DESIGN CONSIDERATIONS FOR SHEAR FAILURE OF FLAT CONCRETE SLABS EXPOSED TO FIRE

Emmanuel Annerel[a,b] and Luc Taerwe[a]

[a] Ghent University, Faculty of Engineering and Architecture, Department of Structural Engineering, Magnel Laboratory for Concrete Research, Technologiepark Zwijnaarde 904, B-9052, Ghent, Belgium; emmanuel.annerel@ugent.be, luc.taerwe@ugent.be
[b] Promat Research and Technology Centre, Bormstraat 24, B-2830, Tisselt, Belgium; e.annerel@prtc.be

Abstract

Underground structures often consist of concrete flat slabs connected to concrete columns. This type of structure is sensitive to shear failure, hence precautions are taken in the design guidelines for ambient condition (such as the Eurocodes) to avoid this brittle failure.
Nevertheless, in the literature some collapses can be found from such type of structures when exposed to fire. Mainly two reasons can be considered. Firstly, the load bearing capacity of the structure is reduced due to losses of strength and stiffness of the constituent concrete and steel materials. This reduction is also highly influenced by possible explosive concrete spalling during fire, by which the concrete section is locally reduced. Secondly, the punching load increases due to the effect of restraint of thermal deformations. As the slab is heated form one side, it is willing to expand and curve towards the fire. Due to its fixed position at the column tops, stresses of restraint actions are introduced in the slab-column connection.
To quantify the risk of punching failure during fire exposure and in order to provide design guidelines, an extensive research was conducted at Magnel Laboratory for Concrete Research at Ghent University, from which the main results are presented in this paper.

Keywords: Eurocodes, Fire, Flat Concrete Slabs, Punching Shear, Spalling, Simplified Model

1 Introduction

Underground structures, such as car parks, are often constructed as flat slabs supported by a grid of columns. These types of structures are sensitive to punching failure, since the slabs have a fairly small thickness and the bending moments and shear forces are locally high due to the point support by the columns. When the load increases, bending cracks will appear first at the top side of the slab, which start from the centre and gradually extend with the load in radial direction. These bending moments at the supports are resisted by the top reinforcement in the tensile zone and by the concrete near the column face, as well as the bottom reinforcement in the compression zone. On the other hand, the actual punching failure occurs as a result of the development of tangential cracking in the concrete slab, starting from the column face and spreading to the top of the slab. In this way, a punching cone is developed. To improve the punching resistance of the slab, specific shear reinforcement can be provided within the critical perimeter to resist these developed shear stresses. To avoid this brittle failure at ambient temperatures, building codes (such as the Eurocodes – EN1992-1-1) provide design guidelines which are based on simplified calculation models.

According to the Eurocodes, the design for the punching resistance at ambient conditions should take into account safety factors on the material and the load. On the other hand, in case of fire, those material and load safety factors are decreased. This approach results in a design resistance at the beginning of the fire which is larger than the design resistance at ambient conditions, whereas the load decreases to a value which is lower than at ambient condition. In this way, a degree of overdesign exists which is helpful in case the structure is exposed to fire and contributes to the rather adequate fire resistance of concrete structures.

Nevertheless, despite this extra margin, some concrete structures collapse partially or completely due to fire (Taerwe & al. 2008). Mainly two reasons can be considered. Firstly, the load bearing capacity of the structure is reduced due to losses of strength and stiffness of the constituent concrete and steel materials. This reduction is also highly influenced by possible explosive concrete spalling during fire, by which the concrete section is locally reduced. Secondly, the punching load increases due to the effect of restraint of thermal deformations. As the slab is heated from one side, it is willing to expand and curve towards the fire. Due to its fixed position at the column tops, stresses of restraint actions are introduced in the slab-column connection. The combination of both can result in collapses well before the calculated fire resistance according to e.g. the Eurocodes is attained. An example can be found in the case of Gretzenbach (Switzerland) on 27 November 2004, where the roof of an underground car park collapsed due to punching failure during fire. Investigations afterwards revealed design and execution mistakes resulting in an overload due to an excessive soil cover thickness and a decreased punching shear capacity.

The problem of punching shear failure of flat concrete slabs exposed to fire has been studied extensively at Magnel Laboratory for Concrete Research of Ghent University in 2010-2012. The study was part of a larger national and multidisciplinary project about fire safety of car parks, from which the results are published in a special issue of Fire Safety Journal (2013). This paper summarizes the results of the real scale punching tests, and translates the project outcomes towards three design considerations, namely a simplified calculation method, the importance of concrete spalling and the risk of premature failure.

2 Punching shear tests

2.1 Test slabs

A total of 6 real scale slabs are tested for punching failure, from which 2 at ambient and 4 at high temperatures. The dimensions of the slab are limited by the dimensions of the available floor furnace being 3 x 6 m, whereas they must also be representative for a real structure to reduce scale effects. The slabs measure 3.2 x 3.5 x 0.25 m and have a connected column stub with size 0.3 x 0.3 x 0.65 m. The concrete cover is 26 mm on the flexural slab reinforcement and 20 mm on the stirrups. According to Table 5.9 of EN 1992-1-2 for flat slabs, the slab without stirrups have a fire resistance of REI 90 minutes (axial distance is 32 mm). However, since for REI 120 an axial distance of the reinforcement of 35 mm is required, the actual fire resistance of the slabs will be close to REI 120. Also, the concrete cover complies with the regular requirements for durability.

