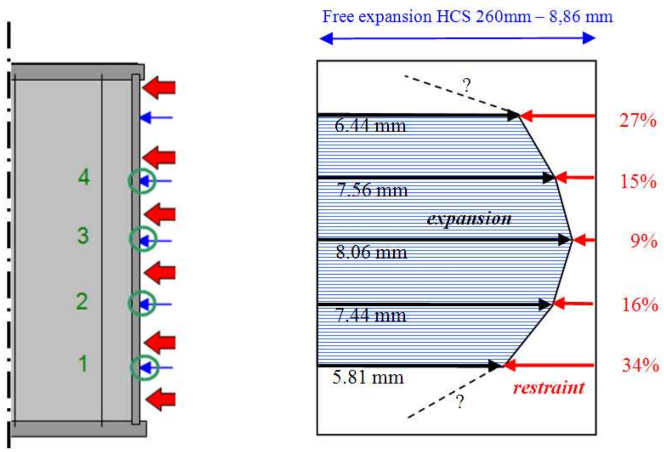


Figure 7.25. Expected restraint on base of measured expansion of the bottom flange

Near the slab ends the expansion is 0.5 · (5.81 + 6.44) = 6.1 mm, this corresponds with 1.9 mm per element.

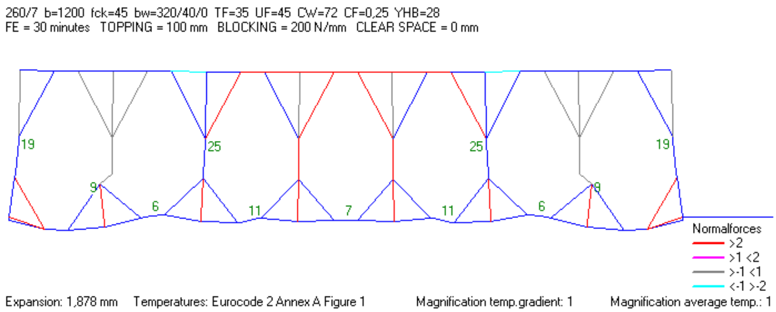


Figure 7.26. Result of analysis of blocked cross-section HC260/7

Calculating backwards, the frame model gives the spring stiffness corresponding with an expansion of 1.9 mm. This backward calculated spring stiffness is 200 N/mm. Similarly a blocking spring of 55 N/mm at mid-span (9% of blocking) can be derived. The average spring stiffness over the length of the slab is then approx. 55 + (200-55)/3 ≈ 100 N/mm. The model shows horizontal cracks in the lower part of the second web; this was also observed with in test R2. Nevertheless REI90 was granted. Note: the difference between the expansion of the under flange of a free element and a blocked cross-section is: 2.7 – 1.9 = 0.8 mm! In Appendix 7.C a summary is shown of the comparison of the observed expansion and the results of the R-tests [7.16] with the results of the Holcofire Frame Model.

7.8. Comparison of blocking effects of different cross-sections with Holcofire model

It is clear from the calculation that concrete and thus the hollow core cross section needs some free space in order to make some initial settlements possible. In order to make a comparison of blocking effects, several parameterized cross-sections are calculated with regard to the three main parameters for blocking:

- Blocking spring stiffness;
- Free space;
- Thickness of the topping.

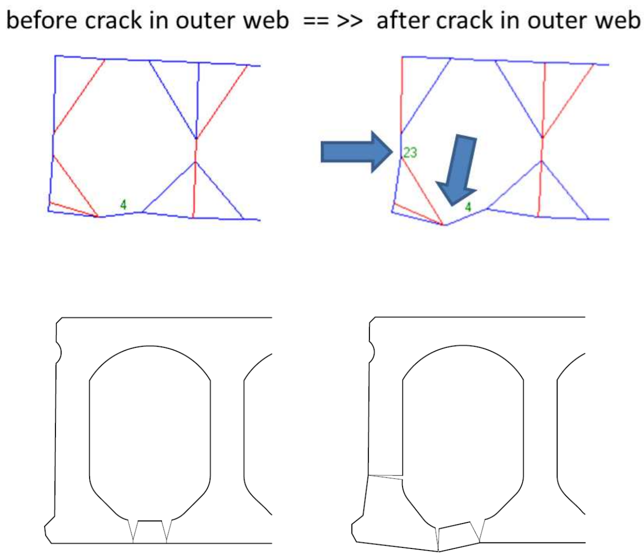


Figure 7.27. Release of the under flange by click-clack effect as shown by frame model (above) and sketched in a cross-section (below)

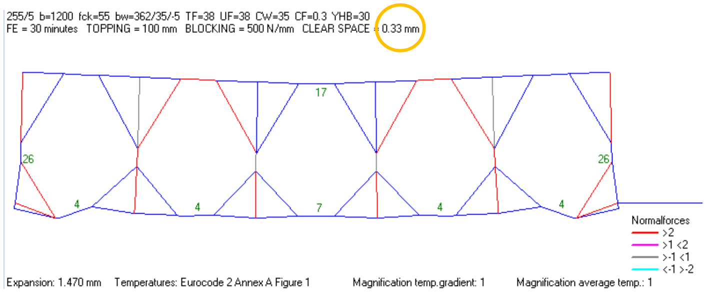
For each free (no blocking spring) cross-section the maximum allowable thickness of the topping is determined on which no horizontal cracks occurs. And then by increasing the blocking spring the needed free space is determined in such a way that no horizontal web cracking or buckling spalling will occur. The results are presented in Appendix 7.A. Under remarks it is indicated whether horizontal web cracking or compression failure occurred. With the ”click-clack effect” is meant that the under part of the outermost web rotates in order to release the under flange from restraint. See Figure 7.27 for an explanation. After this “click-clack effect” no horizontal cracking or buckling spalling can occur anymore.

