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28 FIRE ENGINEERING APPROACH – CASE STUDY "DE POSTHOORN" AT HAMONT

Summary

In 2007, a new transformation and extension of the existing hall was done. Due to a misunderstanding about the regulations, there was no fire protection at all foreseen for the transformation. The new part of the building that contains a stage needed to fulfil the latest legal requirements. In this case, this implies a safety level equal to that associated to a fire resistance of 30 minutes ISO834.This became clear after the stage was fully equipped with all technical installations.

28.1 INTRODUCTION

The building of this case study is an existing candle factory which had already been transformed in the past to a local village hall. Because this previous transformation was performed before implementation of regulations concerning safety and fire resistance, there was at that time no special measure taken about this issue.

In 2007, a new transformation and extension of the existing hall was done. Due to a misunderstanding about the regulations, there was no fire protection at all foreseen for the transformation. The new part of the building that contains a stage needed to fulfil the latest legal requirements (Fig. 28.1). In this case, this implies a safety level equal to that associated to a fire resistance of 30 minutes ISO834.This became clear after the stage was fully equipped with all technical installations.

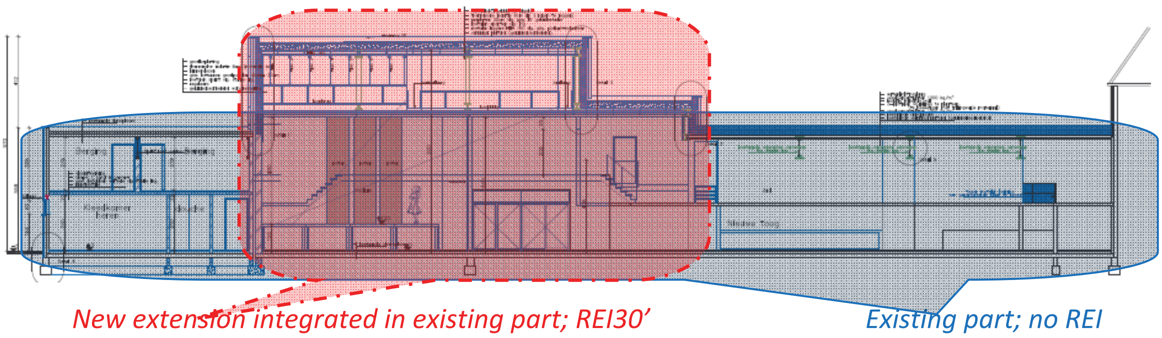

Fig. 28.1 Section over the whole building

Because the renovated hall is located in the centre of the village, also adequate acoustic insulation criteria needed to be fulfilled. Those are the most stringent for the stage and for that reason a massif 30 cm concrete roof was needed. To support this roof two main beams where needed, spaced at about 4,85 m with a span of 22,38.

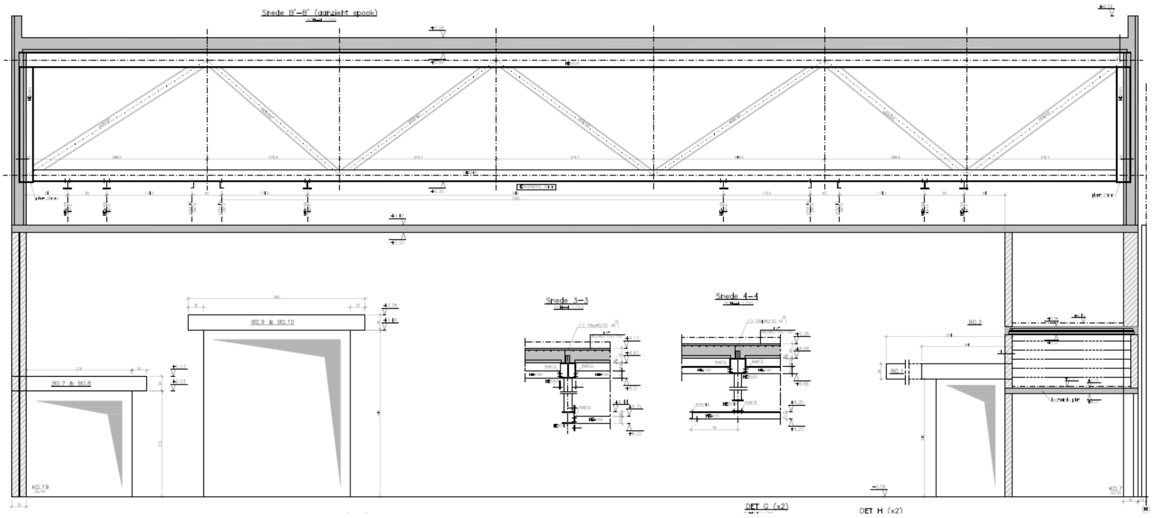

Fig. 28.2 Section of the stage

To avoid a building height higher than 10 m and to make the technical equipment accessible by the aid of catwalks, a steel truss girder with irregular diagonal slopes was chosen. A concrete beam would indeed be rather heavy and couldn't make an access possible with the catwalks because it would be too deep (about 2 m).

28.2 ORIGINAL DESIGN

For the design of the steel truss girders, the framework software "Power frame" from the company Buildsoft was used. Deadweight and variable loads from the roof act on the upper chord of the truss girder; loads are also applied on the secondary beams supporting the catwalk.

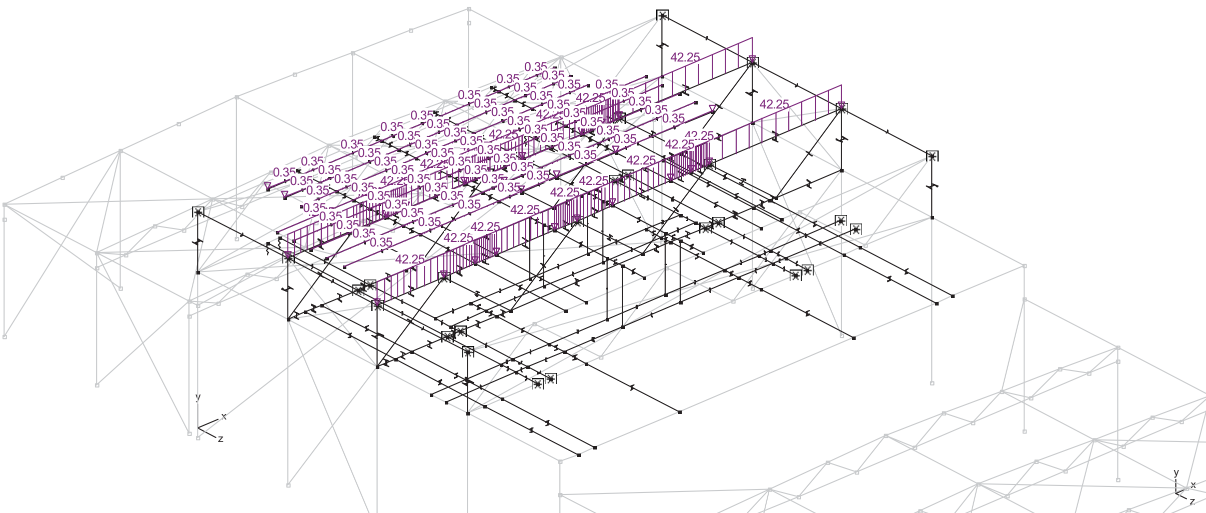

Fig. 28.3 Dead loads acting on the steel structure

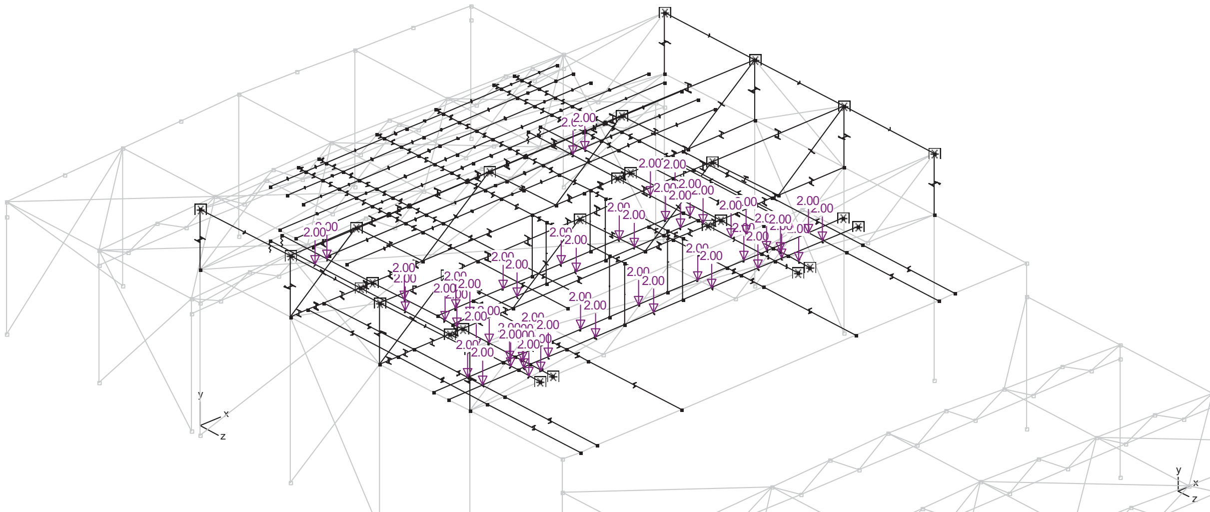

Fig. 28.4 Mobile loads acting on the steel structure, considered as concentrated loads

Effects of wind and of the own weight of the structure were also taken into account. Verification was done for service and ultimate limit states. A summary of the applied loads can be found in Tab. 28.1. Verifications resulted in a resistance ratio of 96.2% and a stability ratio of 98.9%, both number being given for the most critical element in the structure, see Fig. 28.5.

Tab. 28.1 Loads acting on the steel structure

Load	Ψ	kN/m²	Continuity factor	Service load in kN/m	Load in case of fire kN/m
Own weight	1	Variable	-	Variable	Variable
Dead load	1	0,3×25+0,28	1,12	42,25 + tracks	42,25 + tracks
Snow load	0	0,5×0,8=0,4	1,12	2,18	0
Mobile load, maintenance	0	1-A/100 or concentrated 2kN	-	Conc. Loads 2 kN	0
Wind actions	0.2	0,63×(0,75+0,3)	1,0	3,21	0

For the mobile loads, it appeared to be more critical for the catwalks to consider the concentrated load instead of the distributed loads, because of their small surface. For the main beams on the other hand, where the surface is higher than 60 m², the surface load was considered. The dead load is composed by the own weight of the concrete roof slab spanning on four supports+ thermal insulation + watertight membrane. This load is considered with a continuity factor in order to take into account the effect of the three unequal spans in the concrete roof.

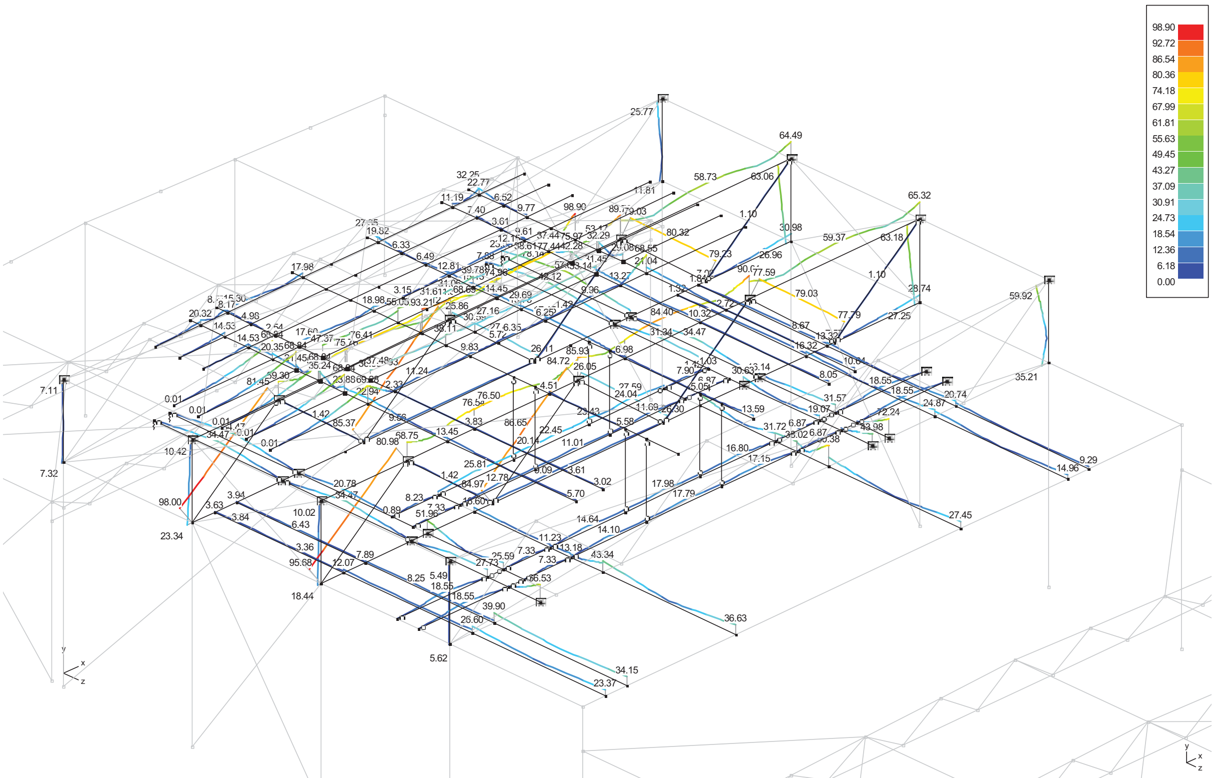

Fig. 28.5 Result of the stability check

28.3 FEASIBILITY STUDY

To get a first idea of the stability in case of fire, the thermo elastic module of the same software was used but it became immediately clear that it is not possible to prove a stability of 30 minutes with a thermo elastic model because the restraint to thermal expansion generates huge stresses. In this building especially, there is significant restraint caused by the concrete elements.

After this first attempt, a more advanced finite element model was used for the analysis of the structure. Concrete walls are modelled with plate and wall elements but without membrane effects. Catwalks are not considered in the model as a steel members but as an extra fixed load of 1,75 kN/m on the upper chord of one truss girder. Only the fact to consider the collaboration between steel and concrete (in bending) we could already improve the resistance and stability verification checks. The values presented in Fig. 28.5 are reduced to36 % and 56 % for the most critical elements and same loads.

The hypothesis is made that concrete will be hardly affected by a 30 minutes fire. Another appreciation of the behaviour of the structure was obtained by affecting the strength and stiffness of the S235 steel in the model mentioned above by the reduction factors mentioned in Table 3.1 of EN 1993-1-2.

This model shows that the secondary elements are working in compression, see Fig. 28.8.

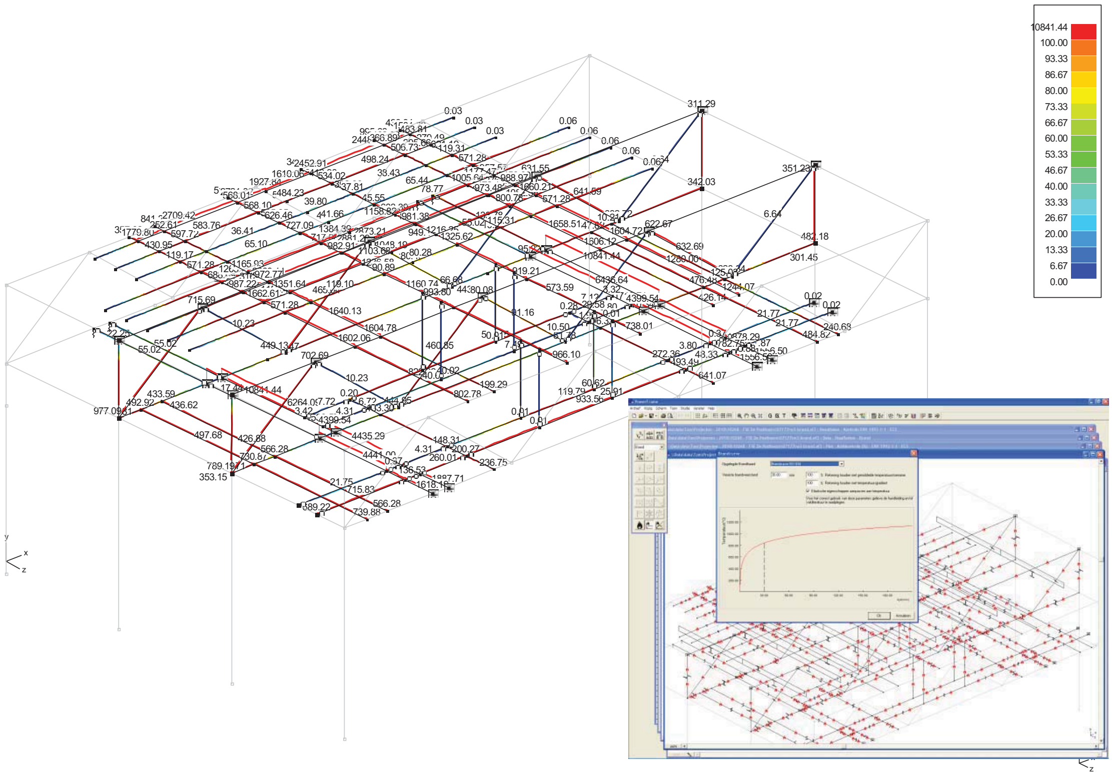

Fig. 28.6 Result of the resistance/stability check with a thermo elastic approach

