Simplified Fire Design Method of Reinforced Concrete Columns According to the Eurocode

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ABSTRACT

Although high-rise buildings are recently being introduced to Ethiopia and fire presents a great threat to these buildings, fire design is a neglected matter in the country. Reinforced concrete columns being important structural components of high-rise buildings, safety against capacity reduction due to additional fire load needs to be considered in design.

Currently the design method for reinforced concrete columns in Ethiopia is based on tabular methods. These methods can only be applied to common dimensions and have limitations. An understanding of the simplified methods and the fundamental behavior of reinforced concrete columns in fire is timely and will be useful to the structural engineer.

In this thesis, one of the simplified methods of the Eurocode, *EN 1992-1-2:2004,* for fire design of reinforced concrete columns is used to evaluate capacity with a step-by-step procedure adopted from literature.

This research will outline the basic principles behind fire design of reinforced concrete columns. Furthermore, the effect of these loads on the capacity of the selected columns will be investigated and compared with experimental results from literature. This paper is prepared to function as an introduction for fire design of reinforced concrete columns and as a basis for further study in Ethiopia.

Keywords: Columns: Fire, design: Concrete, reinforced: Simplified, method: Eurocode

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T	Temperature
A_c	Cross sectional area of concrete
b	Width of cross section
h	Height of cross section
c_b	Depth of neutral axis at balance failure point
c_n	Depth of neutral axis at failure point on failure curve
d	Distance from extreme compression fiber to centroid of tension bar
d′	Concrete cover
E_c	Elastic modulus of concrete
E_s	Elastic modulus of steel reinforcement
E_{sq}	Elastic modulus of steel reinforcement at temperature q
e	Bending eccentricity
e_b	Balanced load eccentricity
e_{mag}	Magnified eccentricity
f_{ck}	Concrete cylindrical strength
f_{yk}	Yield stress of steel reinforcement
$f_{sy,q}$	Yield stress of steel reinforcement at temperature q
d	Moment magnification factor
e	Mechanical strain
e_u	Strain failure criterion of concrete
e_1	Yield strain of concrete
e_{ult}	Ultimate strain of concrete
$e_{s,fi}$	Strain of the reinforcing steel at temperature
k(q)	Reduction factor for a strength or deformation property dependent on the temperature q

I_c	Moment of inertia of cross section
I_s	Moment of inertia of steel
k_c	Reduction factor for concrete
k_s	Reduction factor for steel
M_b	Balanced moment capacity
P_b	Balanced failure load
P_n	Ultimate load

1 INTRODUCTION

1.1 General

"From a long time past, fire has not only constituted a benefit to mankind, but also a serious threat." Many fires have occurred in the past due to restricted knowledge which were severe enough to threthen the survival of whole communities.[1]

Non combustible chimineys were introduced in the mediveal times to control fire at the source.[2]

In 1189 in London stone walls were required to be built on the boundary between buildings to prevent fire from spreading. It was then extended in the 19th centuary to include non-combustible or 'fireproof' floors in certain parts of buildings.[3]

Fire design for buildings was not sufficiently supported by science until relatively recent times.[4] The introduction of high rise buildings in the United States and elsewhere in the world demarked the start of schematic investigations in the fire field.[5]

In traditional fire design building regulations are concerned with safety within and around the building by preventing internal and external fire spread. Compliance with these requirments is ensured by relying on standard tests carried out on individual elements or components. Structural stability is not considered as an issue for safety of occupants. But after 9/11 incident in the United States, the crucial impact of structural stability on safety of ocupants was observed.

[1] L. TWILT AND J. WITTEVEEN, *Trends in fire safety design of buildings, TNO-Institute for Building Materials and Structures, 95*

[2] IStructE, *Introduction to the fire safety engineering of structures, 7*

[3] IStructE, *Introduction to the fire safety engineering of structures, 8*

[4] IStructE, *Introduction to the fire safety engineering of structures, 8*

[5] L. TWILT AND J. WITTEVEEN, *Trends in fire safety design of buildings, TNO-Institute for Building Materials and Structures, 97*

Key structural elements like columns can simultaneously collapse during fire in multi storey buildings and design should be beyond current robustness requirements. Design in such cases should be carried out according to the requirements of the building in terms of risk and consequence. [6] Fire safety engineering now is having a growing impact on how structures are designed, procured and specified. [7]

Although high-rise buildings are recently being introduced to Ethiopia and fire presents a great threat to these buildings, fire design is a neglected matter. Columns being important structural components of these buildings, safety against capacity reduction due to additional fire load needs to be considered.

In this thesis, the design approaches given in the fire sections of the Eurocode for columns will be studied. These along with the detailed science, step-by-step procedure, and experimental results from literature will help evaluate the capacity of columns during fire. The findings can then be an introduction of simplified design methods for fire design of columns in Ethiopia and a basis for further study.

1.2 Statement of the Problem

Little or no attention was given to fire deign in the past. Although it posed great threats it is getting recognition in structural design only in recent years worldwide. Design philosophy of reinforced concrete structures around the world against fire is a new concept that is recently being developed.

In the past in Ethiopia, there wasn't a need for fire design. As the country is now getting introduced to construction of high-rise buildings and congested constructions, fire design can no longer be taken for granted.

[6] T Lennon BEng BA, *Structural fire engineering design: introduction, Digest 484, 2*

[7] IStructE, *Introduction to the fire safety engineering of structures, 7*

Reinforced concrete structural systems are mainly used for high-rise buildings, reinforced columns being the major load component. Therefore, reinforced concrete columns have to satisfy fire design requirements since after using active majors for controlling the fire; structural integrity is the last line of defense.

Fire has an effect on the overall property of these columns. Capacity reduction of columns in design due to fire load needs to be considered because fire catastrophe in columns leads to collapse and poses real threat to human life.

1.3 Objectives

The main objective of this thesis is to evaluate the capacity of reinforced concrete columns during fire according to the Eurocode. And to use the simplified method of the Eurocode for design of columns during fire in accordance with procedure adopted from a journal of structural engineering, *Fire Resistance of Four-Face Heated Reinforced Columns*.

In doing the work the following points are also dealt with:

- The different column design methods for fire given in Eurocode, *EN 1992-1-2:2004*.
- Factors governing fire resistance of reinforced concrete columns.
- Fire contribution on secondary effect of reinforced concrete columns, additional bending caused by fire.
- Deterioration of strength and stiffness of concrete and steel during fire.
- Relationship between different heating curves.
- Effect of temperature on P-M interaction curves.

- Comparison of experimental results with capacity calculated using the proposed simplified procedure.

1.4 Methodology

- Literature review
- Present the simplified method of the Eurocode for design of columns during fire in accordance with adopted procedure according to literature.
- Select representative columns from experimental data from literature.
- Evaluate the capacity of the selected columns using the proposed simplified procedure and examine the results.
- Compare results from simplified procedure with the experimental results.
- Summarize findings

1.5 Application of Results

This research will outline the basic principles behind fire design of reinforced concrete columns. Furthermore, the effect of these loads on the capacity of the selected columns will be investigated. This research is prepared to function as an introduction for fire design of columns and as a basis for further study in Ethiopia.

1.6 Scope

This thesis will focus on fire design of reinforced concrete columns according to Eurocode.

- Eurocode is used for design by developing a simplified procedure for capacity calculation of only 4 face heated uniaxial columns.

- Fire design evaluation is only carried out for reinforced concrete four face heated uniaxial columns.
- Only rectangular columns are considered.
- While performing this research no physical test of columns has been carried out. Experimental data is taken from other literature.

2 LITERATURE REVIEW

2.1 Background

In the last few years there was a significant development in fire engineering. Behaviour of natural fires, behaviour of materials and component and system interactions during fire have been investigated. [8]

Fire safety engineering is concerned with the growth phase of fire. Active protection systems such as early detection, sprinkler systems and smoke control are used to achieve fire safety. [9]

There are two measures to fire, passive measure which contain the fire and prevent structural collapse and active measures which control the fire itself and/or facilitate the evacuation of occupants. These two measures in addition to fire engineering, which provides additional design effort for complex structures, comprise what is termed as the good practice in fire design. [10]

History of a well-ventilated fire with the different periods is shown in the figure below. Fire resistance is a passive measure and serves as a second line defense when fire progresses to fully developed stage. [11]

Flashover as seen in the figure, is the brief transition between the original localized fire and the conflagration of all elements in the compartment. [12]

[8] *T Lennon BEng BA, Structural fire engineering design: introduction, Digest 484, 1*

[9] IStructE, *Introduction to the fire safety engineering of structures, 8*

[10] The Concrete Center, *Concrete and Fire, 10*

[11] IStructE, *Introduction to the fire safety engineering of structures, 9*

[12] IStructE, *Introduction to the fire safety engineering of structures, 18*

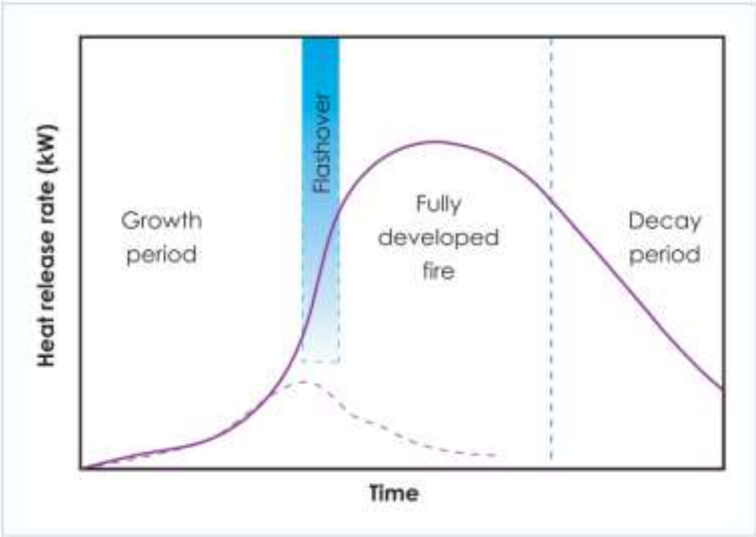

Figure 1: History of a well-ventilated natural fire against rate of heat release. The dotted line represents diminution of fuel before flashover has occurred. [13]

In the beginning, fire analysis was based on ad-hoc testings. Later in order to improve repeatability and reproducibility a need arised for a more standardized test procedure.

Investigations in the past led to the concept of "standard fire curve", standardized relation ship between gas temperature and time which is the basis of traditional structural fire engineering design in many countries.

Standard temperature time cure is the mostly used curve for design. It has no decay phase and doesn't represent any real fire. It is designed to represent the post flash over behavior phase of most fires. [14]

Intially every country developed their own standard curve. With the International Standardization Organization (ISO) these different versions where harmonized into one temperature-time curve which is now accepted world wide. [15]

[13] IStructE, *Introduction to the fire safety engineering of structures, 17*

[14] IStructE, *Introduction to the fire safety engineering of structures, 17*

Regarding materials, concrete in general is non-combustible and has a low conductivity and it serves as a barrier to the spread of fire into adjacent space and within itself. [16] A good fire design for concrete individual elements means measure of fire resistance appropriate to their location, function, load, level of reinforcement and size and shape as mentioned in Eurocode 2. [17]

To prevent failure by exceeding one or more of the relevant fire limit states which are retaining loadbearing capacity, protection from harmful smoke and gases and shield for people from heat, it should be ensured that the temperature of the concrete throughout the section does not reach critical levels. This helps to avoid loss of strength for bending, shear and compression. [18]

Differential temperatures, moisture levels and pore pressures that are caused by heat flow when concrete structures are in fire affect concrete's performance in fire at the three limit states. [19]

On the other hand, Steel losses its strength quickly during fire and hence protection against fire is needed.[20]Average concrete cover should be provided to prevent the reinforcement from reaching critical temperature levels, which is the cause for loss of bond strength between the concrete and the reinforcement. [21]

The Eurocode 2 Part 1-2 *EN 1991 1-2:2004* defines the full analytical procedure for structural fire design as the understanding of the structural system behavior at

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[15] L. TWILT AND J. WITTEVEEN, *Trends in fire safety design of buildings, TNO-Institute for Building Materials and Structures, 100*

[16] The Concrete Center, *Concrete and Fire, 4*

[17] The Concrete Center, *Concrete and Fire, 6*

[18] The Concrete Center, *Concrete and Fire, 6*

[19] The Concrete Center, *Concrete and Fire, 7*

[20] IStructE, *Introduction to the fire safety engineering of structures, 24*

[21] IStructE, *Introduction to the fire safety engineering of structures, 7*

elevated temperatures and the potential heat exposure together with useful effects of active and passive fire protection systems with concern to consequences of failure. [22]

Fire safety engineering is now enabling a gradual transition from a prescriptive based approach to an engineered approach. [23] Recent researches are inclined to performance-based design with hand calculations and advanced computer models rather than code prescribed tables. [24]

2.2 Fire Design

2.2.1 Prescriptive and Performance Designs

There are two ways of achieving fire safety in a building. One is application of building codes of which the majority of solutions are prescribed. Second is using engineered solutions that provide answers where there is deviation from the prescribed. [25]

There are three domains of testing structural ability to carry fire load: time domain, strength domain and temperature domain. Time domain where equivalent time resistance should be greater than the fire duration is usually stated in the codes and standards. The strength and the temperature domains need the some engineering approach where the strength and temperature limit of the structural element should not be exceeded respectively.[26]

[22] EN 1992-1-2:2004 (E), 7

[23] IStructE, *Introduction to the fire safety engineering of structures, 8*

[24] The Concrete Center, *Concrete and Fire, 10*

[25] IStructE, *Introduction to the fire safety engineering of structures, 9*

[26] IStructE, *Introduction to the fire safety engineering of structures, 9*

Fire engineering develops a specific solution for a specific design and is in the contrary from the tradition usage of prescribed data as it creates fire strategy for the specific project.[27]

Performance based design analyzes fire scenarios and allows any design that complies with fire safety requirements. Elements in concrete act as part of a whole rather than in isolation, which is the basic assumption of prescriptive tables, which makes these tables conservative.[28]

With cutting edge software which simulates structural conditions such as cooling patterns, performance based designs are getting popularity these days.[29]

There are two alternative approaches stated in the Eurocode 2 Part 1-2 *EN 1991 1-2* for a project design in fire. The schematic presentation of the design procedures is depicted below.[30]

The prescriptive approach uses nominal fires to generate thermal actions. Whereas, the performance based design focuses on the physical and chemical properties to generate the thermal actions.[31]

Fire resistance period related to applied load level is provided.[32]