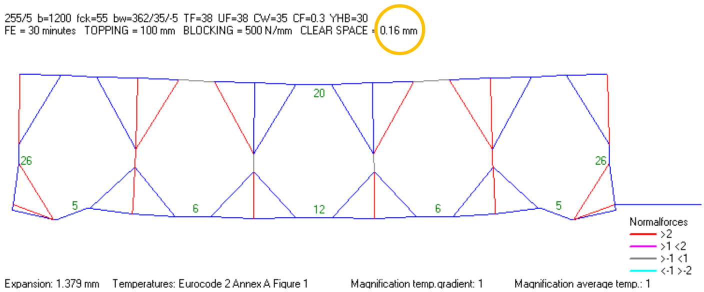
From the results in Appendix 7.A can be recognized that for cross-sections with the higher depths blocking in normal applications will not cause horizontal cracking or buckling spalling because the expected crack width, due to shrinkage in service time, is for these depths enough to prevent horizontal cracking. From the comparison of the results can also be seen that with respect to the thickness of the topping higher cross-sections are not that sensitive for horizontal cracking as the lower depths are. This is because of the fact that for lower depths the webs are relatively more rigid or with other words for the higher depths are the webs more flexible. A relation between the topping thicknesses where horizontal cracking can occur of 25 to 30% of the slab height can be recognized in the results presented in Appendix 7.A.

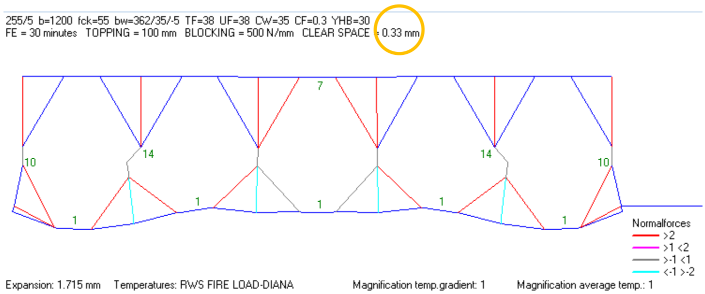
This effect was also seen in the fire test R3. The configuration of the floor in this test was a hollow core floor with a depth of 200 mm completed with a structural topping with a thickness of 50-70 mm. Horizontal cracking was observed at about 13 to 15 minutes. Nevertheless in the R3 test REI90 was granted. Further, the results as presented in Appendix 7.A show that with free spaces which are common for expose class XC1, no horizontal cracking need to be expected. In contrast to this, the limited shrinkage and thus limited free space expectable in XC3 can lead to horizontal cracks.

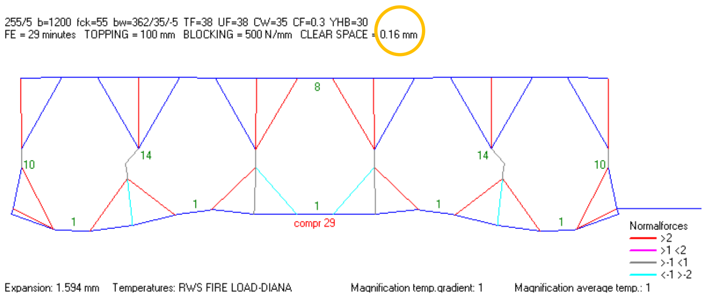
7.9. Assessment of the hollow core cross-section of the Rotterdam fire

The Holcofire Frame Model is used in this section to make an analysis of the hollow core cross section that was used in fire case parking Lloydstraat, Rotterdam as described in [7.13]. But in order to make a calculation, at first reasonable figures must be determined for the parameters “restraint” and “free-space”. In the background report [7.14] conducted by Van den Bos the lateral stiffness of a hollow core slab in a floor field is investigated. A floor slab consisting of prestressed hollow core slabs with a concrete topping is modelled with shell elements in the finite element program DIANA. The report deals specifically with the stiffness of the floor system in the Rotterdam Lloyd building under a local fire. Hence, the Rotterdam floor structure is surrounded by walls and connected to them. The span of the hollow core slab is 10700 mm. The width of the floor is 12600 mm, and the thickness of the topping is 70 mm (in reality it ranged 70-90 mm excluding an asphalt layer of 90-120 mm on the topping). The total height of the walls is 5620 mm. From the simulations it is concluded that the restraint over the height of the under flange is on average is 500 N/mm^2/mm.

<table>
<tr><td></td><td>XC1 (0.33 mm free space)
XC3 (0.16 mm free space)</td></tr>
<tr><td>ISO fire</td><td>




</td></tr>
<tr><td>RWS fire</td><td>




</td></tr>
</table>

Figure 7.28. Analysis with different fire curves and different free-spacings depending of the humidity grade

For the parameter “free-space” we can assume that joints in the floor are filled after 30 days after the hollow core slabs are produced. Then it is read out from the standard EN1992-1-1 that 40% of shrinkage is already established. Hence, 60% of shrinkage will remain and will cause shrinkage cracks in the joints. From EN1992-1-1 Table 3.2 the nominal unrestrained drying shrinkage for C40/50 is:

- XC1: RH= 40%, $\varepsilon_{cd,0} = 0.46\ ^{o}/_{oo}$, then crack width in joint equals 60% · 0.46 · 1200 / 1000 = 0.33 mm;
- XC3: RH= 80%, $\varepsilon_{cd,0} = 0.24\ ^{o}/_{oo}$, then crack width in joint equals 60% · 0.24 · 1200 / 1000 = 0.16 mm.

It is evident that under dry conditions the shrinkage and thus the free space is larger. Hence, for application of the floor under XC1 or XC3 the free-space is 0.33 mm and 0.16 mm, respectively. Now that the parameters are clear, four analysis are calculated, with the environmental conditions and fire conditions as the variables. As in [7.13] was concluded that the rate of heat release was much higher than in case of ISO 834 fire, we will consider in the calculations two fires: the ISO 834 standard fire, and the Rijkswaterstaat (RWS) fire. The latter one has a much higher heat release rate than the ISO fire which suits the purpose of this explanatory study. Hence, the following

Figure 7.28 gives all four calculations. In all calculations only 30 minutes is simulated, and the structural topping thickness is assumed 100 mm and restraint 500 N/mm.