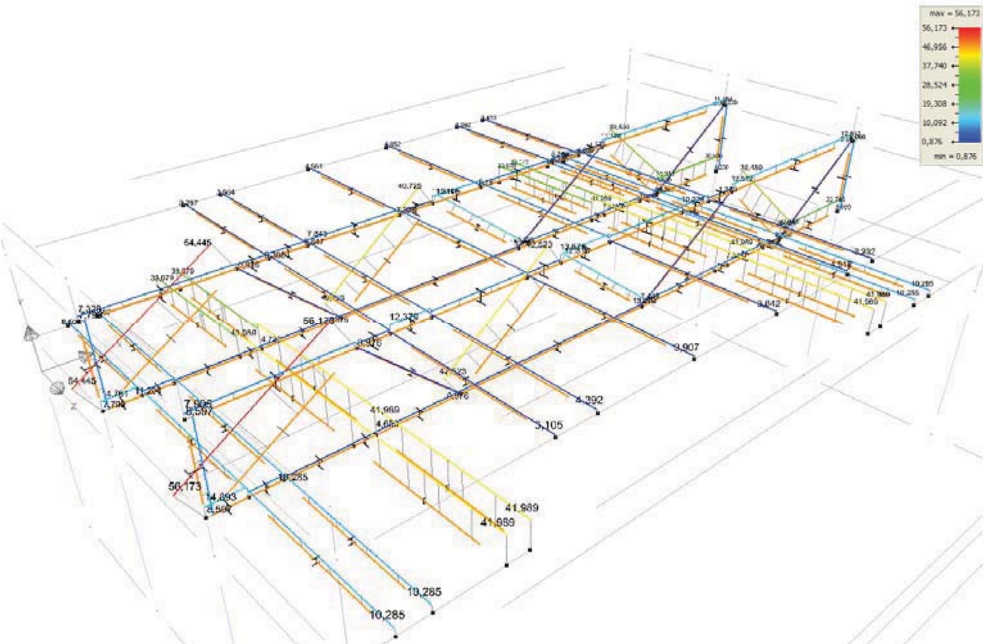

Fig. 28.7 Result of the resistance/stability check

Tab. 28.2 Values from table 3.1 EN 1993-1-2

	$k_{y,\theta} = f_{y,\theta}/f_y$	$k_{p,\theta} = f_{p,\theta}/f_y$	$k_{E,\theta} = E_{a,\theta}/E_a$
700°C	0,230	0,075	0,130
800°C	0,110	0,050	0,090
900°C	0,060	0,0375	0,0675

Tab. 28.3 Temperature in steel after 30 minutes of ISO834 fire

Profile	exposed sides	Temperature °C	Effective yield strength N/mm²	Youngs modulus N/mm²
UPN140	4	825	22,9	17719
HEA160	4	824	22,9	17719
HEB160	4	801	25,9	18900
HEB200	4	784	25,9	18900
HEB240	4	768	34,9	21588
HEB320	3	746	41,1	23436

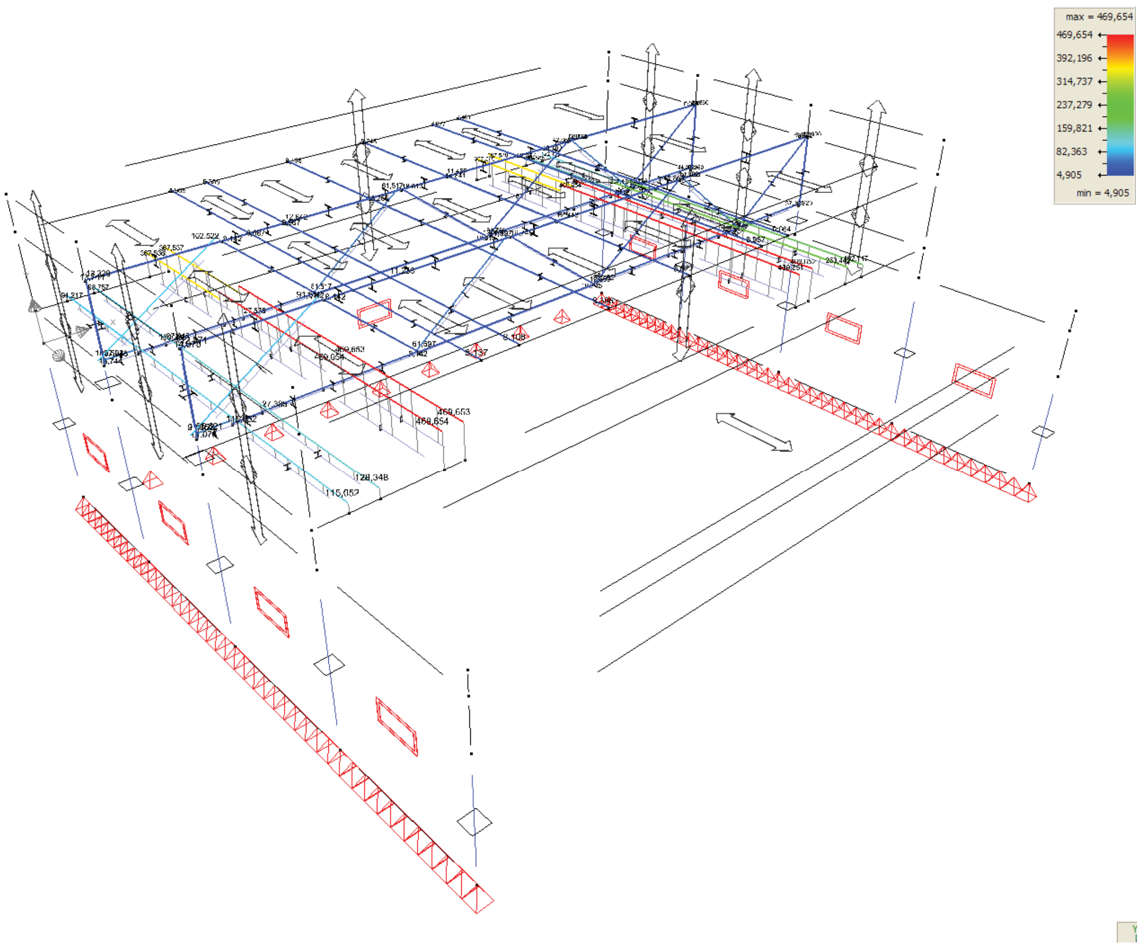

Fig. 28.8 Result of the resistance/stability check with reduced stiffness

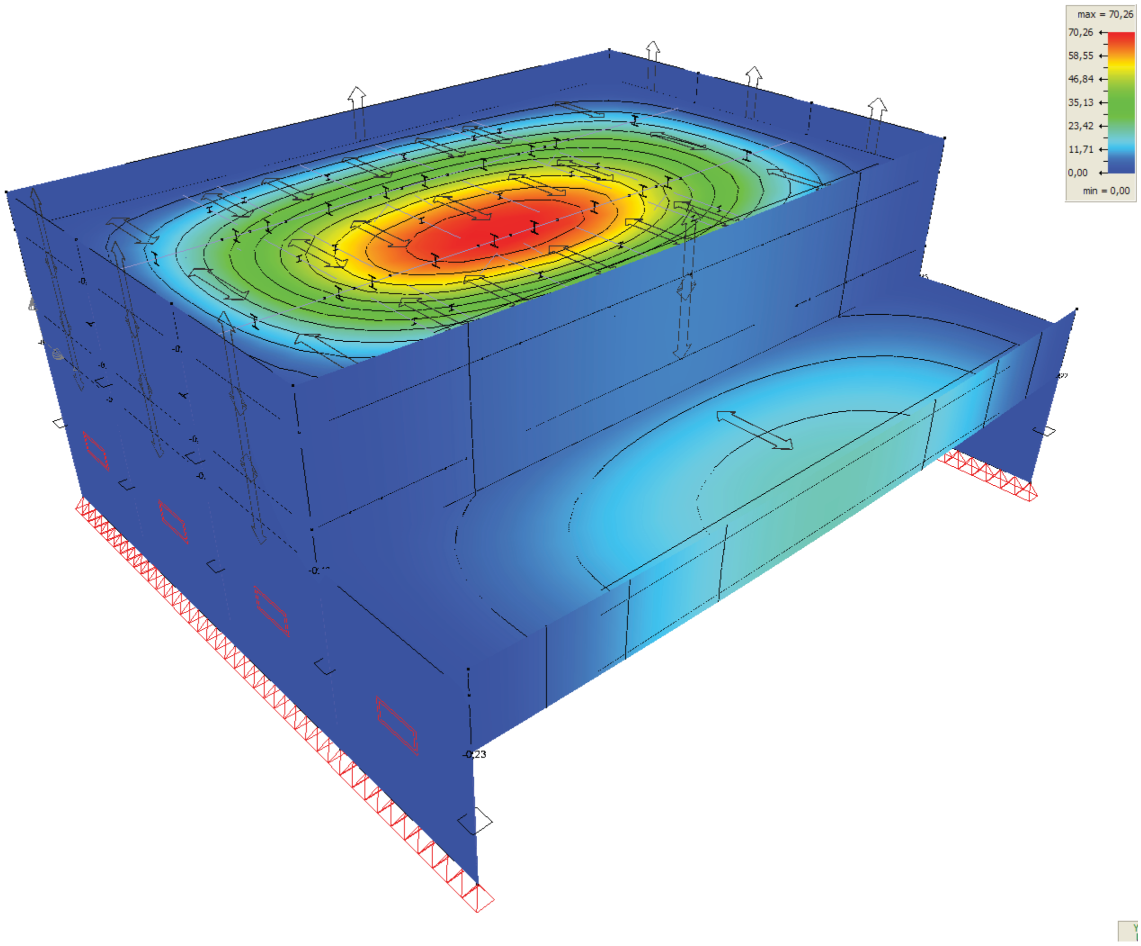

Fig. 28.9 Deflections with reduced steel stiffness

It could be expected that rather high deformations will occur. This is in fact the case as can be seen on Fig. 28.9 where the roof deforms in a membrane like shape. This model showed that the reinforcement calculated from the bending moments on an elastic way is much higher than what had been foreseen: Lower 450/141 mm²/m and upper mesh 150x150x8x8.

28.4 NATURAL FIRE DESIGN

The temperature development in the compartment was estimated with the two zone model Ozone, Cadorin and Franssen (2003), Cadorin et al. (2003). A fire curve obtained by this model is shown on Fig. 28.10, together with the temperature development in a HEB240 steel section subjected to this fire curve.

28.5 FINAL STUDY

The simple analysis model is not able to prove stability of the structural system, even in the case of a natural fire solicitation. With the large deflections that are observed with the simple model, it is clear that tensile membrane action in the concrete slab will take place and should be considered in the load bearing capacity, which is not possible in the simple model (leading to excessive amount of reinforcement). It will thus be necessary to use a more refined structural analysis tool that would be able to take into account the effects of tensile membrane action.This is the case whatever the fire scenario that is being considered, either the ISO834 fire curve or a natural fire curve.

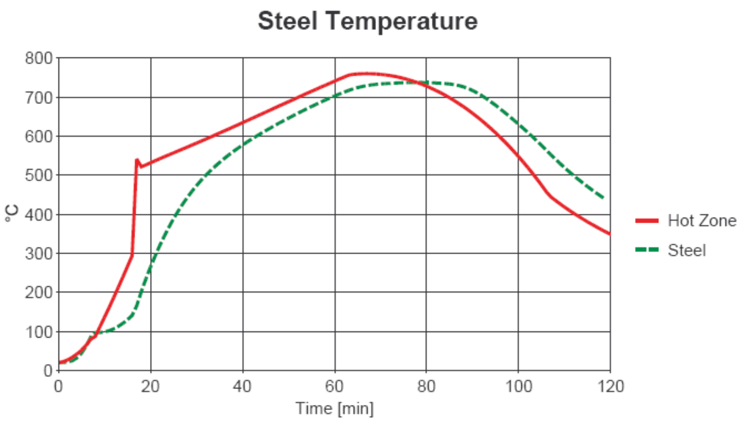

Fig. 28.10 Result of Ozone model

28.6 CONCLUSIONS

The analyses were conducted on a structure that is essentially made of a concrete slab supported by two unprotected steel girders. An analysis based on thermo-elastic model confirmed that such model is totally inadequate when some degree of restraint is involved. A second model was built in which no degradation was considered in concrete and the strength of stiffness of steel where modified according to the reduction factors of Eurocode 3. This model working in small displacements is not able to tackle the tensile membrane action that develops in the slab due to the large displacements that occur when the steel truss girders lose their strength and stiffness.

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29 RECONSTRUCTION OF WAREHOUSES IN BUDAPEST IS THE BIRTH OF CET

Summary

In this paper the case study about the construction of the CET building in Budapest is presented. CET is a mixture of historical and new architecture, it is a 160 meter long "whale" of steel, glass, and aluminium, which is situated among Budapest's historical buildings. The historical part is the rest of the Warehouses, completed in 1881 and remained one of the capital's main distribution centres until World War II. The Warehouses was damaged by heavy bombardments and the area didn't develop anymore. The reconstruction was an idea to save this historical monument. The basic of the fire concept was the performance based fire engineering, because the modern notion of the structure and building shape didn't match with the Hungarian Fire code. The two different main part of the building, a huge atrium like a retail area (the body of the whale) and a multi floors event centre (the head of the whale). Both of the smoke and heat exhaust system of the main part was analysed by the FDS. It was centralized to the circumstances of the evacuation and the temperature of the area around the structures. The result of the new view point brought more new solutions, which make reality from this building in Hungary.

29.1 INTRODUCTION

After the Revolutions of 1848-49 and especially after 1867 the economy of Hungary began to recover and this new period also resulted in a commercial blooming that took place in Budapest. In 1875 a decision has been made. The warehouses had to be positioned under the toll-house. Finally, after an immense amount of disputes, in 1879 the decision was born, namely the building of four storage rooms. The implementation of the plans finished on the 30th of September in 1881. These four buildings have been connected with an elevator that pumped through the corn from the steamships. The whole structure (including the neighbouring warehouses) has been destroyed during the Second World War.

The area is in its present state since 1966; one of the four storage houses has been demolished because it was greatly damaged during the bombings of the Second World War. The elevator and the neighbouring warehouse have also been demolished and Park Nehru opened there in this year.

During the last years it has been brought up that the area should be put to use once again; rebuilding the old, monumental buildings with the help of new architectural elements.

Fig. 29.1 Visualization of CET Budapest

Fig. 29.2 View of CET Budapest (Januar 2012)

Finally, the outline of the new building was born and the dilapidated warehouses have been rebuilt; CET stands there now, a new construct that could become a new symbol of Budapest. There used to be four warehouses at the bank of the River Duna in the district Ferencváros but one of them was totally destroyed during the bombings of the Second Word War.

2 GENERAL BUILDING DESCRIPTION

At the CET Budapest the architect designer is Kas Oosterhuis, the general main contractor WHB Építő Kft.

The name CET refers to the local time zone (Central European Time) and to the whale shape of the building (cet means whale in Hungarian). The first plans were slightly modified since the architects had to round off the whale’s nose and the system of the coating was also modified but these are only minor changes. The space between the two buildings will be covered with a glass cap and this way it will look like the above mentioned animal or something like the waves of a river.

The total territory of the new building is 31000 square metres from which 12500 square metres is the usable area. What you can find here is a room with space for 1200 people that is maintained for programmes, an underground garage with 250 parking spots, galleries, book shops, restaurants, cafes and nearly 200 meter long terraces that are situated at the bank of the River Duna.

Regulations prescribe the 60% retention of the three original storage buildings. A 20 metre part of the buildings' Northern side is going to be demolished so that the future construct will be easier to approach (thanks to a 20x50 metre square). The third (Southern) warehouse will be almost totally removed.

The appropriate linking of the modern architectural forms to the older parts is a great challenge. The entrance that is made up of a glass structure conjugates the two solid brick walls. The arched glass rooftop is the main visual element, it gives an iconic appearance to the core body of the building and it also creates an inner space with a unique atmosphere that is sunlit and protected from the weather conditions. All the steel, aluminium and glass elements of the roof are different and economic to produce (thanks to computer studies). These materials have to be connected with the elements of the old building (namely wood, brick and stone).

The body of the whale will be 160 metres long and the width of the glass rooftop will be 18 metres. The glass rooftop that has a triangle cross-section with its upper part slightly leaning out touches the top of the warehouses (from the North). Heading south it slowly turns into a curve, then overhanging the warehouses it turns into a glass shade that is the whale's head.

29.3 FDS SIMULATION

29.3.1 The Hungarian Law prescribes the following:

On the basis of the Hungarian Fire Code (OTSZ) the heat and smoke exhaust system has to be as big as 3% of the atrium's floor space. Hence, a surface of this size is required for both exhaustion and air inflow. If we would like to set up the exhaust system with the help of mechanical devices then every square meter would receive 2m^3/s air inflow. There are two atriums within the building. One in the 'body' and one in the 'head' of CET. The surface of the 'body' structure's atrium is 1000 square meters, while the 'head' structure has a floor space of 590 square meters.

That is why a 218700 m^3/h suction and a 30, 34 m^2 air inflow space has to be provided in the 'body' and a 126500 m^3/h suction and a 17, 56 m^2 air inflow space is required in the 'head'.

According to Hungarian Fire Code (OTSZ), smoke and heat exhaustion (in the case of mechanical exhaustion) has to be started on the level of the fire and on the levels above the fire level.

Tab. 29.1

According to OTSZ	Suction	surface for air inflow
'Body'	218 700 m^3/h	30,34 m^2
'Head'	126 500 m^3/h	17,56 m^2

29.3.2 Why was it necessary to approach the project from an engineer's perspective?

It was problematic in both the 'body' and the 'head' structure that the required amount of air inflow could not be provided (due to the architectural conception). Only half of the ordered surface was available. The area of the 'head's atrium is interconnected by joists. We were afraid that smoke and heat exhaustion (that is started on both levels) would quicken the upwelling of smoke. It was expected that exhaustion launched on the level above the fire would have a sucking effect on the fire level. This would have led to a faster spreading of smoke.