The slab has reinforcement bars with a diameter of 12 mm, positioned in 2 orthogonal directions with a spacing of respectively 0.10 m at the top and 0.25 m at the bottom (see Figure 1). The column has 8 longitudinal rebars, from which 4 are located at the corners and the other 4 are positioned in between the corner bars. Stirrups with a diameter of 6 mm are used along the height of the column with 60 mm spacing.

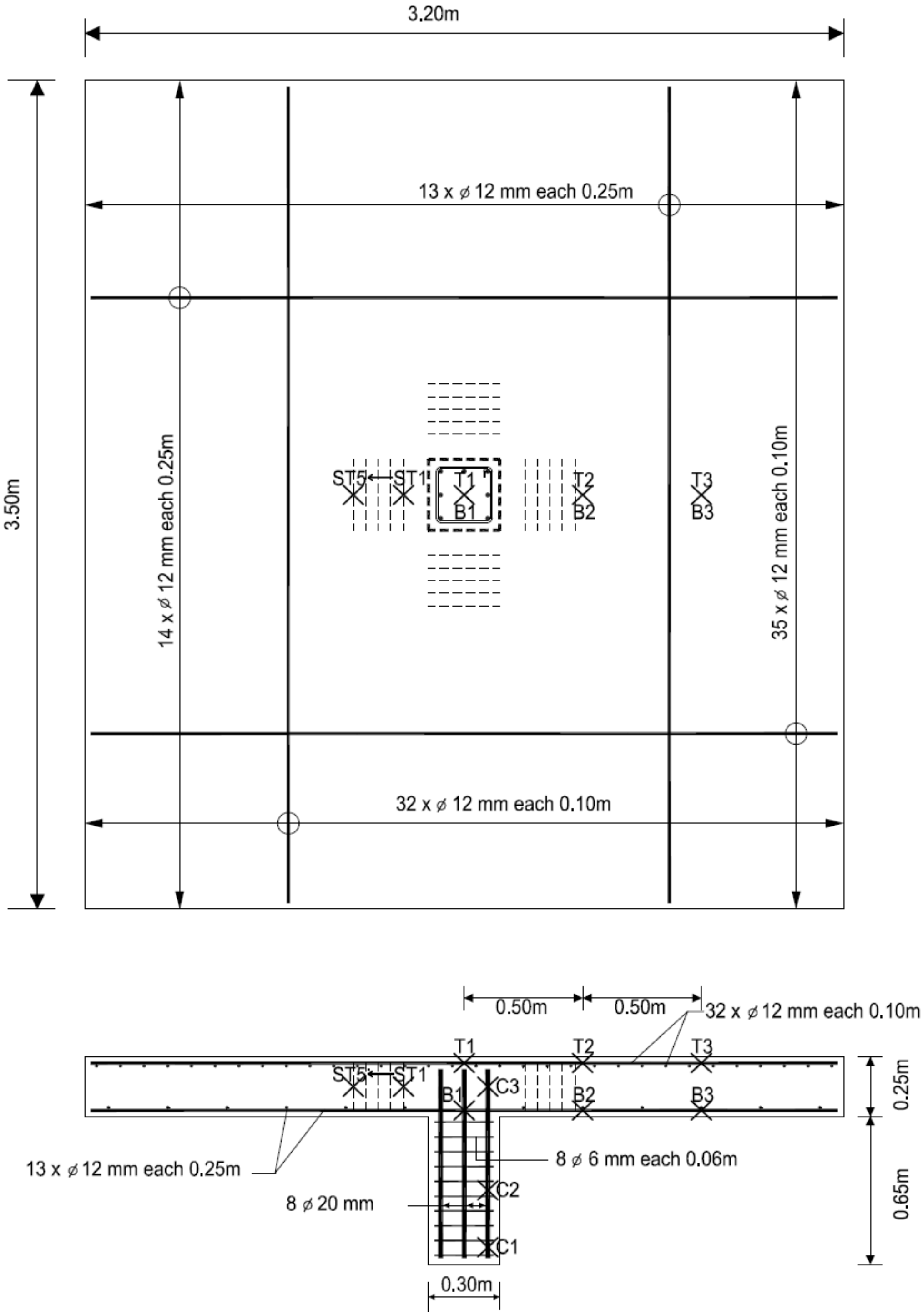

Fig. 1 Geometric characteristics of the test slabs and thermocouple locations

Four types of slabs are cast which differ in amount of shear reinforcement. The first slab has no shear reinforcement (slab S0), the other types have respectively 1 (S1), 2 (S2) and 5 (S5) stirrups at each of the four column sides. These stirrups consist of two U-shaped legs that are connected to the main reinforcement and have a diameter of 6 mm (see Figure 2). The first stirrup is in all cases positioned at 106 mm (0.5 x effective depth of the slab) from the column face. In case of S2, the second stirrup is positioned with a spacing of 106 mm. In case of S5, all the stirrups are placed within 318 mm (1.5 x effective depth of the slab), resulting in intermediate spacings of 53 mm. Figures 2 & 3 illustrate the position of the stirrups for slabs S1 and S5.

Fig. 2 Top view of shear reinforcement of slab S1

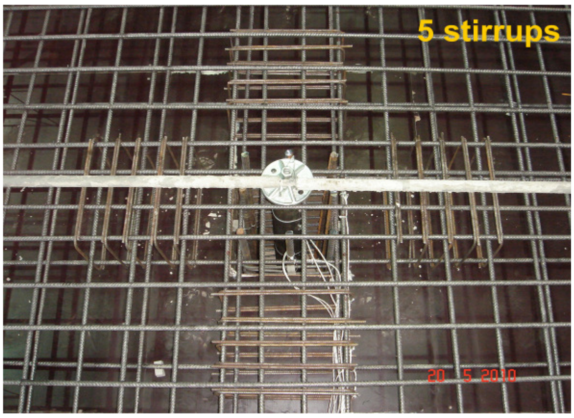

Fig. 3 Top view of shear reinforcement of slab S5

Both the flexural and shear reinforcement are calculated according to EN 1992-1-1 for a flat slab configuration supported by columns in a grid of 6 by 6 meters. From this grid the section around the column with dimensions of 3.2 x 3.5 m is cast and exposed to an ISO 834 fire curve.