From the calculations we can observe the following:

- Under the ISO fire and in XC1 and XC3 between 4 to 12 minutes the topside of the underflange cracks, and at 17 and 20 minutes a crack is initiated at the underside of the topflange. At 26 minutes the outer web cracks at the inside and the ”click-clack” mechanism occurs that releases the cross section. Hence, no horizontal cracks are initiated.
- Under the RWS fire and in XC1 and XC3 directly at 1 minute the topside of the underflange cracks, and at 7 and 8 minutes a crack is initiated at the underside of the topflange. Under XC1 at 10 and 14 minutes horizontal cracks are initiated, while in addition in XC3 also at 29 minutes buckling spalling occurs in the underflange.

Of the four calculations presented, the Rotterdam fire case most resembles the case with RWS curve and XC3 environmental conditions. When the calculated results are compared with the analysis from Chapter 5 [7.13], it is concluded that the combination of limited shrinkage crack widths at the joints (XC3, more blocking) and the extreme fire conditions (more temperature gradient) causes horizontal cracks at the webs and buckling failure of the underflange. These local damages are clearly visible at the photos depicted in [7.13]. Hence, with the Holcofire Frame Model we can clearly explain the local damage on the soffit of the slabs of the Rotterdam fire. Despite, the hollow core floor showed enough redundancy and did not fail under the loads present in the parking garage and fulfilled both R and EI. As stated earlier, redundancy is not accounted for in the Holcofire Frame Model as it models only a cross section of a hollow core in transversal direction. Nevertheless, this section concludes that we can explain the mechanism behind the local damage observed on some hollow core slabs that were present above the seat of the severe fire in Rotterdam.

7.10. Conclusions

The Holcofire Frame Model explains the occurrence of horizontal cracks; as well as for free cross-sections as for restrained cross-sections. Horizontal cracks in the webs can lead to local damage. By means of the magnitude of expansion due to blocking, the difference between blocked and free is marginal. The width of shrinkage cracks can be enough to enable the cross-section to expand without the occurrence of horizontal web cracks or buckling spalling. However we must realize that in unheated spaces, there where concrete structures are designed for environment class XC3, the moisture content will be higher and as a consequence the total shrinkage is lower. By this fact the free spaces will be smaller and the blocking forces in this type of construction larger and therefore the probability of buckling spalling and/or horizontal web cracking higher. See also Figure 7.29 for some photos of spalling, all taken from humid environments. Note that in environment class XC3 the moisture content is high and explosive spalling occurs.

In contrast to the Dutch authors [7.4 to 7.9] Holcofire concludes that blocking due to the topping is not the most decisive parameter in initiation of horizontal cracks. Holcofire concludes that the most decisive parameter in a cross section is the horizontal blocking force resulting from the internal and external structure and the free space. The topping however does play an important role and the critical influence of the thickness of the topping turns out to be dependent on the depth of the hollow core cross section. The thickness of the topping where horizontal cracking can occur is found to be 25 to 30% of the depth of slab.

Because the Holcofire Frame Model is a 2D cross-section approach, it cannot be stated that horizontal web cracks found in this model, or delamination of the under flange as a result thereof, results in a local failure because only cross sectional behaviour is considered and not the behaviour of the total floor.

It is a myth that concrete building structures only need to be cleaned after a serious fire; due to high temperatures in the reinforcement or the prestressing tendons the tensile strength of the steel will remain downgraded after the cooling down of the fire, dependent on the duration of the fire. Investigation and recalculation by a structural engineer must proof that the exposed structure can be re-used for a next period and whether additional provisions are needed.

Different types of concrete structures show similar damage after a severe fire. Some photos found in the internet are shown in Figure 7.29. In these photos buckling spalling, thermal spalling and explosive spalling can be observed on the concrete columns, concrete beams, concrete walls and concrete floors. It must be noticed that all photo’s found in the internet on damage to concrete structures due to fire are car-parkings, both environment conditions and fire conditions are extra ordinary. Looking into the details of hollow core floors at fire some specific behaviour can be noticed. But in general, the properties and the response to fire of hollow core floors is not different than other concrete floors: severe local damage but no failure due to the structural redundancy.

Figure 7.29. Different concrete structures; same type of damage

References Chapter seven

[7.1] TNO Bouw en Ondergrond, Report TNO-034-DTM-2009-00651. Brandwerendheid kanaalplaten, Deelplan A inclusief bijlagen, in opdracht voor BFBN. 24 juli 2009, p. 60 (main report) + p. 122 (Appendix).

[7.2] EN 1365 Fire resistance tests for loadbearing elements - Part 2: Floors and roofs, 1999

[7.3] EN 13501-1 Fire classification of construction products and building elements - Part 1: Classification using data from reaction to fire tests, 2009

[7.4] TNO Bouw en Ondergrond, Report TNO-2007-D-R1236/C. Onderzoek naar het constructieve gedrag tijdens brand van een kanaalplaat zoals toegepast aan de Lloydstraat te Rotterdam, in opdracht voor Veiligheidsregio Rotterdam-Rijnmond, January 2008, pp.12

[7.5] Kleinman, C., 2011, Kanaalplaat in dwarsrichting bij brand. Cement 5 (2011), pp. 6-13

[7.6] Hordijk, D., et al., 2011, Nieuwe maatregelen kanaalplaatvloeren (1), Cement 5 (2011), pp. 20-31

[7.7] Van den Bos, A.A., Joostensz, O., 2009, Brandsimulatie op beton. Cement 7 (2009), pp. 34-37

