Thermo-mechanical analysis of an underground car park structure exposed to fire

Authors: E. Annerel[*, a], L. Taerwe[a], B. Merci[b], D. Jansen[c], P. Bamonte[d], R. Felicetti[d]

[a] Ghent University, Faculty of Engineering and Architecture, Department of Structural Engineering, Magnel Laboratory for Concrete Research, Technologiepark Zwijnaarde 904, B-9052, Ghent, Belgium

[b] Ghent University, Faculty of Engineering and Architecture, Departement of Flow, Heat and Combustion Mechanics, Sint-Pietersnieuwstraat 41, B-9000, Ghent, Belgium

[c] DHV B.V., Business Unit Gebouwen, Larixplein 1, 5616 VB Eindhoven, The Netherlands

[d] Politecnico di Milano, Department of Structural Engineering, Piazza Leonardo da Vinci 32, 20133 Milano, Italy

Emmanuel.Annerel@UGent.be, Luc.Taerwe@UGent.be, Bart.Merci@UGent.be, daan.jansen@dhv.com, patrick.bamonte@polimi.it, roberto.felicetti@polimi.it

* corresponding author: Tel +32 9 264 55 19, Fax +32 9 264 58 45,

Abstract

Underground concrete structures composed of flat slab-column connections are sensitive to punching failure at ambient temperatures. During fire however, due to the restraint of thermal curvatures , an increase of the punching load can occur which could lead to premature collapse. In this paper, this problem is studied numerically by the combination of CFD and mechanical finite element analysis, whereas further considerations are developed by the use of plasticity theory. The basis for the fire scenario used in the analysis is found in the literature as a real fire, as well as the Belgium standard NBN S 21-208-2 for the design of the ventilation system in underground car parks with respect to car fires. The dimensions of the underground car park are also based on a real fire accident that occurred in Gretzenbach and where the structure actually failed due to punchingfollowing a rather small fire. In the analysis, special attention is given to the increase of the axial load on the slab-column connection with temperature. For the mechanical properties and boundary conditions assumed, it is found that during fire the axial load could increase with a factor which, at least in the worst cases, could be close to the safety factor found from large scale fire tests.

Keywords: fire, flat concrete slabs, punching shear, CFD, FE analysis, global structural analysis

1 Introduction

Underground car parks are often built by means of cast in situ flat slabs. Eekert [1] reports that in 2006 in the Netherlands, 83% of the underground car parks are cast in situ, whereas 17% are precast. Flat slabs supported by columns are known to be rather sensitive to punching failure [2]. An experimental study recently carried out at Ghent University shows a loss of punching shear capacity after 120 minutes ISO 834 fire of about 40 %, corresponding to a remaining safety margin of about 1.8-2.2 times the service load. Details of the test program are given in Annerel et al. [3]. During the same experimental campaign, one slab was overloaded with a factor of 1.5-2.0 times the service load during heating and failed after 20 minutes of ISO 834 fire. Such an increase of the load acting on the column may occur during fire when looking at the global structural behaviour, because of the indirect actions ensuing from restrained thermal dilations (and curvatures). Due to sagging of the concrete slabs towards the fire, indirect bending moments are introduced over the supports, resulting in an increase of the load at the column-slab connection (illustrated in Figure 1). An increase of this axial load can also be found when considering the expansion of a column directly exposed to the fire. If this increase exceeds the safety margin of 1.8-2.2, as found for the slabs investigated during the above mentioned experimental program, failure will occur. However, this increase is limited by the possible formation of a yield line pattern, and thus by the attainment of a ductile collapse mechanism. Nevertheless, it is of primary importance that this ductile mechanism can be attained, without premature brittle failures (such as punching, for instance), preceding it. It is therefore of interest to examine how large the latter contributions are expected to be, because this will influence the structural stability of the underground car park.

In this paper, a global structural analysis is performed for a specific case which follows from a literature review. The dimensions of the underground car park studied in this paper are based on those of the Gretzenbach underground car park which was involved in a fire accident that occurred in 2004. In this fire a roof slab failed, because of punching shear, after a rather small and localized fire[4]. A basic analysis of this fire scenario is given by Bamonte et al. [5]. This paper contributes further to this analysis by investigating the effect of different fire loads on the structural behavior. For a realistic evaluation of the thermal input, a review is given in section 2 about the amount of fires and the number of cars involved in real accidents in the recent past. Since the review showed a large scatter on the number of cars involved, the temperature distribution in time and space is calculated for a specific fire sequence scenario documented in the literature. This scenario is based on the Harbour Edge fire [6] and can be considered as severe. The temperatures obtained by means of CFD calculations are then introduced into a finite element model, developed by means of the finite element (FE) package ABAQUS, to highlight the influence of the structural layout on the punching load.

2 Literature review

2.1 Statistical studies of car park fires

Although fire is considered as an accidental load, car park fires frequently occur as can be found from statistical studies. For further details, reference is also made to Merci et al. [7].