[27] The Concrete Center, *Concrete and Fire, 8*

[28] The Concrete Center, *Concrete and Fire, 10*

[29] The Concrete Center, *Concrete and Fire, 10*

[30] *EN 1992-1-2:2004 (E), 9*

[31] *EN 1992-1-2:2004 (E), 8*

[32] *EN 1992-1-2:2004 (E), 43*

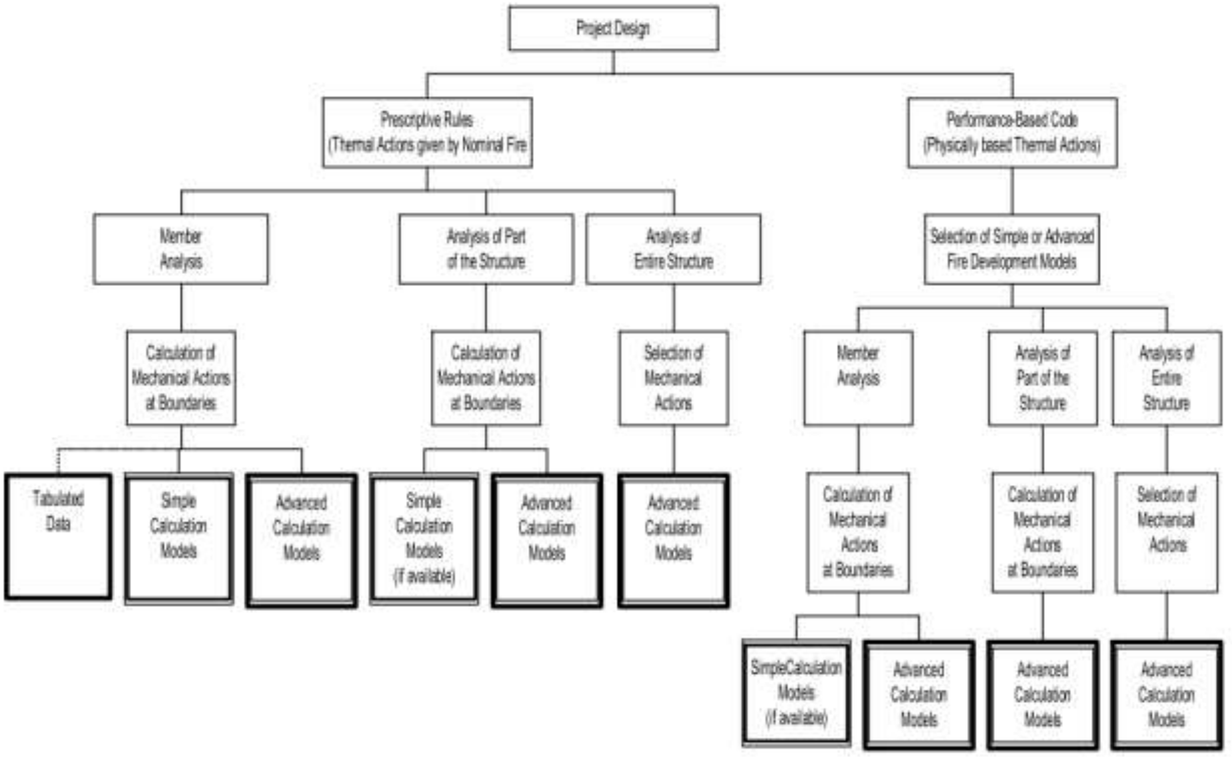


Figure 2: Alternate design procedures

2.3 Fire Exposure

Usually, fire resistance is determined by exposing structures to standard temperature time curves or by using natural fire models.

2.3.1 Standard Temperature-Time Curve

In general for structural analysis fire action is accounted by means of temperature-time curves.

Nominal temperature curves are the most basic form of temperature-time curves that present gas temperature of a member surfaces as a function of time. Nominal temperature time curves assume constant temperature in the whole compartment. These curves don't consider thermal properties of compartments like fire load, ventilation and they don't model cooling phase of the fire.

A more realistic approach is provided by parametric curves where temperature evolution is based on physical parameters of the compartment. A descending branch is also considered for cooling phase representing decay. But temperature is again assumed constant in the whole compartment in this model.

In EN 1991 1-2 three types of nominal temperature-time curves are presented.

a. *Standard temperature-time curves*

b. *External fire curve*

c. *Hydrocarbon curve*

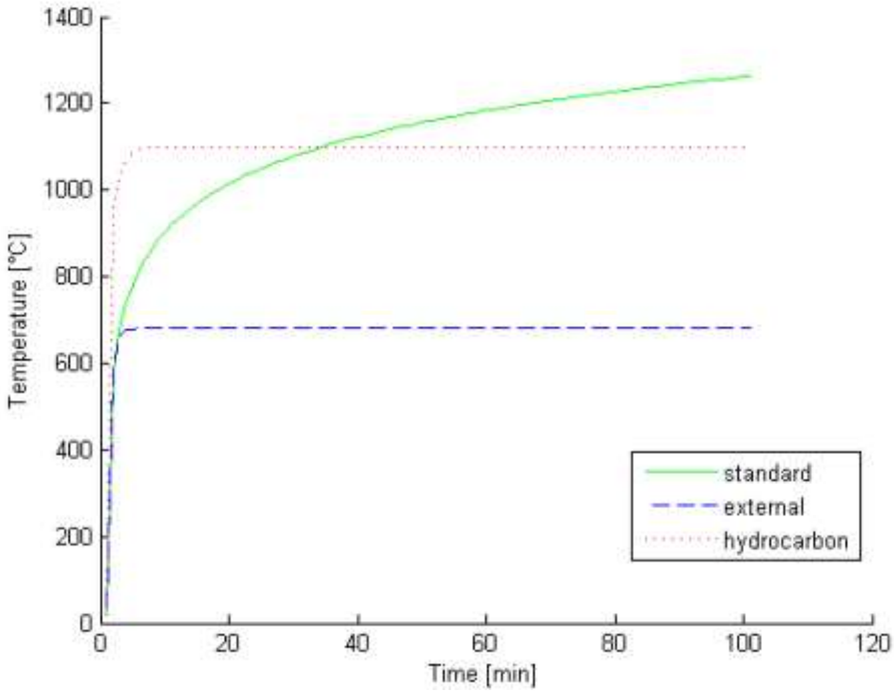


Figure 3: Nominal temperature curves

The standard temperature-time curve or ISO curve is the mostly used temperature-time curve. *"It was developed in the 1930s by summarising data from fires in* *residential, office and commercial buildings. The curve should cover most of the potential courses of fires in common buildings."* [33]

Standardized fire testing goes back to 1903, London. Many heating curves were developed from then on and revisions have been carried out to harmonize all these heating curves through the international standard ISO 834. [34]

The standard temperature-time curve doesn't represent the temperature variation of a real fire. But it can be used in fire resistance test, to check fire resistance of structural members, to analyze behavior at increase temperature, to check resistance of buildings for fire. [35]

Standard temperature-time curve is mainly used for building.

EN 1991 1-2 gives the following expression for standard temperature-time curve.

Equation 1: Standard temperature-time curve equation EN 1991-1-2-2002

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

where:

θ_g is the gas temperature in the fire compartment [^{0}C]

t is the time [min]

The coefficient of heat transfer by heat convection is:

α_c=25W/m^{2}K

[33] J. Zehfuss, D. Hosser, *A parametric natural fire model for the structural fire design, Fire Safety Journal 42 (2007) 115-126,116*

[34] IStructE, *Introduction to the fire safety engineering of structures, 15*

[35] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 11*

2.3.2 Advanced Fire Models

In the early sixties with a better understanding of the fire process and improvement of research facilities a basis for a more detail fire models was led that accounted ventilation conditions. This was the start of a rapid development of more advanced fire models. [36]

Advanced fire modesls in the Eurocode take gas properties, mass exchange and energy exchange into account**.**

Advanced calculation methods can be used to assess a simple structural element, parts of the structure and the whole structure. They provide realistic analysis. As long as the material properties are known advanced calculation methods can be used with any heating curve and any cross section. [37]

Advanced methods involve numerical models for heat transfer from the fire to the structural member and mechanical response models for the response of member or members to the response of the effect of both mechanical and thermal actions. [38]

The application of advanced fire models is not common because of complex and high computational demands.

2.4 Material Properties

2.4.1 Concrete

In general for concrete, its basic property as a building material and its functionality in a structure are the two major components that determine its successful performance in fire. A good fire design for concrete structures includes robustness, which can be

[36] L. TWILT AND J. WITTEVEEN, *Trends in fire safety design of buildings, TNO-Institute for Building Materials and Structures, 97*

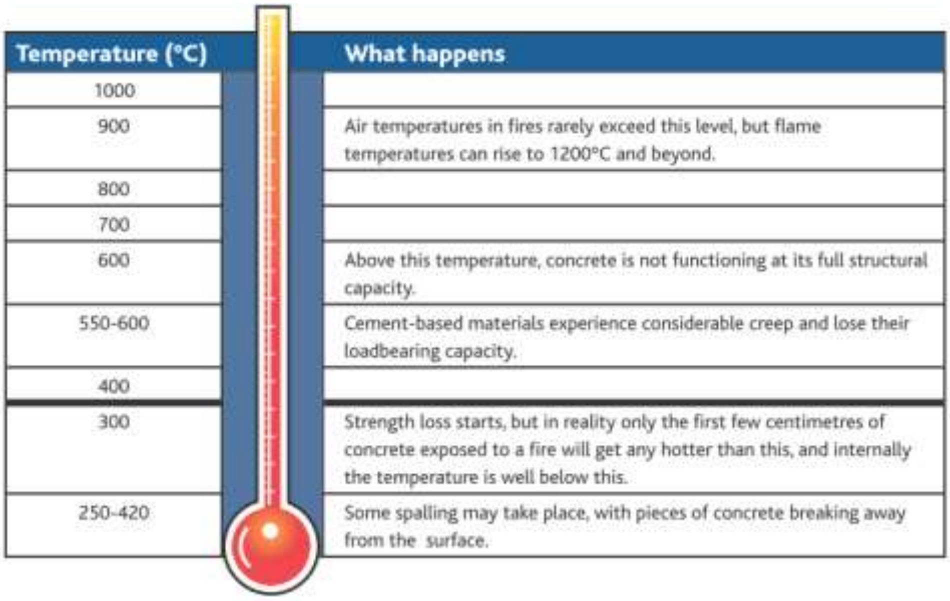
[37] *EN 1992-1-2:2004 (E), 34*

[38] *EN 1992-1-2:2004 (E), 34*

attained by adequate strength, continuity of reinforcement and adequate detailing of connections. [39]

Particular elements of the structure should fulfill its designed function for a period of time in the event of a fire. The function will depend on the element's position and role within the structure. [40]

The following figure briefly describes what happens to concrete at high temperature.



Temperature (°C)	What happens
1000	
900	Air temperatures in fires rarely exceed this level, but flame temperatures can rise to 1200°C and beyond.
800	
700	
600	Above this temperature, concrete is not functioning at its full structural capacity.
550-600	Cement-based materials experience considerable creep and lose their loadbearing capacity.
400	
300	Strength loss starts, but in reality only the first few centimetres of concrete exposed to a fire will get any hotter than this, and internally the temperature is well below this.
250-420	Some spalling may take place, with pieces of concrete breaking away from the surface.

Figure 4: Concrete in fire: physiochemical process [41]

Conductivity, specific heat and thermal elongation are the important qualities that define thermal properties of concrete.

Concrete is good in resisting fire. It has low conductivity. Spalling is the major disadvantage of concrete during fire. [42]

[39] The Concrete Center, *Concrete and Fire, 6*

[40] IStructE, *Introduction to the fire safety engineering of structures, 6*

[41] The Concrete Center, *Concrete and Fire, 5*

[42] IStructE, *Introduction to the fire safety engineering of structures, 27*

In general increased permeability reduces spalling. High performance concretes are vulnerable to spalling because they are dense.[43]

Concrete thermal expansion is complicated due to complex nature the constituent elements of concrete: aggregate, cement paste and rate of heating. Cement paste expands till 150^0C and contracts between 150^0C-400^0C, whereas the aggregate might still be expanding. This creates differential thermal expansion. The reason why concrete doesn't disintegrate at this differential expansion is because of creep of the paste. Above 400^0C creep is significant in concrete, concrete creep consists of the cement paste and aggregate creep.[44]

Both concrete compressive strength and stiffness reduce during fire. The Eurocode gives the reduction factor for both with a distinction between siliceous and calcareous aggregate.

[43] The Concrete Center*, Concrete and Fire, 11*

[44] IStructE*, Introduction to the fire safety engineering of structures, 28*

Table 1: Values for the main parameters of the stress-strain relationships of normal weight concrete with siliceous or calcareous aggregates concrete at elevated temperatures [45]

Concrete temp. θ	Siliceous aggregates			Calcareous aggregates		
	$f_{c,\theta} / f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$	$f_{c,\theta} / f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$
[°C]	[-]	[-]	[-]	[-]	[-]	[-]
1	2	3	4	5	6	7
20	1,00	0,0025	0,0200	1,00	0,0025	0,0200
100	1,00	0,0040	0,0225	1,00	0,0040	0,0225
200	0,95	0,0055	0,0250	0,97	0,0055	0,0250
300	0,85	0,0070	0,0275	0,91	0,0070	0,0275
400	0,75	0,0100	0,0300	0,85	0,0100	0,0300
500	0,60	0,0150	0,0325	0,74	0,0150	0,0325
600	0,45	0,0250	0,0350	0,60	0,0250	0,0350
700	0,30	0,0250	0,0375	0,43	0,0250	0,0375
800	0,15	0,0250	0,0400	0,27	0,0250	0,0400
900	0,08	0,0250	0,0425	0,15	0,0250	0,0425
1000	0,04	0,0250	0,0450	0,06	0,0250	0,0450
1100	0,01	0,0250	0,0475	0,02	0,0250	0,0475
1200	0,00	-	-	0,00	-	-

Concrete compressive strength depends on the grade of concrete. The higher the grade the lesser the strength will be. It also depends on the type of aggregate. 46Concrete starts to lose its compressive strength at around 3500C for siliceous aggregates and at slightly higher temperatures for calcareous and lightweight concrete. [47]

Water/cement ratio or water content is also another factor that affects compressive strength. The larger it is the lower the strength will be. [48]

To test the compressive strength of concrete at elevated temperatures cubic specimens were heated to the predetermined temperature and kept constant in a preheating

[45] EN 1992-1-2:2004 (E), 20

[46] Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 11

[47] IStructE, *Introduction to the fire safety engineering of structures, 29*

[48] Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 11

furnace for six hours to have a uniform temperature distribution. Then the specimens were taken out and put into a strength test furnace and loaded to measure the respective compressive strengths. Figure below shows the failure patterns. For specimens up to 300^{0}C the failure pattern is the same as specimen at normal temperature. But for higher temperatures, there are broken corners and edges and the central part is also reduced. And for temperature greater than 800^{0}C specimen breaks at the end of the loading or breaks into pieces when taken out of the furnace. [49]