[7.8] Van den Bos, A.A., Joostensz, O., 2011, Uitgevoerde FEM-simulaties van de verkennende brandproeven. In: Nieuwe maatregelen kanaalplaatvloeren (1), Cement 5 (2011), pp. 20-31

[7.9] Breunesse, A., 2011, Analyses op basis van eenvoudige modellen. In: Nieuwe maatregelen kanaalplaatvloeren (1), Cement 5 (2011), pp. 20-31

[7.10] EN1992-1-2 Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design, 2011

[7.11] Vos, P. van, 2013, Brandschade aan betonconstructies parkeergarages – Adviesbureau ir. J.G. Hageman B.V., http://www.vexpan.nl/evenementen/lunchbijeenkomsten => Presentatie Hageman - October 3rd, 2013, http://www.vexpan.nl/?wpfb_dl=35 - October 3rd, 2013

[7.12] The concrete Centre, 2004, Concrete and Fire, ISBN 1-904818-11-0

[7.13] Jansze, W., et al., 2013, Fire case Parking Lloydstraat Rotterdam, Retrospective view, new insights and outlook, Chapter 5, this book, 24 January 2014.

[7.14] Bos, A.A. van den, en Joostensz, O., 2010, Hollow core slabs subjected to fire loading – Fire calculation for Rotterdam Parking

[7.15] fib Model Code for Concrete Structures 2010, ISBN 978-3-433-03061-5, 2013

[7.16] Jansze, W., et al., 2013, Fire tests to validate the load bearing capacity under restrained conditions, Chapter 6, this book, 24 January 2014.

Appendix 7.A – Free space depending on blocking due to spring and/or topping

Analysis of different cross-section configurations with Holcofire FrameModel Version 8; 30 minutes fire exposure.
Width of the cross-section is 1200 mm.

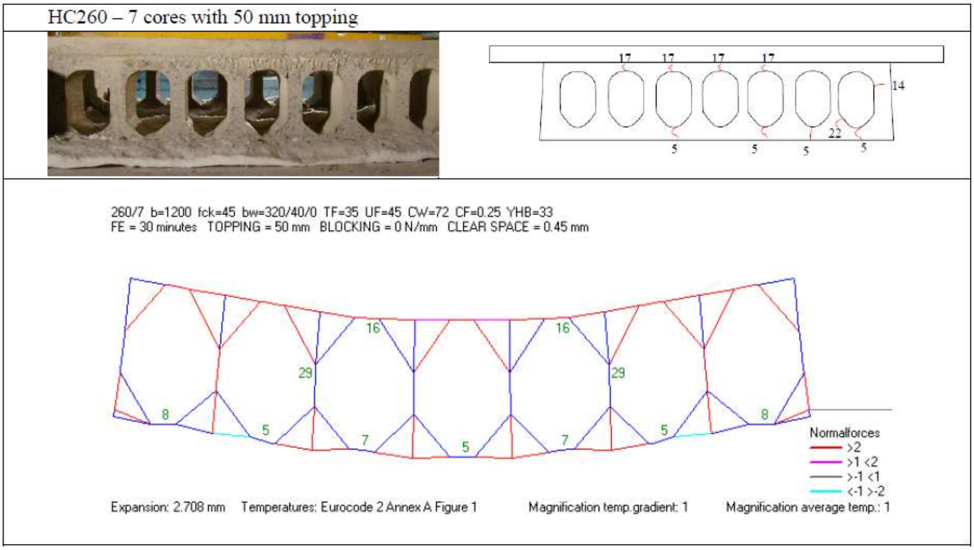
Cross-section	Topping [mm]	Blocking spring [N/mm]	Free space [mm]	Resulting expansion [mm]	Remark
HC400-4	0	2000	0.3	0.7	click clack effect
	100	2000	0.3	0.73	click clack effect
	100	0		2.7	web cracking
	100	300	0.3	1.76	
	80	0		2.8	no web cracking
HC400-5	100	0		2.9	no web cracking
	100	300	0.3	1.7	no web cracking
	100	500	0.3	1.26	click clack and compr.failure
	100	2000	0.5	0.85	click clack effect
HC400-7	100	0		2.3	2nd web cracks
	80	0		2.3	
	80	300	0.7	1.7	
	80	800	0.7	1.38	
	80	2000	0.7	1.1	
HC265-167	100	0		2.96	horizontal web cracking
(X-section G-series)	70	0		2.98	horizontal web cracking
	70	300	0.3	1.62	click clack effect
	70	800	1.2	1.67	click clack effect
	70	2000	1.5	1.69	click clack effect
	0	300	0	1.47	click clack effect
	0	800	0.3	0.94	web cracks and compr.failure
	0	800	0.4	1.09	click clack effect
	0	2000	0.4	0.73	click clack effect
HC255	120	0		2.84	horizontal web cracking
	110	0		2.9	
	100	300	0.3	1.78	click clack effect
	100	800	0.3	1.12	web cracks and compr.failure
	100	800	0.4	1.24	click clack effect
	100	2000	0.8	1.15	click clack effect
	70	300	0.1	1.7	
	70	800	0.3	1.18	click clack effect
	70	2000	0.7	1.07	click clack effect
HC260-7	110	0		2.69	horizontal web cracks
	100	0		2.72	
	100	300	0.4	1.76	
	100	800	0.5	1.26	click clack effect
	100	2000	0.9	1.22	click clack effect
	70	800	0.5	1.27	click clack effect
	70	2000	0.6	0.97	
HC200-7	100	0		3.16	hor web crack
	70	0		3.18	2 middle webs hor.cracks
	50	0		3.2	horizontal web cracking
	70	300	1.7	2.44	
	70	800	1.8	2.16	
	50	300	0.9	2	
	50	800	1.3	1.79	
HC200-11	100	0		3.26	hor web cracking
	70	0		3.23	4 webs compr crack
	70	300	1.5	2.3	4 internal web compr crack
	70	800	2	2.3	4 internal web compr crack
	50	300	1.2	2.15	4 internal web compr crack
	50	800	1.7	2.08	4 internal web compr crack

Appendix 7.B – Comparison Holcofire Frame Model with test results on slices of hollow core slab

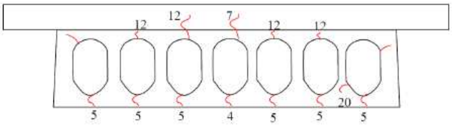
Comparison with test results on slices of hollow core slab with different thicknesses of the topping executed in The Netherlands in July 2010. The aim of these tests was to get information about the relation between the thickness of the topping and the occurrence of horizontal cracks.