On assignment for of the fire department of Paris (France), the Centre Technique Industriel de la Construction Métallique (CTICM) investigated the number and size of car park fires in Paris [8]. In

1997, the fire department reported 327 fires in underground car parks, whereas between 1995 and 1997, 78 fires were reported in car parks above ground level. When the fire was controlled by cars, which was the case for 158 fires, approximately 98% of the fires was restricted to less than 4 cars. Four cars were burning in only 2 cases, while 1 fire was controlled by 5 cars and for 2 fires, even 7 cars were involved. On the other hand, in 32% of all cases, the fire was not caused by the ignition of a car.

Furthermore, a study from New Zealand showed that between 1995 and 2003 in approximately 97% of the fires only 1 burning car was involved [9].

A study carried out by BRE in Great Britain found an occurrence of 3096 car park fires over a period of 12 years, which corresponds to an average of 258 fires each year [10]. Of these fires, 51% started with the ignition of a car, but in most cases no fire spread to other cars was stated.

2.2 Causes of car fires

In Merci et al. [7], an extensive review is presented on car park fires. Car fires are often ignited by arson (in the passenger cabin, the engine block or at the level of the tires) [11] Electrical short circuit is also a possible cause. Different parts of the car are available as fire load, such as the synthetic parts of the body, the interior lining, the rubber tires and the fuel. The fire can be sustained due to the amount of oxygen generally available in car parks. Several parameters affect the evolution of the fire, such as the ventilation conditions of the car itself (broken or open windows) and inside the car park, the position of the car (close to a wall or not), the distance between neighboring cars, wind effects outside the car park, etc. [7]. A fully developed fire can result in a liquid pool fire as well, due to rupture of the fuel tank, with potentially rapid spread of the liquid fuel over a large area.. In the case of a gas tank, an explosion is also possible. However, in most cases the over pressure valve will release the gas causing a jet flame [11].

Fires in underground car parks differ from those in aboveground structures, because the ventilation conditions affectthe temperature development. In open car parks, a certain amount of the produced heat will be lost through the open façades, while in underground car parks the hot gases are not able to leave the 'compartment' without forced ventilation and thus could accumulate. This can cause more rapid fire spread [7]. Obviously, radiation feedback from the heated structure will further increase the heat transfer and cause faster fire spread.

Car fires are localized and the period of high heat release rate is of relatively short duration (order of magnitude about 30 minutes [7, 11]), thus resulting in a temperature distribution which is strongly variable in time and space. When the fire spreads, it moves to (a) neighboring car(s), while the first car(s) start to extinguish. Depending on the distance from the fire, the temperatures strongly vary over the length of a structural element. Thus,, a uniform temperature distribution as supposed for a standard ISO 834 fire test, in most cases does not reflect reality. As a result, differential and restrained thermal dilations can cause other types of damage within the element and at its supports than those typically found in laboratory fire tests. These phenomena are illustrated in section 2.3, while a detailed study of the influence of travelling fires on a concrete frame is given in Law et al. [12].

2.3 Recent car park fires

A short list is given of damages of real car park fire accidents that occurred mostly in the Netherlands during the period 2002-2007:

- Schiphol Airport (The Netherlands). On October 13, 2002, a fire occurred in an aboveground car park for rental cars at Schiphol Airport. Approximately 51 cars burned, due to which a partial collapse of the structure occurred [13]. The structure consisted of massive pretensioned concrete slabs which were supported by concrete T-girders. Fire investigations showed that the fire was due to arson. It was a very large fire, because the car park was fully booked, with only 40 cm spacing between the cars. Furthermore, the cars were relatively new (with high amounts of synthetic materials) and had a full fuel tank (as it is often the case for rental cars).

- Apartment building Geleen (The Netherlands). During the night of 23-24 June 2004 a fire happened in a car park beneath an apartment building in Geleen. Twelve vehicles burned in total. The concrete was heavily damaged with complete cover loss for the slabs, walls and some columns. The structure was repaired with shotcrete and supplementary reinforcement [14].