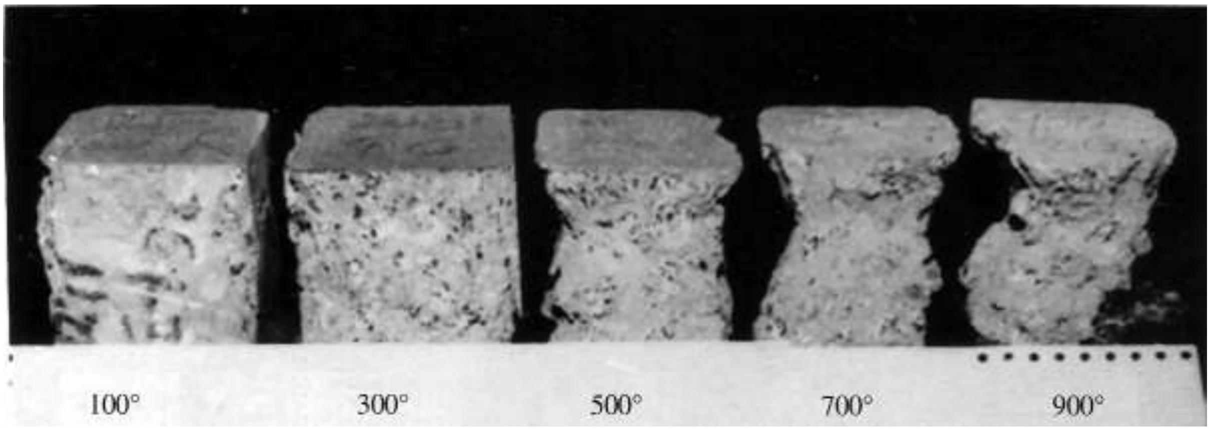


Figure 5: Failure patterns of compressive cubic specimens of concrete at elevated temperatures [50]

The main reasons for the above strength loss are formation of crack and porosity in the interior of concrete due to evaporation, differential thermal expansion and second stress and expansion of coarse aggregate expand and crack at high temperature causing cumulative internal damage at high temperature. [51]

Tensile strength of concrete should normally be ignored, which is conservative. [52]

Strength and deformation properties of uniaxially stressed concrete and reinforcing steel at elevated temperature are determined from stress strain relationships. [53]

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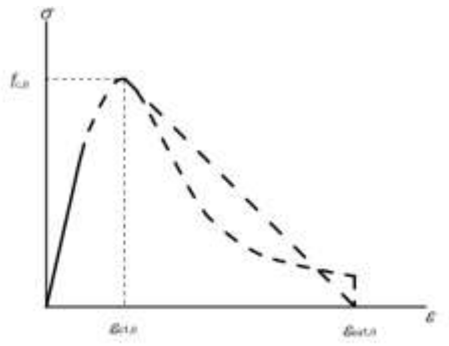
[49] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 8*

[50] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 9*

[51] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 10*

[52] *EN 1992-1-2:2004 (E), 21*



Range	Stress $\sigma(\theta)$
$\varepsilon \leq \varepsilon_{c1,\theta}$	$\frac{3\varepsilon f_{c,\theta}}{\varepsilon_{c1,\theta}\left(2+\left(\frac{\varepsilon}{\varepsilon_{c1,\theta}}\right)^3\right)}$
$\varepsilon_{c1(\theta)} < \varepsilon \leq \varepsilon_{cu1,\theta}$	For numerical purposes a descending branch should be adopted. Linear or non-linear models are permitted.

Figure 6: Mathematical model for stress-strain relationships of concrete under compression at elevated temperatures

Strength gain of concrete during cooling phase of concrete exposed to fire should not be considered. [54]

A test was conducted to measure the deformation and stress strain relationship of concrete. Prism specimens were used to conduct the test. The specimens were preheated and put in to a furnace for deformation test. The result is shown in the following figure. [55]

[53] *EN 1992-1-2:2004 (E), 19*

[54] *EN 1992-1-2:2004 (E), 20*

[55] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 21*

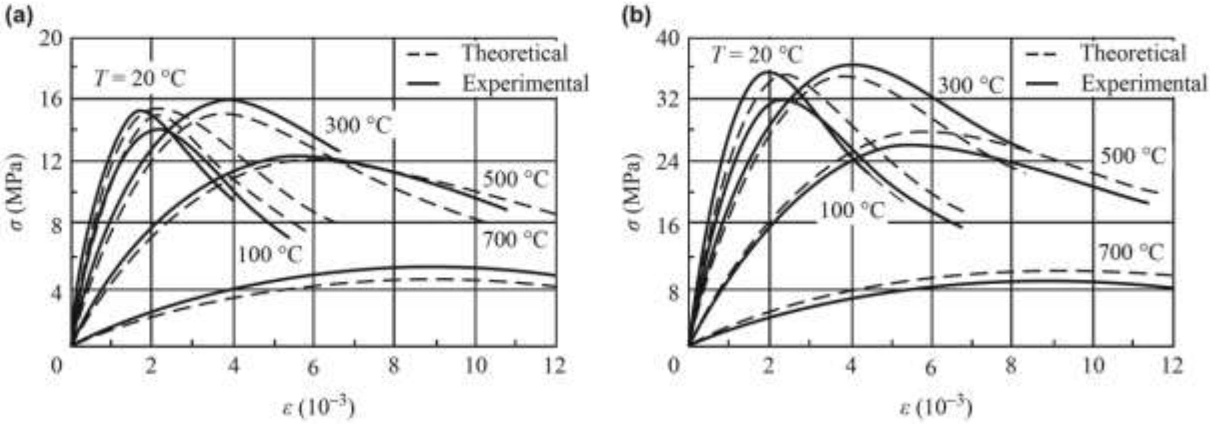


Figure 7: Complete compressive stress–strain curves for concrete at elevated temperatures [56]

It can be seen that the stress strain curve flattens at elevated temperature. The compressive strength decreases while the corresponding peak strain increases. For testing temperature greater than 5000C, cracks are formed in the specimen before loading which reduces the stress scope. As the stress increases the slope of the stress strain decreases because of cracks formed in the surface and interior. And the peak part of the curve tends to flatten. And as the stress extends beyond the peak the deformation increases continuously and the capacity is reduced. [57]

It can be observed from the figure below that when the testing temperature is less than 300^0C an inclined crack appears and when the temperature is greater than 500^0C the most specimens break in the furnace and the angle of the inclined cracks is lesser. [58]

[56] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 21*

[57] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 21*

[58] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 22*

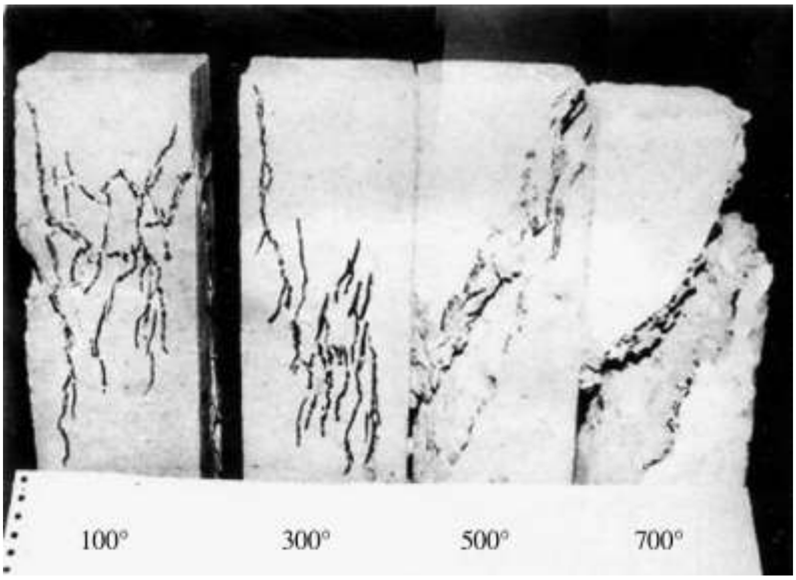


Figure 8: Compressive failure pattern of a prism specimen at elevated temperatures [59]

2.4.2 Steel

Strength and deformation properties are the most important properties of steel.

In the Eurocode like concrete, reduction factors for reinforcement are provided in figures and tables.

Strength and stiffness of steel at high temperatures decrease with a nonlinear stress-strain behavior. [60]

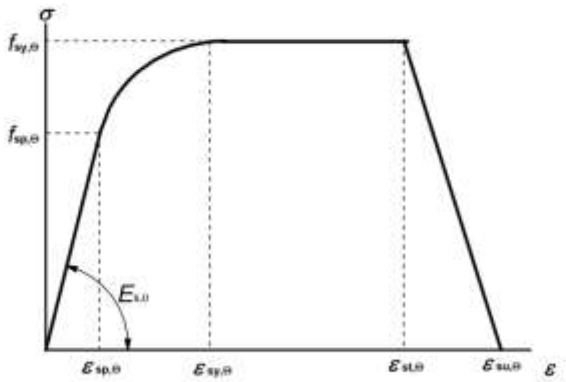
[59] *Experiment and Calculation of Reinforced Concrete at Elevated Temperature, 22*

[60] IStructE, *Introduction to the fire safety engineering of structures, 34*

Table 2: Class N values for the parameters of the stress-strain relationship of hot rolled and cold worked reinforcing steel at elevated temperatures [61]

Steel Temperature θ[°C]	$f_{sy,\theta} / f_{yk}$		$f_{sp,\theta} / f_{yk}$		$E_{s,\theta} / E_s$	
	hot rolled	cold worked	hot rolled	cold worked	hot rolled	cold worked
1	2	3	4	5	6	7
20	1,00	1,00	1,00	1,00	1,00	1,00
100	1,00	1,00	1,00	0,96	1,00	1,00
200	1,00	1,00	0,81	0,92	0,90	0,87
300	1,00	1,00	0,61	0,81	0,80	0,72
400	1,00	0,94	0,42	0,63	0,70	0,56
500	0,78	0,67	0,36	0,44	0,60	0,40
600	0,47	0,40	0,18	0,26	0,31	0,24
700	0,23	0,12	0,07	0,08	0,13	0,08
800	0,11	0,11	0,05	0,06	0,09	0,06
900	0,06	0,08	0,04	0,05	0,07	0,05
1000	0,04	0,05	0,02	0,03	0,04	0,03
1100	0,02	0,03	0,01	0,02	0,02	0,02
1200	0,00	0,00	0,00	0,00	0,00	0,00

[61] EN 1992-1-2:2004 (E), 24



Range	Stress $\sigma(\theta)$	Tangent modulus
$\varepsilon_{sp,\theta}$	$\varepsilon E_{s,\theta}$	$E_{s,\theta}$
$\varepsilon_{sp,\theta} \le \varepsilon \le \varepsilon_{sy,\theta}$	$f_{sp,\theta} - c + (b/a)[a^2 - (\varepsilon_{sy,\theta} - \varepsilon)^2]^{0.5}$	$\dfrac{b(\varepsilon_{sy,\theta} - \varepsilon)}{a\left[a^2 - (\varepsilon - \varepsilon_{sy,\theta})^2\right]^{0.5}}$
$\varepsilon_{sy,\theta} \le \varepsilon \le \varepsilon_{st,\theta}$	$f_{sy,\theta}$	0
$\varepsilon_{st,\theta} \le \varepsilon \le \varepsilon_{su,\theta}$	$f_{sy,\theta}[1-(\varepsilon - \varepsilon_{st,\theta})/(\varepsilon_{su,\theta} - \varepsilon_{st,\theta})]$	-
$\varepsilon = \varepsilon_{su,\theta}$	0,00	-
Parameter *)	$\varepsilon_{sp,\theta} = f_{sp,\theta} / E_{s,\theta}$ $\varepsilon_{sy,\theta} = 0{,}02$ $\varepsilon_{st,\theta} = 0{,}15$ $\varepsilon_{su,\theta} = 0{,}20$ Class A reinforcement: $\varepsilon_{st,\theta} = 0{,}05$ $\varepsilon_{su,\theta} = 0{,}10$	
Functions	$a^2 = (\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta} + c/E_{s,\theta})$ $b^2 = c\,(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})\,E_{s,\theta} + c^2$ $c = \dfrac{(f_{sy,\theta} - f_{sp,\theta})^2}{(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})E_{s,\theta} - 2(f_{sy,\theta} - f_{sp,\theta})}$	

*) Values for the parameters $\varepsilon_{pt,\theta}$ and $\varepsilon_{pu,\theta}$ for prestressing steel may be taken from Table 3.3. Class A reinforcement is defined in Annex C of EN 1992-1-1.

Figure 9: Mathematical model for stress-strain relationships of reinforcing and prestressing steel under at elevated temperatures (notions for prestressing steel "p" instead of "s")

2.5 Reinforced Concrete Columns Exposed to Fire

When columns are subjected to fire, the axial force moment (P-M) interaction diagram contracts.

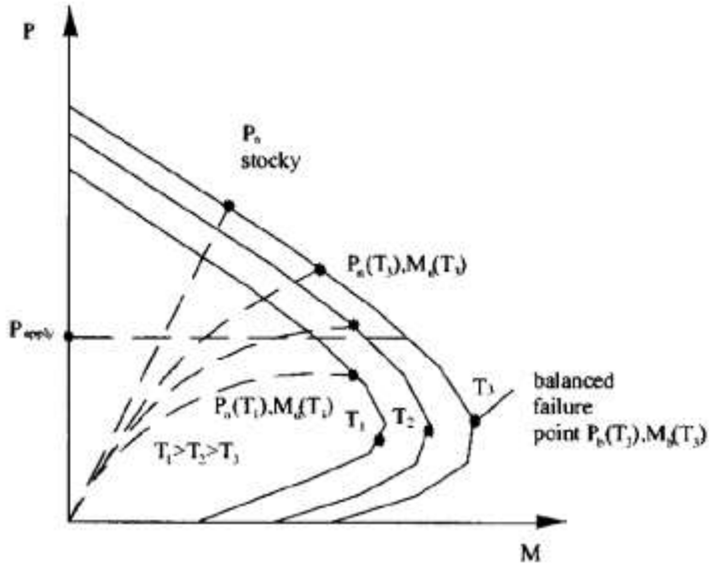


Figure 10: Failure curves of column at different temperatures [62]

Typically reinforced concrete columns are subjected to uniaxial or biaxial bending in addition to axial loading. [63]When a fire accident occurs these columns could be exposed to either one, two, three or four faced heating depending on the column orientation in the fire compartment. [64]Reinforced concrete columns in fire are often subjected to fire induced biaxial bending. [65] Thermal gradient is caused because of 1-, 2-, or 3- sided fire exposures. 1- or 3- sided exposure can result in uniaxial bending and fire on 2 adjacent column sides can cause biaxial bending.

Slenderness and second order effects are the essential mechanical behaviors of reinforced concrete columns, which make fire induced bending crucial.

