UNPROTECTED STEEL IN MULTI-STOREY CAR PARKS
15 min Fire Resistance of Unprotected Steel and Composite Members in a Multi-Storey Car Park System

Prof. Peter Schaumann, Thomas Kirsch

Leibniz University Hannover, Institute for Steel Construction, Hannover, Germany

INTRODUCTION

Visible steel structures are aesthetically appealing and cost effective. These advantages can often not be used because steel constructions must be protected for reasons of the structural fire safety. Due to numerous examinations no fire resistance of the support structure is required in Germany for open car parks. For the reason of high ventilation and low fire load densities even the low fire resistance of unprotected steel is adequate.

Fig. 1: Open multi-storey car park (©GOLDBECK)

Thus, unprotected steel is used by a number of providers of multi-storey car parks in Germany during the last decade (Fig. 1). In some European countries, for example the Czech Republic, the fire resistance level R15 for open car parks is required. The objective of the investigation presented in this paper is to check whether the steel or the composite construction fulfils this requirement. For that purpose the authors calculated the fire resistance time of several steel columns and composite beams applying different structural fire design methods according to the Eurocodes [1, 2, 3].

1 FIRE RESISTANCE REQUIREMENTS IN EUROPEAN OPEN CAR PARKS

A survey of the fire resistance requirements in open car parks in Europe is provided in [4]. It becomes obvious that there is a wide scatter of fire resistance times required in different countries. For example in France there is no differentiation made between open and closed car parks or between car parks above or below ground. So the required fire resistance class is R90 in general. If advanced performance based design methods are applied the requirements to steel members may be reduced. In Finland R60 is required. Other countries for example Great Britain and the Czech Republic accept a shorter fire resistance time of 15 min (R15) for car parks above ground with sufficient ventilation openings (cf. Table G1, [5]).
The use of unprotected steel columns and composite beams in German open car parks is applicable because the building regulations do not require any fire resistance. These boundary conditions have been the reason that in Germany the steel/composite construction has become the standard construction method for open car parks. The company GOLDBECK provided the structural data of its system GOBACAR for the investigations dealt with in this paper. These constructions are designed for room temperature conditions according to the European steel

and composite design codes. The question was whether these unprotected steel members are able to withstand ISO standard fire exposure for short duration of 15 min.

2 CALCULATION METHODS

2.1 Simple Calculation Method

According to the Eurocodes there are two possible simplified assessment methods to calculate the fire resistance of steel and composite members. The first method is based on the critical temperature. The second method bases on the load bearing capacity at elevated temperatures. Due to possible stability problems especially for columns the second method was used for this investigation. For the steel columns the method of EC3 part 4.2.3.5 [2] and for the composite beams the method of EC4 annex E [3] was used, respectively.

The temperature of the members was calculated by a formula of Eurocode 3 part 4.2.5.1 [2] using heat transfer conditions according to EC1 part 3.1 [1]. As some input parameters are temperature dependent, the calculation has to be carried out incrementally.

Schaumann developed an approximation that allows an explicit algebraic formulation of the heating curves. These formulae are very helpful because they decrease the computing time compared to the incremental method. The limitation of the area of validity has to be considered (cf. Table 1).

Table 1: Approximation of temperature increase in an unprotected steel section

$\theta_{a,t} = \dfrac{c_1 \cdot c_2 + c_3 \cdot t^{c_4}}{c_2 + t^{c_4}}$		
$c_1 = \theta_0 = 20°C$	$c_3 = \dfrac{9875}{0.486 + 1.89 \cdot \ln\left(k_{sh} \dfrac{A_m}{V}\right)}$	
$c_2 = 11890 \cdot \left(k_{sh} \dfrac{A_m}{V}\right)^{-1.12}$	$c_4 = 1.253 + 0.071 \cdot \ln\left(k_{sh} \dfrac{A_m}{V}\right)$	
$\left(A_m/V\right)$ [m^{-1}]	Area exposed to fire divided by volume	Min: 25 m^{-1} Max: 300 m^{-1}
t [min]	Time	Max: 30 min
$\Theta_{a,t}$ [°C]	Steel temperature	Max: 700°C

2.2 Advanced Calculation Method

The more sophisticated and presumably more exact way to achieve the fire resistance time for steel and composite members is the advanced calculation method. At the Institute for Steel Construction the software BoFire is used for this task in most cases. BoFire is a finite-element software based on work done by Schaumann [6] in 1984. It allows simulating the load bearing behaviour of two dimensional bars and frames of steel, concrete and composite constructions. Therefore BoFire is a "Level-3"-method concerning EC3 [2] and EC4 [3]. In 2001 the software was modified by Upmeyer [7] to implement actual material properties concerning the Eurocodes and to give a possibility to use design fires instead of the ISO standard fire.