  Car park Gretzenbach (Switzerland). A fire took place on November 11, 2004, in a car park in Gretzenbach. After approximately 90 minutes, the roof of the underground car park collapsed due to punching and 7 firemen died during their intervention. Fire investigation revealed design and execution mistakes resulting in an overload of soil and a decreased punching shear capacity [4]. Because of the occurrence of a clear punching failure and the typical car park geometry, this example will be used as the basis for the geometry of the case study presented in this paper.Apartment building Harbour Edge (The Netherlands). A fire occurred on October 1, 2007, in the open car park of an apartment building (Harbour Edge) in Rotterdam. At the moment of the fire, 7 cars were parked at the level where the fire took place. The fire started near the middle of the first six cars, parked side by side and spread to both sides of the row (two options are given in Figure 2). According to de Feijter et al. [15], it is most likely that the initial fire spread to the second car occurred after 10 minutes and after 12 minutes to the third car. After 22 minutes of fire also the 4th car was involved in the fire. The moments of ignition of the 5th and the 6th car, on the contrary, are somewhat uncertain. Finally, the 7th car which was separated from the group of 6 by an empty space and which was only partially involved in the fire, was not considered to contribute to the fire in terms of HRR in the fire scenario analyses, because it was only damaged, not burnt out. The building consists of prestressed hollow core slabs with a cast-in-situ concrete topping as compression layer. Additional reinforcement is provided in the compression layer as a tension ring to increase the stiffness of the slab. The load is transferred to the foundations via the façades and a central core. The design fire resistance was 120 minutes. According to d Feijter et al. [15], a partial collapse of the structure was observed during the fire. The bottom chord of the hollow core slabs failed above the fire zone. Even after the fire had been extinguished, during the cooling phase, further collapse of construction elements was observed. Anchorage failure of the hollow core slabs near their support was noticed, as well as spalling of the façade. According to Overbeek et al. [16], horizontal cracks occurred in the concrete between the canals, due to the restraint exerted by the compression layer. These horizontal cracks have led to failure of the bottom of the hollow core slabs. Because of the severity of the fire and the well documented reports of the fire investigation, this fire scenario is used in this paper to evaluate the influence of restrained thermal dilations (and curvatures) on the punching load of a concrete flat slab structure.

- 1st car park Hilversum (The Netherlands). A fire took place on November 11, 2007 in a car park in Hilversum, by which 2 cars were involved [17]. The fire damage was limited to the superficial layers and consisted of spalling until beyond the bottom reinforcement, but only in the area of the structure directly above the fire.
- 2nd car park Hilversum (The Netherlands). A fire occurred on November 29, 2007 in a car park in Hilversum (the Netherlands), by which 2 cars and a motorcycle burned. Fire investigation revealed a maximum gas temperature of about 1000°C. Nevertheless, only superficial damage could be detected on the structure [18].

2.4 Structural modeling of concrete slabs

The behavior of flat concrete slabs in fire conditions is a problem that has received a lot of attention in the past. The interest of the scientific community was focused mainly on the membrane actions that develop inside slab elements under fire conditions, as a consequence of the thermal strains.

With reference to this aspect, it is worth recalling that the temperature distribution inside the thickness of a concrete slab is highly non-linear, and the ensuing thermal strains along the thickness are also non-linear. The ensuing total deformations (that are linear along the thickness) can be decomposed into a constant component, namely the average thermal dilation, and the thermal curvature:

- whenever the average thermal dilation is restrained (because of the boundary conditions or because of adjacent portions of slab that are not heated), a state of compression is created;
- because of the large deflections ensuing from the thermal curvatures, the slab carries a significant part of the load by means of tensile membrane forces (as it is the case in ordinary conditions, at impending collapse).

The state of compression has a two-fold effect on the structural behavior:

- an increase of the positive bending moment at mid-span of a slab (i.e. an unfavorable effect), whenever the line of thrust of the membrane action falls above the centroid of the slab section;
- an increase of the bearing capacity in bending due to the beneficial effect of compression on the bending capacity [19, 20].

Whether the first or second effect prevails is difficult to predict, since they both depend on the structural layout, and more specifically on the position of the boundary restraints [21].

As for the development of tensile forces, that is by far one of the most investigated topics regarding reinforced concrete slabs in fire, it requires that the top reinforcement (that remains cold upon one way heating from the slab bottom) is prolonged over the full span length. This is another aspect that brings in a significant dependence on the structural detailing.

Further, concrete elements which are only heated from one side will undergo significant thermal curvatures, resulting in deflections and rotations. These rotations are generally prevented by the fact that slab fields are continuous over the columns, and thus a significant redistribution of the internal forces occurs. This redistribution is typically represented by an increase of the hogging slab moments over the columns. As a consequence, because of equilibrium, also the axial loads on the columns, ensuing from the shear forces at the top of the column, increase (Fig. 1). This increase is generally limited by the bending capacity of the slab, i.e. when the slab section over the column reaches its plastic capacity, the redistribution reaches its limit, and the load on the column cannot increase further [22]. In a certain sense, the plastic capacity of the slab section acts like a "structural fuse", i.e. it prevents the increase of the axial load to attain large values. With typical reinforcement ratios, however, the load increase can still be as high as 40-50% of the service load [5, 23]. This load increase can result in an increased likelihood of punching failure which is one of the key parameters in the design of thin slab structures.

3 Fire scenario

As mentioned in the introduction, the effects of restrained thermal dilations in terms of indirect actions on a flat concrete slab supported by a grid of columns are studied for a fire scenario based on Harbour Edge and a ground plan based on the underground car park in Gretzenbach.

3.1 CFD model

The car park considered in the following measures 22.5 m in length and 17 m in width. The column grid divides the car park into a central span of 8.0 m and two lateral spans of 4.5 m. An opening is positioned at one side of the central span, located over the full width and height. A plan view of the structure is given in Figure 3, from which it is clear the car park can be divided in 15 zones when considering the grid of columns. The cars are parked in zones 2 till 4.