A calcareous concrete with blast furnace slag cement is delivered by a ready mixed concrete plant (cylinder compressive strength at 28 days: 27.9 – 34.5 N/mm², slump 200-220 mm). The 6 slabs are cast in 3 phases: slabs S0 & S1 in casting phase 1, slabs S2 & S5 in casting phase 2 which all four are exposed to fire and slabs S0 & S5 in casting phase 3 which are tested at ambient temperature.

Thermocouples are embedded at different locations of the reinforcement (see crosses in Figure 1).

The moisture content at the time of the fire test is found from cubes which are sealed with aluminium tape at all faces except for one and stored in the neighbourhood of the slabs. For slabs S0 & S1 (casting phase 1) and S2 & S5 (casting phase 2), the moisture content is respectively 3.5% and 4%.

2.2 Test programme

Table 1 gives an overview of the test programme. Testing at ambient conditions consists of loading till failure in subsequent loading steps. Between two loading steps, the condition of the slab is inspected. Testing at fire conditions consists of first loading the slab till service load, followed by a heating regime of 120 minutes according to the ISO 834 fire curve under constant load, after which the slab is loaded till failure. The applied service load is derived from the ultimate design resistance calculated according to EN 1992-1-1.

Table 1
Test programme

Slab designation	Shear reinforcement		Test	Dead weight [kN/m²]	Variable load [kN/m²]	Service load [kN]
	Number of stirrups (each side of column)	Area [mm²]				
S0_20°C	0	0	Ambient	6.25	1.27	270*
S5_20°C	5	1130	Ambient	6.25	5.40	420
S0_fire	0	0	Fire	6.25	1.27	270*
S1_fire	1	226	Fire	6.25	3.09	336
S2_fire	2	452	Fire	6.25	4.91	402
S5_fire	5	1130	Fire	6.25	5.40	420

(*) Sevice load equals (6.25 + 1.27) x 6 x 6 kN/m²

2.3 Test setup

A special loading frame is constructed to allow the concrete slabs to be loaded and tested for punching failure at high temperatures. The difficulty is to have a loading system positioned on top of the slab, since it was not allowed to load from the furnace floor. Therefore, the slab and column have a central opening which allows to position a high strength steel bar (Dywidag) that is anchored at the lower side of the column stub and on top of a hollow hydraulic jack (capacity 1000 kN) which is supported by a series of steel beams (Figures 4 & 5). The load applied by the jack is transferred to the slab by 8 loading points, distributed symmetrically along the perimeter of a circle with a 1.3 m radius. To protect the steel bar and its anchorage from excessive heating during the fire test, the column was insulated by two layers of calcium silicate boards and mineral wool. During the fire tests, two slabs are simultaneously tested.

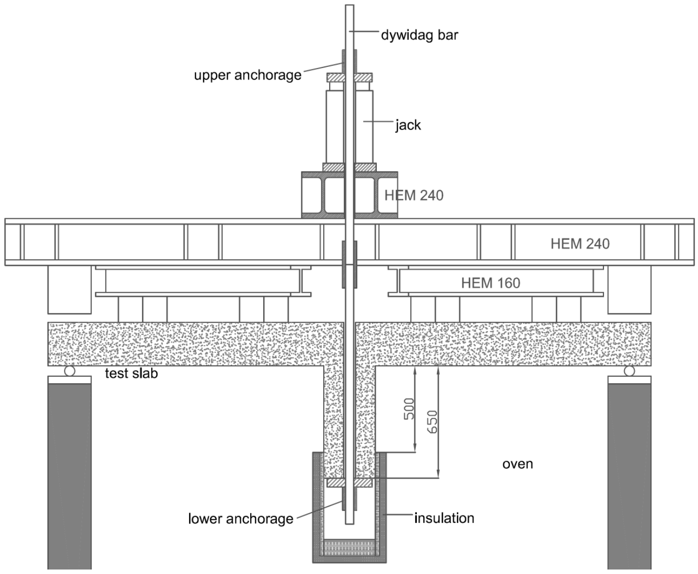

Fig. 4 Cross section of test setup

Fig. 5 Overview of the test setup at high temperatures

During the tests, the vertical displacements are measured by means of LVDT's along a line parallel to the longest span of the slab located in the middle of the shortest span. The radial strains (in the plane of the slab) are also measured by strain stirrups in 2 directions making an angle of 45° and are used to follow the development of cracks during the tests.

Because of limitations of the fire testing infrastructure, the supporting edges of the slabs are not restrained and only a short column stub of about 650 mm is provided. In this way, the fire tests take only into account the effect of the reduction of strength and stiffness of the constitutive materials on the punching resistance.