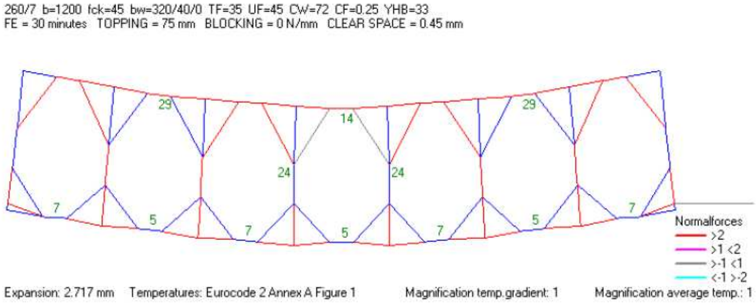
The samples consists of hollow core slabs with 7-cores; 260 and 400 mm height; thickness op the topping 50, 75 and 100 mm. The structural topping was additional connected to the hollow core with glue-anchors. See photo:

Figure 30. Production of the samples of slices of hollow core slabs

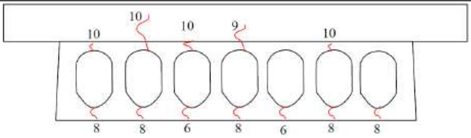


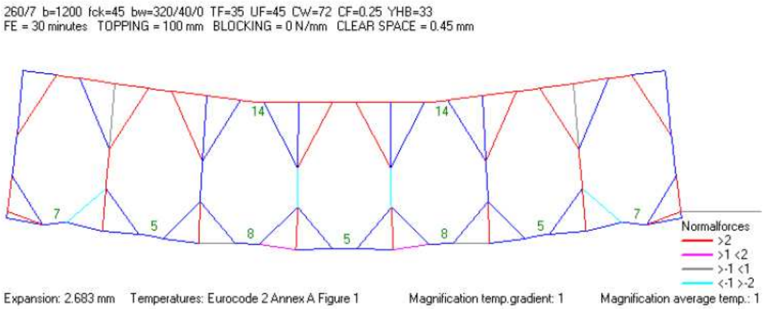
HC260 – 7 cores with 75 mm topping



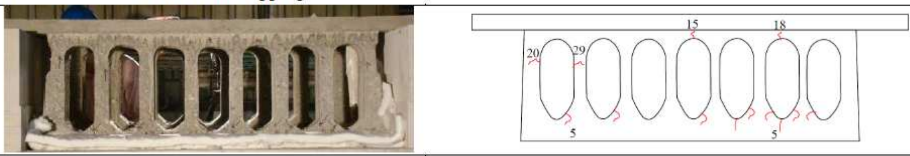


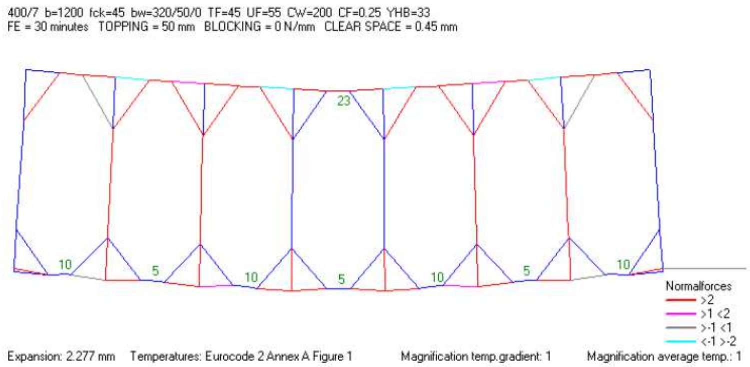
HC260 – 7 cores with 100 mm topping



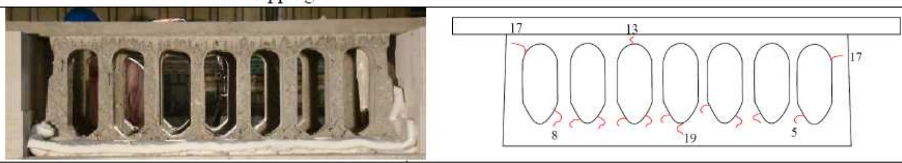


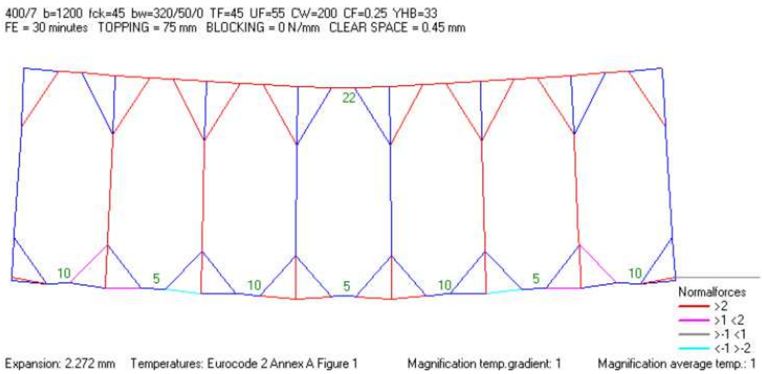
HC400 – 7 cores with 50 mm topping



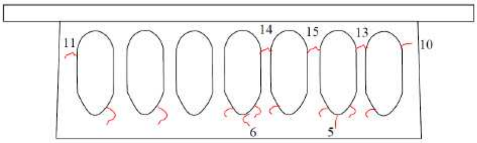


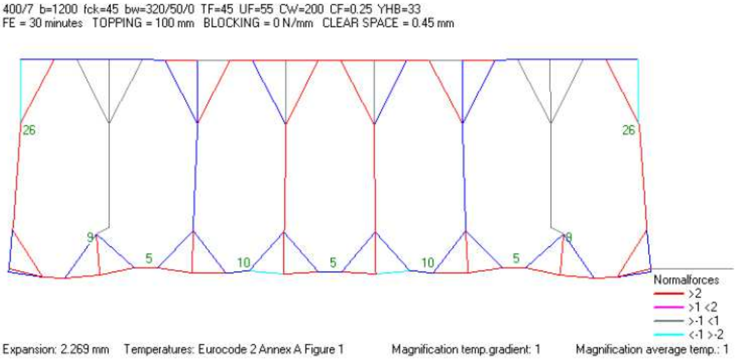
HC400 – 7 cores with 75 mm topping





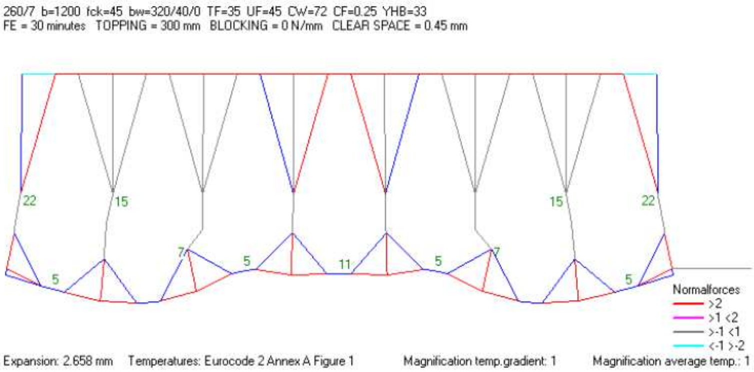
HC400 – 7 cores with 100 mm topping





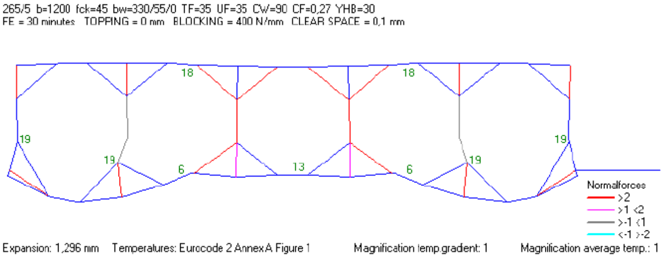
HC260 – 7 cores with 300 mm topping – slab 1200 x 1200

Webcracking after approx. 10 minutes



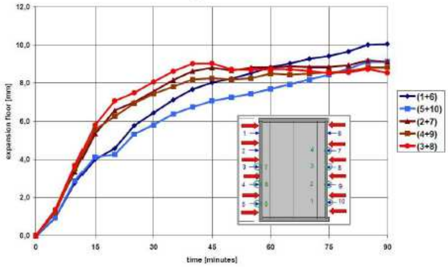