The fires are modeled in Fire Dynamics Simulation (FDS), version 5.4.1 (Windows 64 bit version) to determine the thermal load on to the structure. The model is extended in front of and above the opening to minimize the effects of the assumed boundary conditions at the grid edges. The exterior is 14 m wide (thus covering 3 meters at each side of the opening), 3 meters deep and 1.4 m higher than the car park and covers 1.5 m above the ceiling. Figure 4 shows a geometrical representation of the CFD model.

In the model, the heat transfer between the indoor gases and the structure is taken into account. The walls, floor, ceiling and the columns are all simulated as concrete with constant material properties (k= 2.3 W/mK, c= 0.88 J/kgK, ρ= 2300 kg/m^{3-}). The walls, floor and ceiling are assumed to be 0.28 m thick. The columns are assumed to be square, with a side of 0.4 m.

The car park is naturally ventilated. At the beginning of the fire no air flows are assumed in the CFD model. As a result of the fire, thermal flows develop during the fire which results in ventilation through the opening at the front side of the car park. Hot gasses leave the room at the upper side and fresh air enters the room at floor level.

The cars are represented by adiabatic blocks of 4 m length, 1.75 m width and 1 m height. The heat is released at the upper sides of the blocks. The heat release rate per unit area is gradually increased, without changing the size of the fire surface on top of the blocks. The radiative fraction and the soot yield are chosen in accordance with the Belgian Standard and are respectively 35% (radiation) and 22% (soot) [24].

The calculation grid consists of a uniform grid of cells of 125 mm length, 125 mm width and 100 mm height, resulting in 735.744 cells. It should be mentioned that the purpose of the CFD calculations is to have an adequate order of magnitude and spatial distribution of the temperature field, rather than having the best quality of the CFD calculation. A best quality is not necessary since the temperatures will be averaged over a larger grid size for the structural calculations (see section 4). In this way, the figures illustrating the temperature distribution (Figures 7, 8 and 12) are not the direct CFD output, but already the averaged result over a structural grid of 1.5 x 1.7 m^2.

3.2 HRR curves and derived fire scenarios

For the determination of the HRR (= Heat Release Rate) of a fire, measurement results or standardized reference curves can be used. Two often used reference curves for single car fires can be found in van Oerle et al. [25] and Schleich et al. [26], studies about thrust ventilation and closed car parks respectively, further referred to as TNO and ECSC curves. These two curves show a similar temporal development, starting slowly and then developing into flashover. For the TNO curve, the maximum HRR is 6 MW, which occurs after 16 minutes and remains constant for 3 minutes. The maximum HRR of the ECSC curve is 8.3 MW which occurs after 25 minutes and is a peak value, i.e. there is no plateau at the maximum value.

Both curves are based on multiple measurements. The TNO curve was developed as an average HRR, while the curve of the ECSC is based on the most severe fire which was measured during the research program. It is also recommended that this reference curve must be adjusted for the second and subsequent burning cars, whenever a multiple car fire scenario is defined. The adjustment for subsequent burning cars is required, because the heat-up time is assumed to be 5 minutes shorter. Thus, for those cars the maximum HRR occurs after a subsequent 20 minutes. The Belgian Standard NBN S21-208 uses the TNO reference curve [24]. In this standard, the curve is slightly modified, since the fire burns out faster. In Figure 5, the TNO and the ECSC curve are depicted, together with the adjusted ECSC curve for the second and subsequent cars; the NBN curve is shown as well.

Based on these fire curves, the following 3 fire scenarios are defined:

- Fire scenario 1: a fictitious fire scenario of subsequent burning of 6 cars according to the NBN HRR curve and calculated by means of the CFD model. This fire scenario is based on Bamonte et al. [5], where the temperatures are calculated by a zone model and the HRR of an Austin Maestro car is used.

- Fire scenario 2: the same fictitious fire scenario, but the derived temperatures are the gas temperature near the car park ceiling averaged over each slab field as calculated from the CFD model by the commando 'Upper Temperature' in FDS.
- Fire scenario 3: the Harbour Edge fire sequence with ECSC HRR curve and calculated by means of the CFD model.Figure 2 shows 2 possible fire sequences, designated hereafter as fire scenarios 3a and 3b.

For each fire scenario, the HRR-curve used and also the calculated spatial variation of the temperature field is discussed in the following sections.

3.2.1 Calculated surface temperatures for fire scenario 1

The first fire scenario consists of subsequent burning of the available 6 cars, starting at the first car and fire spread to the adjacent car every 15 minutes (cf. Fig 4 for location of the cars). The resulted HRR as function of time is depicted in Figure 6.

Based on the HRR curve, the temperature near the ceiling is calculated in FDS. Figure 7 shows the ceiling temperatures for a duration of 60 and 120 minutes.

3.2.2 Calculated surface temperatures for fire scenario 2

The second fire scenario uses the FDS output of fire scenario 1, but averages the temperatures in the 15 zones discussed in section 3.1 by means of the commando 'Upper Layer' of FDS. Figure 8 illustrates the calculated ceiling temperatures, again for 60 and 120 minutes of fire duration. The peak temperatures are lower for both fire durations with respect to fire scenario 1.