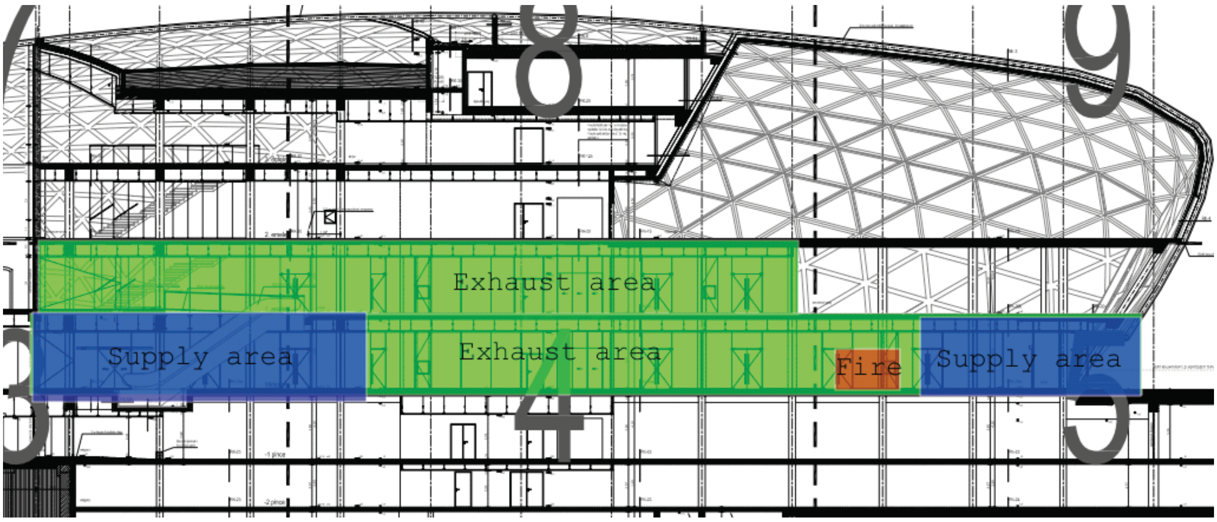

Fig. 29.3 The Smoke control zones

29.3.3 Analysis of the 'body' and the 'head'

The area for fresh air inflow was smaller than the one that is ordered by the fire code (in both the 'body' and in the 'head' as well). In the body this surface was only 10, 06 m^2 instead of the ordered 30, 34 m^2 and in the 'head it was 14, 4 m^2 instead of the required 17, 56 m^2. In Hungary you must have the permission of the National General Directorate for Disaster Management (Országos Karasztrófavédelmi Főigazgatóság) if you would like to implement a heat and smoke exhaust system that does not suit the requirements of the fire code. In order to acquire such a permission you need to justify that the system you would like to set up can function reasonably. We examined this operability with the help of computerized fire-simulation. The base data of such a simulation has to be set up in accordance with the authorities. These data are the following: the heat release of the fire, the location of the fire, the smoke-generating ability of the fire, and the duration of the survey.

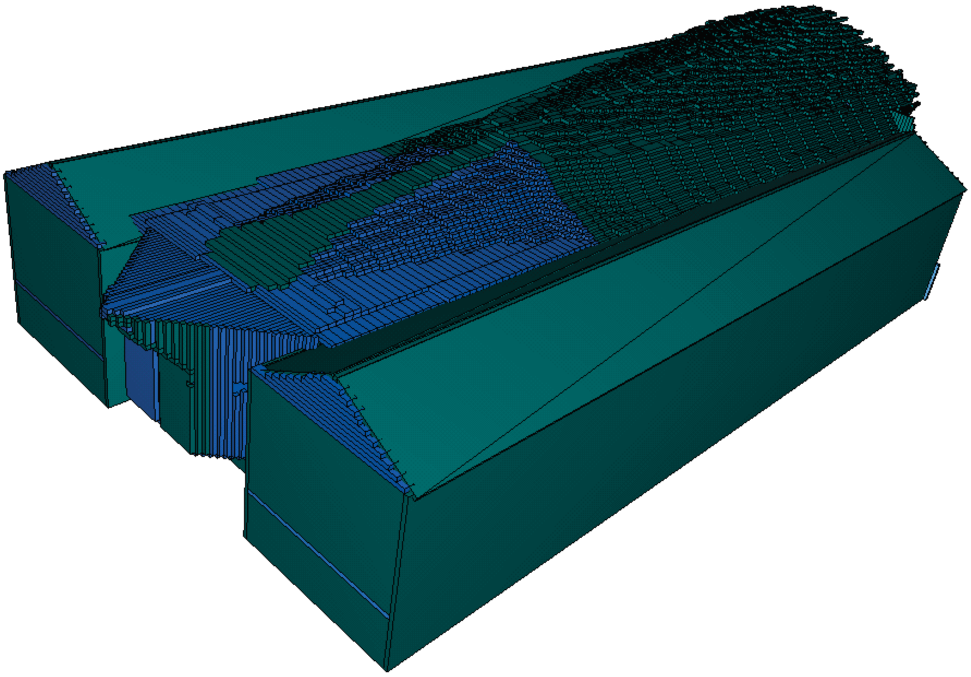

Fig. 29.4 ‘body’ structure from a distance

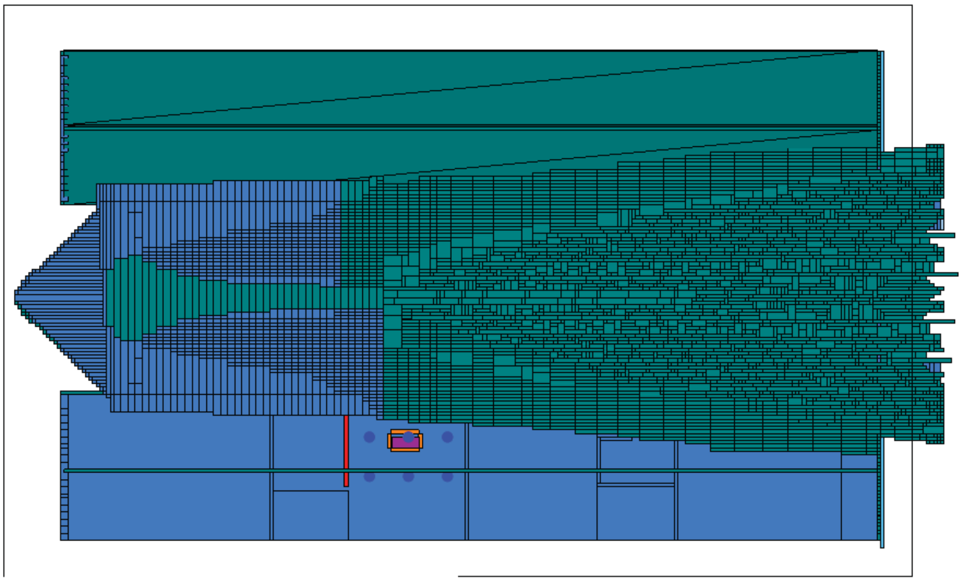

Fig. 29.5 Location of the fire on the first floor of the ‘body’ (the fire is marked by the purple square)

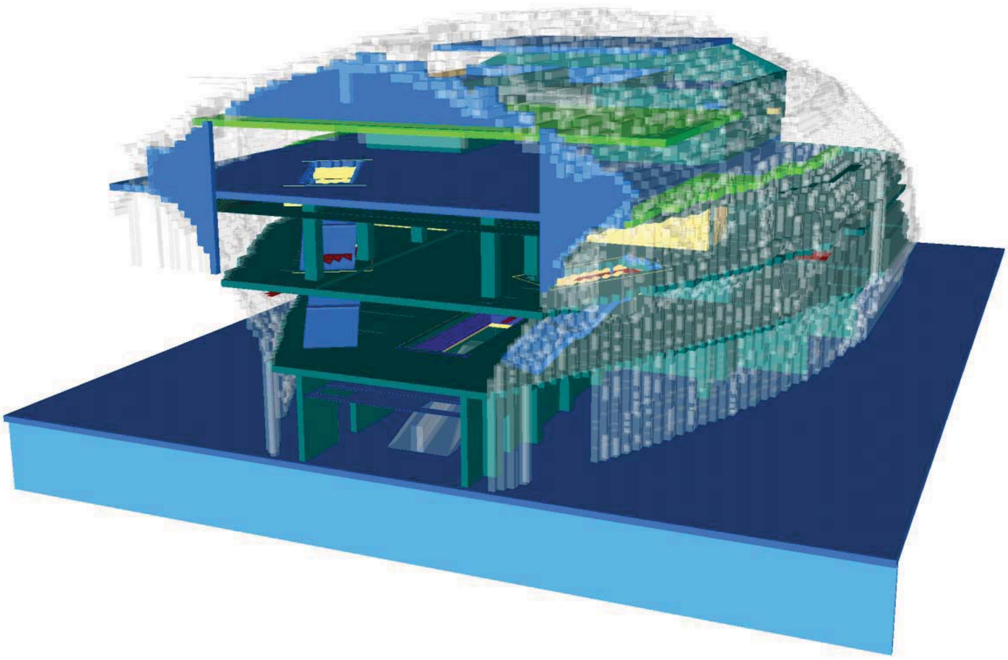

Fig. 29.6 The structure of the 'head'

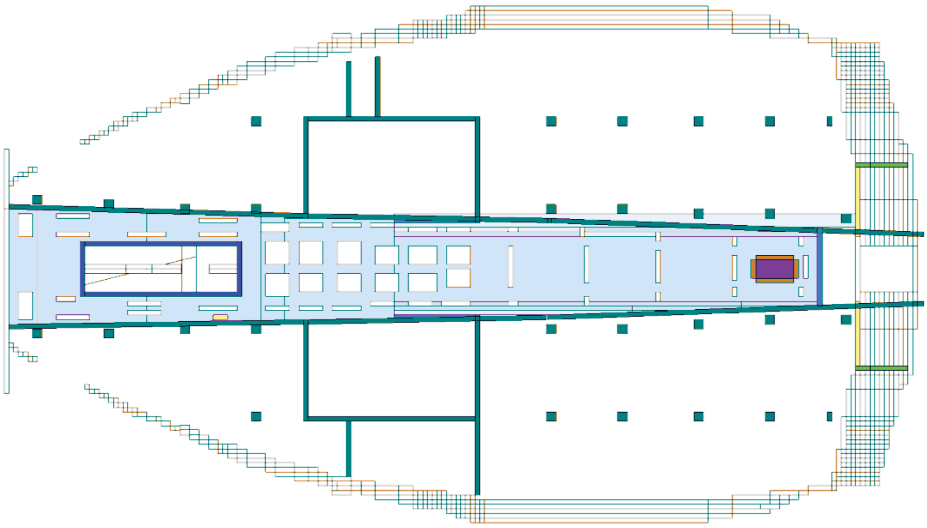

Fig. 29.7 Location of the fire in the 'head' structure on the ground floor (the fire is marked by the purple square)

Smoke could exit the 'body' structure so that no problem was caused and visibility on the escape routes did not worsen. During a simulation authorities in Hungary examine the shift within visibility that is to be calculated with the help of *extinction coefficient.* In Hungary a heat and smoke exhaust system has to provide a 25 meter visibility on the escape routes and a 10 meter visibility for the firemen (at the time of their arrival).

S: *visibility)*[m]

KS: *light occlusion* KS=3; *light emission* KS=8

K:extinction coefficient [1/m]

The distances of visibility can be set up based on this (with the help of the extinction's values).

Tab. 29.1. Distances of visibility for light-occlusive materials (their extinction coefficient differs)

K extinciós koeff. (extinction *coefficient)* [1/m]	S láthatóság (*visibility*) [m]
0,1	30
0,12	25
0,15	20
0,17	17,6
0,2	15
0,25	12
0,3	10

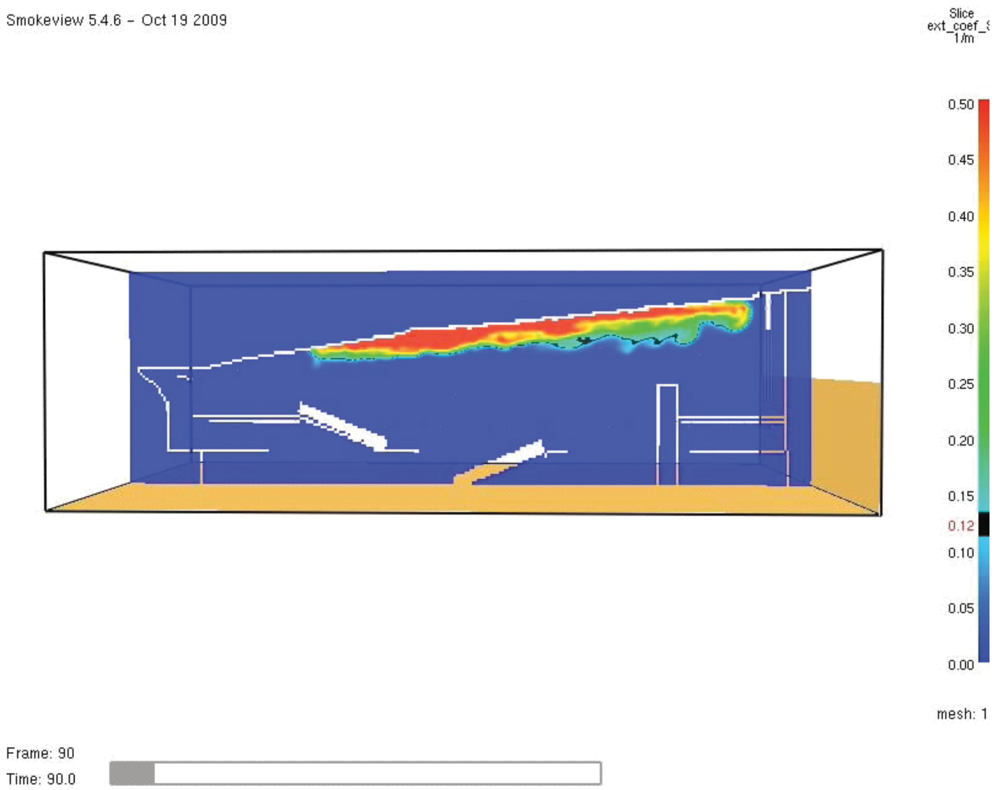

Fig. 29.8 Extinction within the 'body' structure in the 90th second

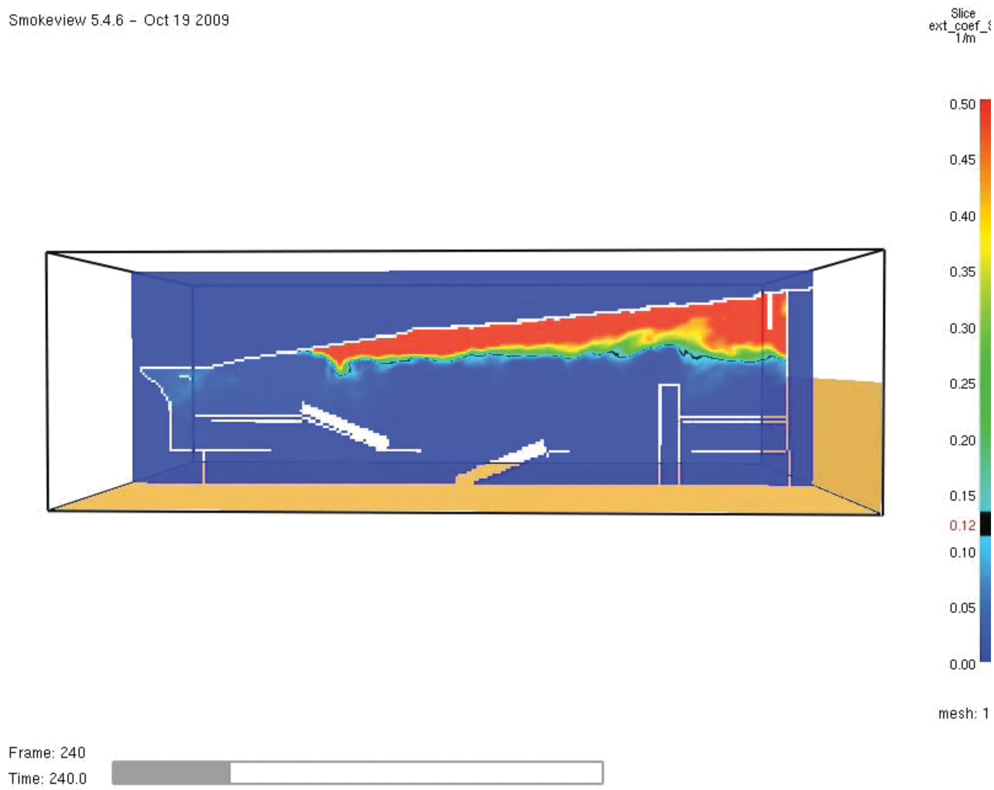

Fig. 29.9 Extinction within the ‘body’ structure in the 240th

Fig. 29.10 Extinction within the ‘head’ structure in the 90th second

Fig. 29.11 Extinction within the 'head' structure in the 240th second

The levels of the 'head' were filled with smoke too fast so we had to come up with another possible solution to the problem.

29.3.4 A possible solution to the problem

We refused to increase the surface of air inflow because most probably this was not the factor that caused the faster spread of smoke. Instead, we made two minor modifications in the heat and smoke exhaust system. First, the system only begins its work on the level of the fire and we also built in smoke barriers (35 cm at the stairs).

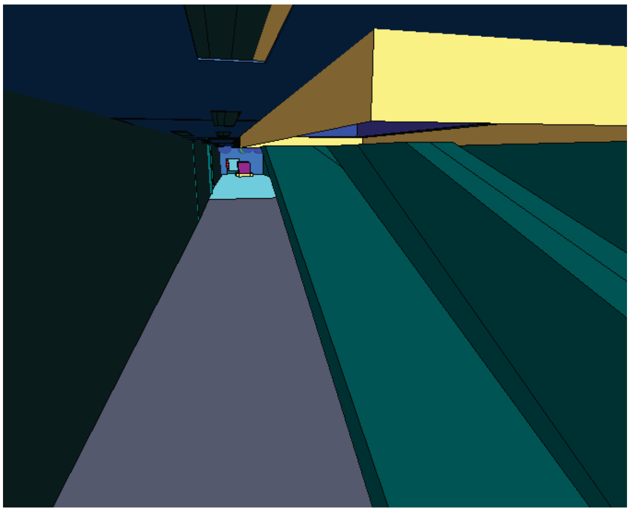

Fig. 29.12 Smoke barriers in the 'head'

29.3.5 Analysis of the possible solution

With the help of the smoke barriers the efficiency of the heat and smoke exhaust system increased greatly and the rate at which smoke is spreading has decreased as well.