[62] K.H. Tan, Y. Yao*, Fire Resistance of Four-Face Heated Reinforced Concrete Columns, J. Struct. Eng.2003.129: 1220-1229*

[63] Venkatesh Kodur, Nikhil Raut*, A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

[64] K.H. Tan, Y. Yao*, Fire Resistance of Reinforced Concrete Columns Subjected to 1-, 2-, and 3- Face Heating, J. Struct. Eng.2004.130: 1820-1828*

[65] Venkatesh Kodur, Nikhil Raut*, A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

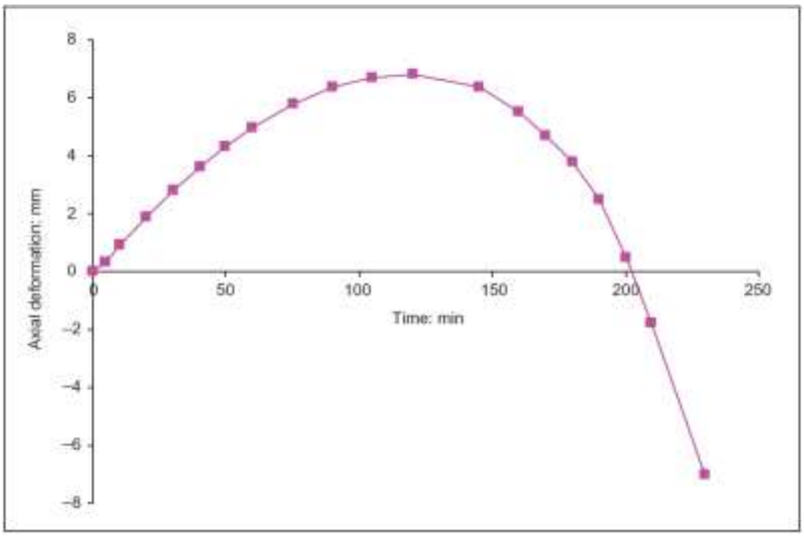


Figure 11: Thermal gradient after 45min into fire exposure in RC columns CE0, CE1, CE2, CE3 and CE4 [66]

It can be seen in the figure that for the most part of the fire duration the column expands. This expansion is due to thermal expansion of steel and concrete. Then the column starts contracting when the steel looses its strength as it is in the outer part of the section and is primarily affected by the temperature. A progressively increasing load is then redistributed to the concrete until the strength of the concrete also deteriorates and ultimately leads to the failure of the column.

Significant stiffness and strength reduction causes large deformation right before failure.

2.5.1 Factors Governing Fire Resistance

- Concrete cover thickness

This has been well established and been recognized in most codes and standards. Fire resistance increases with increased cover thickness. Its obvious that cover provides shield against temperature and deterioration of strength. [67]

[66] V. K. R. Kodur, N. K. Raut, *Design equation for predicting fire resistance of reinforced concrete columns, Structural Concrete 2009 10 No 2*

- Aggregate type

Of the two commonly used aggregates, carbonate aggregates provide higher fire resistance than siliceous aggregates. This is because of high heat capacity and low thermal conductivity of carbonate aggregates.

- Load ratio

Defined as the ratio of axial load acting on the column to the load carrying capacity of the column at ambient temperature. It is significant because it represents the stress extent of the column. Load ratio is directly proportional to fire resistance.[68]

- Slenderness ratio

Buckling becomes serious with deteriorating stiffness that comes from increased fire exposure duration. Slenderness ratio defined as the ratio of the effective length to the radius of gyration of the column has a direct influence on fire resistance.[69]

- Reinforcement ratio

Overall, the reinforcement ratio has a moderate effect on fire resistance of columns. However, an increase in reinforcement ratio for the same area of column leads to lower fire resistance. This is because steel loses it strength fast than concrete. [70]

- Effect of face exposure

Columns could be subjected to fire attacks in 1-, 2-, 3- or 4-faces due to their location in the building's structural layout. [71]

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[67] Venkatesh Kodur, Nikhil Raut, *A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

[68] V. K. R. Kodur, N. K. Raut, *Design equation for predicting fire resistance of reinforced concrete columns, Structural Concrete 2009 10 No 2*

[69] V. K. R. Kodur, N. K. Raut, *Design equation for predicting fire resistance of reinforced concrete columns, Structural Concrete 2009 10 No 2*

[70] V. K. R. Kodur, N. K. Raut, *Design equation for predicting fire resistance of reinforced concrete columns, Structural Concrete 2009 10 No 2*

[71] K.H. Tan, Y. Yao, *Fire Resistance of Reinforced Concrete Columns Subjected to 1-, 2-, and 3- Face Heating, J. Struct. Eng.2004.130: 1820-1828*

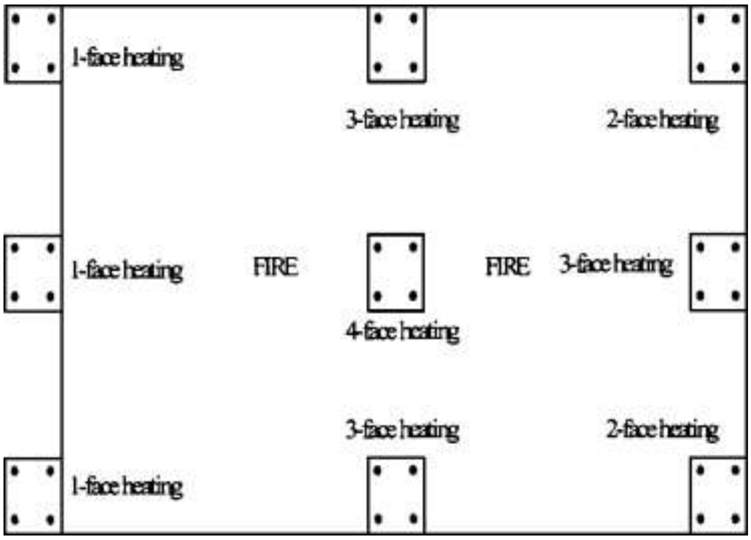


Figure 12: Columns under different thermal boundary conditions in a compartment [72]

Fire resistance analysis was carried out on fifteen columns of three different sizes and results were used to investigate the effect of exposure conditions on fire resistance. The findings indicate that fire resistance of columns exposed to fire in 1- or 3- faces have lower resistance than columns exposed to fire on all 4 sides. The resistance is even lower for columns exposed to fire in 2 adjacent sides. This is due to the fact that columns exposed to fire in 1- or 3- directions experience thermal gradient symmetric to one of the column's axes whereas columns exposed to fire in 2 adjacent sides experience asymmetric thermal gradient in both axes. [73]

[72] K.H. Tan, Y. Yao, *Fire Resistance of Reinforced Concrete Columns Subjected to 1-, 2-, and 3- Face Heating, J. Struct. Eng.2004.130: 1820-1828*

[73] Venkatesh Kodur, Nikhil Raut, *A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

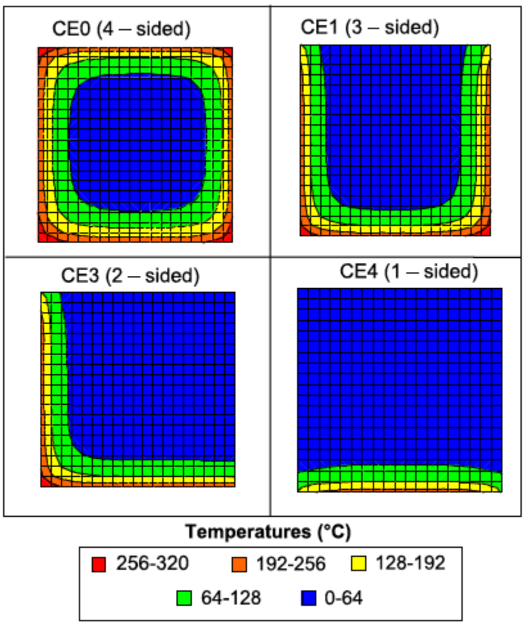


Figure 13: Thermal gradient after 45min into fire exposure in RC columns CE0, CE1, CE2, CE3 and CE4 [74]

The thermal gradients resulting in the column after 45min of fire exposure can be seen in the figure above. CE1, CE2, CE3 and CE4 are the IDs of the columns tested.

The gradient is symmetric along both axes for columns exposed on all four sides. For columns exposed in 1- or 3- sides the thermal gradient is only symmetrical in one of the axes, leading to uniaxial bending. For columns exposed to fire on 2 adjacent sides, thermal gradient is not symmetric on both axes therefore it leads to biaxial bending. However, columns exposed to fire on 2 opposite sides have the highest resistance because there is symmetry on both axes. In addition, the temperature rise in the cross section is lower as compared to 4- side fire exposure. [75]

[74] Venkatesh Kodur, Nikhil Raut, *A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

[75] Venkatesh Kodur, Nikhil Raut, *A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

- Effect of concrete strength

Results from fire tests indicated that fire resistance decreases with increased concrete strength. And for concrete strength above 70MPa the fire resistance decreases much more due to fire induced spalling. Concrete strength is indirectly related to permeability. Therefore fire resistance decreases with decrease in permeability.[76]

- Effect of bi-eccentric loading

Effect of eccentricity can be significant during fire because of reduced stiffness and strength of the column due to increased temperature during fire. The reduced strength and stiffness leads to increased lateral deflection which produces additional moment on the column and then causes additional deflection. And this effect is amplified even more when the column is subjected to eccentricity about both axes.[77]

2.5.2 Design According to Eurocode

For design of reinforced columns the Eurocode gives the choice of use of tabular, simplified or advanced methods.

a. Tabulated methods

Tabulated methods are developed from approximate conservative assumptions that can be used only for the most common structural members. They are based on empirical data proved by conducted tests and experience. Thus are prescriptive in a sense that they specify recommended geometric sizes of members and reinforcement

[76] Venkatesh Kodur, Nikhil Raut, *A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

[77] Venkatesh Kodur, Nikhil Raut, *A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending, Engineering Structures 41 (2012) 428-443*

cover that can only be applied to assure structural safety and can only be used in standard fire. [78]

Based on standard fire.

Tables are simple and used for common cases.

Tables provided in the Eurocode 2 are more flexible than the prescriptive tables as they are provided based on load ratio, i.e. the load for fire situation to the corresponding force for normal temperature. [79]

Specific to determining fire resistance of columns, two methods, Method A and Method B are provided in the Eurocode 2 Part 1-2 (EN 1991 1-2). Tables for braced structures only are provided for both methods that can be used in conjunction with set of rules for each. Both methods can only be used to assess columns in braced structures. [80]

Classes R30, R60, R90, R120, R180 and R240 are used to describe the fire resistance of structures, columns. Where R and the number stand for resistance and the number of minutes the structure should carry the load respectively.

In using these tables no further checks are required for shear and torsion and anchorage details. Spalling is also included in the design.

Method A has limitations on effective length, first order eccentricity and amount of reinforcement. A reduction factor is also employed to account for load combinations, compressive strength and bending including second order effects. Tabular data is given for circular or rectangular columns in terms of ratio of minimum column dimension to axis distance of the main bars for columns exposed on more than one

[78] *EN 1992-1-2:2004 (E), 38*

[79] The Concrete Center, *Concrete and Fire, 7*

[80] *EN 1992-1-2:2004 (E), 42*

side and columns exposed on one side. Empirical expressions are also given to calculate for values not given in the tables.

Method B limits load level, first order eccentricity under fire to account for axial load and first order bending for the load level of the column under normal temperature. Slenderness under fire is also limited. Second order effects have also been taken into account. Tabular data is then given for circular or rectangular columns in terms of minimum column dimension and axis distances based on mechanical ratio and load level.

b. Simplified calculation method

According to EN 1991 1-2 simplified cross-section methods determine the ultimate load bearing capacity of a heated cross section. [81]

There are two methods under the simplified calculation method for member analysis that can be used to assess heated cross section and determine the resistance to axial load and bending moment and their combination. The 500^0C isotherm method and the Zone method. [82]

The 500^0C isotherm method is applicable with both standard and parametric fires. But the Zone method is only applicable for standard fires and it's recommended to use small sections and slender columns.

In the 500^0C isotherm method, concrete section exposed to temperature greater than 500^0C is assumed to be ineffective and ambient temperature design is carried out for the remaining section. This method can only be applied to standard fire exposure. [83]

[81] EN 1992-1-2:2004 (E), 30

[82] EN 1992-1-2:2004 (E), 60

[83] EN 1992-1-2:2004 (E), 31

EN 1991 1-2 gives the following for values of the reduction factors for concrete and steel.

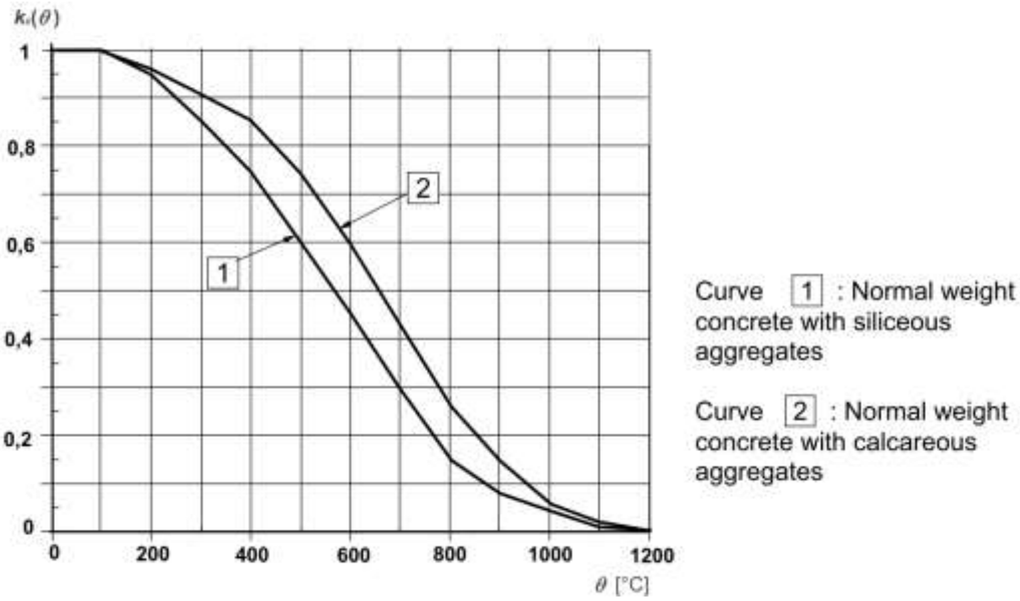


Figure 14: Coefficient $k_c(\Theta)$ allowing for decrease of characteristics strength (f_{ck}) of concrete [84]