3 CALCULATION OF FIRE RESISTANCE OF COMPOSITE BEAMS

The study included two different composite beams and twelve columns belonging to the GOBACAR system. The members of the construction system and the applied loads were taken out of the static calculation for a completed car park in Dresden. The two beams consist of a steel section and a concrete slab with a thickness of 103 mm and an effective width of 2500 mm. According to Fig. 2 the first section comprises a hot rolled beam (IPE400a) while the second comprises a welded beam. The slab is connected to the steel section by headed studs. The materials of both composite beams are equal. Steel grade S355J2G3 and concrete grade C35/45 are used. The headed studs have a maximum tensile strength of 450 N/mm².

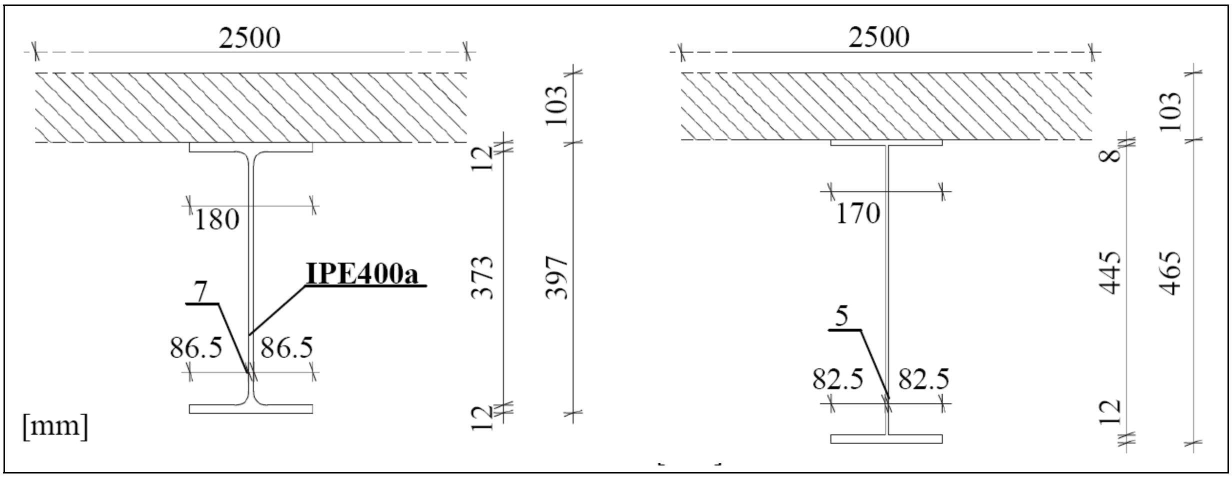

Fig. 2: Dimensions of composite beam with IPE400a profile (left) and welded profile (right)

According to the load, the assumption of the characteristic dead load is obligatory. The live load is normally multiplied by the combination factor Ψ_2 in case of fire. For traffic areas a value of Ψ_2=0.3 is taken for this in general. Alternatively a more realistic approach of applying the characteristic values of the live load limited to the parking bay area was conducted. In this approach live loads on the lane between the slots were not taken into account. The resulting bending moments are nearly identical. For the further investigation the bending moment calculated by the realistic approach was used. This led to a total design moment of 315 kNm for the hot rolled composite beam (IPE400a) and due to less self weight 311 kNm for the welded one. For the reason that the assembly of the car park is symmetric, the beams can be used for most parts of a storey. Only a few beams in the ramp area are shorter and for that reason not relevant. The load on the beams is also practically equal for each storey. So it was possible to calculate just one beam exemplarily for all beams in the car park.