2.4 Test results at ambient temperature

One slab without stirrups (S0_20°C) and one slab with 5 stirrups (S5_20°C) are tested for punching shear at ambient temperatures. The slabs are loaded in subsequent loading steps of about 50 kN till failure. After each loading step, the slab is inspected for cracks which are indicated on the slab surface by a marker. The first visible cracks appear at the top of the slab at about 100 kN for the slab without stirrups (Figure 9), and at 250 kN for the slab with 5 stirrups at each column side. These cracks start from the centre of the slab and increase in size and number with increasing load. Those radial cracks are formed due to the bending of the slab over the column. Yielding of the longitudinal steel bars will influence this crack pattern. While those bending cracks increase in size and number, tangential cracks start from the column-to-slab connection and a punching cone starts to develop. Punching failure occurs when this cone is fully developed over the thickness of the slab and the cone is literally punched through the slab, as is illustrated in Figure 8.

During the loading test the slab is bending over the column. In this way, the slab is only supported at its four corners. Figure 6 shows the maximum vertical displacement of the slab with increasing load (situated at the centre of the slab). For a given load step, the deformations are larger for slab S5 than for S0. As is visible on the graph, the failure load of about 1000 kN is 2.5-3.7 times larger than the service load according to EN 1992-1-1.

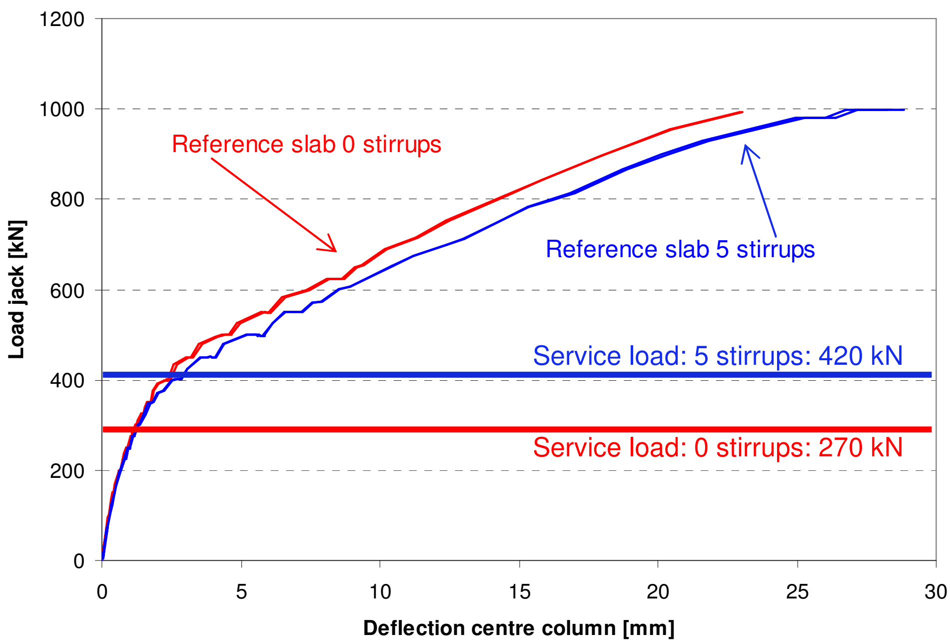

Fig. 6 Load-deflection curve of slabs S0 and S5 tested at ambient temperature

The test setup was designed for a maxium loading of 1000 kN, since this was the capacity of the largest hydraulic jack available that had a central hole necessary for the pulling bar. Unfortunately, slab S5 did not fail immediately at this maximum load of 1000 kN. Eventually it failed after having the maximum load of 1000 kN for 50 minutes. When extrapolating along the slope of the load-deflection curve, an ultimate punching load for S5 can be estimated to be about 1050-1100 kN. The exact failure load of S0 is 998 kN. For the slabs S1 and S2, the punching resistance is not experimentally determined, but could be assessed at about 1000 kN given the test results for S0 and S5.

2.5 Test results at high temperatures

Two fire tests were executed. Each time, two slabs were loaded till service load and heated according to ISO834 curve. This load and temperature curve was held for 120 minutes, after which - and in case the slab did not failed yet - the load was increased till punching failure to determine the remaining capacity.

The results are given in Table 2.

Table 2
Test results of fire tests

Slab	Service load (EN 1992-1-1) [kN]	V(20°C) [kN]	V(T) [kN]	V(T)/V(20°C) [-]
S0	270	998	588	0.59
S1	336	about 1000	627	0.63
S2	402	about 1000	-	-
S5	420	about 1050	-	-

The first column depicts the slab designation, the 2nd column the service load attained during the fire test, the 3rd column presents the results for the test slabs at ambient temperatures, whereas the 4th column presents the remaining punching resistance after 120 minutes ISO834. With respect to the punching strength at ambient condition, slabs S0 and S1 have a remaining punching capacity of about

60% after 120 minutes of ISO 834 fire curve (5th column).

Due to a technical failure in the second fire test, the exact load applied on slabs S2 and S5 is not known. However, based on the measured deflections during the loading period prior to heating (loading till service load), it is estimated that slab S2 was loaded with a factor of 1.5-2 times the service load, whereas slab S5 was only loaded for 50-150 kN. Because of this overload, slab S2 failed due to punching after 20 minutes of ISO 834 fire.

Figures 7 and 8 show the damage to the slabs after fire and loading till failure. A symmetrical punching cone is observed, as well as spalling of the column corners and the bottom of the slab. More details about the spalling damage is given in section 4.

Fig. 7 Punching cone after fire test of slab S1 (top view)

Fig. 8 Punching cone after fire test of slab S1(bottom view)

The thermocouple readings show that the top reinforcement of slabs S0, S1 and S5 stayed below 90°C. The temperature of the stirrups which is measured at mid height (about 125 mm from exposed surface) increased beyond 600°C for all slabs. Due to this high temperature, the tensile strength of the stirrups is expected to be reduced by approximately 50%. From the observations, it is clear that the temperature of the stirrups is influenced by the loss of concrete cover and cracking, putting the stirrups directly exposed to the fire.