This calculation method could be compared with the findings of a zone model, since they both result in average temperatures for each zone. Working with a zone model would simplify the calculation model, but it is not sure that the calculated temperatures are correct. Therefore, to solve this question, additional temperature analysis of the car park are performed with the zone model CFAST. To improve the accuracy of the calculations, the car park is divided in 15 connected compartments as indicated in Figure 3. The cars are parked in the 2nd, 3rd and 4th compartment. Temperature differences of about 200°C are found between the CFD and the zone model (Figure 9). This indicates that despite the fact that the individual compartments are connected in the zone model, no interaction between adjacent fires is taken into account. Based on this observation, zone models seem not to be able to predict the temperature development of fires travelling over several compartments.

3.2.3 Calculated surface temperatures for fire scenario 3

The ECSC reference curve is used as input in FDS. Figures 10 and 11 illustrate the total HRR of fire scenarios 3a and 3b. It is noticeable that the peak value of the total HRR is reached between 25 and 29 minutes of fire, by which the first three cars are burning almost simultaneously. Indeed, due to subsequent ignition with a delay of only 5 minutes, the burning of the first cars results in a larger value than found from a single burning car. For fire scenario 3a, the maximum value is 22.6 MW, while for fire scenario 3b 20.2 MW is obtained. These values are about 3 times the HRR of fire scenarios 1 and 2. Since fire scenario 3a results in the most severe temperature effects, this HRR is used to study the influence on the structural behaviour of the underground car park in section 4.

Figure 12 depicts the calculated ceiling temperatures for 60 and 120 minutes of fire duration. With respect to fire scenario 1, the values of the Harbour Edge fire are larger at 60 minutes, respectively 763°C against 816°C. The opposite is visible at 120 minutes of fire where 303°C is obtained by fire scenario 1 and 215°C by fire scenario 3a.

3.3 Comparison of the different fire scenarios

Figures 13 and 14 illustrate the calculated gas temperatures near the ceiling above the different cars, respectively for fire scenarios 1 and 3a. Between both fire scenarios, differences can be noticed in time, but the maximum temperature level reached is for both cases about 1100°C above all cars.

Figure 13 clearly illustrates the effect of subsequent car burning with fire spread times of 15 minutes to another car. As a result, a time difference of 75 minutes between the first ignited (car 1) and the last burning car (car 6) is noticed. When the fire of the last car is fully developed, the ceiling temperature above the first car drops to less than half of its maximum value. Regarding the structural analysis presented in section 4.2, the temperature effect on the slab-column connection P1 is close to the temperature curve of car 1, whereas connection P4 corresponds to car 6.

On the other hand, for fire scenario 3a, the temperature peaks are less pronounced as the curves are closer to each other. The maximum temperatures occur between 20 and 60 minutes of heating.

Based on the temperature curves, no large differences in total heat exposure are found between the slab-column connections considered, except for the slab-column connections P1 and P4 which are located at the corners of the heating zone, resulting in a 2-sided fire exposure, instead of a 3-sided exposure for the intermediate columns.

4 Thermo-mechanical analysis

4.1 Introduction

The thermo-mechanical model is carried out by means of the FE code ABAQUS. The mechanical model is deliberately kept simple to make the case study general as the focus is on designer-friendly approaches. In this way, the peculiarities of the problem are highlighted and general trends can be derived. On the other hand, introducing nonlinear properties would result in a more realistic behaviour of the structure as cracking and crushing of the concrete is considered. But this would bring, considering the purpose of the paper, too many parameters into play.

The structure is modelled with a layered shell element that allows to properly account for the heat transfer through the thickness (= 0.28 m, as in the real case) of the slab. The nature of the problem is such that the thermal and mechanical analysis do not need to be coupled. Only the thermal field has an influence on the structural behaviour, through the decay of the mechanical properties and the thermal dilations, but not vice versa. This assumption holds if no spalling occurs and is clearly instrumental in simplifying the analysis, since a fully-coupled hygro-thermo-mechanical analysis would be necessary to simulate spalling properly. Such a model is highly demanding in terms of computational time and would unnecessary complicate the analysis with respect to the scope of the paper. Moreover, there is no generally accepted spalling criterion, something that brings in further complications. Therefore, the thermo-mechanical problem was addressed by means of a sequentially coupled analysis. The heat transfer problem was first solved and the temperatures inside the slab (i.e. at the centroid of each layer) were determined. The results of this preliminary thermal analysis are then used as an input for the subsequent mechanical analysis, together with the applied loads, to simulate the effect of the fire on the structure.

In fire scenarios 1 and 3a, the input data of sections 3.2.1 and 3.2.3 consisted of the ceiling temperature, since the structural element was part of the CFD simulations (see also section 3.1). The structure was subdivided on the basis of a grid with 150 points (10 points over the width and 15 points over the length of the car park), in order to reproduce the same layout used in the simulation of the fire scenario. In these two cases, the heat transfer was modeled by simply imposing the intrados temperature at the lower face of the concrete slab, and by considering the temperature evolution over the thickness as a pure conduction problem.