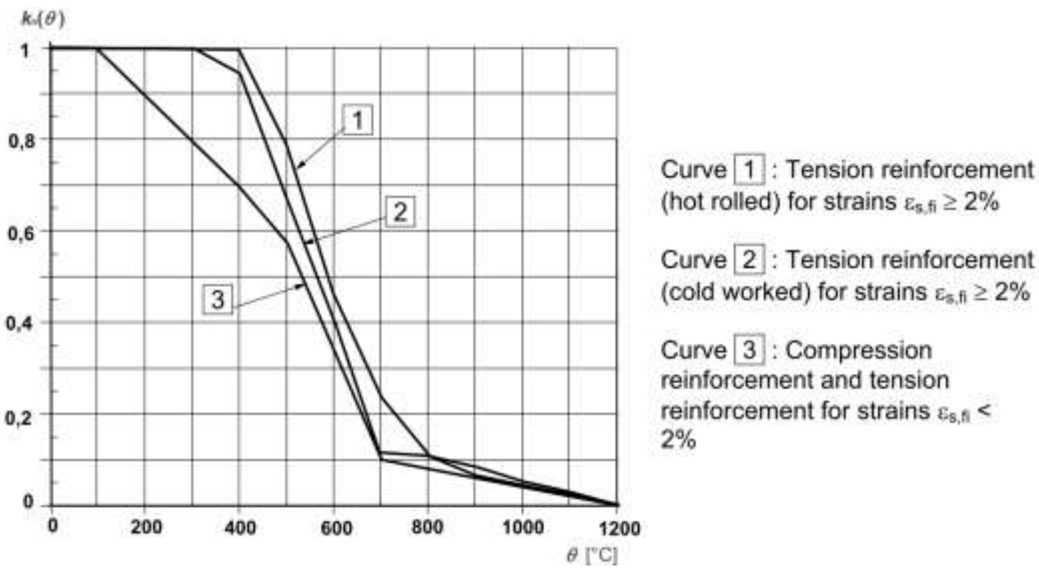


Figure 15: Coefficient $k_s(\Theta)$ allowing for decrease of characteristics strength (f_{yk}) of tension and compression reinforcement (Class N) [85]

[84] EN 1992-1-2:2004 (E), 72

[85] EN 1992-1-2:2004 (E), 33

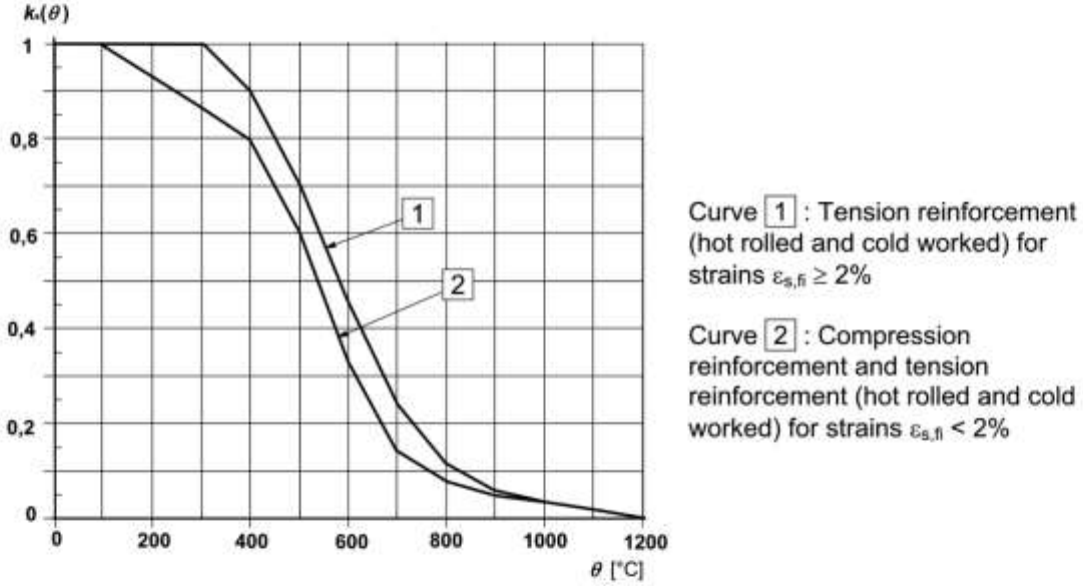


Figure 16: Coefficient $k_s(\Theta)$ allowing for decrease of characteristics strength (f_{yk}) of tension and compression reinforcement (Class X) [86]

In the Zone method, the cross-section is divided in to more than three zones and each zone is assessed with a relevant reduction for concrete and reinforcement at elevated temperature to a simplified cross section. This method can only be applied to standard temperature-time curve. Although more accurate than the 500^0C method the Zone method is very rigorous. [87]

No specific guideline to account for bending caused by the fire thermal gradient.

c. *Advanced calculation*

According to EN 1991 1-2 advanced calculation methods shall provide a realistic analysis of structures exposed to fire. They shall be based on fundamental physical behavior leading to a reliable approximation of the expected behavior of the relevant structural component under fire conditions. They include calculation models for the

[86] EN 1992-1-2:2004 (E), 33

[87] EN 1992-1-2:2004 (E), 76

determination of the development and distribution of the temperature within structural members (thermal response model) and the mechanical behavior of the structure or any part of it (mechanical response model). Advanced calculation methods maybe used with any heating curve and any type of cross section.[88]

Computer programmes are neccesary.

[88] EN 1992-1-2:2004 (E), 76

3 SIMPLIFIED DESIGN METHOD

3.1 General

In this thesis, a simplified design procedure for reinforced concrete columns under uniaxial bending during four-face fire heating is adopted. The step-by-step process is adopted from a journal of structural engineering, *Fire Resistance of Four-Face Heated Reinforced Columns* in accordance with the European codes.

The simplified design procedure in the journal is described as an extension of the American Concrete Institution (ACI) code 318-99 (ACI 2000) for column design at ambient temperature. Three major steps are used in this simplified method. First is the determination of strength and stability reduction factors for concrete and steel at elevated temperatures. Then, it is the calculation of balanced failure point of the column. Balanced failure point is calculated to determine whether the applied load eccentricity is small or large. Lastly, the actual failure load taking into account the slenderness of the column is determined by trial and error.

Temperature effects on concrete and steel with regard to strength and stability of the columns are quantified based on numerical simulations using a software called SAFIR. In addition, three case studies comprising a total of 64 reinforced concrete columns were then analyzed to verify the approach. The experimental results showed that the extended approach was not only accurate and consistent but also conservative. [89]

As mentioned, the procedures discussed above are adopted in this document in the Eurocode context and are followed to design selected sample column for the purpose of this thesis and finally will be compared with the experimental results of the paper.

[89] K.H. Tan, Y. Yao, *Fire Resistance of Four-Face Heated Reinforced Concrete Columns, J. Struct. Eng.2003.129: 1220-1229*

Temperatures of individual reinforcing bars in tension and compression steel is read from temperature profiles given in the code. As the heating experienced differs, corner bars experience heating from two faces while middle bars experience heating only from one face. Hence, reduction factors are also given specific for corner and middle bars differently.

After determining the temperatures of individual steel reduction factors for modulus of elasticity and yield strength are calculated using formulas in the Eurocode, EN 1992-1-2:2004 (E).

Concrete reduction factor for fire specified in EN 1992-1-2:2004 could not be used, as they don't address the overall simplified strength reduction in the cross section. As temperature distribution in concrete is non-uniform these reduction factors can only be used with the simplified methods of the Eurocode or with advanced models. Therefore the 500^0 isotherm method which is a simplified design method is used to take this into account and reduce the cross section of the effective concrete. For the remaining cross section concrete retains its original strength and deformation properties.

After taking the necessary reductions for concrete and steel, balanced failure point is calculated as done in the EN 1992-1-1 column design method. Lastly, failure point is determined by taking into account slenderness effects using the EN 1992-1-1 nominal stiffness method and using trial and error process.

3.2 Material Reduction Factors

As temperature increases during fire exposure, strength and stiffness of steel and concrete materials decreases. Material reduction factors are used to take this in to account.

3.2.1 Steel

Strength reduction factor for reinforcement steel is different for corner and middle bars. Corner bars experience direct heating from adjoining sides while middle bars experience heating from only one side. This can be taken in to account while determining temperature of individual bars.

For compression reinforcement and tension reinforcement for which $\varepsilon_{s,fi}<2\%$ EN 1992-1-2:2004 gives the following reduction factor corresponding to specific temperature.

Equation 2: Compression reinforcement strength reduction EN 1991-1-2-2002

$k_s(\theta) = 1.0$ for $20^0C \leq \theta \leq 100^0C$

$k_s(\theta) = 0.7 - 0.3 (\theta - 400)/300$ for $100^0C < \theta \leq 400^0C$

$k_s(\theta) = 0.57 - 0.13 (\theta - 500)/100$ for $400^0C < \theta \leq 500^0C$

$k_s(\theta) = 0.1 - 0.47 (\theta - 700)/200$ for $500^0C < \theta \leq 700^0C$

$k_s(\theta) = 0.1 (1200 - \theta)/500$ for $700^0C < \theta \leq 1200^0C$

Table is given in chapter 2 for parameters of the stress-strain relationship of reinforcing steel according to EN 1992-1-2:2004.

3.2.2 Concrete

There is non-uniform temperature distribution in concrete and the compressive strength at elevated temperature given in the Eurocode is not applicable to the entire cross section. Hence for the concrete the cross section is reduced as done in the 500^0C method while keeping the initial value of strength and deformation property of the concrete.

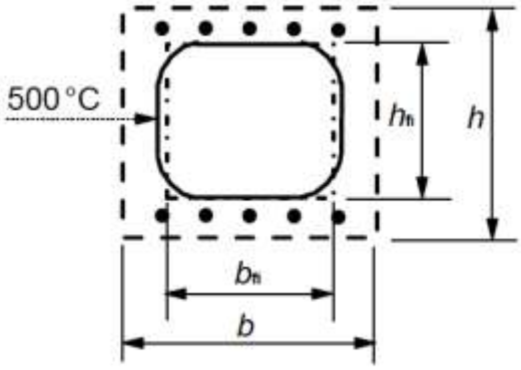


Figure 17: Reduced cross-section of reinforced concrete beam and column, fire exposure on four sides (beam or column) [90]

And for strain for strain failure criterion the following table is used from literature.

Table 3: Values of $\varepsilon_u(T)$ [91]

Concrete temperature (°C)	20	100	200	300	400	500	600	700	800	900	1,000
t_{ISO} (min)	0	0.16	0.29	0.69	1.45	2.95	5.87	11.6	22.7	44.3	86.5
$\varepsilon_u(T) \times 10^{-3}$	3.0	4.0	5.0	6.5	8.0	10.0	13.0	14.5	15.0	15.5	15.5

3.3 Cross section Balanced Failure Point

At elevated temperature unless the cross section is reduced, the transformation factor and the shapes of non-linear stress blocks are different from those at ambient temperature because the strength of the outer fiber deteriorates rapidly under fire attack. Thus compressive concrete stress in the outer fiber is assumed zero. The concrete strength reduction factor is not enough for column under axial load and bending, therefore in addition to strength reduction of the concrete the stress block is also transformed as shown below.

[90] *EN 1992-1-2:2004 (E), 73*

[91] K.H. Tan, Y. Yao, *Fire Resistance of Four-Face Heated Reinforced Concrete Columns, J. Struct. Eng.2003.129: 1220-1229*

Both compressive strength and stiffness of uniaxially stressed concrete decrease during fire. In addition if there is bending there should be an additional reduction to the cross section.

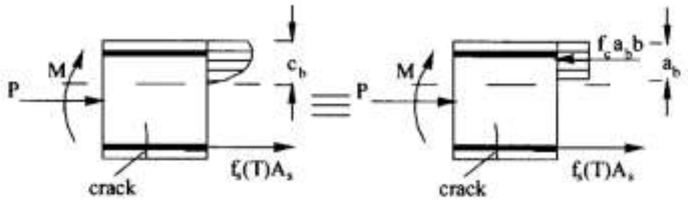


Figure 18: Transformation of stress block at ambient temperature [92]

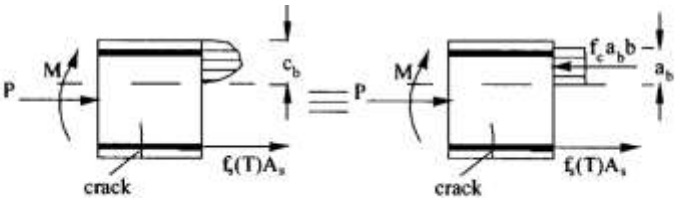


Figure 19: Transformation of stress block at elevated temperature [93]

But for concrete section where cross section has been reduced to effective cross-section this will not be necessary.

Therefore, the balanced failure point load and moment can be calculated as follows:

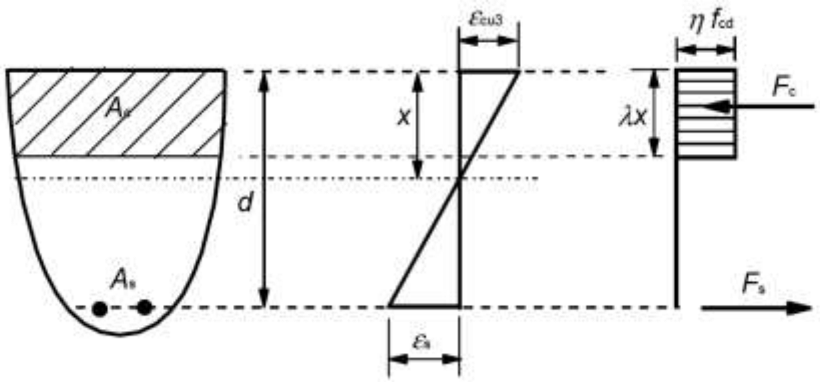


Figure 20: Rectangular stress distribution [94]

[92] K.H. Tan, Y. Yao, *Fire Resistance of Four-Face Heated Reinforced Concrete Columns, J. Struct. Eng.2003.129: 1220-1229*

[93] K.H. Tan, Y. Yao, *Fire Resistance of Four-Face Heated Reinforced Concrete Columns, J. Struct. Eng.2003.129: 1220-1229*

Equation 3: Balanced failure load

$P_b(T)= f_{ck}\lambda x b_{eff}+ A'_s f'_s(T)- A_s f_y(T)$

Equation 4: Balanced failure moment

$M_b(T)= f_{ck}\lambda x b_{eff}()+ A'_s f'_s(T)()- A_s f_y(T)()$

3.4 Slenderness

Using Eurocode nominal stiffness method, *EI* the nominal stiffness of slender compression members with arbitrary cross section is calculated.