In contradiction to the upward bending of the slabs for the tests at ambient conditions, the slabs were bending towards the fire, whereas an upward displacement of the corners (up to 20-30 mm) was observed.

3 Punching shear model under fire conditions

3.1 Punching shear model

The experimental data was used to verify a draft design model that takes into account the punching shear resistance at high temperatures. Details of this model, as well as a further verification by means of punching tests at fire conditions by Kordina (1993) is reported in Annerel, Lu & Taerwe (2013b), whereas in this paper a summary of the model is given.

In EN 1992-1-1 a simplified punching model is given to design concrete structures at ambient temperatures. This method is based on the assumption that the slab fails when inclined cracking starts near the column face and reach the top of the slab. Hence, a punching cone can be found for which an angle of 26.6° is adopted. The perimeter that is found as the intersection of this punching cone with the plane of the upper reinforcement is called basic control perimeter (u_1). This model is now taken as the basis for a refined zone method that allows to calculate the punching shear resistance of flat RC slabs - with and without shear reinforcement - at high temperatures, and can be found in Equation 1.

$$V_{\max} = \eta_a 0.18k(100\rho_1 f_{ck})^{1/3} u_1 d + 1.5(d / S_r) A_{sw} f_{yk,ef}(T_{h,ef}) \sin\alpha \tag{1}$$

where:

η_a is the average reduction factor of the punching shear strength of the slab. Firstly, the cross section of the slab is divided into small layers (thickness of 10 mm), for which the temperatures is computed at each time step of interest (thermal properties of EN1992-1-2 are taken, 0.7 emissivity for concrete, convection coëfficiënt of 25 at the heated surface). Secondly, the average reduction factor is calculated by the reduction factor of each layer $\eta(T_i)$: $\eta_a = \frac{1}{n}\sum_{i=1}^{n}\eta(T_i)$, where T_i is the temperature of layer i $(i = 1......n)$

k $= 1 + \sqrt{\frac{200}{d}} \leq 2$ with d in mm

ρ_l tensile reinforcement ratio of the slab [-]

f_{ck} characteristic compressive concrete strength [N/mm²]

u_1 basic control perimeter [mm]

d effective depth of the slab [mm]

S_r radial spacing of perimeters of shear reinforcement [mm]

A_{sw} area of one perimeter of shear reinforcement around the column [mm^2]

$f_{yk,ef}$ effective characteristic yield strength of the punching shear reinforcement [N/mm²]

$T_{h,ef}$ effective temperature of the stirrups [°C]

α angle between the shear reinforcement and the plane of the slab [°]

Compared to the Eurocode model for ambient conditions, the following adjustments are done to obtain the model at high temperatures:

- A global reduction factor η_a is introduced in $V_{Rd,c}$. It is decided not to reduce the compressive strength itself, because it is not known if the shear strength relation as function of compressive strength also exists at high temperatures. The decrease of $V_{Rd,c}$ as function of temperature, is taken according to the tensile strength loss curve as given in EN 1992-1-2.
- The factor 0.75 of $V_{Rd,cs}$ is dropped.
- The partial safety factors (γ_s and γ_c) are taken as 1 according to EN 1992-1-2
- $\sigma_{cp} = 0$ (In this project, as there is no compression stress from the lateral directions of the slab)
- The decrease of the yield strength of the stirrups with temperature is implemented by $f_{y,ef}(T_{h,ef})$. The temperature $T_{h,ef}$ is taken at the concrete depth given by the effective tension area according to EN1992-1-2. The effective tension area is that part of the cross section which is in tension just before development of the first crack. In EN 1992-1-1, equations are given to calculate the depth of the effective area, resulting for this project in h_{ef} = 80 mm.

3.2 Calculation results

Figures 9 and 10 show the loss of punching resistance upon heating, given as the ratio of the punching strength at high temperatures V(T) and at ambient temperature V(20°C).

For the ambient condition, the punching resistances calculated according to EN1992-1-1 for slabs S0, S1, S2 and S5 are respectively, 747, 816, 885 and 1090 kN. In this calculation, the used concrete compressive strength is 35 N/mm², the concrete Young's modulus is 31000 N/mm² and the yield strength of the flexural and shear reinforcement is 500 N/mm². It is noted that a good agreement is found with the experimental value for slab S5, whereas for slab S0 the calculated punching strength is about 250 kN lower than the experimental value.

Figure 9 shows the punching strength loss in case no spalling is considered. After 120 minutes of ISO 834 fire, a punching strength loss of up to 20-30% is found. These losses are smaller than the experimental value of about 40% (found for slabs S0 and S1).