Appendix 7.C – Comparison of the results of the test series R with the Holcofire Frame Model

Test R1: Hollow core slabs HC255/5 – 100 mm topping
Remark: No further analysis with regard to horizontal cracking because shear-bending interaction failure was noticed.

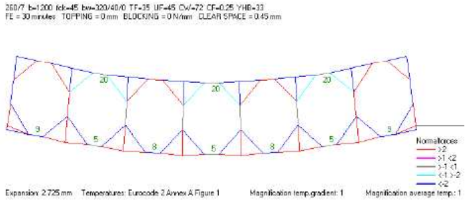
Test R4: Hollow core slabs HC265/5-167 - no topping
Remark: no measurements of horizontal displacements; after 21' a loud bang was heared.
Overview of the expected restraint: Analysis done: search with Holcofire Frame Model the springstiffness at which horizontal cracking occurs:  Result: time of horizontal cracking at 19 minutes, this is close to the observed time of horizontal cracking, that was 21 minutes. Derived magnitude of blocking spring: 400 N/mm (/mm)

Test R2: Hollow core slabs HC260/7 – 100 mm topping

Measured horizontal expansion:



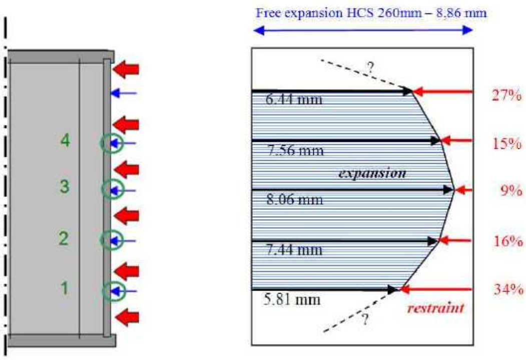
Expansion of free cross-section:



For test floor with width of 3.9 meter:

Free expansion = 2.725 · 3.9 / 1.2 = 8.86 mm

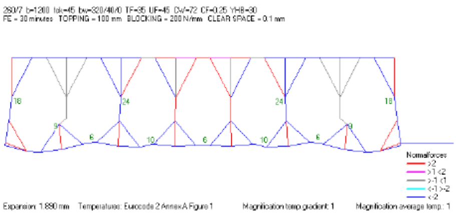
Overview of the expected restraint:



Near the support:

Average expansion per element:

0.5 · (5.81+6.44) · 1.2 / 3.9 = 1.88 mm

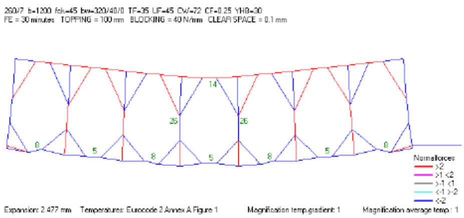


Derived magnitude of blocking spring: 200 N/mm

At midspan:

Average expansion per element:

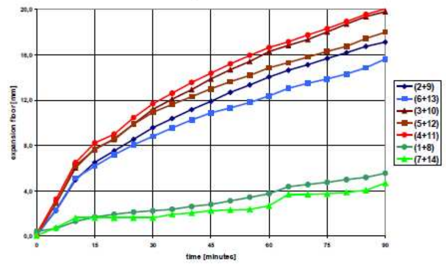
8.06 · 1.2 / 3.9 = 2.48 mm



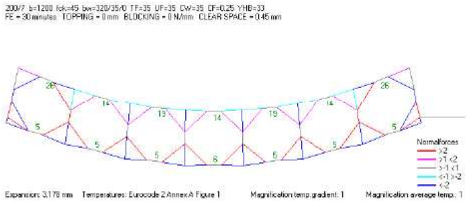
Derived magnitude of blocking spring: 40 N/mm

Test R3: Hollow core slabs HC200/7 – 50/70 mm topping

Measured horizontal expansion:

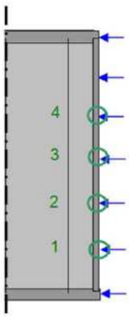


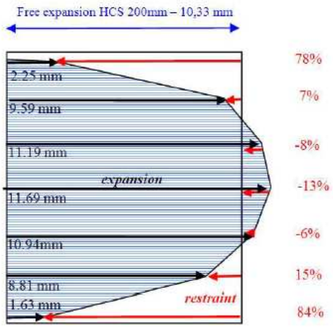
Expansion of free cross-section:



For test floor with width of 3.9 meter:
Free expansion = $3.178 \cdot 3.9 / 1.2 = 10.33$ mm

Overview of the expected restraint:

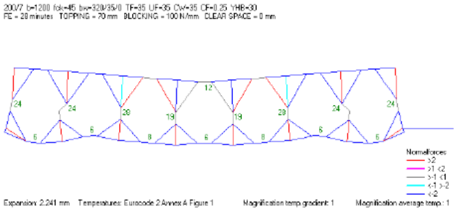




Near the support:

Average expansion per element:
$0.5 \cdot (8.81+9.59) \cdot 1.2 / 3.9 \cdot 10.33 / 11.69^{*} = 2.5$ mm

* = correction to free section



Derived magnitude of blocking spring: ≅ 100 N/mm

At midspan:

Average expansion per element:
Negligible: ≅ 0 mm ≅ free cross-section

Derived magnitude of blocking spring: 0 N/mm

Chapter Eight

8

Lessons Learned

Lessons learned on prestressed concrete hollow core floors exposed to fire

8.1. Holcofire lessons learned

The Holcofire lessons learned are discussed in more detail on the next page. For the intermediate conclusions per topic the reader is referred to the conclusions at the end of the respective chapters.

Holcofire lesson learned #1.
Product meets regulations and requirements

1.1 The precast hollow core floor and precast external wall applied in the Rotterdam car park still met the R criteria (loadbearing resistance) and EI criteria (separating function) after the fire was extinguished.

1.2 In real fires the hollow core floor proves to be more redundant thanks to the floor system effect. The fire duration time is a result of the robustness of the floor and alternative load paths present in the floor.

1.3 The fire resistance time of a hollow core slab floor can be established by testing it in accordance with the applicable testing standards EN1363-1 and EN1365-2. It is a widely accepted procedure to test only a single specimen and to approve it if it is able to withstand fire for the required duration without failure (REI-criteria).

1.4 When designing building elements, fire safety is achieved by specifying a safe value at the loading side (duration of the fire) taking into consideration the fact that the risk of fire occurring in practice has a low probability of occurrence.

1.5 All available regulations and requirements for hollow core slab floors under ambient conditions and under fire conditions have been derived from and verified on the basis of real experiments, which is more than can be said for many other structural products.

Holcofire lesson learned #2.

Product performs well when exposed to fire

2.1 The 162 independent fire test results from the Holcofire database confirmed that, if the resistance models currently available (EN1168, EN1992-1-2, EN1363-1, EN1365-2) are strictly followed, 94.5% of the fire tests can be fully explained.

2.2 The fire tests G series and the 42 fire test results from the Holcofire database with shear and anchorage failure confirm the model in EN1168 Annex G for shear and anchorage capacity under fire conditions at various heights. It does not account for the positive contribution of the system effect.

2.3 For hollow core slabs on flexible support, fire conditions do not reduce the shear capacity more severely. The shear capacity for flexible supports can be calculated using EN1168 Annex G in the same ways as done for rigid supports.

2.4 The fire tests R series and the Holcofire frame model clearly explain the phenomena of horizontal cracking and under flange spalling due to restraints, but also confirm the conclusion that these are not failure mechanisms.

2.5 Overall, the past performance of 1,000 million m^2 of hollow core floors in Europe confirms the good performance of the hollow core floor under fire conditions. There are no cases known where the safety of people and structural stability were jeopardised.

Holcofire lesson learned #3.

Scale of fire in car park in specific cases is more severe than standard fire

3.1 The simulation of the fire with dynamic software FDS5 yielded better insights in the fire development in the Rotterdam car park compared to static CaPaFi calculation made by Efectis. Contrary to CaPaFi, FDS5 takes into account the real dimensions of the structure and the ambient environmental conditions which greatly impact the calculated results.

3.2 The fire in Rotterdam was much more severe than an ISO fire simulated in laboratory conditions due to the consecutive burning of six cars. The local damage to the floor was increased due to the high moisture content in the concrete floor and (dynamic) force from the fire boat that extinguished the real fire after 45 minutes.