Equation 5:Flexural rigidity

$EI = K_c E_{cd} I_c + K_s E_s I_s$

Where:

E_{cd} is the design value of the modulus of elasticity of concrete

I_c is the moment of inertia of concrete cross section

E_s is the design value of the modulus of elasticity of reinforcement

I_s is the second moment of area of reinforcement, about the centre of area of the concrete

K_c is a factor for effects of cracking, creep etc,

K_s is a factor for contribution of reinforcement

[94] EN 1992-1-1, 36

Provided ρ > 0.002:

$K_s = 1$

$K_c = k_1 k_2 / (1 + \varphi_{ef})$

Where:

ρ is the geometric reinforcement ratio, A_s/A_c

A_s is the total area of reinforecement

A_c is the area of concrete section

φ_{ef} is the effective creep ratio

k_1 is a factor which depends on concrete strength class

k_2 is a factor which depends on axial force and slenderness

$$k_1 = \sqrt{f_{ck}/20}$$

$$k_2 = n\frac{\lambda}{170} \le 0.20$$

where:

n is the relative axial force, $N_{Ed}/(A_c f_{cd})$

λ is the slenderness ratio

If the slenderness ratio λ is not defined, k_2 may be taken as

$k_2 = n0.3 \leq 0.20$

Provided ρ > 0.01:

K_s=0

K_c=0.3/(1+0.5φ_{ef})

The column buckling load N_B based on nominal stiffness:

Equation 6:Buckling load

$$N_B = \pi^2 EI/l_0^2$$

To include P-Δ effects the moment magnification factor is η introduced:

Equation 7:Moment magnification factor

$$\eta = 1 + \frac{\frac{\pi^2}{c_o}}{\frac{N_B}{N_u} - 1}$$

For equal end eccentricities in the same direction

e_{mag}= ηe, e is the load eccentricity

The value of e_b, eccentricity at balanced failure point is calculated.

e_b= M_b/ P_b

Then by comparing the applied load eccentricity e with e_b, the failure mode of the column can be identified. Then different neutral axis depth, c_n, are used to determine the failure point at temperature T by comparing M_n and M_{mag}.

M_{mag} =P_nηe.

3.5 Summary

The simplified procedure is briefly summarized below.

1. Determine the temperature field of the cross section.
2. Reduce the cross section.
3. Determine the temperatures of individual reinforcement.
4. Reduce strength and stiffness of reinforcement.
5. Calculate balanced failure load.
6. Calculate buckling load using nominal stiffness method.
7. Calculate actual failure load by trial and error.

3.6 Design Examples

The extended ACI calculation method in the literature was used to predict the resistance of rectangular reinforced columns. The calculation method was then checked against test results from three labs. It was then concluded that the method could predict the resistance of the columns accurately.

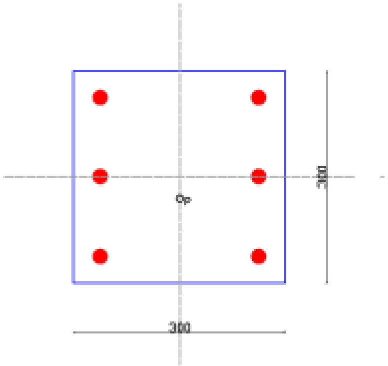
In this section five representative columns are taken from literature and designed according to the method in the EN 1992-1-2:2004 discussed in the previous chapter.

3.6.1 Column 1

The properties of the column are: Ac=0.3m x 0.3m= 0.09m^2, 6ϕ20, L=3.76m, d'=38mm, e=30mm, pinned at both ends, f_{ck}=24.1MPa, f_y=487MPa and E_s (20)= 2x10^{5}MPa.

The design of the column according to the method in the *EN 1992-1-2:2004* with procedures and steps as discussed in the previous chapter will be carried out and finally be compared with test results.

Section



Step 1:

Reduction Factors

Concrete

Cross section reduction

Using Fig A:13 of EN 1992-1-2:2004 for temperature profile, the cross section heated beyond 500^0C is reduced as shown below.

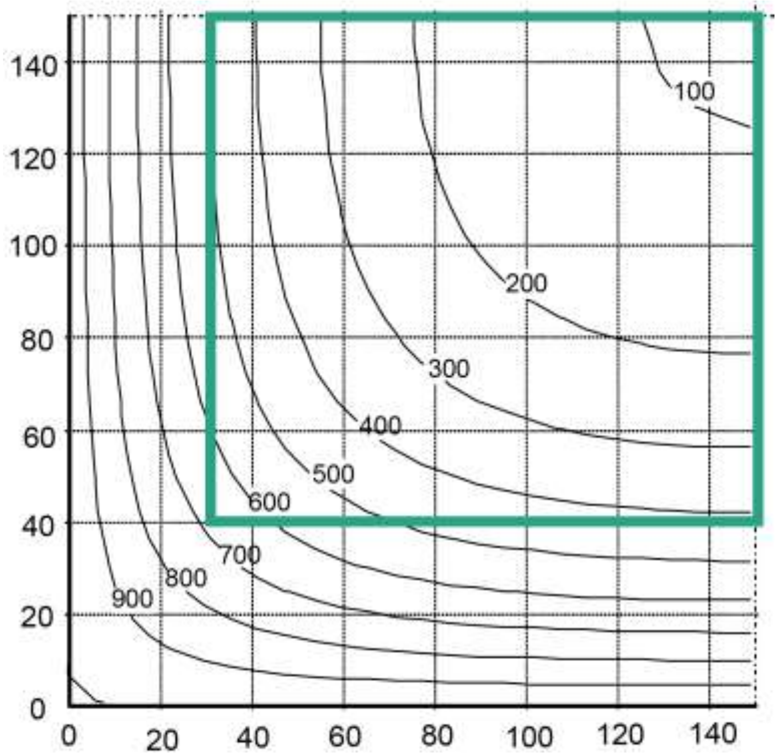
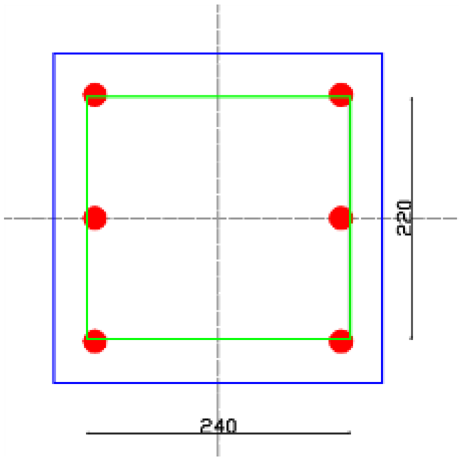


Figure 21: Temperature profiles (^{0}C) for a column, h x b =300 x 300 –R90 [95]

Shown in green is the reduced effective area, which is the approximation of the 500^{0}C isoline to a square effective cross section.

[95] *EN 1992-1-2:2004 (E), 69*

Reduced cross section



Since cross section is reduced no additional reduction is required for strength and modulus of elasticity of concrete.

Reinforcement

Determining the respective temperature of the reinforcement bars

Using Figure 21 the temperature of the reinforcement bars is determined depending on cover, cross section and exposure time.

Temperature of Reinforcement Bars

Corner bars: 650^0C

Middle bars: 400^0C

Using 4.2.4.3 of EN 1992-1-2:2004 reduced strength of the reinforcement due to the temperature rise is determined.

Tension reinforcement

Steel Temperature θ[°C]	$f_{sy,\theta}/f_{yk}$ hot rolled	$f_{sy,\theta}/f_{yk}$ cold worked	$f_{sp,\theta}/f_{yk}$ hot rolled	$f_{sp,\theta}/f_{yk}$ cold worked	$E_{s,\theta}/E_s$ hot rolled	$E_{s,\theta}/E_s$ cold worked
1	2	3	4	5	6	7
20	1,00	1,00	1,00	1,00	1,00	1,00
100	1,00	1,00	1,00	0,96	1,00	1,00
200	1,00	1,00	0,81	0,92	0,90	0,87
300	1,00	1,00	0,61	0,81	0,80	0,72
400	1,00	0,94	0,42	0,63	0,70	0,56
500	0,78	0,67	0,36	0,44	0,60	0,40
600	0,47	0,40	0,18	0,26	0,31	0,24
700	0,23	0,12	0,07	0,08	0,13	0,08
800	0,11	0,11	0,05	0,06	0,09	0,06
900	0,06	0,08	0,04	0,05	0,07	0,05
1000	0,04	0,05	0,02	0,03	0,04	0,03
1100	0,02	0,03	0,01	0,02	0,02	0,02
1200	0,00	0,00	0,00	0,00	0,00	0,00

Result is interpolated for corner bars with temperature 650^{0}C:

$$f_{sy,\theta}/f_{yk} = 0.35 \text{ and } E_{s\theta}/E_s = 0.22$$

For temperature 400^{0}C:

$$f_{sy,\theta}/f_{yk} = 1 \text{ and } E_{s\theta}/E_s = 0.7$$

Compression Reinforcement

$$ks(\theta) = 0.7 - 0.3(\theta - 400)/300 \text{ for } 100^0\text{C} < \theta \le 400^0\text{C}$$

$$ks(400) = 0.7 - 0.3(400 - 400)/300 = 0.7$$

$$ks(\theta) = 0.1 - 0.47(\theta - 700)/200 \text{ for } 500^0\text{C} < \theta \le 700^0\text{C}$$

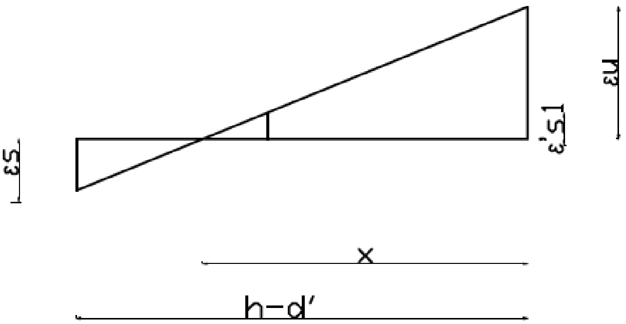
$$ks(650) = 0.1 - 0.47(650 - 700)/200 = 0.22$$

Step 2:

Balanced Failure Point

$$P_b(T) = f_{ck} l\, x b_{eff} + A'_s f'_s(T) - A_s f_y(T)$$

For balanced failure point:



For ISO 834 for 86min temperature is around 1000⁰C

Concrete temperature (°C)	20	100	200	300	400	500	600	700	800	900	1,000
t_{ISO} (min)	0	0.16	0.29	0.69	1.45	2.95	5.87	11.6	22.7	44.3	86.5
$\varepsilon_u(T) \times 10^{-3}$	3.0	4.0	5.0	6.5	8.0	10.0	13.0	14.5	15.0	15.5	15.5

$$e_s = e_y$$

$$e_y = \frac{f_y}{E} = \frac{487N/mm^2 * 0.35}{2*10^5 N/mm^2 * 0.22} = 3.87*10^{-3}$$

$$C_b = (h - d')\frac{e_u}{e_u + e_s}$$

$$e_u = 15.5*10^{-3}$$

$$C_b = (300mm - 38mm)\frac{15.5*10^{-3}}{(15.5*10^{-3} + 3.87*10^{-3})} = 210mm$$

Therefore middle bar is in compression.

$$e'_{s1} = e_u\frac{(C_b - d')}{C_b}$$

$$e'_{s1} = 15.5*10^{-3}\frac{(210 - 150)}{210} = 4.4*10^{-3}$$

For middle bar in compression:

$$\varepsilon_y = \frac{487N/mm^2 * 0.7}{2*10^5 N/mm^2 * 0.7} = 2.4*10^{-3}$$

$$\varepsilon'_{s1}(T) > \varepsilon_y(T)$$

$$f'_{s1}(T) = f_s(T) = 0.7*487N/mm^2 = 341N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 15.5*10^{-3} \frac{(210 - 38)}{210} = 12.7*10^{-3}$$

For corner bars in compression:

$$\varepsilon_y = \frac{f_y}{E} = \frac{487N/mm^2 * 0.22}{2*10^5 N/mm^2 * 0.22} = 2.4*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$

$$f'_s(T) = f_s(T) = 0.22*487N/mm^2 = 107N/mm^2$$

$$f_y(T) = 487N/mm^2 * 0.35 = 170N/mm^2$$

$$A'_s = A_s = 628mm^2$$

$$P_b(T) = 24.1N/mm^2 * 0.8 * 210mm * 220mm + 628mm^2 * 107N/mm^2 +$$

$$628mm^2 * 341N/mm^2 - 628mm^2 * 170N/mm^2$$

$$= 891kN + 67kN + 214kN - 107kN$$

$$= 1065kN$$

$$M_b(T) = 891kN(46mm) + 67kN(112mm) - 214kN(0) - 107kN(112mm)$$

$$= 41kNm + 7.5kNm - 12kNm$$

$$= 36.5kNm$$

Calculation of Euler load:

$$N_B = \frac{\pi^2 * EI}{L^2}$$

$$EI = K_c E_c I_c + K_s E_s I_s$$

$$\rho > 0.01$$

$$K_s = 0$$

$$K_c = 0.3/(1 + 0.5\varphi_{ef})$$

$$K_c = 0.3$$

$$EI = 0.3 * 31000 N/mm^2 * 220mm * (240mm)^3/12$$

$$= 2.36 * 10^{12} Nmm^2$$

$$N_B = \frac{\pi^2 * 2.36 * 10^{12} Nmm^2}{(3760mm)^2} = 1648kN$$

$$\delta = \frac{1}{1 - \frac{P_b}{N_B}} = \frac{1}{1 - \frac{1065}{1648}} = 2.8$$

$$e_{mag} = \delta * e = 2.8 * 30mm = 57mm$$

$$e_b = \frac{M_b}{P_b} = \frac{36.5kNm}{1065kN} = 34.3mm$$

$$e_{mag} > e_b$$

Therefore, the neutral axis depth is smaller than that at balanced failure. The eccentricity is defined as large eccentricity.

The column tends to fail in steel yielding mode.

Step 3:

Failure Point

Trial 1: C_n=160mm

$$e_u = 160mm \frac{3.87 * 10^{-3}}{(300mm - 38mm - 160mm)} = 6.07 * 10^{-3}$$

$$\varepsilon'_{s1} = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_{s1} = 6.07*10^{-3} \frac{(160 - 150)}{160} = 3.8*10^{-4}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$

$$f'_{s1}(T) = 3.8*10^{-4}*0.7*2*10^5 N/mm^2 = 53N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 6.07*10^{-3} \frac{(160 - 38)}{160} = 4.6*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$

$$f'_s(T) = f_s(T) = 0.22*487N/mm^2 = 107N/mm^2$$

$$P_n = 24.1N/mm^2*0.8*160mm*220mm + 628mm^2*107N/mm^2 +$$

$$628mm^2*53N/mm^2 - 628mm^2*170N/mm^2$$

$$= 679kN + 67kN + 33kN - 107kN$$

$$= 672kN$$

$$M_n = 679kN(46mm) + 67kN(112mm) - 33kN(0) - 107kN(112mm)$$

$$= 31kNm + 8kNm - 12kNm$$

$$= 27kNm$$

$$M_{mag} = \frac{672*30}{1 - \frac{672}{1648}} = 34$$

P_n should be lesser

Trial 2: C_n=130mm

$$\varepsilon_u = 130mm \frac{3.87*10^{-3}}{(300mm - 38mm - 130mm)} = 3.8*10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$

$$\varepsilon'_{s1} = \frac{3.8*10^{-3}*20}{130} = 5.8*10^{-4}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$

$$f'_{s1}(T) = 5.8*10^{-4}*0.22*2*10^5 N/mm^2 = 25N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 3.8*10^{-3}\frac{(130-38)}{130} = 2.7*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$

$$f'_s(T) = f_s(T) = 0.22*487N/mm^2 = 107N/mm^2$$

$$P_n = 24.1N/mm^2*0.8*130mm*220mm + 628mm^2*107N/mm^2 -$$

$$628mm^2*25N/mm^2 - 628mm^2*170N/mm^2$$

$$= 551kN + 67kN - 16kN - 107kN$$

$$= 495kN$$

$$M_n = 551kN(46mm) + 67kN(112mm) - 16kN(0) - 107kN(112mm)$$

$$= 25kNm + 8kNm - 12kNm$$

$$= 21kNm$$

$$M_{mag} = \frac{495*30}{1-\frac{495}{1648}} = 21$$

After a few iterations, the column failure load converged to 495kN.