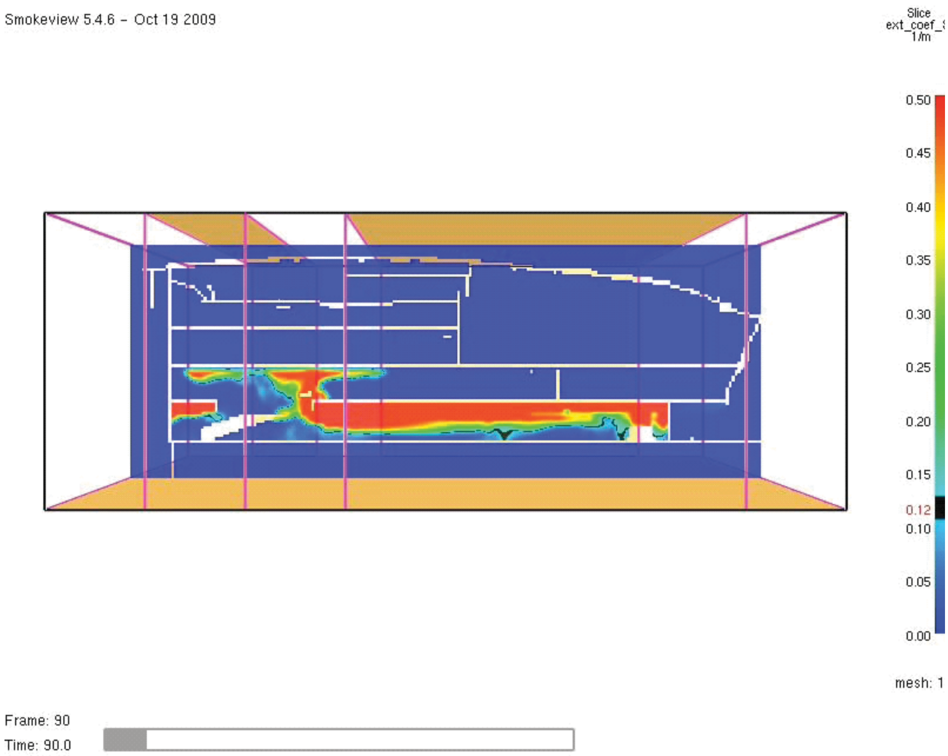

Fig. 29.13 Extinction in the ‘head’ after its modification in the 90th second

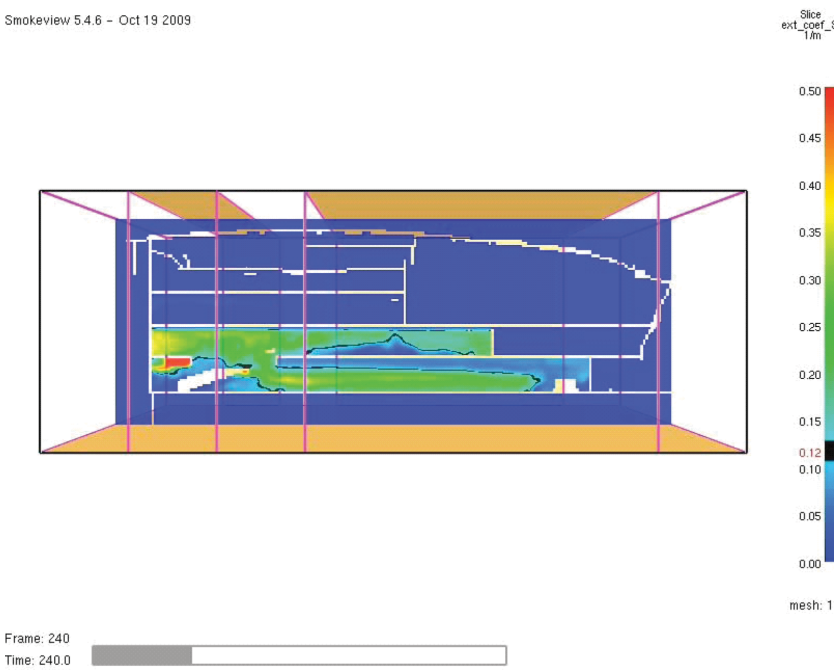

Fig. 29.14 Extinction in the ‘head’ after its modification in the 240th second

29.3.6 Results

It can be stated that although the system (of air inflow) does not suit the fire code, this does not cause any difficulties during an evacuation/emergency. We have found that the heat and smoke exhaust system within the 'head' structure was not appropriate (based on the results of the primary test). It was rather probable that the fast spread of the smoke was not caused by the cutting back of the air inflow so we have made two modifications in the system. Namely, the exhaust system only started to work on the level of the fire and a 35 cm smoke barrier was integrated at the floor space (next to the stairs) so that it may slow down the spread of the smoke. Thanks to these modifications it can be claimed that the new system slowed down the rate at which the smoke is spreading and this way the heat and smoke exhaust system has become much more efficient.

Table 3

According to the model	Suction	Surface provided for air inflow
'Body'	218 700 m^3/h	10,06 m^2
'Head'	126 500 m^3/h	14,40 m^2

29.4 FIRE RESISTANCE INVESTIGATION OF THE STRUCTURE

In Hungary every building structure has to meet a requirement of class of fire risk and critical limit for fire resistance, depending on number of floors and fire resistance degree. The rules are laid in the Hungarian Fire Code (OTSZ, 28/2011. (IX.6.) issued by Ministry for Home Affairs).

The CET building contains partly historical old brickworks and also modern structures. The modern shell structure made from steel and glass has loadbearing and separating function as well and is partial nearly vertical.

In order to find out if fire design of CET building was appropriately fire resistance tests of its modern building structure had to be carried out in the accredited Fire Protection Laboratory of ÉMI Nonprofit Ltd.

29.4.1 Walls

The vertical part of the shell structure was investigated as a wall in the vertical testing furnace.It was loaded and the steel frame was coated with fire-proof painting. The examination of critical limit for fire resistance of the structure was carried out in according to the regulations of the MSZ EN 13651: 2000 standard.

Fig. 29.15 Test model of the wall construction before the fire resistance test

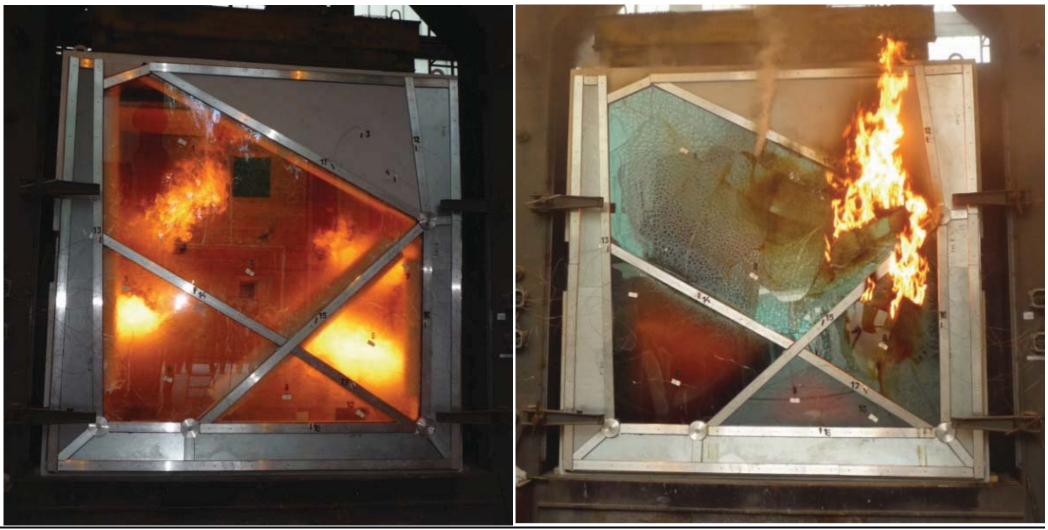

Fig. 29.16 Test model of the wall construction during the fire resistance test

Fig. 29.17 Tested wall model after the fire resistance test

(left – unexposed side, right – fire exposed side)

29.4.2 Ceiling

The horizontal part of the shell structure was investigated as a ceiling in the horizontal testing furnace. It was loaded and the steel frame was coated with fire-proof painting and its flexibility was under scrutiny as well. The examination of critical limit for the fire resistance of the structure was carried out in accordance with the regulations of the MSZ EN 13652: 2000 standard.

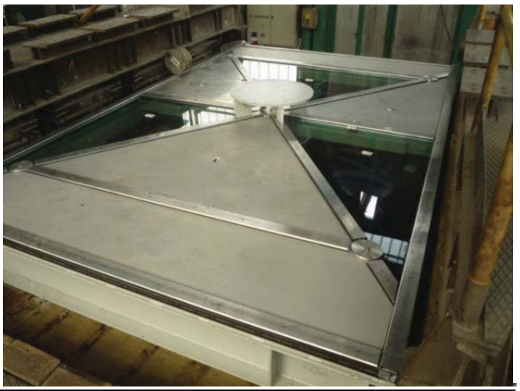

Fig. 29.18 Test model of the ceiling before the fire resistance test

Fig. 29.19 Test model of the ceiling during the fire resistance test

Fig. 29.20 Tested ceiling model after the fire resistance test

29.5 SUMMARY

Based on the information above, it can be claimed that those buildings which have a special structure and implementation require studies concerning their fire resistance (laboratory study) and fire simulations (FDS) as well. If a structure has a critical fire resistance value that is lower than the required value, then additional protective steps are required (such as integrating sprinkler heads or a smoke control system). Such protective arrangements in the case of CET are the coating of the steel frames (that have the required cross-section) with fire-proof paint and the fact that the filling elements are made of fire-proof glass and solid, insulated materials. It can be stated that CET's heat and smoke exhaust system is safer and more economical than what is prescribed by the laws. In Hungary one is only allowed to implement a heat and smoke exhaust system that is different from the one prescribed by the law, after proving that the system is able to function appropriately. Computational simulation is one of such justification methods (but it should be noted that the base data is always set by the authorities).

Acknowledgement

The authors would like to thanks

-OKF (National General Directorate for Disaster Management).

-Architect designer: Kas Oosterhuis

-General main contractor: WHB Építő Kft.

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30 EXAMINATION, ASSESSMENT AND REPAIR OF FIRE DAMAGED RC STRUCTURE OF „REFINERY-OKTA“ IN SKOPJE

Summary

In November 2008 a part of the structure of "Refinery–Okta" in Skopje, Macedonia, was in fire and the last three spans were completely burned. According to the damages recorded in situ, it was found out that fire caused severe damage to the reinforced concrete bearing structure. For realization of the repair project of the RC structure, nonlinear and transient thermal analysis, nonlinear stress-strain analysis of the frame structure and experimental determination of the residual concrete strength after action of fire were recommended. Results obtained by this analysis are presented in this paper. The global repair recommendations are given, also.

30.1 INTRODUCTION

Three years ago a part of the structure of "Refinery-Okta" in Skopje, used for primary processing of crude oil, was in fire. The burning process was supported by the crude oil and gasoline. The fire started in the second span of the open structure, but quickly spread to the first three spans. Fire duration, according to the witnesses, was about one our, but the caused effect indicates that it lasted longer and that temperature in the fire section was up to 1100^{o}C. Fire was extinguished by using water and foam.

30.2 EXAMINATION AND ASSESSMENT OF FIRE DAMAGED RC STRUCTURE

30.2.1 Basic data about structure

The object was built in 1978 and the technical project documentation was done by a design bureau from Moskva, Rusia. The load-bearing structure is classic skeletal reinforced concrete structure (Fig. 30.1, 30.4). Special cooling systems are placed on the slab. The reinforced concrete frames are placed on 6.0m. Basic frame elements are:

- columns S1, dimensions 60 x 80 cm,
- beams G1, dimensions 55 x 100 cm,
- crane beams G2, dimensions 35 x 70 cm.

The beams that support the reinforced concrete full slab (8.0 cm thick), are placed in longitudinal direction:

- beams B1(at the edge), dimensions 30 x 60 cm,
- beams B2, dimensions 45 x 60 cm,
- beams B3, dimensions 30 x 60 cm.

Reinforcement details were available from the technical project documentation. The design value of concrete compressive strength, for all elements, is f_c = 30Mpa. All columns and beams have ribbed bar reinforcement, type A III (according to our regulation RA 400-500). Stirrups are from mild reinforcement Ø8, type A I (according to our regulation GA 240-360). For the slabs reinforcement type AI and AIII is used.

Data on the design loads, as well as static and dynamic calculations, were not available.

30.2.2 Visual inspection of structure

Based on the data gathered through detailed visual survey of the bearing structural elements, the following characteristic damages of beams and slabs were recorded: change of concrete color; fissures and cracks inside the concrete mass; cracks along main reinforcement, crushing of concrete and falling off of concrete parts along the edges of linear elements up to the reinforcement. White color of the surface concrete layers was caused by the chemical reaction between the water, used for extinguishing the fire, and the dehydrated carbonate aggregate (Fig. 30.1, 30.2). This phenomenon is followed by expansion of concrete mass up to 44% and causes cracks in concrete mass (Bazant & Kaplan, 1996).

Due to the openness of the space and the continuous supply of oxygen and oil, the flames spread high under the slab and the columns were exposed to lower temperatures. These elements were not visually damaged, cracks were not recorded and the concrete color was rose-red, that indicates temperatures less then 600^{o}C.

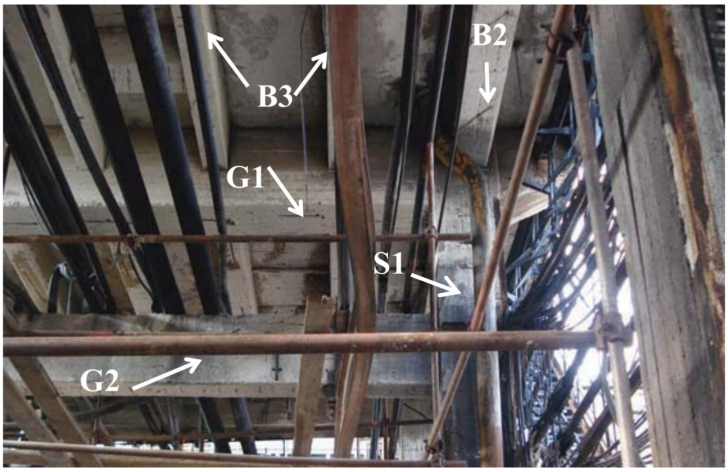

Fig. 30.1 Disposition of the beams in the second spam and change of concrete color due to chemical reaction

30.2.3 Experimental determination of the residual concrete strength

Temperature over 400°C causes reduction of the compressive strength and other mechanical properties of concrete and this process is irreversible (the strength of concrete does not recover in the cooling phase). The mechanical properties of hot welded steel (reinforcing bars) decrease as well, but in the cooling phase they increase again. According to these statements and for realization of the repair project of the RC structure, experimental and numerical determination of the residual concrete strength was recommended. Based on previous Schmidt hammer testing results, the locations of the eight concrete specimens, taken only from lateral side of the RC beams, were defined (Fig. 30.2).

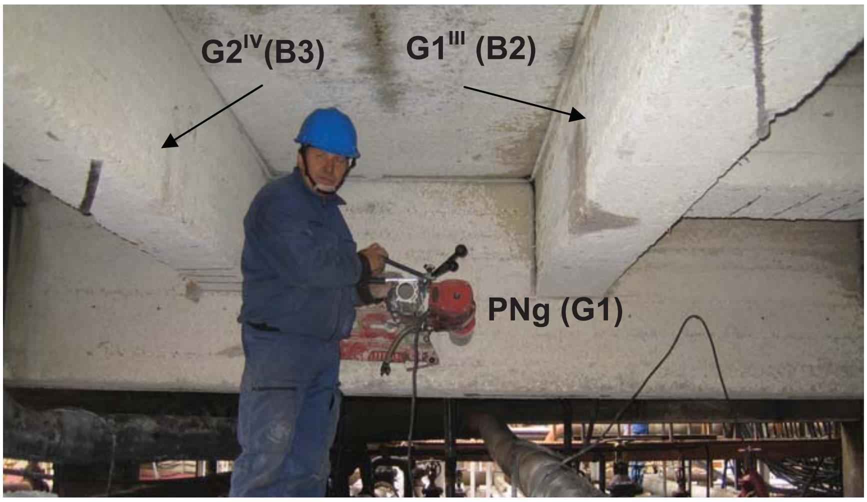

Fig. 30.2 Locations where the concrete specimens were taken from the beams of the second spam and change of concrete color due to chemical reaction

The concrete specimens were tested at the Testing laboratory of the Civil Engineering Faculty in Skopje. Before testing all the specimens were divided in two slices. The deteriorated (burned) slices had small height (3-6 cm) and rough surface (Fig. 30.3). Resulting average value for the residual concrete strength of burned slices was $f_{c,r}$ = 9.24 Mpa, (Cvetkovska & Lazarov, 2008). Lower values were expected for the concrete layers at the bottom of the beams (directly exposed to the flames) but, because of the position of the reinforcement, specimens were not taken from this location. This statement and the testing results were confirmed by the numerical thermal analysis of the fired beams. Hammer testing, testing results of the concrete specimens taken from locations that were not fired, as well as test results of the unburned slices, confirmed that the compressive strength of the carbonate aggregate concrete before the action of fire was f_{co}= 30 MPa.

