International Journal of Progressive Sciences and Technologies (IJPSAT)
ISSN: 2509-0119.

http://ijpsat.ijsht-journals.org
Vol. 27 No. 1 June 2021, pp.292-298

Fire Resistance Assessment of Reinforced Concrete Slabs According to the Eurocodes

Thien Anh Tran

Department of Civil Engineering, The University of Danang - University of Science and Technology, Vietnam

***Abstract* - Reinforced concrete slabs are sensitive structural concrete elements to the effects of fire. This paper investigates the fire resistance of reinforced concrete slabs exposed to a standard fire using the simplified calculation mehods in the Eurocodes. The effects of tension reinforcement ratio, concrete cover, and concrete strength on the fire resistance of reinforced concrete slabs are assessed. The results show that the fire resistance of reinforced concrete slabs increases as the tension reinforcement ratio increases. The increase in concrete cover also improves the fire resistance of slabs, but up to a certain point where the decrease in the effective depth of cross-section becomes significant, the fire resistance of reinforced concrete slabs will be reduced. Meanwhile, high concrete strength slightly improves the fire resistance of reinforced concrete slabs.**

***Keywords* - reinforced concrete, slab, fire resistance, fire design, Eurocode.**

I. Introduction

The resistance of structural elements to the effects of fire is an important requirement of any structural and fire safety design. There are several building codes and standards pertaining to structural design for fire such as those from the United States, European Union members, Canada, Australia, New Zealand, and Japan: ASCE/SEI/SFPE 29-05 [1], ACI 216.1-07 [2], IBC 2018 [3], ASTM E119 [4], Eurocode 1 EN 1991-1-2:2002 [5], Eurocode 2 EN 1992-1-2:2004 [6], CSA A23.3-04 [7], AS 3600-2018 [8], NZS 3101-2006 [9], the Building Standard Law of Japan [10]. These countries have vigorous research activities on behavior and design of structures under fire [11]. This study presents the general principles and design methods in the Eurocodes, and investigates the fire resistance of reinforced concrete slabs exposed to a standard fire using the simplified calculation mehods in Eurocode 2 EN 1992-1-2:2004. The effects of tension reinforcement ratio, concrete cover, and concrete strength on the fire resistance of reinforced concrete slabs are calculated and assessed.

II. General Principles And Design Methods According To The Eurocodes

2.1. Nominal temperature-time curves

The nominal temperature-time curves present the relation between the gas temperature in the compartment and the time. No parameters related to the compartment characteristics are considered in the nominal curves. Eurocode 1 EN 1991-1-2:2002 [5] provides the following models.

The standard temperature-time curve (ISO 834) is given by:

$$\theta_g = 20 + 345\log_{10}(8t+1) \tag{1}$$

where θ_g (°C) is the gas temperature in the compartment; t (min) is the time.

The external fire curve and the hydrocarbon temperature-time curve are given in (2) and (3), respectively.

$$\theta_g = 20 + 660(1 - 0.687e^{-0.32t} - 0.313e^{-3.8t}) \tag{2}$$

$$\theta_g = 20 + 1080(1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}) \tag{3}$$

2.2. Load combinations involving fire

The general format of effect of actions is calculated from:

$$E_{d.fi} = \sum_{j\geq 1} G_{kj} + P + A_d + \psi_{fi} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{4}$$

where P is the characteristic value of the prestressing force; A_d represents the indirect actions due to the fire; ψ_{fi} is the combination factor for frequent or quasi-permanent values given either by $\psi_{1,1}$ or $\psi_{2,1}$.

As a simplification to (4), the effects of actions maybe obtained from a structural analysis for normal temperature design as:

$$E_{d.fi} = \eta_{fi} E_d \tag{5}$$

where E_d is the design value of the corresponding force or moment for normal temperature design, for a fundamental combination of actions; n_{fi} is the reduction factor for the design load level for the fire situation.

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{6}$$

where G_k is the characteristic value of a permanent action; $Q_{k,1}$ is the principal variable load; γ_G is the partial factor for a permanent action; $\gamma_{Q,1}$ is the partial factor for variable action 1. Examples of the variation of the reduction factor η_{fi} versus the load ratio $Q_{k,1}/G_k$ and different values of the combination factor $\psi_{1,1}$ are shown in Fig. 1.