For fire scenario 2, the data obtained in section 3.2.2 are used as gas temperature of the upper (i.e. the “hot”) layer for each of the 15 zones of the car park. In this case, the heat transfer process inside the slab is modelled by enforcing convection (with coefficient = 35 W/[m^2·K], see EN 1991-1-2 [27]) and radiation (emissivity of exposed surface = 0.70, see EN 1991-1-2) between the ambient atmosphere and the concrete slab.

The columns are modeled as point supports. This assumption implies that no bending moments nor shear forces are transmitted between the slab and the columns. Moreover, since the columns are not included in the thermal analysis, their thermal elongations are neglected. This assumption can be considered reasonable, because the structure under investigation is assumed to consist of a rather thin concrete slab with massive concrete columns. Therefore, the deflections of the slab ensuing from the thermal curvatures (and the ensuing indirect actions) are by far larger than the elongations of the columns (whose core remain cold and thus yield only limited thermal strain). Finally, the linear supports provided by the surrounding walls along three boundaries of the slab are considered to provide no rotational restraint to the slab. Moreover, they do not restrain the horizontal displacements, which means that the slab is free to expand.

4.2 Results of the structural analysis

The average size of the mesh is 0.50 m (maximum size = 0.50×0.66 m in the central portion; minimum size = 0.50×0.50 m in the lateral portions), a size that is a good compromise between computational effort and accuracy of the results. The main properties of the concrete in ambient conditions are the modulus of elasticity (E = 30000 MPa), Poisson's ratio (ν = 0.20), and the coefficient of thermal dilatation (α = 9.00E-6°C^{-1}). The structure is subjected to a self-weight load of the slab of 7 kN/m² and a permanent load due to the soil on top of the slab equal to 10 kN/m².

4.2.1 Preliminary analysis

A first preliminary analysis is carried out to determine the most critical parts of the structure. For the sake of simplicity, the evolution of the modulus of elasticity and the coefficient of thermal expansion with temperature was not taken into account and fire scenario 2 (averaged temperature in each of the 15 zones) was used. As for the stiffness of the structure that plays a role in the magnitude of the indirect actions, two assumptions are considered:

- Model 1: the modulus of elasticity is kept constant at the ambient temperature value E_{20}. This first model allows to highlight the general trends in terms of structural behavior, despite its characteristics are far from reality.
- Model 2: the same as model 1, but the shear modulus G is put equal to $E_{20}/8$, in order to take into account the possibility of a reduced torsional stiffness because of cracking.

These two basic models are used as preliminary models to highlight the general trends of the structural behaviour and to clarify the role played by the two-way load-bearing ability of the structure.

Regarding a possible failure due to punching during fire, Figure 15 depicts the calculated axial forces acting on the columns. The results are only shown for columns P1-P5, which are the closest to the fire and thus most affected. It is clear that the columns most adversely affected by the fire are P1 and P4: both show a very significant increase of the axial force ($N_{fire,max} = 3.16 \cdot N_{20}$ and $2.62 \cdot N_{20}$ respectively in model 1, and $N_{fire} = 2.73 \cdot N_{20}$ and $2.25 \cdot N_{20}$ respectively in model 2). Notice that this expected increase of the axial load is much larger than the punching resistance that was recently measured experimentally, namely 1.8-2.2 times the initial load [3]. However, it should be kept in mind that this increase is significantly influenced by the fact that the stiffness values are kept constant throughout the whole fire duration, and equal to the value in ambient conditions. Finally, it is worth noting that the differences between models 1 and 2 are of minor importance (i.e. the two-way load transfer ensuing from the torsional stiffness), if compared to the relative effects of the fire.

4.2.2 Advanced analysis

Based on the preliminary results in the following the attention will be limited to columns P1 and P4 only . Two further models are considered, to take into account the effects of high temperatures on the stiffness of the structure:

- <u>Model 3:</u> the modulus of elasticity is decreased as function of temperature according to EN 1992-1-2 standard [27];
- <u>Model 4:</u> the modulus of elasticity at ambient temperature is reduced by 75% to take into account the combined effects of cracking and temperature-induced stiffness decay. This rather crude assumption is based on Kordina [22], where on the basis of several tests on continuous slabs in fire conditions an effective stiffness equal to 20-25% of the initial (uncracked) stiffness is proposed for reinforced concrete slabs and to 50-90% for prestressed concrete slabs.

Again, the assumptions of the two models are simple, but instrumental in highlighting different effects. Since in model 3 the stiffness reduction is temperature-dependent (via the modulus of elasticity), the role of the mutual restraints by the different parts of the structure comes into play (i.e. the colder parts restrain the portions that are directly exposed to fire). In model 4, on the contrary, there is no distinction between the different parts of the structure as the stiffness reduction is calibrated on real tests. However, it is probably more close to capturing the role played by nonlinear phenomena such as cracking. In both models, the coefficient of thermal dilatation is considered variable with temperature according to the provisions given in EN 1992-1-2 for siliceous concrete.