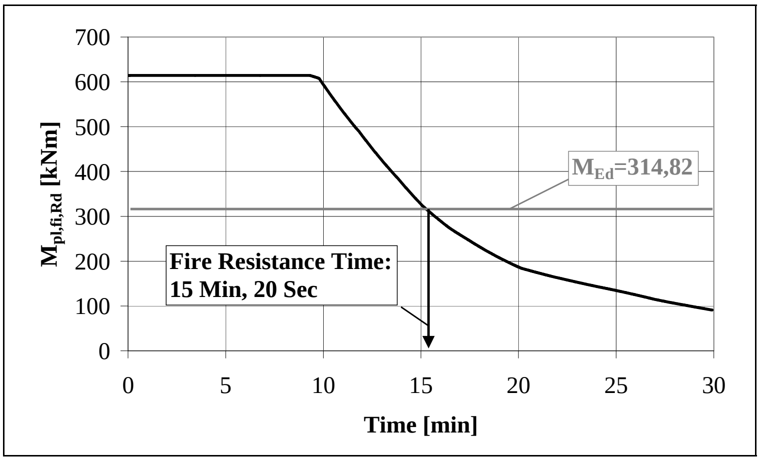

Fig. 3: Time dependent plastic moment capacity of composite beam with hot rolled section IPE400a

As described before, the temperature was calculated using the incremental method according to EC3 part 4.2.5.1 [2]. The new explicit formulae for calculation of the steel temperature were also used and verified. As the next step the procedure concerning EC4 annex E [3] was used. So the cross sectional area of any part was multiplied by the yield stress decreased by the reduction factor for steel members $k_{y,\Theta}$ concerning Table 3.2 of EC4 [3]. In this way a resulting force was found for the web and both flanges. By calculating the dependent compressive force of the concrete slab and the inner lever arm, the plastic bending moment was found for every increment. That way the exact time when the plastic moment decreased to a value less than the applied moment of 315 kNm was determined to 15 min and 20 sec (cf. Fig. 3). The plastic moment at 15 min is 328 kNm. So the fire resistance class R15 is reached by the composite beam with the IPE400a section.

Because the flanges and the web of the welded profile are thinner than the parts of the IPE400a profile, the heating of the member is faster. This leads to a fire resistance time of only 14 min and 5 sec. Because of this result advanced calculation methods were applied.

The thermal material properties, the heat transfer coefficient α=25 W/m²K and the emissivity coefficient ε_r=0.8 were taken from the Eurocodes. The configuration factor Φ was set to 1.0 according to EC1 [1]. The plastic moment capacity after 15 min calculated by BoFire is 251 kNm. This is less than the needed plastic moment of 311 kNm and even less than the plastic moment of 272 kNm calculated by the simple calculation method of EC4 [3]. Reason for the difference is a difference in the steel temperatures. The shadow factor k_{sh} taken into account in the simplified calculation method causes lower steel temperatures than the heat transfer assuming a configuration factor Φ = 1.0 in the advanced calculation method. This conflicts with EC4-1-2 cl.4.1(3) which reads: “Tabulated data and simple calculation models should give conservative results compared to relevant tests or advanced calculation models.” The shadow factor k_{sh} has been established to consider the fact that not every part of the open section is equally and directly exposed to the fire. Concerning advanced calculation methods, there is a possibility to reduce the heat flux into the member by reducing the configuration factor Φ, but there is no default value given in the Eurocodes for this purpose. Improvement may be based on experimentally checked configuration factors Φ < 1.0. For the time being the fire resistance time calculated by advanced calculation methods is less than calculated by the simple method in this case.

4 CALCULATION OF FIRE RESISTANCE OF STEEL COLUMNS

In contrast to the beams there are differences in the load and thus in the cross section of the columns for the different storeys. In higher car parks the columns of the ground storey need to have a higher load bearing capacity. There are also four different types of columns in any storey. For this reason at least twelve types of columns were analysed including car parks with 2, 4 and 6 storeys. The most interesting four columns are shown in Table 2.

Table 2. Column types for different positions and loads

Number of storeys	Position of column	Normal force [kN]		Bending moment[kNm]		Profile
		at 20°C	in case of fire	at 20°C	in case of fire	
2	Edge column	311	202	9	6	HE-140A
	Internal column	614	399	7	5	HE-180A
6	Edge column	932	606	9	6	HE-220A
	Internal column	1841	1197	7	5	HE-280A

The columns with highest load utilisation ratios are the internal columns while the columns exposed to the maximum bending moment are the edge columns. The two other types of columns are located in the ramp area and there number is lower.