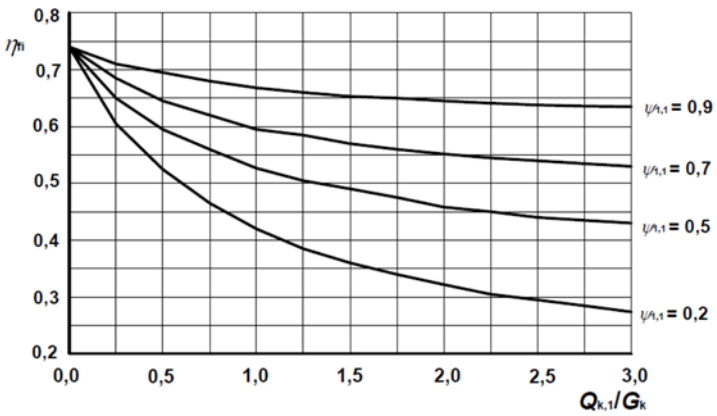

Fig. 1 Variation of the reduction factor with the load ratio $Q_{k,1}/G_k$ *(Eurocode 2)*

2.3. Tabulated data method

The method gives recognized design solutions for the standard fire exposure up to 240 minutes. In the tables, minimum values of the cross-sectional dimensions and the axis distance of the main reinforcement are given. The data has been derived based on an empirical basis confirmed by experience and theoretical evaluation of tests. The tables are obtained for the more common structural elements, using approximate conservative assumptions. Tables exist for the design of slabs, beams, tensile members, columns, and walls, including both load-bearing and non load-bearing.

An advantage of this method is that the designer can quickly verify whether the structural elements are acceptable under fire conditions. It should be noted that the data is obtained for normal weight concrete made with siliceous aggregates. When calcareous aggregates are used, the values for the minimum cross-sectional dimensions may be reduced by 10%. Although the tabulated data method is very convenient for the designer, it only gives approximate and conservative assessment on the fire resistance of the structural elements.

2.4. Simplified calculation methods

Simplified calculation methods may be used to determine the ultimate load-bearing capacity of a cross-section at elevated temperatures which is then compared with the action effect applied on the structural elements. The basis of the simplified calculation approaches are the temperature profiles in the cross-section of structural elements. Eurocode 2 EN 1992-1-2:2004 [6] provides several temperature profiles for various cross-sections with siliceous aggregates exposed to a standard fire.

500ºC isotherm method:

The method is applicable to a standard fire exposure and any other time heat regimes causing similar temperature fields in the fire exposed member. It comprises a general reduction of the cross-section size with respect to a heat damaged zone at the concrete surfaces. The thickness of the damaged concrete is made equal to the average depth of the 500ºC isotherm in the compression zone of the cross-section. Damaged zone which is concrete with temperatures in excess of 500ºC is assumed not to contribute to the load bearing capacity of the member.

In this method, conventional calculation methods for the reduced cross-section are used to determine the ultimate load-bearing capacity of structural elements with the reduced strength of the reinforcing bars due to the elevated tempertures (Fig. 2).

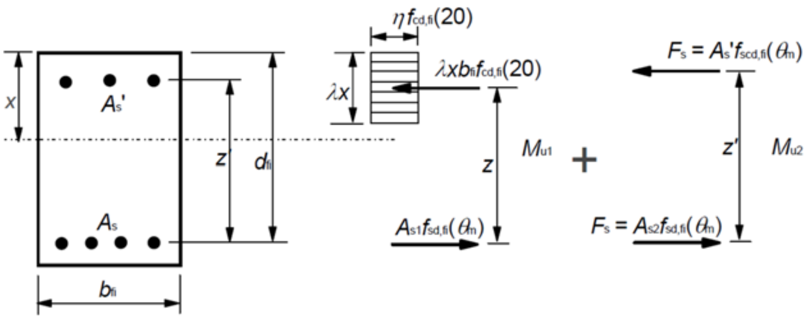

Fig. 2 Calculation of load-bearing capacity of a cross-section for a general flexural element (*Eurocode 2*)

where d_{fi} is the effective depth of the effective cross-section; A_s is the area of tension reinforcement; A_s' is the area of compression reinforcement; $f_{cd,fi}(20)$ is the design value of compression strength of concrete in the fire situation at normal temperature, $f_{cd,fi}(20) = \frac{f_{ck}}{\gamma_{c,fi}}$; $f_{sd,fi}(\theta_m)$ is the design value of the tension reinforcement strength in the fire situation at mean temperature θ_m in that layer. The factors λ and η are determined based on the rectangular stress distribution as follows:

$$\lambda = 0.8 \quad \text{for} \quad f_{ck} \le 50MPa \tag{7}$$

$$\lambda = 0.8 - (f_{ck} - 50)/400 \quad \text{for} \quad 50 < f_{ck} \le 90MPa$$

$$\eta = 1.0 \quad \text{for} \quad f_{ck} \le 50MPa \tag{8}$$

$$\eta = 1.0 - (f_{ck} - 50)/200 \quad \text{for} \quad 50 < f_{ck} \le 90MPa$$

Zone method:

In this method, the cross-section is divided into a number $(n \geq 3)$ of parallel zones of equal thickness where the mean temperature, the corresponding mean compressive strength and modulus of elasticity of each zone are assessed. The fire damaged cross-section is represented by a reduced cross-section ignoring a damaged zone of thickness a_z at the fire exposed sides.

The mean reduction coefficient for a particular section may be calculated as:

$$k_{c,m} = \frac{(1-0.2/n)}{n}\sum_{i=1}^{n} k_c(\theta_i) \tag{9}$$

where n is the number of parallel zones in width w; m is the zone number.

The width of the damaged zone a_z for slabs, beams or members in plane shear may be calculated using the following expression:

$$a_z = w\left[1 - \frac{k_{c,m}}{k_c(\theta_M)}\right] \tag{10}$$

where $k_c(\theta_M)$ denotes the reduction coefficient for concrete at point M.

III. FIRE RESISTANCE OF REINFORCED CONCRETE SLABS

The fire resistance of simply supported reinforced concrete slabs exposed to a standard fire are determined using the simplified calculation mehods in Eurocode 2. The slabs have the thickness of 220mm and span of 7m. The concrete self-weight is 25kN/m^3. The tension reinforcement has the characteristic yield strength f_{yk} of 500MPa. The superimposed dead load and live load are 1kN/m^2 and 3kN/m^2, respectively. The effects of tension reinforcement ratio, concrete cover, and concrete strength on the fire resistance of the simply supported reinforced concrete slabs are determined and evaluated in the following sections.

3.1. Fire resistance versus tension reinforement ratio

The relation between the fire resistance T (min) of the slab and its tension reinforcement ratio μ (%), corresponding to various values of a (mm), is presented in Fig. 3. Distance a is the axis distance of tension reinforcement from the nearest exposed surface. Concrete C20/25 is used in which the characteristic compressive cylinder strength of concrete at 28 days f_{ck} is 20 MPa.

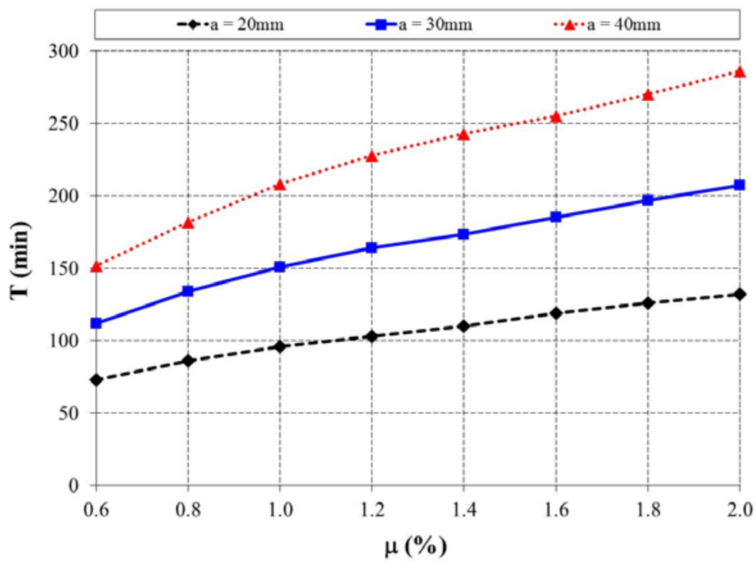

Fig. 3 Fire resistance versus tension reinforcement ratio

The figure shows that the calculated fire resistance of the slab T increases quite significantly as the tension reinforcement ratio increases. When the tension reinforcement ratio increases from 0.6% to 2.0%, the fire resistance T increases approximately 80%

and 90% at the axis distance of 20mm and 40mm, respectively.

The figure also indicates that the fire resistance of the slab T increases quickly when the axis distance increases. When the axis distance doubles from 20mm to 40mm, the fire resistance T increases 2.1 and 2.2 times, corresponding to the tension reinforcement ratio of 0.6% and 2.0%, respectively.

3.2. Bending moment capacity versus axis distance

Fig. 4 shows the relation of the bending moment capacity $M_{Rd,fi}$ (kNm) versus the axis distance a (mm) after two hours of standard fire exposure, corresponding to different levels of tension reinforcement ratio. Concrete C20/25 is used.