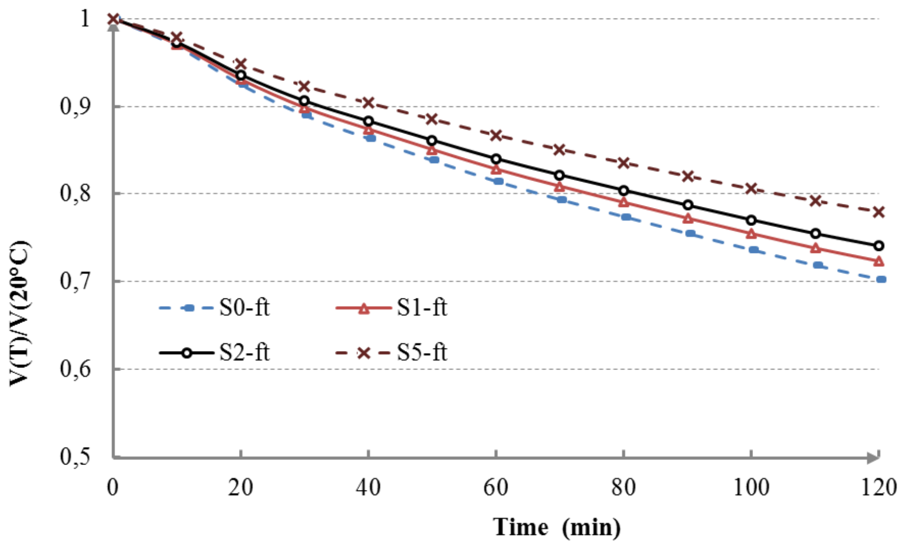

Fig. 9 Evolution of the punching shear resistance upon heating

To obtain a better agreement with the fire test results, the experimentally observed spalling of the concrete cover of the bottom reinforcement is introduced in the calculation as a gradual loss of the outer layers. This spalling is adopted as the gradual removal of the concrete layers that exceed a temperature of 350°C from the thermal and mechanical calculation. This spalling criterion was proposed by Kodur & Dwakat (2008) on the basis of a number of tests. This process is continued till the total concrete cover of 25 mm is removed. Figure 10 shows the calculation results, from which the influence of the loss of the bottom concrete cover on the punching shear resistance is clear. An additional punching shear loss due to spalling of about 10-15% is found. At 120 minutes, the punching strength loss reaches about 40% for the slabs with 0 and 1 stirrups at each side of the column, which is in good agreement with the fire tests.

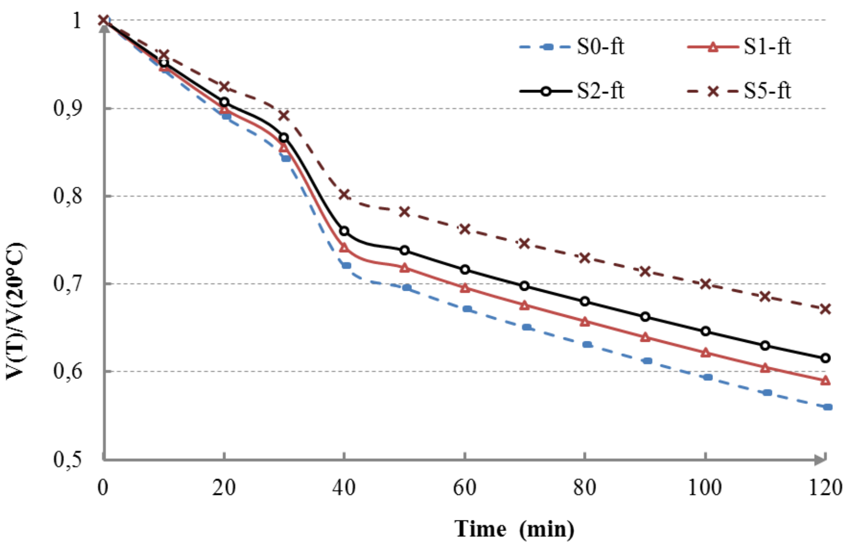

Fig. 10 Evolution of the punching shear resistance upon heating when spalling is considered

4 Spalling

Concrete spalling occurred at the column corners near the slab surface (see figure 8). Furthermore, a total loss of the concrete cover of the bottom reinforcement of the slabs was found in a region of about 800-1000 mm surrounding the column, resulting in naked bottom reinforcement and stirrups. Closer to the column stub, maximum spalling depths of up to 100 mm were measured. This spalling damage was observed for all slabs tested for fire. Figure 11 illustrates the typical spalling behaviour, as observed for slab S0.

Fig. 11 Spalling damage of slab S0

Contributions to the observed spalling, could be found in the initial moisture content of the concrete at the start of the fire of about 3.5-4.0%, as well as the expected high compressive stresses near the concrete surface. Those compressive stresses are a result of both the mechanical loading to service load at the beginning of the fire test, and the restraint of the thermal expansion of the outer heated layers by the inner less heated layers.

From the fire test observations and the simplified calculation model, it is clear that spalling of the bottom concrete cover contributes to a large deal to the loss of punching shear resistance during fire. To avoid structural problems in flat slab-column connections, appropriate measures for reducing spalling should be adopted. It was not investigated which measures would be sufficient, either polypropylene fibers or additional reinforcement, or passive fire protection.

5 Calculation safety with respect to effect of restraint actions

In the introduction it was explained that the Eurocode will generally take a decrease of the load into account in the case of the accidental (fire) condition. In this section, it is illustrated that due to restraint actions of thermal deformations of the heated elements, the load can increase, resulting in potential risk for premature failure.

5.1 Increased punching load upon fire

Firstly, the load can increase due to restraint of the thermal deformation of the concrete slab. Because of one sided heating and the low conductivity of concrete, a thermal gradient will appear over the slab thickness. Consequently, the outer concrete layers will expand more than the inner layers, which will result in a thermal curvature towards the fire. At the column-slab connection, these rotations are restrained, resulting in an increased supporting moment, and thus also an increase of the punching shear load (illustrated in figure 12).