The buckling length of all columns for calculation at 20°C is 2.7 m. This is equal to the height of every storey. In case of fire the static system changes for the ground storey to 0.7 times of the length at 20°C. This leads to a buckling length of 1.89 m.
The constant temperature field of the columns was calculated by the simple method of EC3 [2]. The calculation of the fire resistance was carried out by comparing the load bearing capacity to the applied load using the calculation method of EC3 part 4.2.3.5. The method implies the lateral and the lateral torsional buckling by decreasing the resistance against normal forces. The temperature dependent load capacity was determined for every timestep. So in Table 3 the time is given at which the load capacity becomes less than the applied load. This time is defined as the fire resistance time.

Table 3. Calculated fire resistance time and required profiles (R15) for different column types

Number of storeys	Position of column	Used profile	Calculated fire resistance time	Required profile (R15)	Calculated fire resistance time
2	Edge column	HE-140A	10:01	HE-200A	15:50
	Internal column	HE-180A	11:15	HE-220A	15:00
6	Edge column	HE-220A	12:33	HE-260A	15:45
	Internal column	HE-280A	13:03	HE-320A	15:53

The fire resistance time for all actual used profiles is less than 15 min. This is caused by the high load utilisation factor of nearly 1.0 calculated at 20°C. The advanced calculation by BoFire leads to a shorter fire resistance time than the calculation by the simple method of EC3 [2]. This is caused by the use of the shadow factor k_{sh}, again.
The minimum required profile to reach the aim of R15 is also given in Table 3. For all columns there are profiles needed which are minimum two classes bigger then the already used. This leads to the assumption that it is cheaper to use fire protection materials instead of increasing the size of the steel columns. However, it is possible to use unprotected steel columns for car parks if the fire resistance class R15 is required.

5 CONCLUSIONS

The calculation of the fire resistance time by the simple and advanced calculation methods showed that the more sophisticated method leads to a more conservative solution in this case. This is presumably caused by the shadow factor k_{sh}. This factor is not applicable to advanced calculation methods. The average decrease of the steel temperatures caused by the shadow factor is determined to 95% for the columns (k_{sh}=0.85) and 93% for the composite beams (k_{sh}=0.7). It is obvious that a calculation with advanced methods can not compensate this reduction of temperatures unless taking into account this shadow effect. So the calculated fire resistance time was not increased for the analysed members.
The exact calculation of the fire resistance time of the analysed members by simple calculation methods according to the Eurocodes showed that it is possible to meet the fire resistance class of R15 for composite and steel members. To reach this aim it is necessary that the steel part of the members has either a minimum thickness or a maximum load utilisation factor. It is shown that the investigated hot rolled IPE beam can be used without any fire protection for car parks with a required fire resistance class of R15. In contrast the welded beam with thinner web and flanges is not reaching the required fire resistance class. Concerning the columns the load utilisation factor at 20°C is very high for economic reasons and the fire resistance time is less than 15 min. The class R15 can be reached by using profiles with an increased cross sectional area (overdesign).

6 SUMMARY AND ACKNOWLEDGEMENT

The standard construction method for multi storey car parks in Germany is the steel and composite construction. This is caused by the fact that no structural fire resistance is required according to the building regulations for open car parks. In some other European countries, for example the Czech Republic, a fire resistance class of R15 is obligatory. This leads to the question whether the steel structure of German car park systems withstands this short fire exposure. So the fire resistance time of two composite beams and twelve steel columns was calculated in this investigation.

The calculation of the steel temperature was conducted by the incremental simple method of Eurocode 3 and by an approximated explicit formulation. It has become obvious that the approximated formulae are adequate to calculate the steel temperature in its application range.

Because the required fire resistance class was not reached in any case, the advanced calculation method BoFire was applied. The fire resistance time calculated by the advanced calculation method was found to be more conservative than calculated by the simple method in this case. Reason for this is the shadow factor k_{sh} in the simplified calculation method. An adequate reduction of the heat transfer is not regulated for practical use when advanced calculation methods are applied. So the calculated fire resistance was not increased for the analysed members.

The calculation of the fire resistance time by simple calculation methods according to the Eurocodes showed that it is possible to gain a fire resistance time of 15 min for composite and steel members. One of the investigated composite beams reached the class R15, while the second failed for the reason of a thinner web and flanges.

The load utilisation factor of the analysed columns at 20°C is very high for economic reasons. Thus the fire resistance time was determined to less than 15 min. It is also shown that it is possible to reach the class R15 by increasing the cross sectional area of the columns (overdesign).

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