The figure indicates that the calculated bending moment capacity first increases quickly as the axis distance increases from 30 to 40mm, then remains almost the same at the axis distance ranging from 40 to 50mm, and finally decreases quickly when a increases from 50 to 90mm. This trend is very similar for different levels of tension reinforcement ratio in the range of 0.6% and 1.8%. This is because the fire resistance of slabs is first improved when increasing the axis distance, then it is declined due to the decrease of the effective depth of the cross-section. As the tension reinforcement ratio triples from 0.6% up to 1.8%, the bending moment capacity $M_{Rd,fi}$ increases around 2.7 and 2.4 times at the axis distance of 30 and 90mm, respectively.

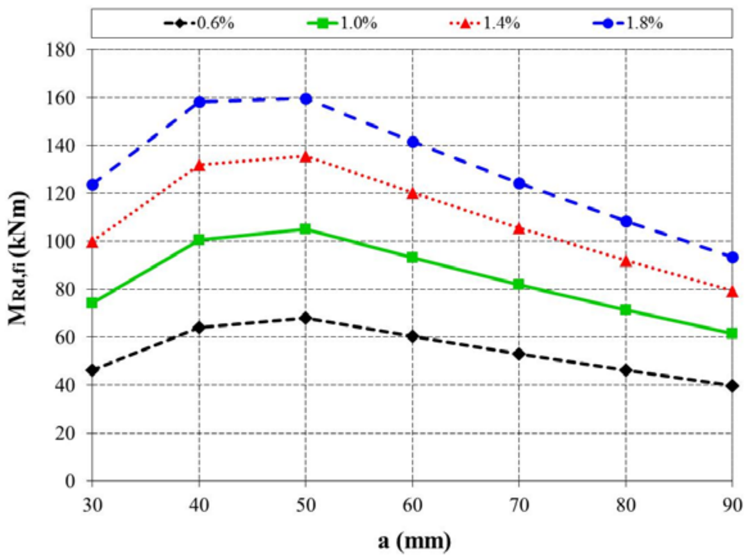

Fig. 4 Bending moment capacity versus axis distance

3.3. Fire resistance versus axis distance

The relation between the fire resistance T (min) of the slab and its axis distance a (mm) is presented in Fig. 5. Concrete C20/25 is used and the tension reinforcement ratio is 0.8%. The figure indicates that the fire resistance T increases substantially as the axis distance increases from 30 to 70mm, then the increase is slowed down and T decreases when the axis distance exceeds 80mm. The fire resistance T at the axis distance of 70mm is 2.3 times of that at the axis distance of 30mm.

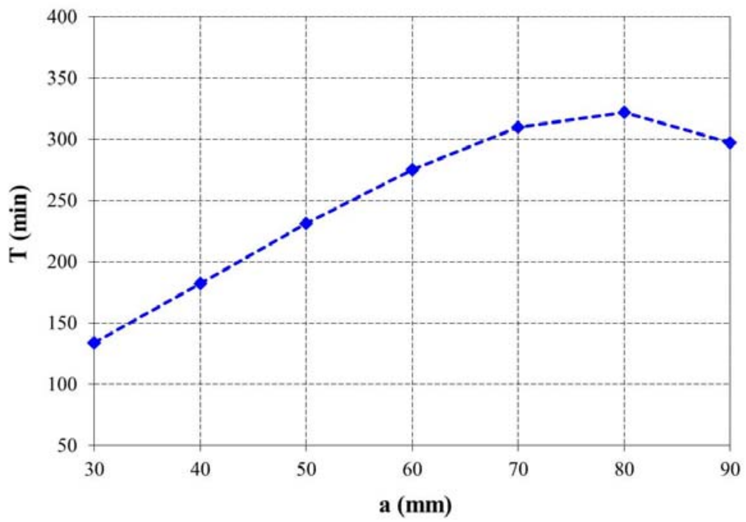

Fig. 5 Fire resistance versus axis distance

3.4. Bending moment capacity versus concrete strength level

Fig. 6 shows the relation of the bending moment capacity $M_{Rd,fi}$ (kNm) versus the concrete strength f_{ck} (MPa) which is the characteristic compressive cylinder strength of concrete at 28 days. The bending moment capacity is calculated after two hours of standard fire exposure.