Fig. 30.3 Concrete specimens before testing

30.2.4 Numerical determination of the residual concrete strength

Thermal response of the fired structure (Fig. 30.4) has been investigated analytically, too. Elements geometry; support conditions; concrete cover thickness; type of aggregate; compression strength of concrete; steel ratio and defined fire scenario were taken into account while the nonlinear and transient temperature field and the concrete strength reduction in the cross section of the elements exposed to fire were determined.

The computer program FIRE, (Cvetkovska, 2002), was used to solve this problem and the following assumptions were made:

- Fire was modeled by a single valued gas temperature history and in this case ISO 834 fire model was used. According to data gathered in situ it was assumed that the maximum fire temperature of 1100^{o}C was reached at the moment t= 1.5hours and after that the cooling period started. This temperature was reached only in the zones under the slab, but in the lower zones it was approximately 50% less than the maximum one.
- Temperature dependent material properties were known (recommended in EC2).
- The fire boundary conditions were modelled in terms of both convective and radiating heat transfer mechanisms.
- The easy heat penetration, after cracks had appeared, or some parts of the cross section had crushed, was neglected.

The results of the nonlinear thermal analysis are presented by graphs. The isotherms in the cross section of the beam B2, at moment t = 1.5 hour, are presented on Fig. 30.5a. The beam was fired only from the bottom and lateral sides but, because the space was open, at the opposite side (over the slab) the temperature was proportional to the temperature under the slab (approximately 25% of the temperature reached under the slab) and the beam was heated from both sides, but not with the same intensity.

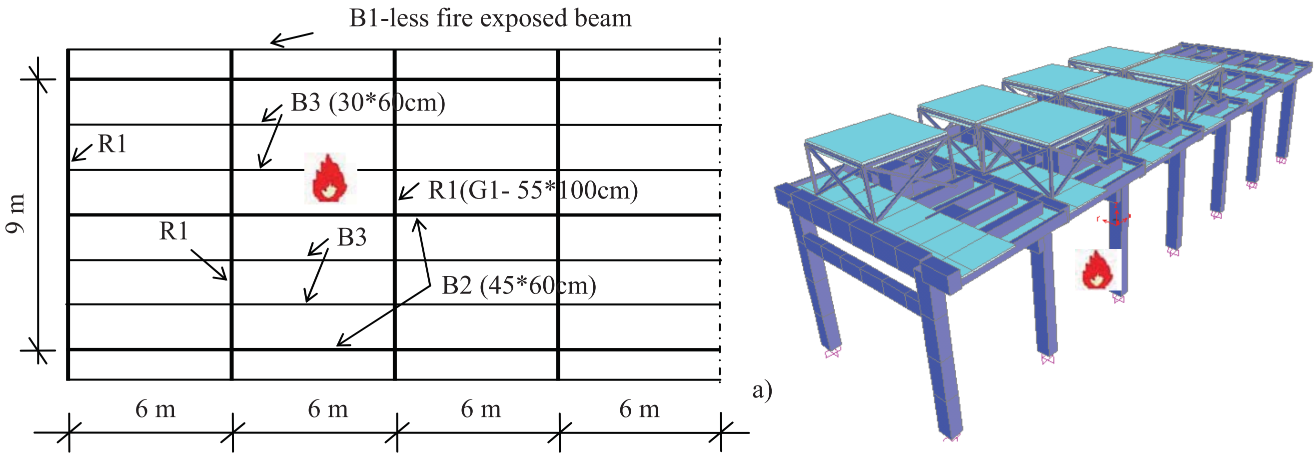

Fig. 30.4 a) Schematic presentation of the fired structure

b) Model of structure in SAP2000

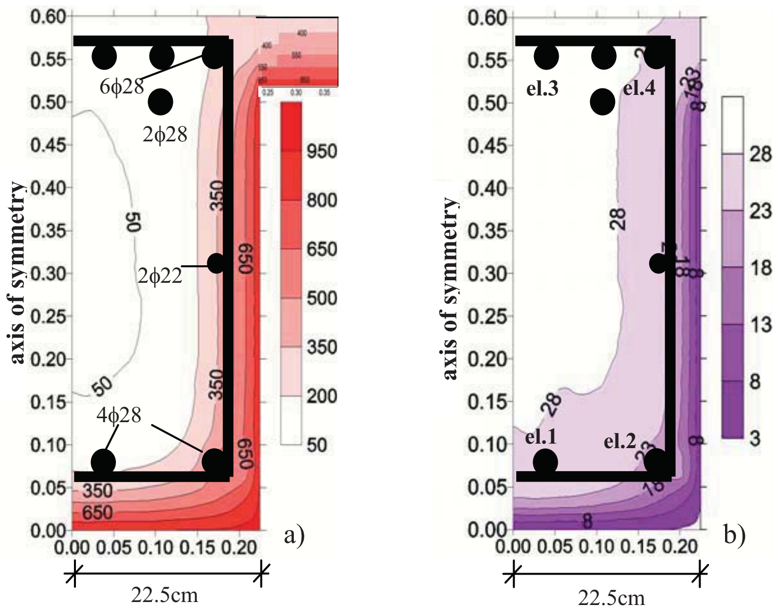

Fig. 30.5 a) Isotherms in the cross section of the beam B2 (t=1.5hours),

b) residual concrete strength after cooling phase

The residual concrete strength after cooling period is presented on Figure 5b. Calculation results indicate that on the side of the fire, in 3-4 cm thick layer, which is 25 - 30% of the cross section of the beam, the strength reduction is significant and the residual strength of concrete is 10 Mpa in average. These results correspond well with experimental results obtained by laboratory testing of specimens taken from the same RC beam. In the cross section core the strength of concrete is not reduced.

While the elements were built, the stirrups were not well tied for the reinforcing bars and almost in all beams there are thick concrete layers between the stirrups and the bars. The bars are placed high in the cross section and the concrete cover thickness from the bottom side is 6-7 cm. This has adversely effect from the aspect of the internal lever arm and the bearing capacity of the beams, but in case of fire it helps the temperatures of the reinforcing bars to be lower than in case when the concrete cover thickness is only 2-3 cm. The time dependant temperatures of the reinforcing bars of the beam B2 are presented on Fig. 30.6.

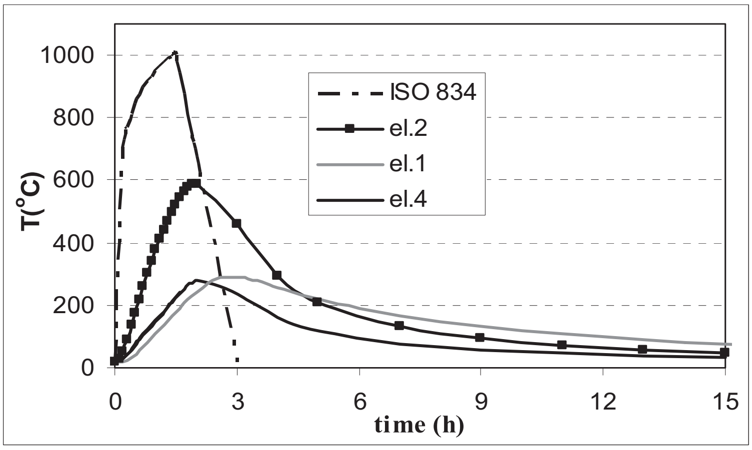

Fig. 30.6 Time dependent temperatures of the reinforcing bars of the beam B2 which is exposed to fire from three sides

The beams B3 and the beams G1 (Fig. 30.1), that are part of the reinforced concrete frames and supports beams B2 and B3, were exposed to fire from three sides, too. Calculation results indicate similar temperature distribution as for the beams B2. In 3-4 cm thick layer, which is near 20% of the cross section of the beam G1, the residual concrete strength is 10 Mpa in average.

The beams G2, that support the crane, were fire exposed from all sides, symmetrical temperature field in the cross section of the beams was generated and near 30% of the cross section has got concrete strength less then 20 Mpa (10 Mpa in average). Results are presented on Fig. 30.7 a,b.

The time dependant temperatures of the reinforcing bars of the beam G2 are presented on Fig.30.8.

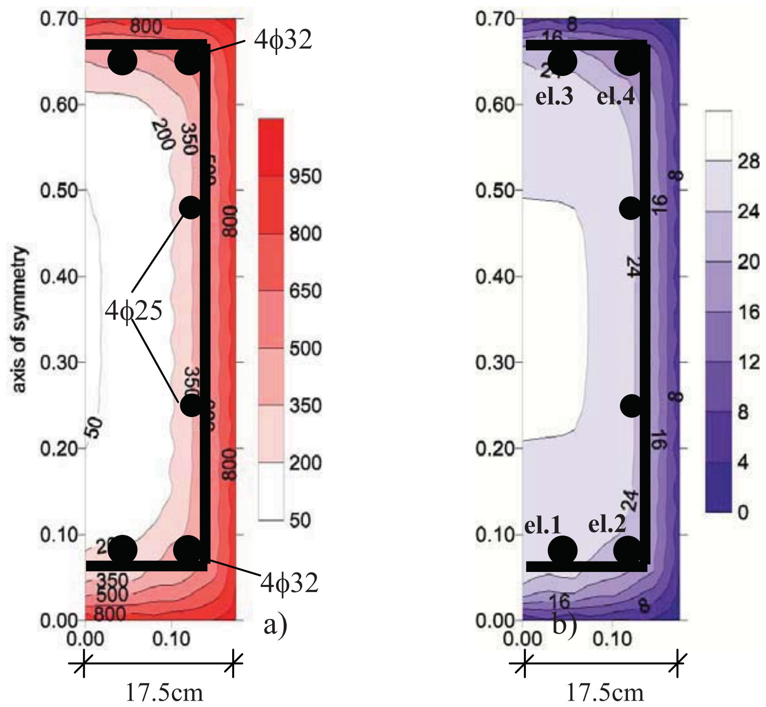

Fig. 30.7 a) Isotherms in the cross section of the beam G2 (t=1.5hours),

b) residual concrete strength after cooling phase

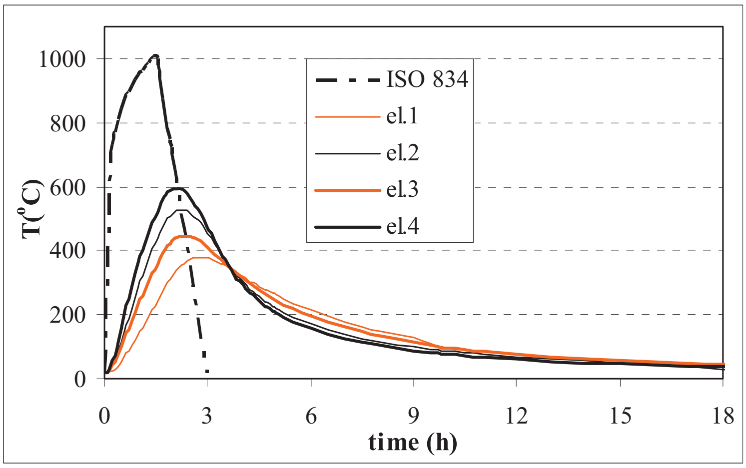

Fig. 30.8 Time dependent temperatures of the reinforcing bars of the beam B2 which is exposed to fire from three sides

30.3 REPAIR OF FIRE DAMAGED REINFORCED CONCRETE STRUCTURE

30.3.1 Nonlinear stress-strain analysis of the fire exposed reinforced concrete structure

Data on the design loads, as well as static and dynamic calculations of the bearing structure, were not available from the project documentation. For the needs of the Repair project additional calculations for normal temperatures (without fire action) were conducted. For that purpose the program SAP2000 was used (Fig. 30.4b). There was lack of data on the intensity of the dynamic loads of the cooling system placed on the slab and they were not taken into account, but the structure was controlled on seismic loads. The conclusion confirmed that the bearing structure was well designed and there were sufficient reserves for adoption of the dynamic impact of the cooling system.

The response of the bearing structure, while exposed to extremely high temperatures during fire action, as well as in the cooling phase, was predicted by the program FIRE, (Cvetkovska, 2002). This program carries out the nonlinear transient heat flow analysis (modulus FIRE-T) and nonlinear stress-strain response associated with fire (modulus FIRE-S). The solution technique used in FIRE is a finite element method coupled with time step integration. The program accounts for: dimensional changes caused by temperature differences, changes in mechanical properties of materials with changes in temperature, degradation of sections by cracking and/or crushing and acceleration of shrinkage and creep with an increase of temperature. The used analysis procedure does not account for the effects of large displacements on equilibrium equations.

Fig. 30.9a presents discretization of beams B2 and B3 (supported by frames R1), Fig. 30.9b presents discretization of frame R1 (axial symmetry is used).

Fig. 30.9c presents discretization of the cross section of the elements (isoparametric finite elements with four node are used). Period of 30 hours was analyzed. The cooling phase was important for defining the residual bearing capacity of the structure.

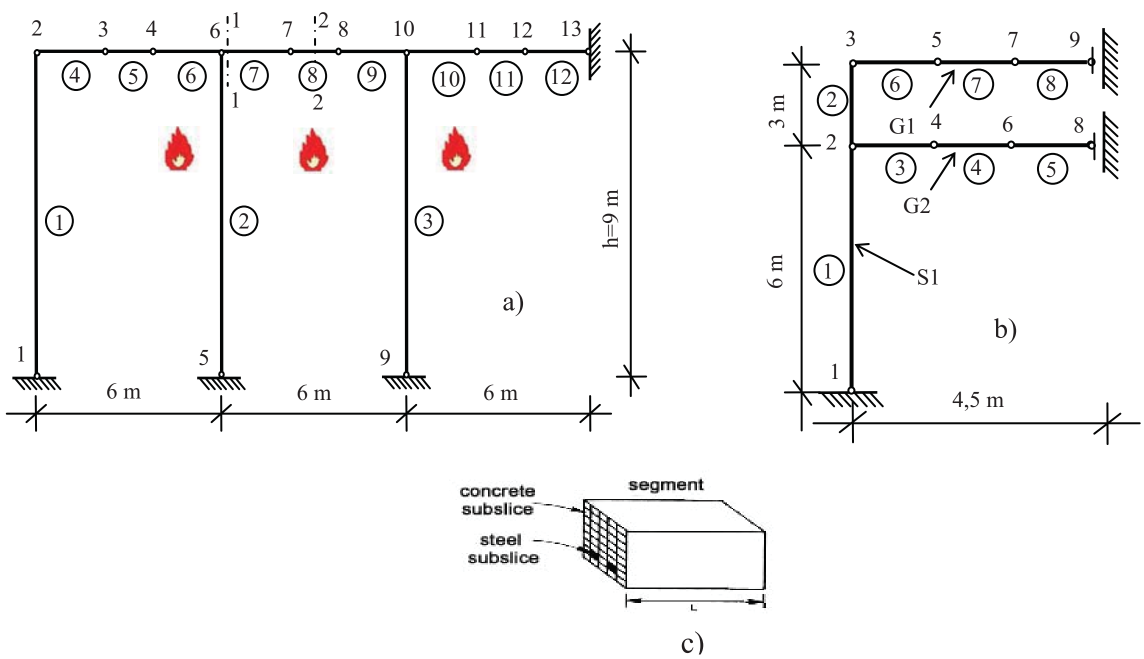

Fig. 30.9 Schematic presentation of the fired structure and discretization of the elements:

a) longitudinal beams B2 and B3, b) main frame R1, c) discretization of the cross section

The columns S1 (60 × 80 cm) are symmetrically reinforced with 20ϕ32 (f_y (20°C) = 400 Mpa). Before the action of fire the compressive stress in concrete core was 1.2 Mpa, that was only 4% of f_c (20°C), and the stresses in all reinforcing bars were 64Mpa, that was only 16% of f_y (20°C). Stress redistribution was caused due to the high temperature difference between the surface layers and inner layers. This redistribution was highly expressed at the moment when the fire was extreme (t = 1.5 hours), but it didn't cause cracking, or crashing of concrete, nor yielding of the reinforcing bars. Only the upper parts of the columns were exposed to fire with the same intensity as beams and slabs were, therefore critical stresses occurred only in the cross sections close to the slabs, but the extreme values were +363 Mpa, or 90% of $f_y(T)$, at the inside and -250 Mpa, or 63% of $f_y(T)$, at the outside of the column cross section. After the cooling phase the stresses were back to 30% of yielding strength of the bars for room temperature. Therefore the repair of the RC columns was not recommended.

The bottom and lateral sides of the beams B2, B3 and G1 were directly exposed to fire. They became hotter than the top sides and tended to expand more. This differential heating caused the ends of the elements to tend to lift from the supports thus increasing the reactions. This action resulted in a redistribution of the bending moments. The negative moments increased, while the positive moments decreased and tended to become negative. After time t = 0.5 h, the negative moments began to decrease again.

The concrete, directly exposed to fire, developed large compression stresses due to thermal gradients. At the ends of the beams fire had the same effect as the uniform load did, therefore the concrete at the bottom side of the cross section crushed, while concrete at the top side of the cross section cracked. The effect was opposite at the middle of the span.

The thick concrete layers (6-7 cm) on the bottom side of the beams protected reinforcement from high temperatures, but after a time the bars which were close to the fire became hotter, and opposite (Fig. 30. 6, 8). The increase of negative moments at the ends of the beams was accommodated, but the redistribution that occurred was sufficient to cause yielding of the top bars which were in the corners. In the same cross section, the reinforcing bars which were on the side of the fire were all the time in compression by the action of the fire and the negative bending moment (el.2 on Fig. 30.10, 12). The yield strength was reduced due to the high temperatures, hence the reinforcement started to yield very soon. Large plastic deformations occurred at the end of the heating period, so during the cooling phase the reinforcement changed the sign and residual stresses in tension occurred. The residual tension stresses were significant for bottom bars placed at corners of beams B2 and B3 and their values were up to 90% of the yielding strength at room temperature (Fig. 30.10). Beams G1 were in a better situation. The residual tension stresses were not higher then 50% of the yielding strength at room temperature (Fig. 30.12), therefore the repair of the beams G1 was not recommended.