The effect of the restraint of the thermal curvature of slabs is studied in Annerel & al. (2013). A finite element analysis is performed on a typical underground parking structure consisting of a flat slab supported by two rows of square columns and four perimeter walls. A reasonable structural slab slenderness of 23 is considered. A fire scenario of subsequently burning of 6 cars is used, resulting in maximum temperatures of about 1150°C and a total fire duration of close to 120 minutes. Only the slab is modelled, whereas the column tops are considered as fixed supports in vertical direction. In case a temperature dependent elastic modulus is used, an increase of the axial load at high temperatures of up to 2.3 times the axial load at ambient temperatures is found for the worst loaded columns. The model does not consider concrete cracking and rebar yielding which could reduce the restraining effect. Also, Kordina (1993) reports for a specific fire scenario an increase of the column load to a value of between 1.15 and 1.70 times the axial load at ambient condition, depending on the location in the grid of 6 by 6 m.

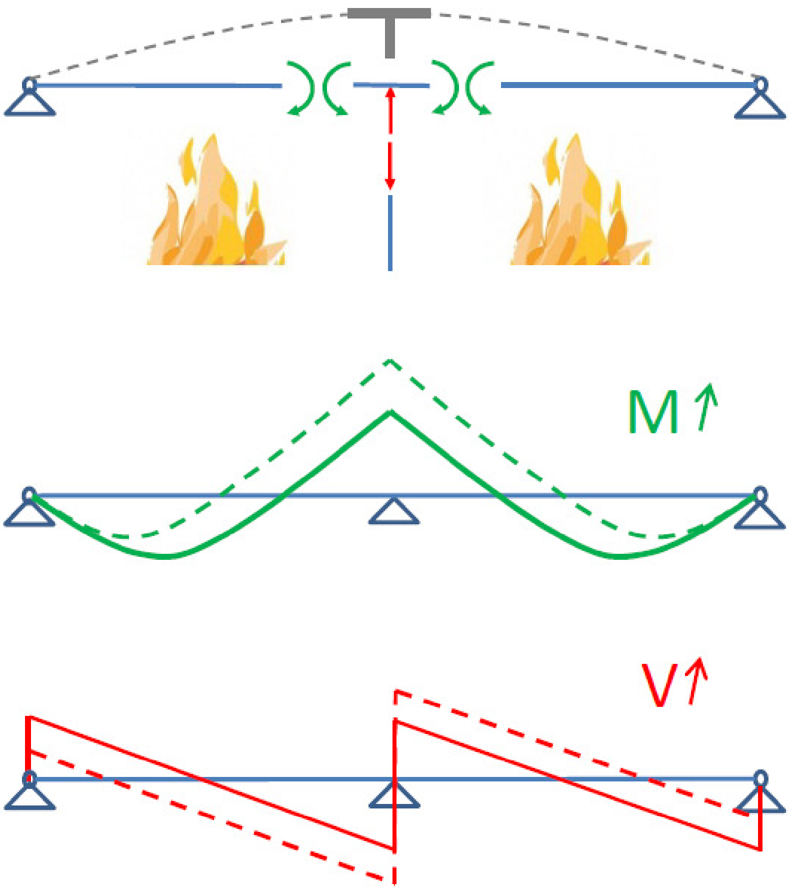

Fig. 12 Increase of axial load due to restraint of thermal deformations

Secondly, an increase of the axial load at the support can be introduced during fire due to a restraint of the thermal expansion of the column. When concrete columns are heated, they will expand. However, this expansion will be restrained by the weight and stiffness of the surrounding structure, resulting in a reaction force in the column-slab connection (illustrated in figure 12). The resulting displacement of the column top can be obtained by the free thermal expansion which should be reduced by several load and temperature dependent strains, namely creep, instantaneous stress-related strain and transient strain. These load dependent strains are often grouped together as Load Induced Thermal Strain (Khoury & al. 2007) and have a major influence because for load ratios of about 40% of the initial compressive strength, the resulting expansion is very limited (Anderberg & Thelandersson 1976, Schneider 1976). In the international literature, these phenomena have been studied since a long time resulting in several applicable models (Khoury & al. 2007, Anderberg & Thelandersson 1976, Schneider 1976, Tao 2008, Annerel & Taerwe 2011). Martins & Rodrigues (2010) found an increase of the axial load due to the restraining force from experiments with factors ranging from 1.02 to 2.64 depending on the longitudinal reinforcement ratio, the slenderness of the column, the restraint level, the load level and the load eccentricity. Martins & Rodrigues (2010) also mentions that restraint of the thermal elongation of reinforced concrete columns does not affect the fire resistance of the column significantly, but the increased axial load should be redistributed to the surrounding structure. This redistribution of the forces might be a problem for flat concrete slabs given their sensitivity to punching shear.

5.2 Assessment of risk at premature failure

In order to assess the risk at premature failure of the flat slab-column connection under investigation, the experimental results of section 2 can be compared to the increase of the axial loads found from finite element analysis, as given in section 5.2.

Table 3 summarizes the punching test results for the four slabs. The 3rd column presents the results for the test slabs at ambient temperatures, whereas the punching resistance at high temperature is given in the 4th column. With respect to the service load, the actual punching resistance at ambient conditions is about 2.5-3.7 times higher (5th column). Those values can be considered as a kind of safety factor taken into account by the design procedure. After 120 minutes of ISO 834 fire curve, this safety factor dropped till 1.87 and 2.17, respectively for the slabs S0 and S1 (6th column). Thus, based on the calculated increase of the axial load during fire for the given scenario of section 2.3, punching failure is expected to occur as the load action of 2.3 times the value at ambient conditions is larger than the remaining resistance safety factor.