The figure indicates that the calculated bending moment capacity only increases slightly as the compressive strength of concrete increases, especially with low tension reinforcement ratio. When the compressive strength of concrete increases from 20 up to 90MPa, the bending moment capacity only increases 6% and 23% at the tension reinforcement ratio of 0.6% and 1.8%, respectively.

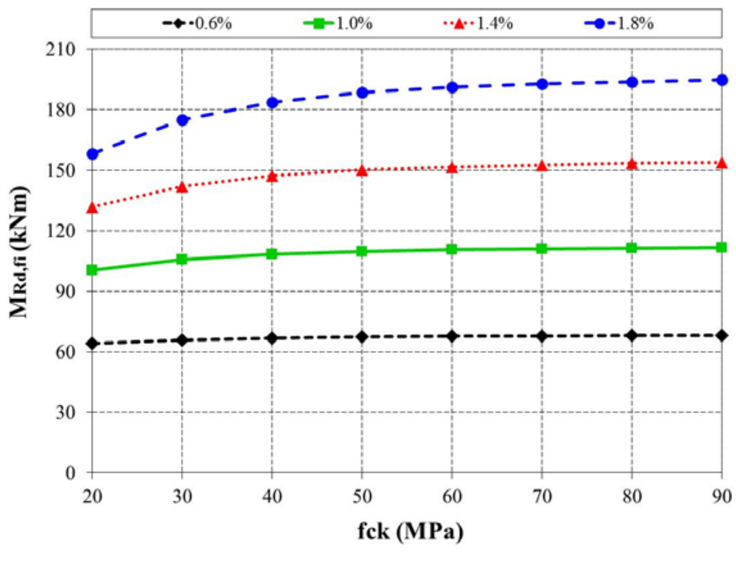

Fig. 6 Bending moment capacity versus concrete strength level

IV. Conclusion

An assessment on the fire resistance of reinforced concrete slabs exposed to a standard fire was carried out using the simplified calculation mehods in the Eurocodes. The simply supported slabs have the thickness of 220mm and the span of 7m. The concrete self-weight is 25kN/m^3. The tension reinforcement has the characteristic yield strength of 500MPa. The superimposed dead load and live load are 1kN/m^2 and 3kN/m^2, respectively. From the study, the following conclusions can be drawn:

- The fire resistance of the slab increases quite significantly as the tension reinforcement ratio increases. When the tension reinforcement ratio increases from 0.6% to 2.0%, the fire resistance T (min) increases approximately 80÷90% at the axis distance ranging between 20 and 40mm.
- The fire resistance T as well as the bending moment capacity $M_{Rd,fi}$ of the slab increase as the axis distance increases, but up to a certain point where the decrease in the effective depth of cross-section becomes significant, the fire resistance of the slab will be reduced.
- The bending moment capacity at elevated temperatures only increases slightly as the compressive strength of concrete increases, especially with low tension reinforcement ratio. When the compressive strength of concrete increases from 20 up to 90MPa, the bending moment capacity only increases 6% and 23% at the tension reinforcement ratio of 0.6% and 1.8%, respectively.

REFERENCES

[1] American Society of Civil Engineers (2007) "ASCE/SEI/SFPE 29-05 Standard Calculation Methods for Structural Fire Protection," Reston, VA, USA.

[2] American Concrete Institute (2007) "ACI 216.1-07/TMS 0216-07 Code requirements for Determining fire Resistance of Concrete and Masonry Assemblies," Farmington Hills, MI.

[3] International Code Council (2018) "International Building Code," Country Club Hills, IL, USA.

[4] ASTM E119 (2012) "Standard Test Methods for Fire Tests of Building Construction and Materials," American Society for Testing and Materials, West Conshohocken, PA.

[5] EN 1991-1-2 (2002) "Eurocode 1: Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire," European Committee for Standardization (CEN), Brussels, Belgium.

[6] EN 1992-1-2 (2004) "Design of concrete structures. Part 1-2: General rules — Structural fire design," European Committee for Standardization, Brussels, Belgium.

[7] Canadian Standards Association (2004) "CSA A23.3-04 Design of Concrete Structures," Toronto, ON, Canada.

[8] Standards Australia (2018) "AS 3600-2018 Concrete structures," Sydney, NSW, Australia.

[9] New Zealand Standards (2006) "NZS 3101: Concrete Structures Standard - Part 1: The Design of Concrete Structures 2006, with amendments Aug. 2008,", Wellington, NZ.

[10] Building Center of Japan (2011) "The Building Standard Law of Japan," Tokyo, Japan.

[11] NIST Technical Note 1842 (2014) "Structural Design for Fire: A Survey of Building Codes and Standards".