3.6.2 Column 2

The properties of the column are: Ac=0.3m x 0.3m= 0.09m^2, 6ϕ20, L=4.76m, d'=38mm, e=30mm, pinned at both ends, f_{ck}=24.1MPa, f_{yk}=487MPa and E_s (20)= 2x10^{5}MPa.

Material reduction factors and the balanced failure load is the same as the prior example.

$$P_b(T) = 1065kN$$

$$M_b(T) = 36.5kNm$$

Calculation of Euler load:

$$N_B = \frac{\pi^2 * EI}{L^2}$$
$$EI = K_c E_c I_c + K_s E_s I_s$$
$$\rho > 0.01$$
$$K_s = 0$$
$$K_c = 0.3/(1+0.5\varphi_{ef})$$
$$K_c = 0.3$$
$$EI = 0.3 * 31000 N/mm^2 * 220mm * (240mm)^3/12$$
$$= 2.36 * 10^{12} Nmm^2$$

$$N_B = \frac{\pi^2 * 2.36 * 10^{12} Nmm^2}{(4760mm)^2} = 1028kN$$
$$\delta = \frac{1}{1 - \frac{P_b}{N_B}} = \frac{1}{1 - \frac{1065}{1028}} = 28$$
$$e_{mag} = \delta * e = 28 * 30mm = 840mm$$
$$e_b = \frac{M_b}{P_b} = \frac{36.5kNm}{1065kN} = 34.3mm$$
$$e_{mag} > e_b$$

Therefore, the neutral axis depth is smaller than that at balanced failure. The eccentricity is defined as large eccentricity.

The column tends to fail in steel yielding mode.

Step 3:

Failure Point

Trial 1: C_n=140mm

$$e_u = 140mm \frac{3.87 * 10^{-3}}{(300mm - 38mm - 140mm)} = 4.44 * 10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$

$$\varepsilon'_{s1} = \frac{4.44*10^{-3}*10}{140} = 3.17*10^{-4}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$

$$f'_{s1}(T) = 3.17*10^{-4}*0.22*2*10^5 N/mm^2 = 14N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 4.44*10^{-3}\frac{(140-38)}{140} = 3.2*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$

$$f'_s(T) = f_s(T) = 0.22*487N/mm^2 = 107N/mm^2$$

$$P_n = 24.1N/mm^2*0.8*140mm*220mm + 628mm^2*107N/mm^2 -$$

$$628mm^2*14N/mm^2 - 628mm^2*170N/mm^2$$

$$= 594kN + 67kN - 9kN - 107kN$$

$$= 545kN$$

$$M_n = 545kN(46mm) + 67kN(112mm) - 9kN(0) - 107kN(112mm)$$

$$= 25kNm + 7.5kNm - 12kNm$$

$$= 20.5kNm$$

$$M_{mag} = \frac{545*30}{1-\frac{545}{1028}} = 35$$

P_n should be lesser.

Trial 2: C_n=100mm

$$\varepsilon_u = 100mm\frac{3.87*10^{-3}}{(300mm - 38mm - 100mm)} = 2.39*10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$

$$\varepsilon'_{s1} = \frac{2.39*10^{-3}*50}{100} = 1.2*10^{-3}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$

$$f'_{s1}(T) = 1.2*10^{-3}*0.22*2*10^5 N/mm^2 = 53N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 2.39*10^{-3}\frac{(100 - 38)}{100} = 1.48*10^{-3}$$

$$\varepsilon'_s(T) < \varepsilon_y(T)$$

$$f'_s(T) = 1.48*10^{-3}*0.22*2*10^5 N/mm^2 = 65N/mm^2$$

$$P_n = 24.1N/mm^2*0.8*110mm*220mm + 628mm^2*79N/mm^2 -$$
$$628mm^2*44N/mm^2 - 628mm^2*170N/mm^2$$
$$= 466kN + 50kN - 28kN - 107kN$$
$$= 381kN$$
$$M_n = 466kN(46mm) + 50kN(112mm) - 28kN(0) - 107kN(112mm)$$
$$= 22kNm + 6kNm - 12kNm$$
$$= 16kNm$$

$$M_{mag} = \frac{381*30}{1 - \frac{381}{1062}} = 18$$

Trial 3: C_n=90mm

$$\varepsilon_u = 90mm\frac{3.87*10^{-3}}{(300mm - 38mm - 90mm)} = 2.03*10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$

$$\varepsilon'_{s1} = \frac{2.03*10^{-3}*60}{90} = 1.35*10^{-3}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$

$$f'_{s1}(T) = 1.35*10^{-3}*0.22*2*10^5 N/mm^2 = 60N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 2.03*10^{-3}\frac{(90 - 38)}{90} = 1.17*10^{-3}$$

$$\varepsilon'_s(T) < \varepsilon_y(T)$$

$$f'_s(T) = 1.17 * 10^{-3} * 0.22 * 2 * 10^5 N/mm^2 = 51N/mm^2$$

$$P_n = 24.1N/mm^2 * 0.8 * 90mm * 220mm + 628mm^2 * 51N/mm^2 -$$
$$628mm^2 * 60N/mm^2 - 628mm^2 * 170N/mm^2$$
$$= 382kN + 32kN - 38kN - 107kN$$
$$= 269kN$$
$$M_n = 382kN(46mm) + 32kN(112mm) - 38kN(0) - 107kN(112mm)$$
$$= 18kNm + 4kNm - 12kNm$$
$$= 10kNm$$
$$M_{mag} = \frac{269 * 30}{1 - \frac{269}{1028}} = 10$$

3.6.3 Column 3

The properties of the column are: Ac=0.3m x 0.3m= 0.09m^2, 6ϕ20, L=4.76m, d'=38mm, e=30mm, pinned at both ends, f_{ck}=30.7MPa, f_{yk}=462MPa and E_s (20)= 2x10^{5}MPa.

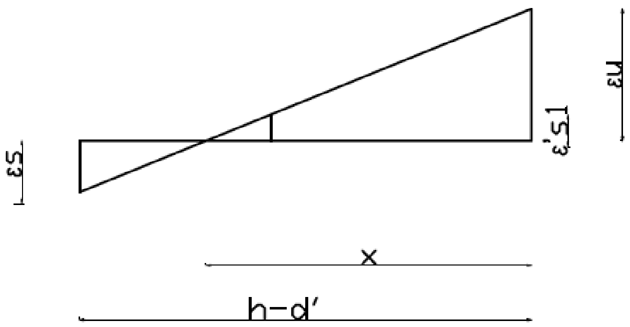
Since the section properties are the same with the previous sample, the material reduction factors are the same.

Step 2:

Balanced Failure Point

$$P_b(T) = f_{ck}\lambda x b_{eff} + A'_s f'_s(T) - A_s f_y(T)$$

For balanced failure point:



$$\varepsilon_s = \varepsilon_y$$

$$\varepsilon_y = \frac{f_y}{E} = \frac{462N/mm^2 * 0.35}{2*10^5 N/mm^2 * 0.22} = 3.68*10^{-3}$$

$$C_b = (h - d')\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_s}$$

$$\varepsilon_u = 15.5*10^{-3}$$

$$C_b = (300mm - 38mm)\frac{15.5*10^{-3}}{(15.5*10^{-3} + 3.68*10^{-3})} = 212mm$$

Therefore middle bar is in compression.

$$\varepsilon'_{s1} = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_{s1} = 15.5*10^{-3}\frac{(212 - 150)}{212} = 4.5*10^{-3}$$

For middle bar in compression:

$$\varepsilon_y = \frac{462N/mm^2 * 0.7}{2*10^5 N/mm^2 * 0.7} = 2.3*10^{-3}$$

$$\varepsilon'_{s1}(T) > \varepsilon_y(T)$$

$$f'_{s1}(T) = f_s(T) = 0.7 * 462N/mm^2 = 323N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 15.5*10^{-3}\frac{(212 - 38)}{212} = 12.7*10^{-3}$$

For corner bars in compression:

$$\varepsilon_y = \frac{f_y}{E} = \frac{462N/mm^2 * 0.22}{2*10^5 N/mm^2 * 0.22} = 2.3*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$

$$f'_s(T) = f_s(T) = 0.22*462N/mm^2 = 102N/mm^2$$

Calculation of Euler load:

$$N_B = \frac{\pi^2 * EI}{L^2}$$

$$EI = K_c E_c I_c + K_s E_s I_s$$

$$\rho > 0.01$$

$$K_s = 0$$

$$K_c = 0.3/(1+0.5\varphi_{ef})$$

$$K_c = 0.3$$

$$EI = 0.3*31000N/mm^2 * 220mm*(240mm)^3/12$$

$$= 2.36*10^{12} Nmm^2$$

$$N_B = \frac{\pi^2 * 2.36*10^{12} Nmm^2}{(4760mm)^2} = 1028kN$$

$$\delta = \frac{1}{1-\frac{P_b}{N_B}} = \frac{1}{1-\frac{1310}{1028}} = 3.6$$

$$e_{mag} = \delta * e = 3.6*30mm = 108mm$$

$$e_b = \frac{M_b}{P_b} = \frac{49kNm}{1310kN} = 37mm$$

$$e_{mag} > e_b$$

Therefore, the neutral axis depth is smaller than that at balanced failure. The eccentricity is defined as large eccentricity.

The column tends to fail in steel yielding mode.

Step 3:

Failure Point

Trial 1: C_n=140mm

$$\varepsilon_u = 140mm \frac{3.68*10^{-3}}{(300mm - 38mm - 140mm)} = 4.22*10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$

$$\varepsilon'_{s1} = \frac{4.22*10^{-3}*10}{140} = 3.01*10^{-4}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$

$$f'_{s1}(T) = 3.01*10^{-4}*0.22*2*10^5 N/mm^2 = 13N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 4.22*10^{-3}\frac{(140 - 38)}{140} = 3.07*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$

$$f'_s(T) = f_s(T) = 0.22*462N/mm^2 = 102N/mm^2$$

$$P_n = 30.7N/mm^2*0.8*140mm*220mm + 628mm^2*102N/mm^2 -$$
$$628mm^2*13N/mm^2 - 628mm^2*162N/mm^2$$
$$= 756kN + 64kN - 8kN - 102kN$$
$$= 710kN$$
$$M_n = 756kN(46mm) + 64kN(112mm) - 8kN(0) - 102kN(112mm)$$
$$= 35kNm + 7kNm - 11kNm$$
$$= 31kNm$$
$$M_{mag} = \frac{710*30}{1 - \frac{710}{1028}} = 69kNm$$

P_n should be lesser.

Trial 2: C_n=100mm

$$e_u = 100mm \frac{3.68*10^{-3}}{(300mm - 38mm - 100mm)} = 2.27*10^{-3}$$

$$e'_{s1} = \frac{e_u s}{C_b}$$

$$e'_{s1} = \frac{2.27*10^{-3}*50}{100} = 1.14*10^{-4}$$

$$e'_{s1}(T) < e_y(T)$$

$$f'_{s1}(T) = 1.14*10^{-3}*0.22*2*10^5 N/mm^2 = 50N/mm^2$$

$$e'_s = e_u \frac{(C_b - d')}{C_b}$$

$$e'_s = 2.27*10^{-3}\frac{(100 - 38)}{100} = 1.4*10^{-3}$$

$$e'_s(T) < e_y(T)$$

$$f'_s(T) = 1.4*10^{-3}*0.22*2*10^5 N/mm^2 = 62N/mm^2$$

$$P_n = 30.7N/mm^2*0.8*100mm*220mm + 628mm^2*62N/mm^2 -$$
$$628mm^2*50N/mm^2 - 628mm^2*162N/mm^2$$
$$= 540kN + 39kN - 31kN - 102kN$$
$$= 446kN$$
$$M_n = 540kN(46mm) + 39kN(112mm) - 31kN(0) - 102kN(112mm)$$
$$= 25kNm + 4kNm - 11kNm$$
$$= 18kNm$$

$$M_{mag} = \frac{446*30}{1 - \frac{446}{1028}} = 24kNm$$

Trial 3: C_n=80mm

$$\varepsilon_u = 80mm \frac{3.68*10^{-3}}{(300mm - 38mm - 80mm)} = 1.6*10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$

$$\varepsilon'_{s1} = \frac{1.6*10^{-3}*70}{80} = 1.4*10^{-3}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$

$$f'_{s1}(T) = 1.4*10^{-3}*0.22*2*10^5 N/mm^2 = 61N/mm^2$$

$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$

$$\varepsilon'_s = 1.6*10^{-3}\frac{(80-38)}{80} = 8.4*10^{-4}$$

$$\varepsilon'_s(T) < \varepsilon_y(T)$$

$$f'_s(T) = 8.4*10^{-4}*0.22*2*10^5 N/mm^2 = 37N/mm^2$$

$$P_n = 30.7N/mm^2*0.8*80mm*220mm + 628mm^2*37N/mm^2 -$$

$$628mm^2*61N/mm^2 - 628mm^2*162N/mm^2$$

$$= 432kN + 23kN - 38kN - 102kN$$

$$= 315kN$$

$$M_n = 432kN(46mm) + 23kN(112mm) - 38kN(0) - 102kN(112mm)$$

$$= 21kNm + 3kNm - 11kNm$$

$$= 13kNm$$

$$M_{mag} = \frac{315*30}{1-\frac{315}{1028}} = 13kNm$$

3.6.4 Column 4

The properties of the column are: Ac=0.3m x 0.3m= 0.09m^2, 6ϕ20, *L*=4.76m, *d´*=38mm, *e*=15mm, pinned at both ends, f_{ck}=30.7MPa, f_{yk}=462MPa and E_s (20)= 2x10^{5}MPa.