Figure 16 shows the results in terms of axial force in columns P1 and P4 as a function of the fire duration. For the sake of simplicity, only fire scenario 2 is considered. It is clear that, compared to model 1 (= constant stiffness and constant coefficient of thermal dilation during the whole fire duration) from the preliminary analysis, the axial forces exhibit a significantly smaller increase. For example, taking as a reference the maximum axial force in column P1 obtained from model 1 ($N_{fire,max} = 3.16 \cdot N_{20}$), the peak axial force (at approximately 90 minutes of fire duration) is reduced by approximately 23% in model 3, and by round 50% for model 4. Moreover, the differences between the two columns are significantly smaller in model 4, as should be expected, since the higher the stiffness decay (i.e. the lower the stiffness), the lower the indirect actions induced by the dilation. However, it is worth noting that the increase of the axial force in columns P1 and P4 is still rather high, i.e. $N_{fire,max} = 1.50 \cdot N_{20}$ (on the average) in model 3, and $N_{fire,max} = 2.30 \cdot N_{20}$ in model 4. Such an increase of the axial load holds a potential likelihood for punching failure, as it reaches the experimentally measured punching resistance (1.8-2.2 times N_{20}).

Figures 17a and 17b illustrate the comparison in terms of fire scenario, for columns P1 and P4 separately. For the same fire scenario (i.e. continuous, dashed or dash-dotted lines), the higher curve refers to model 3 and the lower to model 4. The general trend which results from the two figures is that with model 3 there are no major differences in terms of peak axial force in the two columns, On the contrary, model 4 makes the slab much more sensitive to the initial localized effects due to car ignition, as is visible by the small peaks in the curves. Even in this case, with a significant reduction of the bending stiffness and the related decrease of indirect actions, there is a significant increase of the axial loads on columns P1 and P4, and thus of the punching action on the slab. In more detail, column P1 experiences an axial load in fire $N_{fire,max} \approx 2.50N_{20}$ in model 3, with the three fire scenarios yielding very close results, whereas in model 4 the increase of the axial load ranges from 60% (fire scenario 1 and 2) to 90% (fire scenario 3a). As for column P4, $N_{fire,max}/N_{20}$ ranges from 2.10 (fire scenario 2) to 2.40 (fire scenario 3a) in model 3, whereas in model 4 the increase of the axial load ranges from 45% (fire scenario 2) to 95% (fire scenario 3a). Clearly, such axial load increases would still be close, at least in the worst cases, to the punching resistance found from experiments [3].

Finally, when extrapolating previous findings of the author [5], it is worth noting that the general trends observed in the structural analysis would be confirmed when using a more refined material model. However, the overall stiffnes of the structure would further reduce when taking into account concrete cracking/ crushing and steel yielding. In this way, the structure is less sensitive to the development of indirect actions.

4.3 Considerations on the possible failure mechanisms

The analyses shown in the previous sections clearly highlight that the dilations ensuing from fire result in a significant increase of the axial forces on the columns. As previously explained (see section 2.4), this increase is accompanied by a redistribution of the internal forces. Due to this redistribution an increase of the bending moments at the supports (negative moment) is introduced, accompanied by a decrease of the bending moments at midspan (positive moment). Going more into detail, with reference to Figure 18, it can be assumed that the structure will undergo a significant redistribution of the bending moments along the x-direction, for two main reasons: (a) most of the load is transferred to the supports through bending in the x-direction; and (b) the fire is localized mainly in the left lane of the car park.

The redistribution of the bending moments in the x-direction is limited by the possibility of a structural failure due to bending. Considering once again that the left lane of the structure is most affected by fire, this has a two-fold effect: (a) a reduction of the flexural capacity at mid span (positive bending moment; due to the heating of the tensile reinforcement) and (b) increase of the moment at the support (negative bending moment). Thus, a possible (and likely) failure mechanism is shown in Figure 18 where the positive yield lines are plotted as continuous lines and the negative yield lines as dashed lines. The bottom of Figure 18 shows a likely sequence of formation of the yield lines at section A-A. The attainment of this failure mechanism and the consequent redistribution of the bending moments in the x-direction, limits the increase of the axial force in the columns, because when the failure mechanism is attained, no further load redistribution is possible. It is worth noting that even though the strips running in the x-direction above the columns are more reinforced than the slab spans, a failure mechanism with yield lines in the x-direction is to be considered unlikely because (a) of the dominant structural behavior of the structure under consideration (namely bending in the x-direction); and (b) of the elevated number of yield lines such a collapse pattern would imply. As a matter of fact, the failure mechanisms characterized by few yield lines (i.e. “simpler” collapse mechanisms) aregenerally characterized by lower ultimate loads.