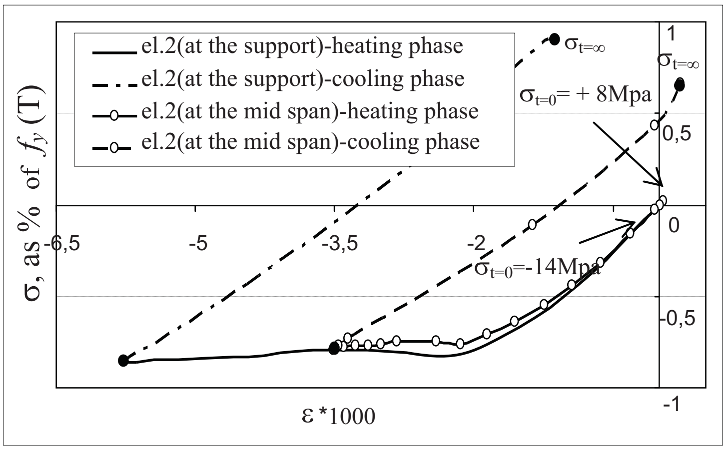

Fig. 30.10 Time dependent stresses of reinforcement no.2 of beam B2, for cross section at support and mid span, as a percent of yielding strength at corresponding temperature

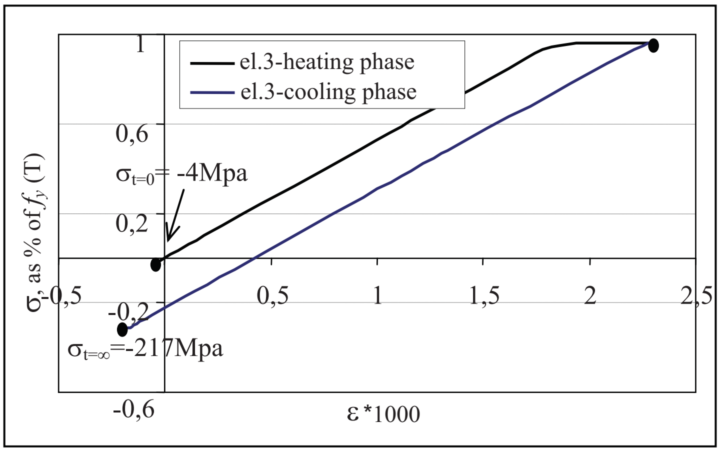

Fig. 30.11 Time dependent stresses of reinforcement no.3 of beam B2, for cross section at mid span, as a percentage of yielding strength at corresponding temperature

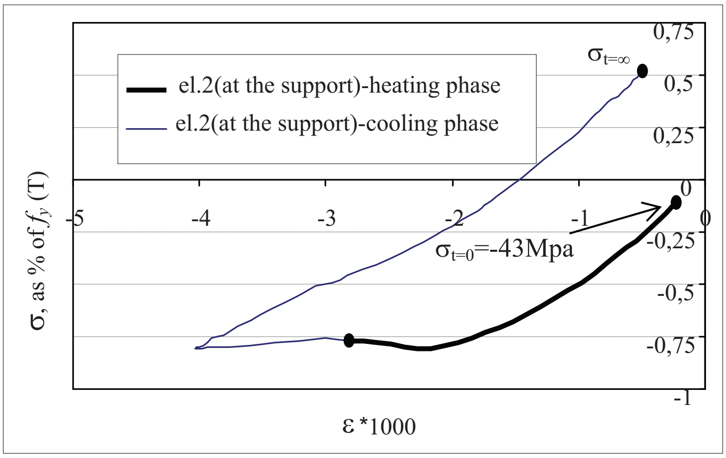

Fig. 30.12 Time dependent stresses of reinforcing bar no.2 of beam G1, for cross section at support, as a percent of yielding strength at corresponding temperature.

The stress-strain diagram for the mid span top reinforcement (temperatures less then 200^{o}C) is as for room temperatures (Fig. 30.11).

Crane beams G2 were exposed to fire from all sides, it caused symmetrical cross section temperature field (Fig. 30.7) and temperature induced stresses were lower than for the other beams. During the cooling period the stresses in reinforcement changed the sign too, but the residual stresses were less then 30% of the yielding strength for steel at room temperatures. Therefore the repair of the beams G2 was not recommended.

30.3.2 Suggested repair of fire damaged structural elements

The aim of the repair suggested in this project was to obtain nearly the same strength, stiffness, deformability and ductility of the repaired elements and their cross sections as they had before the fire action.

The first step of the suggested repair of beams B2 and B3 (in longitudinal direction) was elimination of the deteriorated (burned) concrete layers up to the reinforcement (Fig. 30.13). For that purpose 3-4 cm thick layers, from the lateral side, and 6-7 cm thick layers, from the bottom side, were removed. Before casting the opened sections were cleaned with running water.

Fig. 30.13 Elimination of the deteriorated (burned) concrete layers up to the main reinforcement

The yield reinforcement at the corners of the beams, with total area of 12 cm^2, was “covered” by additional reinforcement (4Ø20, RA 400/500), placed under the existing stirrups (Fig. 30.14). The additional open stirrups Ø8/20 cm (GA 240/360) were welded to the existing stirrups. Welding between the new ribbed reinforcement and existing smooth stirrups was not recommended for two reasons: the additional reinforcement had higher yield strength and the existing smooth stirrups had residual plastic deformations. The jacketing with new 3cm thick concrete layer was made with sprayed concrete MB 30.

As it was shown before, the only suggested repair of beams G1 and G2 (RC frames R1) was in replacing the "burned" concrete layers with new one.

The results obtained by the thermal analysis of the RC slabs and the detailed visual survey in-situ led to a conclusion that the concrete strength was reduced only from the bottom side of the slabs where concrete is in tension and does not influence upon the bearing capacity of the slabs. The fact that the slab's span was only 1.5 m, additionally confirmed that the suggested repair of the slabs should be in replacing the deteriorated concrete layers by new one.

Fig. 30.14 Repair of the beam B2 at the second spam.

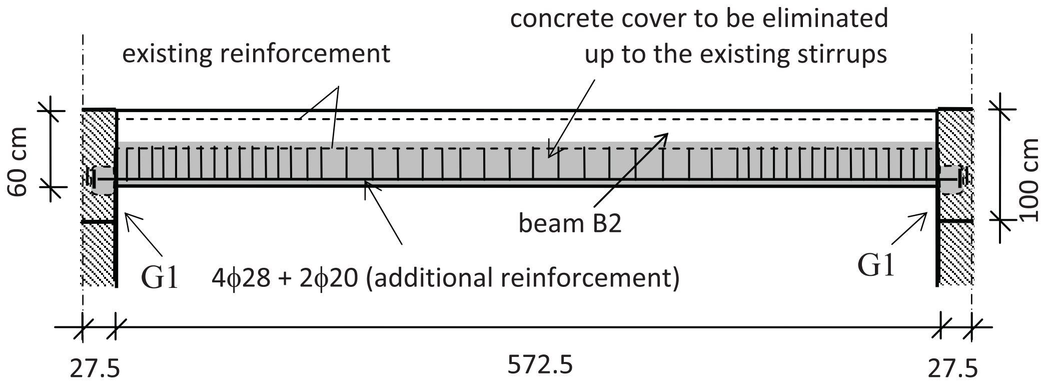

Fig. 30.15 Repair details of beam B2.

Fig. 30.16 Additional stirrups of beam B2, welded for the existing one.

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31 EXAMINATION, ASSESSMENT AND REPAIR OF RC STRUCTURE OF BUILDING DAMAGED IN FIRE

Summary

A twenty storey building structure was in fire and two apartments at the seventh and eighth floor were completely burned. According to the damages recorded in situ, it was found out that fire caused severe damage to the reinforced concrete bearing structure. Based on the data gathered through detailed visual survey of the bearing structural elements, nonlinear and transient thermal analysis, nonlinear stress-strain analysis of the frame structures and experimental determination of the residual concrete strength after action of fire were recommended. Results obtained by these analysis show that during the fire action, as well as in the cooling period, the strength and stiffness of structural elements were continually reduced and adequate repair of the damaged elements has to be made.

31.1 INTRODUCTION

In February 2005 a twenty storey building structure on “Nikola Parapunov str. no. 3, Skopje, Macedonia” was in fire and two apartments at the seventh and eighth floor were completely burned. Primary, the fire was caused by gas explosion, but the synthetic materials in the apartment were additional fire load, so very high temperatures were reached and the fire time was more than four hours.

According to the damages recorded in situ, it was found out that fire caused severe damage to the reinforced concrete bearing structure, while the interior and the installations were completely destroyed. Based on the data collected through detailed visual survey of the bearing structural elements (columns, beams, slabs and RC walls), the following characteristic damages were recorded:

- change of concrete color (red, grey-yellow, yellow, Fig. 31.1)
- fissures and cracks inside the concrete mass (Fig. 31.1d)
- cracks along main reinforcement in columns, beams and slabs (Fig. 31.2)
- crushing of concrete and falling off of concrete parts along the edges of linear elements up to the reinforcement (Fig. 31.2d)

Fig. 31.1 Change of concrete color and characteristic damages of RC elements, recorded in situ

Fig. 31.2 Cracks along main reinforcement in columns, beams and slabs, recorded in situ

According to that situation, the following steps were recommended:

- Control of the element geometry; concrete cover thickness; type of aggregate; compression strength of concrete and the number, type and diameter of the built-in reinforcement, according to the design, as well as control of the loads used in the design calculations.
- Experimental determination of the residual concrete strength after action of fire.
- Nonlinear transient thermal analysis and nonlinear stress-strain analysis of the frame structures exposed to fire and assessment of the degree of damage, according to the results obtained by this analysis.

31.2 EXPERIMENTAL AND NUMERICAL DETERMINATIO OF THE RESIDUAL CONCRETE STRENGTH

The possibility for adequate repair of the damaged elements and the measures that have to be done in that case, directly depend on the level of the damages caused during the fire action, as well as in the cooling period. One of the most important factors that directly influence the repairing possibility is the residual compressive strength of concrete. The mechanical properties of the reinforcement decrease as well, but in the cooling phase they increase again.

Temperature over 400^0C causes irreversible reduction of the compressive strength and other mechanical properties of concrete. The compressive strength of concrete does not recover in the cooling phase because of initial degradation and chemical decomposition of the cement past. The residual compressive strength of concrete should be determinate by laboratory tests of specimens taken from the RC elements exposed to fire, but very often this procedure is impractical because additional destruction of the damaged elements is not advisable. Because of the position of the reinforcement in the surface layers of the cross section, taking the specimens from columns and beams is more complicated and is not advisable. In such cases the problem can be solved by using a numerical procedure, based on the nonlinear transient heat flow analysis and the nonlinear stress-strain analysis. When the residual compressive concrete strength is numerically determined, it’s value is assumed to be the same as the value that corresponds to the maximum concrete temperature.

31.2.1 Experimental determination of the residual concrete strength

The experimental testing of the residual concrete strength after the fire action was the first step of the suggested measures and was done by the Institute for Materials Testing and Development of New Technologies “Skopje”-Skopje. According to the previous Schmidt hammer testing results, the locations of the eight concrete specimens, taken only from RC walls and RC slabs, were defined. Hammer testing of concrete elements from apartments that were not fired confirmed that the compressive strength of the concrete before the action of fire was between f_c=30MPa and f_c=40MPa.

The concrete specimens taken from the RC walls (B8, V7, G7, D7, E7 in Tab. 31.1) were exposed to fire from one side. The corresponding surface layers (3-5cm thick) had changed the color (red, grey-yellow, yellow) and were more deteriorated than the inner layers (fig.3a). The RC slabs over the 7th floor were fire exposed from both sides, but they were covered with 1cm thermal isolation and 4cm lean concrete (Fig. 31.1c and Fig. 31.3b), that directly influenced upon the cross section temperature field, therefore the specimens taken from these slabs (T2, MS, CII) were deteriorated only from one side (the bottom side), too.

Fig. 31.3 Deterioration of surface layers of specimens

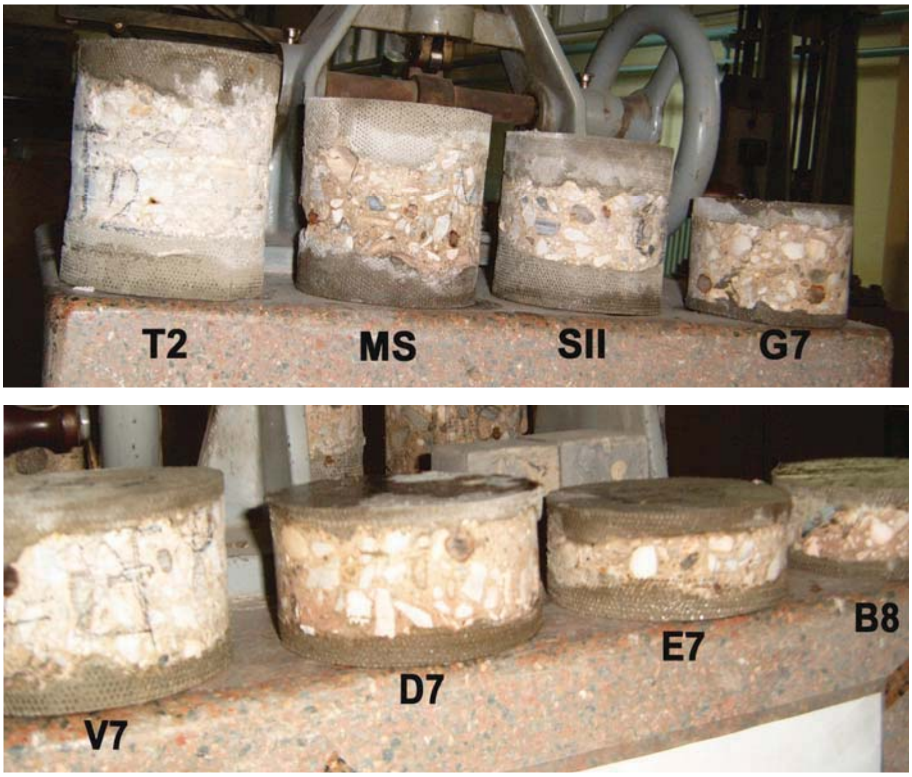

Fig. 31.4 Deteriorated concrete specimens, prepared for testing

Before testing all the specimens were divided in two slices. The deteriorated (burned) slices had small height (3-6cm) and rough surface therefore they were specially prepared by adding plaster layers (fig.4). In that case the measured values for the compressive concrete strength were reduced with coefficients depending on the shape and height (h) of the deteriorated concrete specimens. Test results are presented in Tab. 31.1.

Tab. 31.1 Concrete strength testing results

No.	Position	Dimensions of kerns (cm)			density (kg/m^3)	Test load (KN)	Concrete compressive strength (MPa)		Age of Concrete (years)	Design concrete strength according to MKS U.M1.048
		total H	h (testing)	D			cylinder	reduced to cube 20/20/20		
1	RC wall (B8)	15	9.9	9.9	2214	200	**26.0**	26.5	22	**20.0**
	(after fire expose)		3.5		2150	390	**15.8***	16.1		**12.0**
2	RC slab (T2)	10 +7cm plaster	10	9.9	2300	215	**28.0**	28.6	22	**21.4**
	(after fire expose)		/		/	/	**/**	/		**/**
3	RC slab (MS)	18	10.3	9.9	2340	260	**33.8**	34.5	22	**26.0**
	(after fire expose)		6.5		2310	145	**12.5***	12.8		**9.5**
4	RC slab (SII)	18	9.6	9.9	2315	295	**38.4**	39.1	22	**29.4**
	(after fire expose)		4		2269	160	**13.0***	13.3		**10.0**
5	RC wall (V7)	25	9.9	9.9	2117	272	**35.4**	36.1	22	**27.0**
	(after fire expose)		5		2138	265	**14.5***	14.8		**11.0**
6	RC wall (G7)	19.4	10	9.9	2179	195	**25.4**	25.9	22	**20.0**
	(after fire expose)		3.5		2168	290	**14.5***	14.5		**11.0**
7	RC wall (D7)	18	10	9.9	2212	236	**30.7**	31.3	22	**24.0**
	(after fire expose)		3.5		2168	290	**14.5***	14.8		**11.0**
8	RC wall (E7)	16	10	9.9	2259	246	**31.9**	32.6	22	**25.0**
	(after fire expose)		3		2225	380	**14.1**	14.4		**10.5**

* Values are reduced with coefficients depending on the shape and height (h) of the deteriorated concrete specimens

31.3 NUMERICAL DETERMINATION OF THE RESIDUAL CONCRETE STRENGTH AND ASSESSMENT OF THE DEGREE OF DAMAGE

Thermal and structural response of four-bay, five-story reinforced concrete frame (only one part of the whole frame with defined support conditions) exposed to fire scenario at the two floors only, has been investigated analytically (Fig. 31.5). Elements geometry; support conditions; concrete cover thickness; type of aggregate; compression strength of concrete; steel ratio and defined fire scenario were taken into account while the nonlinear and transient temperature field and the concrete strength reduction in the cross section of the elements exposed to fire were determined.