3.3 Real fires in car parks are accidental, severe and unpredictable, and will due to restraints, always cause local damage to any flooring structure - hollow core floors as well as other precast floors, cast in-situ floors and composite solutions.

3.4 In car parks, the travelling fire concept should be used instead of the well-known ISO compartment fire concept. This ISO fire is unlikely to happen for a fire compartment measuring 2,100 m^2 over several floors, while the many cars in the car park are more likely to induce a travelling fire which, by definition, is more severe than an ISO fire.

3.5 Fire Safety Engineering is a performance-based approach that, compared to the prescriptive based approach of an ISO fire, should be advocated by the concrete industry in order to arrive at a more realistic fire load for the structure.

8.2. Technical summary and conclusion

In concrete floor construction the precast hollow-core slab is a very successful product. This success is largely attributed to its highly efficient design, structural efficiency and lean production method. Every year around 20 to 25 million square metres of precast concrete hollow-core floors are erected in Europe. The estimated total stock of hollow-core floors cuurently installed in Europe is 1,000 million square meters. Experiences with past performance of hollow-core floors confirm that under fire conditions hollow-core floors have excellent fire resistance.

Some cases of premature shear failure in fire tests in the years 2000s and the lack of theoretical shear capacity models under fire, led to reluctant clients, although in practical applications shear hardly governs floor design. End of 2007, a heavy car park fire in the parking garage in the just completed apartment building in the Lloydstraat in Rotterdam revamped again the attention of regulatory institutions on the fire resistance of hollow-core floors. The structure in the Lloydstraat was locally severly damaged, but all REI requirements were met, failure of the structure was not the case in this situation. Both incidents damaged the good image of the hollow-core slab among clients and authorities in some European countries.

The European project “Holcofire” was therefore initiated by the BIBM with the objective to get full understanding of the behaviour of prestressed concrete hollow-core slab floors under fire conditions in order to regain full acceptance for the application of hollow-core slabs under fire conditions. The Holcofire project consists of meta-analysis, laboratory fire tests, finite element simulations, and calculations conducted by experts in the field of precast hollow-core floor construction and fire testing.

A database covering the years 1966-2010 was set up with 162 fire test results in order to perform a meta-analysis over the fire tests. It is concluded in that 94.5% of the database can be fully explained with the design models and requirements stated in the available European standards (EN1992-1-2, EN1168, EN1363-1, EN1365-2). The other 5.5% is dealt with in the Holcofire study as a specific research subject.

In performed Holcofire tests (the G series) the shear formula presented in EN 1168/A3 Annex G was compared with 42 fire test results from the database and the Holcofire fire tests G1 to G7. It is concluded that with this EN1168 Annex G formula for shear and anchorage resistance under fire, the designed hollow-core floor is safe for the ultimate limit state in the accidental situation. In a subsequent desk-study it is concluded that the EN1168 Annex G formula can also be used to determine the shear and anchorage resistance under fire for hollow-core floors on flexible supports.

The Rotterdam fire case has been analysed in a retrospective view, with in-depth analyses leading to new insights. In a finite element simulation of the real fire of Rotterdam with FDS5 software led to the conclusion that the fire was far more severe than an ISO fire due to the travelling characteristics of the fire, leading to a 33% rise in temperature above car

1 in 20 minutes and a threefold temperature increase rate. A clear explanation of the successive phases of delamination of the bottom flange is given.

In performed Holcofire tests (the R series) the capacity of hollow-core slabs under restrained conditions was investigated. The overall conclusion reached is that high floor restraints due to horizontal blocking and thick topping can lead to buckling spalling and horizontal web cracking. However these are concluded not to be failure mechanisms. Under the design load and well anchored strands, the fire resistance was still met through the structural redundancy and alternative load paths in the hollow-core slab floor.

The Holcofire Frame Model clearly shows that the initiation of horizontal web cracks and buckling spalling at the soffit can clearly be simulated with a limited number of parameters. It is concluded that it is not the thickness of the structural topping, but the magnitude of transversal restraint that has the most influence on both phenomena. Shrinkage cracks and dilatations in hollow-core floors used in practice are enough to keep these transversal blocking effects at such a low level that horizontal web cracking and buckling spalling of the bottom flange are unlikely. This explains why these local damages are only seldom observed in practice. And when observed, they are incidents, like the Rotterdam fire case where the fire was far more severe than an ISO fire.

Real fires in car parks are accidental, severe and unpredictable and due to blocking effects will always cause local damage to any flooring structure; hollow-core floors, but also other precast floors, cast in-situ floors and composite solutions.

The Holcofire study concludes that the proven track record of more than 1,000 million m^2 of hollow-core floors in Europe plus the extensive testing of hollow-core slabs in laboratories and the analysis of the real fire in the Rotterdam incident confirm once again that hollow-core floor systems meet all regulatory, quality and safety requirements. The Holcofire lessons learned are, firstly, that the product meets regulations and requirements; secondly, that the product performs well when exposed to fire; and thirdly, that in specific cases real fires in car parks are far more severe than standard fires. Based on the knowledge and experiences gained in this European project carried out by experts and reported on in this book, there is no need anymore for further fire testing and modelling. Society can continue to rely fully on the structural solid performance of floors consisting of hollow-core slabs.

The BIBM Holcofire end report published in this book shall be, if needed, the bases for each country to make reviews and/or follow-up research for the application of floors consisting of hollow-core slabs. It is the BIBM Holcofire's opinion that further research-testing on hollow-core slabs with regard to fire resistance is not necessary. All available regulations and requirements for hollow-core slab floors under ambient conditions and under fire conditions have been derived and verified on the basis of real experiments.

If approval fire tests are necessary, it is recommended or even mandatory to follow the existing European standards for fire testing on floors (EN1363-1, EN1365-2). These standards apply to any floor structure and thus also for hollow-core slabs.