Considering a slab strip in the x-direction, centered on one of the columns P2, P3 or P4 and 4.50 m wide, it can be studied as a one way carrying element (see bottom of Figure 18). The load acting on the column at incipient collapse can be established by enforcing equilibrium through the principle of virtual works, both for the part involved in the collapse mechanism (the left lane) and the remaining part of the structure. Except for the negative yield line running along columns P1-P5, the remaining part of the structure can still be considered to be in the elastic range.

If the two spans in the x-direction are indicated as L1 (= 4.50 m) and L2 (= 8.00 m), the width of the slab strip in the y-direction as L_y (= 4.50 m) and the total load as p (= 17.00 kN/m^2), the total force acting on the column is given by Equation 1:

$$N_{P2} = N_{P3} = N_{P4} = [p\cdot(L_1/2 + L_2/2) + M_{pl}^-/L_1 + M_{pl}^-/L_2 - M_{el}/L_2]\cdot L_y \qquad (1)$$

where M_{pl}^- and M_{el} are the average negative plastic moment per unit length along the negative yield line and the average negative moment per unit length over the second support, respectively, on the length L_y. Note that the average positive plastic moment, as well as the position of the positive yield line, have no influence on the load acting on the column. Moreover, whether the positive yield line can develop or not depends on structural parameters such as span length, applied load, …. A significant role is played, on the contrary, by the average top reinforcement. Assuming the values given in Figure 18, the average reinforcement ratio is

$$\rho_{av} = 0.45\text{-}0.55\%$$

and the corresponding plastic moment (assuming f_{yk} = 500 MPa, total depth h = 0.28 m, gross cover = 0.04 m, effective depth d = 0.24 m, z = 0.9d and L_y = 4.50 m) is

$$M_{pl-} = 570\text{-}620 \text{ kNm}$$

Note that, following the tabulated data of EN 1992-1-2 (section 5.7), one-way slabs with a gross cover (= axis distance) of 40 mm, and a total depth of 280 mm (≥ 120 mm) have a fire endurance REI = 120 minutes.

By applying Equation (1), the total load acting on the column is determined as

$$N_{tot} = 690\text{-}725 \text{ kN}$$

Therefore, the increase of the axial load on the column with respect to ambient temperature (= 580 kN, see also Figures 15-17) is as high as 19-25%. However, higher values (≈ 40%) can be obtained if the average reinforcement ratio is increased to 0.75%.

4.4 Discussion

The preliminary analyses carried out under different fire scenarios clearly show that a significant increase of the axial forces acting on the columns (and therefore of the punching action on the slab) is to be expected, even if the stiffness of the structure is significantly reduced (see model 4 in Figures 17a and 17b). This increase is not unlimited, but stops when the distribution of plastic zones is such that a flexural failure mechanism takes place. In the case under consideration, the most likely yield line pattern at collapse is the one ensuing from the (prevailing) bending in the x-direction. Depending on the reinforcement ratio for negative bending, the increase of the axial force could range from 20 to 40%. Of course, to make sure that the failure mechanism in bending can be attained, and thus that the collapse is ductile, possible brittle mechanisms (such as a punching failure) should be avoided.

5 Conclusions

- Review of statistical studies reveals that generally less than 4 cars are involved in car park fires, however, cases of up to 5-7 cars and even 51 cars are reported.
- A fire in an underground car park is simulated with a CFD model. Six cars are involved in the fire. In total 3 fire scenarios are studied with differences in the sequence and time of fire spread between two adjacent cars, and the adopted HRR curve. The maximum temperature on the slab-column connections is about 1100°C and is reached for every column, and each fire scenario. For fire scenario 3a, all temperature peaks are reached between 20 and 60 minutes of heating, whereas for fire scenario 1, due to subsequent heating the peaks are pronounced and not grouped.
- The structural impact of the assumed fire scenarios is studied with 4 different models for the modulus of elasticity. In all cases, an axial load increase can be found that is close, at least in the worst case, to the punching resistance found from experiments. Therefore, punching

failure during fire should be thoroughly studied when designing flat slab-column connections.

- The most affected flat slab-column connections are P1 and P4, which correspond to two-sided heated columns instead of 3-sided for the other columns.
- From the structural analysis, it is found that the studied fire scenario has an important impact on the increase of the axial load. The worst case is observed when the temperature peaks at the column tops are grouped together (fire scenario 3a), e.g. the middle car is first heated from where the fire spreads to the neighboring cars.
- The model of the Young's modulus is of major importance. As could be expected, the highest increase of the axial load is found when the modulus of elasticity is kept constant to its initial value. Introduction of a decreased shear modulus equal to 1/8th of the elasticity modulus is of limited influence, since only a small drop of the axial load is found. On the other hand, significant lower values are found when reducing the modulus of elasticity with temperature, or when using a constant value of 0.75 times the initial value. In the latter case, the slab is much more sensitive to the initial localized effects due to car ignition. It is noticed that even with such a significant reduction of the bending stiffness and related decrease of indirect actions, an important increase of the punching action on the slab is found.

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