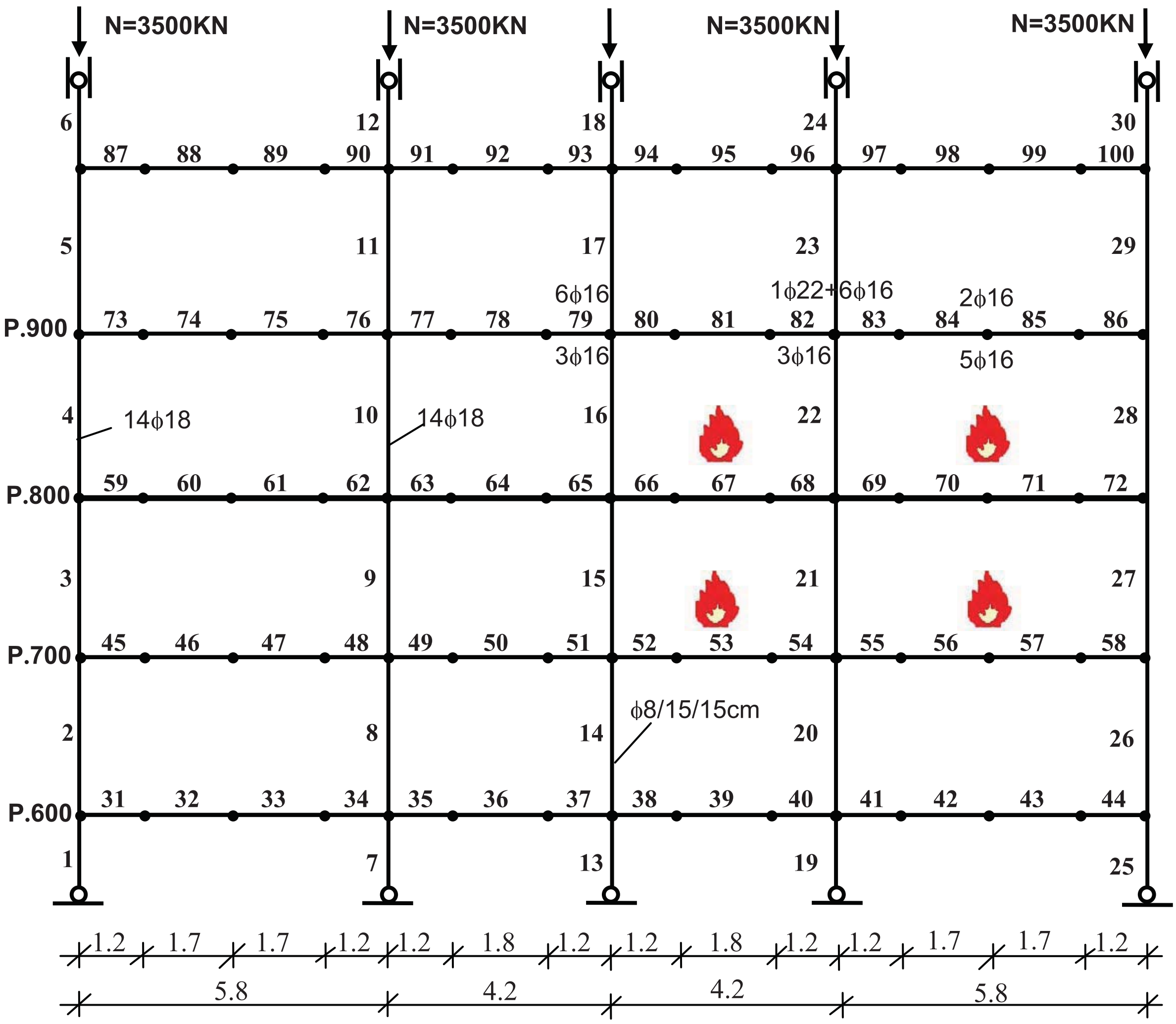

Fig. 31.5 Schematic presentation of the fired frame (elements discretization)

The computer program FIRE was used to solve this problem and the following assumptions were made:

- Fire was modeled by a single valued gas temperature history and in this case ISO 834 fire model was used. According to data gathered through detailed visual survey of the burned structural elements and the change of concrete color it was assumed that the maximum fire temperature of 1000^{o}C was reached at the moment t=1.2hour and after that the cooling period started.
- Temperature dependent material properties were known (recommended in EC2)
- Two dimensional heat transfer was assumed.
- The fire boundary conditions were modeled in terms of both convective and radiating heat transfer mechanisms. For the surfaces directly exposed to fire the coefficient of convection was assumed h_c=25W/m^2 ^{0}C and for the unexposed surfaces h_c=9W/m^2 ^{0}C, as it is recommended in Eurocode 2, part 1.2.
- No contact resistance to heat transmission at the interface between the reinforcing steel and concrete occurred.
- The easy heat penetration, after cracks had appeared, or some parts of the cross section had crushed, was neglected.

Results obtained by the thermal and static analysis of this frame structure show that fire had the most negative influence upon the elements from level 800 (over the 7th floor). The fire intensity was less on the 8th floor and the degree of damage was less too.

The most damaged column is incorporated into the wall, so it was exposed to fire only from the inside of the compartment, but the temperature on the other side (in the hall) was raising proportionally to the temperature in the fire compartment (the fire flames were coming out through the open door) and the heating was from the both sides, but not with the same intensity. The dimensions of the cross section of this column (fig.6) are 60×60cm, the compressive strength of concrete before action of fire was f_c=40MPa. It is symmetrically reinforced with 18ϕ16. The yield strength of the reinforcing bars is f_y(20^{o}C)=240Mpa. Before the action of fire the column was loaded by axial force N=3000KN and the compressive stress was 8 Mpa (20% of f_c).

The concrete strength was reduced as a result of high temperatures in the surface layers of the cross section (fig.6a). Calculation results indicate that on the side of fire, in 4-5cm thick layer, which is 17% of the cross section of the column, the strength reduction is significant and the residual strength of concrete is 16Mpa in average (Fig. 31.6b). These results correspond well with experimental results obtained by laboratory testing of kerns taken from the nearest RC wall (B8, Tab. 31.1). In the core of the cross section the strength of concrete is not reduced.

Stress redistribution was caused due to the high temperature difference between the surface layers and inner layers (Fig. 31.7a and Fig. 31.7b). During the cooling phase, when the temperature in the cross section layers decreased, the negative elongation was not proportional to positive elongation when temperature increased, so it caused longitudinal cracks along the main reinforcement (Fig. 31.2c). These cracks were visually noticed on the surface of all columns, but the depth was not defined.

The reinforcing bars were close to fire and they were all the time in compression by the axial force and the action of fire. The yield strength was reduced due to the high temperatures, so the reinforcing bars started to yield very soon and high plastic deformations were noticed. During the cooling phase the stresses in reinforcement changed the sign and residual stresses in tension occurred, although they were loaded in compression (Fig. 31.8).

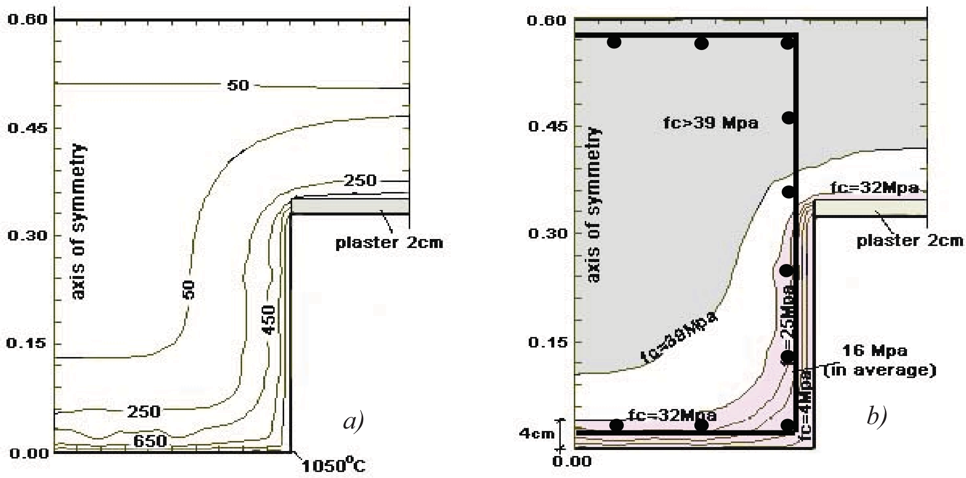

Fig. 31.6 a) Temperature distribution; b) Residual concrete strength

in the cross section of the most damaged column

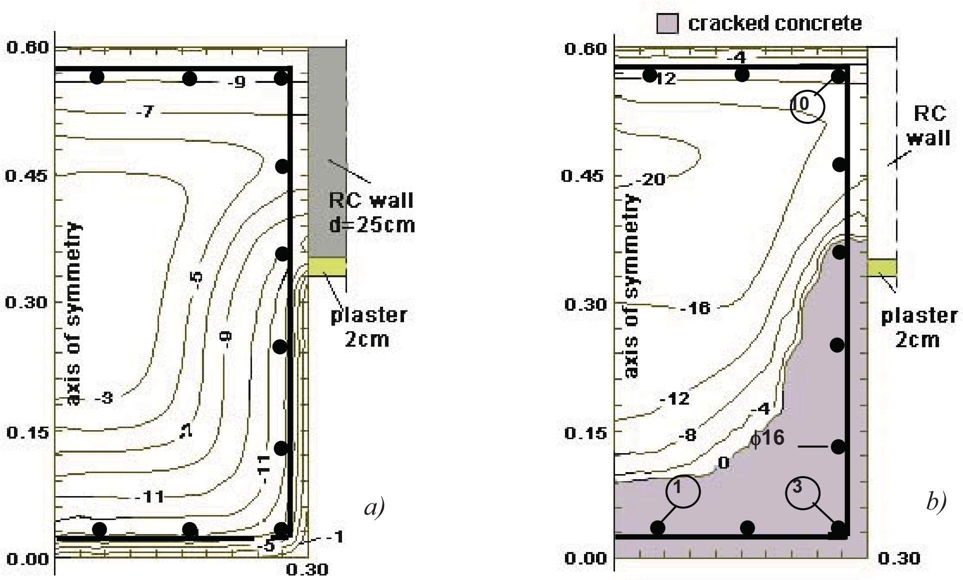

Fig. 31.7 Stress distribution in the cross section of the most damaged column

a) at max. temperatures; b) after the cooling period

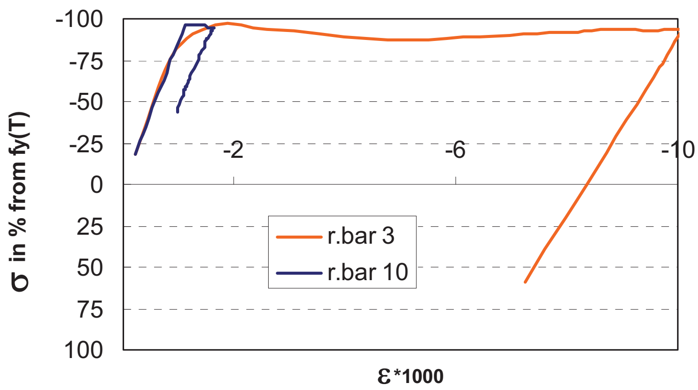

Fig. 31.8 Stresses in the reinforcing bars no.3 and no.10 of the most damaged column, as a percent of yielding strength at corresponding temperature

Columns S2 and S3 were exposed to fire almost from all sides, so it caused almost symmetrical cross section temperature field. In the cross section surface layers, as a result of high temperatures, the concrete strength was reduced. Calculation results indicate that in 4-5cm thick layer, which is 30% of the cross section of the column, the strength reduction is significant and the residual compressive strength of concrete is 15Mpa in average (Fig. 31.9b).

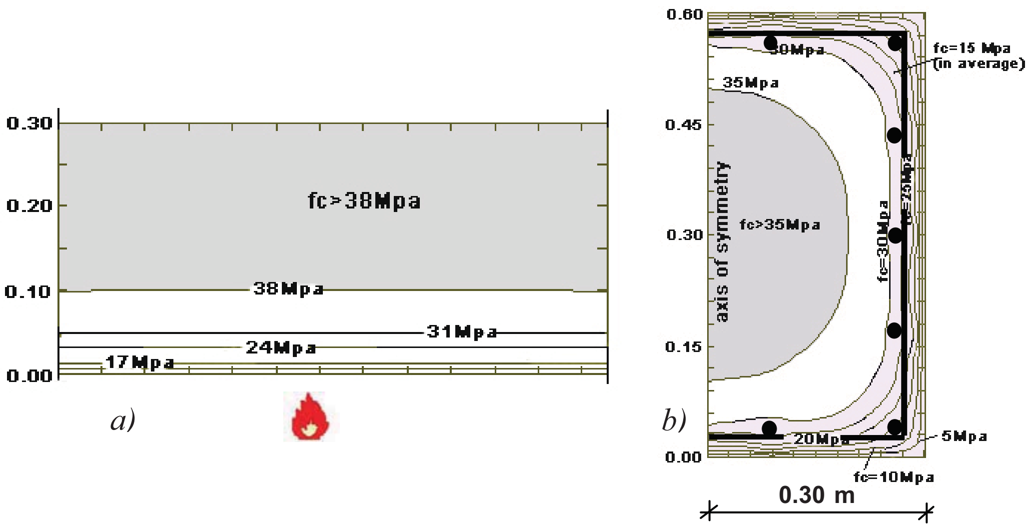

Fig. 31.9 Residual concrete strength in cross section of
a) RC wall exposed to fire from inside; b) RC column exposed to fire from all sides

Results obtained by the thermal and static analysis of the frame structure show that fire had the most negative influence upon the beam elements from level 800 (over the 7th floor). These results correspond well with experimental results obtained by laboratory testing of specimens taken from the elements from that floor and with visually recorded changes in concrete color (V7, G7, D7, E7, Tab. 31.1). The fire intensity on the 8th floor was less then that of the 7th floor and the degree of damage was less too (for B8 the reduction of concrete strength is less then for the other specimens). According to the defined degree of damage, beams and columns from the 7th and 8th floor have to be adequately repaired.

Beams from level 800 (over the 7th floor) were fire exposed from both sides, but from the upper side, as a part of the RC slabs, they were covered with 1cm thermal isolation and 4cm lean concrete (Fig. 31.1c), that directly influenced upon the cross section temperature field (Fig. 31.10a) and the concrete strength reduction (Fig. 31.10b). Beams from level 900 (over the 8th floor) were fire exposed only from the bottom side.

Stress redistribution was caused due to the high temperature difference between the surface layers and inner layers. During the cooling phase, when the temperature in the cross section layers decreased, the negative elongation was not proportional to positive elongation when temperature increased, so it caused longitudinal cracks along the main reinforcement. These cracks were visually noticed on the surface of all columns and beams, but the depth was not defined.

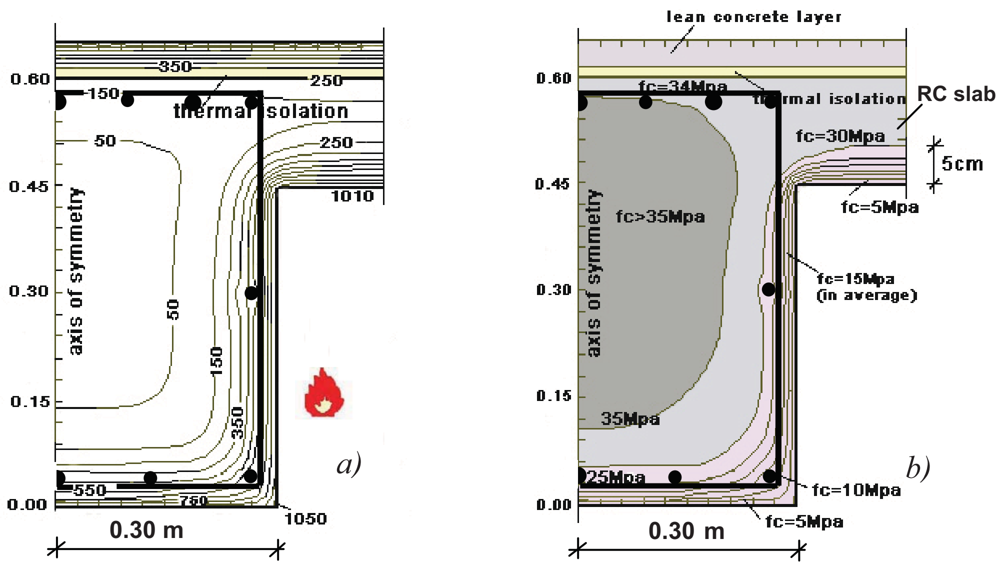

Fig. 31.10 a) Temperature distribution; b) Residual concrete strength in cross section of beam and corresponding RC slab

During the fire period large deformations occurred in beam elements from level 700 (under the fire compartment) although they were not heated because of the thermal isolation and concrete cover over the RC slabs. During the cooling period all cracks were closed and there were no additional residual stresses.

The results obtained by the thermal analysis of the RC slabs type OMNIA, the detailed visual survey in-situ and the experimental results obtained by laboratory testing of specimens (MS, SII, Tab. 31.1) lead to a conclusion that the concrete strength is reduced only in the bottom layers where concrete is in tension and has no influence on the bearing capacity of the slabs. In the upper 10cm thick compressed layers the concrete strength is not significantly reduced, therefore the recommended repair will be only surface finishing of the bottom of the slabs.

The RC walls were fire exposed only from the inside of the compartment, the temperature didn't penetrate deep in the cross sections (fig.9a) and the strength reduction was significant only in the concrete cover layers (3.0-3.5cm), therefore the repair of the RC walls is not recommended. These results correspond well with experimental results obtained by laboratory testing of specimens B8, G7, D7, E7 (Tab. 31.1) taken from the RC walls from the 7th and 8th flour. It is not the case only for the specimen V7 (Fig. 31.3a) taken from the RC wall in the room where the fire started and the temperature was highest.

31.4 SUGGESTED REPAIR OF FIRE DAMAGED STRUCTURAL ELEMENTS

The aim of the repair suggested in this project is to obtain nearly the same strength, stiffness, deformability and ductility of the repaired elements and their cross sections as they had before the fire action. The bearing capacity of the column cross section can be determine from the interaction diagrams: moment-axial force (M–N), moment-curvature (M–φ) and axial force-curvature (N–φ). The main interaction diagram for the beam cross section is moment-curvature (M–φ).

The M-N interaction diagrams of the most fire damaged column before and during the fire action, after the cooling phase and after the suggested repair are presented on Fig. 31.11, Fig. 31.12 and Fig. 31.13.