Table 3
Ratios of punching resistance with respect to service load

Slab	Service load (EN 1992-1-1) [kN]	V(20°C) [kN]	V(T) [kN]	V(20°C)/ Service load [-]	V(T)/Service load [-]
S0	270	998	588	3.7	2.17
S1	336	about 1000	627	3.0	1.87
S2	402	about 1000	-	2.5	-
S5	420	about 1050	-	2.5	-

Due to a technical failure in the second fire test, the load applied on slabs S2 and S5 is not known, but could be retraced as given in section 2.5 to a factor of 1.5-2 times the service load for S2 and 50-150 kN for S5. Because of this overload, slab S2 failed due to punching after 20 minutes of ISO 834 fire. This observation is in agreement with the fire accident at Gretzenbach, mentioned in the introduction.

It is concluded that a global structural analysis which takes into account the thermal restraints is necessary to verify the fire safety design of underground parking structures.

6 Conclusions

- The slabs tested in the first fire test (0 and 1 stirrups) did not fail under the full service load (according to EN 1992-1-1) and 120 minutes of ISO 834 fire. Increasing the load till failure, showed a residual punching capacity of about 60%.
- Calculations based on the EN1992-1-2 punching model show adequate agreement with the fire test results if spalling of the bottom cover is considered and if the loss of compressive strength is according to the steeper degradation curve as given in EN1992-1-2 for the tensile strength.
- Following the ISO 834 fire curve, spalling occurred at the bottom of the flat slabs and corners of the column stub. Spalling depths of up to 100 mm are found. In a region of about 0.8-1 m around the column stub, the concrete cover of the bottom reinforcement is lost and the bottom part of U-shaped stirrups is visible.
- Loss of the concrete cover due to spalling contributes to an additional loss of up to 10-15% of the punching shear resistance, based upon the simplified calculation model. To avoid structural problems, it is advised to take measures to reduce the spalling risk. It was not investigated which measures are sufficient, such as an adequate amount of polypropylene fibers, additional

reinforcement, or the use of passive fire protection.

- The experimental result of S2 (overloaded during fire) and the increase of the load calculated by FEM found in literature explain the risk for punching failure of a flat slab-column connection exposed to fire. It is concluded that a global structural analysis which takes into account the thermal restraints is necessary to verify the fire safety design of underground parking structures.

Acknowledgements

This research is funded by IWT Flanders through the SBO project 080010: 'Fundamental design approaches for improvement of the fire safety in car parks'.

References

Anderberg, Y., Thelandersson, S. (1976), Stress and deformation characteristics of concrete at high temperatures: 2. experimental investigation and material behaviour model, Bulletin 54, Lund Institute of Technology, Lund, Zweden.

Annerel, E., Taerwe, L. (2011), Evolution of the strains of traditional and self-compacting concrete during and after fire, Materials and Structures, Vol. 44, Issue 8, pp. 1369-1380.

Annerel, E., Lu, L., Taerwe, L. (2013a), Punching shear tests on flat concrete slabs exposed to fire, Fire Safety Journal, Vol. 57, pp. 83-95.

Annerel, E., Taerwe, L., Merci, B., Jansen, D., Bamonte, P., Felicetti, R. (2013), Thermo-mechanical analysis of an underground car park structure exposed to fire, Fire Safety Journal, Vol. 57, pp. 96-106

Annerel, E., Lu, L., Taerwe, L. (2013b), Simplified Approach for Punching Shear in Fire conditions, Studies and Researches, Annual Review of Structural Concrete, Vol. 32, pp.263-278.

EN 1992-1-2 (2004), Eurocode 2: Design of Concrete Structures – Part 1-2: General Rules – Structural Fire Design, CEN, Brussels, Belgium.

EN 1992-1-1 (2004), Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings, CEN, Brussels, Belgium.

Fire Safety Journal (2013), Special issue: Fire and Explosion Safety in Large Car Parks without Sprinklers, Vol. 57.

Khoury, G.A., Anderberg, Y., Both, K., Fellinger, J., Høj, N.P., Majorana, C. (2007), fib bulletin 38: Fire Design of Concrete Structures – Materials, Structures and Modelling, State-of-the art Report, International Federation for Structural Concrete (fib TG 4.3.1), Lausanne, Switzerland.

Kodur V.K.R. and Dwaikat M.B. (2008). "Flexural Response of Reinforced Concrete Beams Exposed to Fire". *Structural Concrete*, Vol. 1, No. 9, 45-54.

Kordina, K. (1993) Flat slabs under fire – Redistribution of the internal forces and punching tests, Institut für Baustoffe, Massivbau und Brandschutz, Technische Universität Braunschweig, Germany, CEN/TC 250/SC 2/PT 1-2 Doc N35.

Martins, A.M.B., Rodrigues, J.P.C. (2010), Fire resistance of reinforced concrete columns with elastically retrained thermal elongation, Engineering Structures, Vol. 32, pp. 3330-3337

Schneider, U. (1976), Behaviour of concrete under thermal steady state and non-steady state conditions, Fire and Materials, Vol. 1, pp. 103-115.

Taerwe, L., Bamonte, P., Both, K., Denoël, J.-F., Diederichs, U., Dotreppe, J.-C., Felicetti, R., Fellinger, J., Franssen, J.-M., Gambarova, P.G., Høj, N.P., Lennon, T., Meda, A., Msaad, Y., Ožbolt, J., Periškić, G., Riva, P., Robert, F., Van Acker, A. (2008), fib bulletin 46: Fire Design of Concrete Structures – Structural Behaviour and Assessment, State-of-the art Report, International Federation for Structural Concrete (fib TG 4.3.2), Lausanne, Switzerland.

Tao, J. (2008), Mechanical performance and thermal-mechanical constitutive model of self-compacting concrete at high temperature, PhD, Ghent University, Belgium - Tongji University, China.