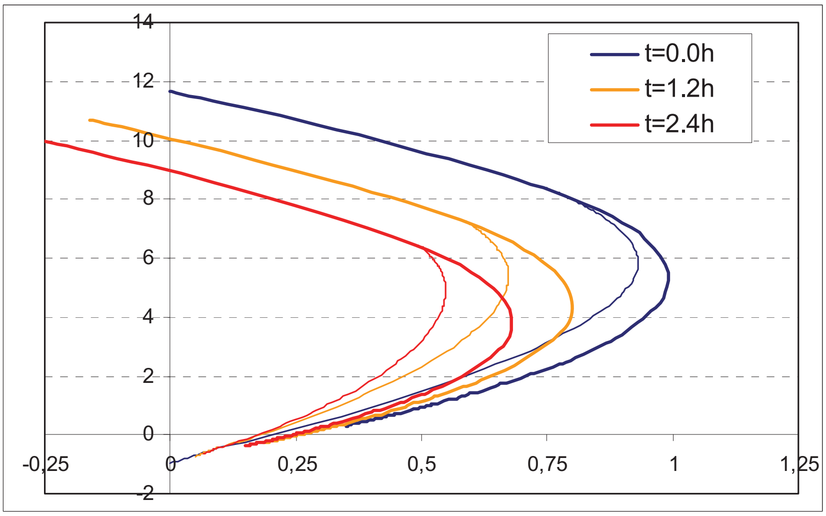

Fig. 31.11 Interaction diagram "bending moment-axial force" for column S4
(before action of fire and at the moment of max. temperature)

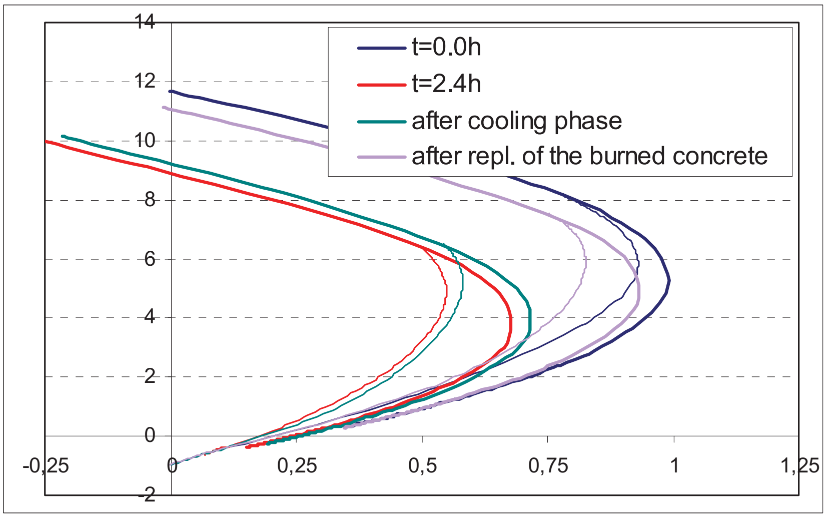

Fig. 31.12 Interaction diagram “bending moment-axial force” for column S4 (before action of fire, at the moment of max. temperature, after cooling phase and after replacement of 5cm thick layer of burned concrete)

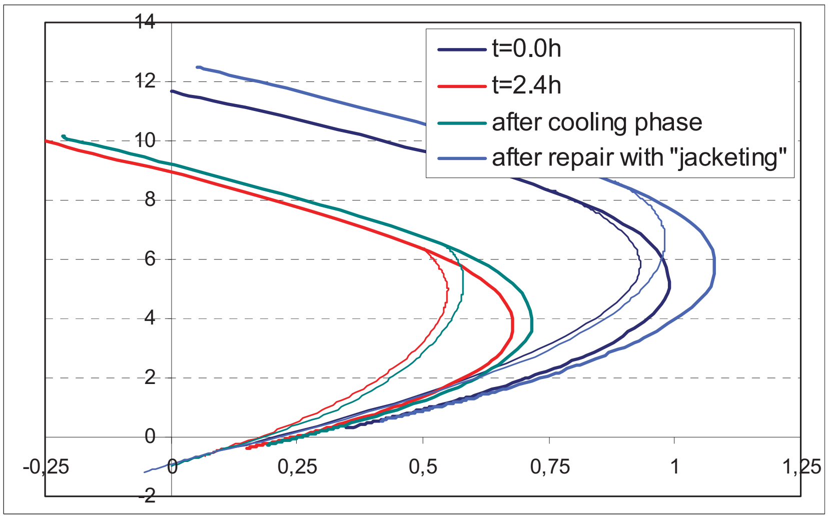

Fig. 31.13 Interaction diagram “bending moment-axial force” for column S4 (before action of fire, at the moment of max. temperature, after cooling phase and after suggested repair with “jacketing”)

The interaction diagrams indicate negligible increase in bearing capacity and ductility of the cross sections after the cooling phase. Replacement of the deteriorated 4-5cm thick concrete layers with new concrete layers has a negligible effect too, therefore the suggested repair is: replacement of the deteriorated 4-5cm thick concrete layers with new concrete layers; addition of main reinforcement and stirrups, addition of new lateral 3cm thick concrete layers. This method is well known as “**jacketing**” and it is most effective if the new layers are made from all sides (the jacket is closed). In this case, due to the elements position, jacketing is not possible from all sides and the suggested solution is jacketing of columns only from three sides, or partly from the fourth side but the jackets will not be completely closed, and for beams jacketing is only from the three sides.

The suggested repair of the most damaged column is presented in Fig. 31.14. The repair consists of: elimination of the “burned” 3-4cm thick concrete layers; addition of main reinforcement (4ϕ18, RA 400/500-2) in the corners of the cross section and at the contact with the RC wall; addition of stirrups (ϕ10/7.5/15cm, GA 240/360) and addition of new lateral 3cm thick concrete layers. Due to the specific geometry open stirrups are used and they have to be welded to the main reinforcement in the RC wall (according to the situation in-situ). Welding between the new ribbed reinforcement and existing smooth reinforcement is not recommended for two reasons: the additional reinforcement has higher yield strength and the existing smooth reinforcement has residual plastic deformations.

The repair of the beam elements consists of: elimination of the “burned” 3-4 cm thick concrete layers; addition of reinforcement in the two corners of the cross section (2ϕ20, RA 400/500-2); addition of stirrups and new lateral 3cm thick concrete layers (Fig. 31.15). Due to the specific geometry open stirrups are used and they have to be welded to the existing stirrups at the middle of the cross section.

The jacketing has to be made with **self compacting** concrete MB35, or **torcrete** concrete MB30.

The recommended repair for the RC slabs will be only surface finishing of the bottom of the slabs, and repair of the RC walls is not necessary.

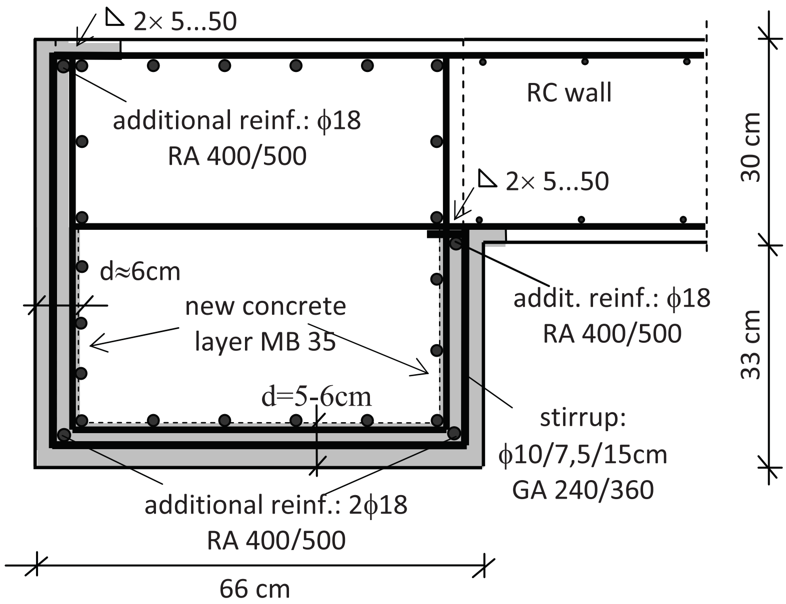

Fig. 31.14 Repair of RC column by “jacketing”

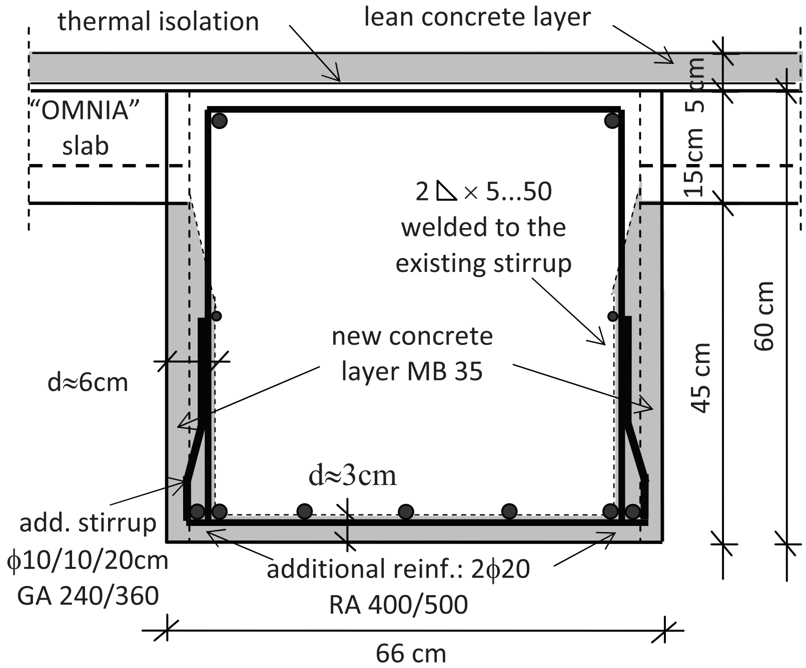

Fig. 31.15 Repair of RC beam by "jacketing"

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32 SAFETY ASSESSMENT OF STEEL STRUCTURE OF THE SINGLE-STOREY INDUSTRIAL BUILDING AFTER A LOCAL FIRE

Summary

Single-storey, two-bay industrial steel-framed building with columns rigidly mounted in the foundation and roof trusses pivotally supported on the columns was subjected to a local fire. Some polypropylene tanks stored in piles near the extreme pillars of the left side of the building has completely burned. Unfavourable events occurring during the action of heat on these above mentioned two steel columns have had a direct impact on the entire static system, and thus – on the whole structural elements located not only in the right bay (crane beams, cold-rolled purlins, roof and wall bracing systems, skylights, etc.) but the rest of the building, as well. Steel construction of the hall was not protected against the fire temperatures at all.

Main structural components of the building have been subjected to high temperatures of varying degrees, depending on their location in the structural system. None detailed data are available regarding either the time-length of the fire itself or the distribution of the temperature field. Local Fire Department has not provided this sort of information. Further increase of internal temperature was observed during the flashover phase after the unsealing of finishing layers of the roof covering and after the bursting of the oblong side windows in the roof skylights.

The approximate temperature of the fire environment was estimated based on the analysis of available source materials and studies on this topic. During the study (following the information provided by available literature) it was adopted that the combustion temperature of polypropylene (PP) can reach 673.80^{o}C. This means that the significant part of steel structural components located in the zone of high temperatures during the fire have had greatly reduced strength values. Basing on the impact analysis of high temperatures a required range of modernization for steel framework has been specified.

32.1 GENERAL INFORMATION

32.1.1 General description of the building

The property described in this paper is a single-storey, two-bay industrial hall equipped with two gantry cranes and skylights in the roof. The height of the building measured to the ridge of trusses is equal approximately +9.945 m, and to the ridge of skylights reaches +12.035 m. The spam of naves, measured in axes of columns is equal L= 2 x 18,0 m. Totally, the main steel load-bearing structure of the building consists of 17 two-bay steel frames.

The construction of the roof was design in form of flat roof trusses made of thin-walled elements, which were produced of unalloyed carbon steel, grade St38U-2 due to DIN standards. The roof trusses were technologically divided into halves and assembled during erection. The bolted assembly connections localised in a mid-spam of upper and lower chords have been designed as pre-stressed with high-strength bolts.

Steel columns, designed as two-chord laced built-up members with the cross-section dimensions dependent on the location in the structure, have been made of thin-walled elements produced of unalloyed carbon steel, grade St38U-2 due to DIN standards. These columns have been composed of two parts of different cross-sectional dimensions: the upper one that supports the roof trusses, and the lower part which supports the crane beams. In a roof slope some transverse bracing systems, distributed at every 6 fields, have been mounted. No longitudinal roof bracing systems have been provided. Bracing members were made of thin-walled angles manufactured of steel grade St38U-2. The roof bracings of the same sort are provided for skylights. Roof purlins were designed as continuous beams made of thin-walled members.

The project of the building has not contained any calculations or guidelines for fire protection of steel structure.

32.1.2 Description of structure after fire

The fire took place on 1st of August 2007. In a result, few lacquer tanks made of polypropylene which were located near the outer longitudinal wall, were completely burnt, (Fig. 32.1.1, 32.1.2).

Fig. 32.1.1

Fig. 32.1.2

Unfortunately, no data are available, how long the fire lasted, and what temperature prevailed in the hall during a fire. As it was mentioned before, the steel structure of the object was not protected against high temperatures at all. The upper parts of two neighbouring columns, located near the fire source were strongly damaged. Some structural components of columns such as diagonals and posts have been locally deformed and destroyed, (Fig. 32.1.3).

Fig. 32.1.3

Under the influence of high temperature also some significant deformations of components in two roof trusses, localised nearest to the fire source occurred, (Fig. 32.1.4). Partial deformation of some diagonals and fragments of upper chords was also found in other roof trusses, more distant from the source of fire.

Fig. 32.1.4

32.2 ANALYSIS OF THE STRUCTURE SUBJECTED TO FIRE

32.2.1 Evaluation criteria

Fire Service Department has not defined the range of temperatures that prevailed in the hall during a fire - there are completely no data on this subject. Reliable estimation of the actual temperature of fire environment is very difficult and could be done only with the help of advanced numerical tools. Based on the analysis of source materials and available studies on the subject (references) it may be determined that the combustion temperature of combustion of polypropylene (PP) can reach 673.80°C, (Tab. 32.2.1).

Tab. 32.2.1 Maximum temperature of the hot zones in a fire compartment

POMIESZCZENIE	Maks. temp. strefy gor acej [°C]							
	Dd	PA	PE	PMMA	PP	PU	PS	PWC
pokój z pożarem	609,3	662,5	682,1	554,6	673,8	667,8	839,0	423,2

32.2.2 Strength analysis

Thermal interactions result in significant changes in mechanical properties and strength of structural steel. The reduction factors for the stress-strain relationship for steel at elevated temperatures, quoted after Eurocode 3: EN 1993-1-2 are given in Tab. 32.2.2. Linear interpolation may be used for values of the steel temperature intermediate to those given in table.

For the temperature level reaching a value of approximately 673.8°C the change of these parameters, in relation to ambient temperature, averages:

- approx. 18% (elastic modulus at elevated temperature $E_{a,\Theta,}$ relative to E_a),
- approx. 10% (proportional limit at elevated temperature $f_{p,\Theta,}$ relative to f_y),
- approx. 30% (effective yield strength at elevated temperature $f_{y,\Theta,}$ relative to f_y).

Tab. 32.2.2 Reduction factors for stress-strain relationship of carbon steel at elevated temperatures

Steel temperature θ_a	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
500 °C	0,780	0,360	0,600
600 °C	0,470	0,180	0,310
700 °C	0,230	0,075	0,130

This leads to the conclusion that a significant part of steel structure components located in the zone of high temperatures, as a result of the fire has greatly reduced or completely lost its capacity.

The result of the significant reduction of the load-bearing capacity of structure is well seen in form of local deformations and damages of members and components.

In the absence of detailed data on the temperatures that prevailed inside the object during the fire during the analysis it was assumed that:

- the highest temperatures prevailed in the zone located just below the ceiling,
- the temperature near the source of fire ranged within the limits 400-600 °C,
- all the bolted assembly connections localised in a mid-spam of upper and lower chords, designed as pre-stressed with high-strength bolts, have been subjected to destructive influence of high temperatures.

32.2.3 Residual strength of steel structures after fire

Although the ad hoc capacity of steel structures decreases due to strength loss of structural steel when the temperature rises the residual strength of steel seems to recover quite well after cooling down, as it was proved in tests (Kirby at al., 1986 & Outinen, 2007). The residual strength of steel after cooling down depends on many parameters, such as: original properties of the steel grade, the maximum reached temperature, loading history, degree of deformation, etc. The general conclusion from the limited amount of experimental results concerning the residual strength of structural steel after heating leads to the statement, that if the elements of steel structure are not deformed in fire, the strength of steel will likely be still adequate, but all the connections, surface coating etc. Have to be checked thoroughly. A rough limit is drawn to about 600°C after which permanent loss of strength seems to take place. According to BS 5950-8:2003 hot finished steels and cast steels can be re-used after fire when the deformations remain within the acceptable tolerances for straightness and shape. For cold finished steel grades that remain within tolerance, it's recommended to assume they have approx. 90% of the original strength.

32.3 CONCLUSIONS

1. The design project of the presented building haven't included any calculations, or guidance on fire protection, and steel structure was not protected against fire at all.
2. Fire Service has not specified the temperature field distribution in fire-stricken areas. In Polish realities they are not used to do it, in general.
3. Based on Eurocode 3 one can determine in an approximate way some changes of the significant mechanical parameters and strength of steel at high temperature, which allow the estimation of the real load-bearing capacity and safety of structures as well. This possibility particularly applies to the structural elements more distant from the source of fire.
4. Structural elements located close the source of the fire, and in a lesser extent, the remaining steel structure have lost their load-bearing capacity.
5. The pre-stressing force in high-strength bolts used in assembly connections of roof trusses has been degraded under the influence of high temperature.

Taking the items given above into consideration:

- In case of new buildings (including also existing ones, for which the change of function is possible during their lifetime) it should be obligatory to determine how the high temperature may influence the structure, and execute the project design taking into account the real fire-load density and specifying the necessary fire protection,
- State Fire Service should at least in approximate range determine the temperature field distribution in fire-stricken premises.

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