Since the section properties and length of the column are the same with the previous sample: the material reduction , balanced failure point and buckling load are the same.

$$P_b(T) = 1310kN$$
$$M_b(T) = 49kNm$$
$$N_B = 1028kN$$

Trial 1: C_n=155mm

$$\varepsilon_u = 155mm \frac{3.68*10^{-3}}{(300mm - 38mm - 155mm)} = 5.33*10^{-3}$$

$$\varepsilon'_{s1} = \varepsilon_u \frac{(C_b - d')}{C_b}$$
$$\varepsilon'_{s1} = 5.33*10^{-3} \frac{(155 - 150)}{155} = 1.7*10^{-4}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$
$$f'_{s1}(T) = 1.7*10^{-4}*0.7*2*10^5 N/mm^2 = 24N/mm^2$$
$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$
$$\varepsilon'_s = 5.33*10^{-3} \frac{(155 - 38)}{155} = 4.02*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$
$$f'_s(T) = f_s(T) = 0.22*462N/mm^2 = 102N/mm^2$$

$$P_n = 30.7N/mm^2 * 0.8 * 155mm * 220mm + 628mm^2 * 102N/mm^2 +$$
$$628mm^2 * 24N/mm^2 - 628mm^2 * 162N/mm^2$$
$$= 837kN + 64kN + 15kN - 102kN$$
$$= 814kN$$
$$M_n = 837kN(46mm) + 64kN(112mm) + 15kN(0) - 102kN(112mm)$$
$$= 39kNm + 7kNm - 11kNm$$
$$= 35kNm$$
$$M_{mag} = \frac{814 * 15}{1 - \frac{814}{1028}} = 57kNm$$

Trial 2: C_n=120mm

$$\varepsilon_u = 120mm \frac{3.68 * 10^{-3}}{(300mm - 38mm - 120mm)} = 3.1 * 10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$
$$\varepsilon'_{s1} = \frac{3.1 * 10^{-3} * 30}{120} = 7.75 * 10^{-4}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$
$$f'_{s1}(T) = 7.75 * 10^{-4} * 0.22 * 2 * 10^5 N/mm^2 = 34N/mm^2$$
$$\varepsilon'_s = \varepsilon_u \frac{(C_b - d')}{C_b}$$
$$\varepsilon'_s = 3.1 * 10^{-3} \frac{(120 - 38)}{120} = 2.1 * 10^{-3}$$

$$\varepsilon'_s(T) < \varepsilon_y(T)$$
$$f'_s(T) = 2.1 * 10^{-3} * 0.22 * 2 * 10^5 N/mm^2 = 47N/mm^2$$

$$P_n = 30.7N/mm^2 * 0.8 * 120mm * 220mm + 628mm^2 * 47N/mm^2 -$$
$$628mm^2 * 34N/mm^2 - 628mm^2 * 162N/mm^2$$
$$= 648kN + 30kN - 21kN - 102kN$$
$$= 555kN$$
$$M_n = 628kN(46mm) + 30kN(112mm) - 21kN(0) - 102kN(112mm)$$
$$= 29kNm + 3kNm - 11kNm$$
$$= 21kNm$$
$$M_{mag} = \frac{555*15}{1-\frac{555}{1028}} = 18kNm$$

P_n should be greater

Trial 2: C_n=130mm

$$e_u = 130mm\frac{3.68*10^{-3}}{(300mm - 38mm - 130mm)} = 3.6*10^{-3}$$

$$e'_{s1} = \frac{e_u s}{C_b}$$
$$e'_{s1} = \frac{3.6*10^{-3}*20}{130} = 5.5*10^{-4}$$

$$e'_{s1}(T) < e_y(T)$$
$$f'_{s1}(T) = 5.5*10^{-4}*0.22*2*10^5 N/mm^2 = 24N/mm^2$$
$$e'_s = e_u\frac{(C_b - d')}{C_b}$$
$$e'_s = 3.6*10^{-3}\frac{(130-38)}{130} = 2.5*10^{-3}$$

$$e'_s(T) > e_y(T)$$
$$f'_s(T) = f_s(T) = 0.22*462N/mm^2 = 102N/mm^2$$

$$P_n = 30.7N/mm^2 * 0.8 * 130mm * 220mm + 628mm^2 * 102N/mm^2 -$$
$$628mm^2 * 24N/mm^2 - 628mm^2 * 162N/mm^2$$
$$= 702kN + 64kN - 15kN - 102kN$$
$$= 649kN$$
$$M_n = 702kN(46mm) + 64kN(112mm) - 15kN(0) - 102kN(112mm)$$
$$= 32kNm + 7kNm - 11kNm$$
$$= 28kNm$$
$$M_{mag} = \frac{649*15}{1-\frac{649}{1028}} = 26kNm$$

P_n should be greater

Trial 3: C_n=135mm

$$\varepsilon_u = 135mm\frac{3.68*10^{-3}}{(300mm - 38mm - 135mm)} = 3.9*10^{-3}$$

$$\varepsilon'_{s1} = \frac{\varepsilon_u s}{C_b}$$
$$\varepsilon'_{s1} = \frac{3.9*10^{-3}*15}{135} = 4.3*10^{-4}$$

$$\varepsilon'_{s1}(T) < \varepsilon_y(T)$$
$$f'_{s1}(T) = 4.3*10^{-4}*0.22*2*10^5 N/mm^2 = 19N/mm^2$$
$$\varepsilon'_s = \varepsilon_u\frac{(C_b - d')}{C_b}$$
$$\varepsilon'_s = 3.9*10^{-3}\frac{(135-38)}{135} = 2.8*10^{-3}$$

$$\varepsilon'_s(T) > \varepsilon_y(T)$$
$$f'_s(T) = f_s(T) = 0.22*462N/mm^2 = 102N/mm^2$$

$$P_n = 30.7N/mm^2 * 0.8 * 135mm * 220mm + 628mm^2 * 102N/mm^2 -$$
$$628mm^2 * 19N/mm^2 - 628mm^2 * 162N/mm^2$$
$$= 729kN + 64kN - 12kN - 102kN$$
$$= 679kN$$
$$M_n = 729kN(46mm) + 64kN(112mm) - 12kN(0) - 102kN(112mm)$$
$$= 34kNm + 7kNm - 11kNm$$
$$= 30kNm$$
$$M_{mag} = \frac{679 * 15}{1 - \frac{679}{1028}} = 30kNm$$

3.6.5 Result Comparison

In this section, a comparison between experimental results and evaluation using the proposed method is carried out. Experimetal results are taken from a renowned source, Technical University of Braunschweig.

Column	b (mm)	h (mm)	Rebar (mm)	L (m)	f_{ck} (N/mm^2)	f_{yk} (N/mm^2)	End	d (mm)	e (mm)	P_{app} (kN)	t_{test} (min)	P_{cal} (kN)	Percentage Difference
1	300	300	6ϕ20	3.76	24.1	487	P-P	38	30	710	86	495	-2.15
2	300	300	6ϕ20	4.76	24.1	487	P-P	38	30	650	63	269	-3.81
3	300	300	6ϕ20	4.76	30.7	462	P-P	38	30	650	80	315	-3.35
4	300	300	6ϕ20	4.76	30.7	462	P-P	38	15	740	85	679	-0.61
5	300	300	6ϕ20	4.76	38.2	404	P-P	38	5	1224	48	1556	3.32

4 DISCUSSION

In structural design, fire is accounted by using temperature time curves. Nominal temperature time curves are the most basic curves used, of which Standard fire cure or ISO curve is the mostly used curve for fire design. Like all nominal temperature curves, Standard temperature curve or ISO curve doesn't take temperature variation in the compartment into account i.e. it assumes constant temperature in the whole compartment. In addition, fire load, ventilation and cooling phase are also not taken into account. Despite this, Standard temperature curve or ISO curve is used in fire resistance tests. It is also used to check resistance of structural members or building during fire and to analyze behavior of structures during increased temperature.

Fire design can be done in two ways, prescriptive i.e. using tables and prescribed solutions that can provide solution when there is a deviation from the prescribed i.e. specific solution for a specific design. Results with prescriptive methods like tables maybe safe at times but could produce results that are too safe and uneconomical.

When we come to material properties, concrete thermal expansion is complicated. Aggregate used, cement paste and rate of heating complicate its property. Cement paste expands till 150^0C and contracts between 150^0C-400^0C, whereas the aggregate might still be expanding. This creates differential thermal expansion. And concrete doesn't disintegrate at this differential expansion because of creep of the paste. Above a certain temperature creep is significant in concrete, concrete creep consists of the cement paste and aggregate creep.

For concrete, modulus of elasticity reduces during fire but unlike steel the strength and stiffness deterioration is related to all elements of the cross section due to the non-uniform temperature distribution as its obvious that the outer fibers are the highly affected.

As for steel, strength and stiffness reduction factor is different for corner and middle bars. Corner bars experience direct heating from adjoining sides while middle bars experience heating from only one side. This can be taken in to account while determining temperature of individual bars.

For reinforced concrete columns, the Eurocode, EN 1992-1-2:2004, gives a choice of tabular methods i.e. tables and equations, simplified methods and advanced methods for fire design. The current most commonly used design practice for evaluating resistance of reinforced concrete columns is based on tabular methods. The tables are a function of column width and axis of main bars. Two tabular methods are specified, Method A which gives fire rating provisions for columns with options for exposure in one side and more than one side and Method B which gives results in terms of load and mechanical reinforcement ratio.

Both methods can only be used for most common dimensions. Although fast and easy to use they don’t take into account significant factors like fire scenario and concrete strength in determining column resistance. Different fire scenarios, exposures in different faces can cause uniaxial or biaxial bending and concrete strength could also affect resistance. Eccentricity limitation is also an additional concern while using these tables.

On the other hand advanced models can be realistic but very complex in nature. They require computer programs and high computational methods that can be difficult for daily use.

This makes simplified methods very important and a helpful tool for the designer.

In EN 1992-1-2:2004 there are two simplified methods, 500^0C isotherm and the Zone method. The Zone method is recommended for small sections and slender columns.

The 500^0C isotherm method assumes concrete section exposed to temperature greater than 500^0C as ineffective and the remaining section is designed with normal temperature design method.

In the Zone method the cross section is divided in zones and each zone is assessed with specific reduction for concrete and reinforcement. It is more accurate than the 500^0C but due to the above it can be very rigorous.

Simplified methods in EN 1992-1-2:2004 specify the importance of second order effects caused by the fire action but do not specify how to take these into account in detail.

Both strength and stiffness of concrete and steel materials reduces during fire. While dealing with reinforced concrete columns, these columns might be subjected to uniaxial or biaxial bending in addition to axial load. And when in fire these columns can be heated in one, two, three or four sides based on their location in the fire compartment. The different faces of heating cause thermal gradient in the column, which in turn causes fire induced bending. One or two-sided exposure can result in additional uniaxial bending whereas fire on two adjacent sides can cause biaxial bending.

During fire, reinforced concrete columns expand for the most part of the fire duration due to expansion of steel and concrete. The steel is highly affected by temperature increase due to fire because it is in the outer surface and is easily attacked by fire. Then the column starts to contract as the temperature of the steel increases and loses its strength. The load is then redistributed to the concrete until the concrete also loses its strength. This significant stiffness and strength reduction can also lead to large deformation, and columns that weren't susceptible to second order effects during normal temperature can be susceptible during fire because of this. Buckling becomes serious with deteriorating stiffness that comes from increased fire exposure duration. The force bending moment, P-M interaction diagram contracts during fire due to this.

In general, the strength of the outer fiber of concrete deteriorates due to fire and average reduced compressive strength of concrete can be used. And for column under bending the cross section has to also be reduced, as the stress block distribution is different from that of the same section at normal temperature. In some cases compressive strength of the outer fiber is assumed zero or the cross section is reduced to account for this reduction.

The main aim of this paper was to present the simplified method for fire design of reinforced concrete columns and also show the relevance of fire design.

A simplified approach for fire design of uniaxial four face heated reinforced concrete columns is provided. As mentioned above, while designing columns different faces of fire exposures could arise and columns are structural elements where second order effect affects capacity. Hence this is taken into account.

The method is a step-by-step procedure extracted from the Eurocode. The EN 1992-1-2:2004, 500^0C simplified method is used together with the EN 1992-1-1:2004 column design method for ambient temperature to come up with a more simplified and detailed fire design method for reinforced concrete column design. The step-by-step procedure is adopted from a journal of structural engineering, *Fire Resistance of Four-Face Heated Reinforced Columns.*

Three major steps are used, determination of reduced strength and stiffness of concrete and steel, balanced failure point and the actual failure load.

Temperatures of individual bars in tension and compression is read from temperature profiles for both corner and middle bars. Modulus of elasticity and yield strength are modified with respective reduction factors. The concrete section is reduced with 500^0C isotherm method and no additional reduction is included for concrete strength and deformation properties.

Then balanced failure load is calculated as would in normal temperature design. Lastly failure load is determined by taking slenderness into account and by using the nominal stiffness method of the Eurocode.

The Eurocode method while calculating the flexural ridigty is a bit sensitive as creep coefficient is variable. For temperature less than 500^0C creep might not have any significant effect but as temperature increases or if other methods are used that do not discard concrete section with temperature greater than 500^0C, creep effect need to be studied.

In the examples studied, the ultimate load and bending moment carrying capacity was successfully determined. It can be seen from the comparison result in section 3.6.5 that the experimental result and results with simplified method are close for some cases. However, it can also be seen that as the length of the column increases, comparison between column 1 and 2, this method can be a little conservative. This could be due to significant cross section reduction. As column 2 is more sensitive to second order effects. And for column with smaller eccentricity, it might overestimate capacity. Approximate results with experiment were achieved for relatively shorter columns i.e. column 1.

5 SUMMARY

Fire design of reinforced concrete columns is especially important in high-rise buildings.

In general, there are two methods for structural fire design, prescriptive and performance. In recent years structural design is going towards performance based design as it allows freedom to the designer not to mention that, it is also economical.

The most common design method for reinforced concrete columns is tabular methods that are more prescriptive in nature. This makes simplified methods helpful tools because of simplicity of use and performance based nature. There major factors governing fire resistance of reinforced concrete columns that much attention hasn't been given to in the past, with the help of simplified methods these factors can be taken into account.

In designing reinforced concrete columns for fire, the material properties of concrete and steel need to be modified. In addition for section subjected to bending the stress block also needs to be modified. But there are limitations in the Eurocode on the availability of temperature profiles in determining cross sectional and reinforcement bar temperatures.

Comparison results for the four different columns from the Technical University of Braunschweig, with result from the simplified procedure were consistent and indicated that the method is approximate for some cases and at times a little conservative.

It is worth mentioning that specifically as the length of the columns increases the capacity with the simplified procedure reduced significantly. Hence, the simplified method could be conservative for slender columns. And for columns with only compression it might overestimate capacity. But it gave approximate results for relatively shorter columns.

The nominal stiffness method, which was used to determine the buckling load, is found to be sensitive to creep. Furthermore, concrete beyond 500^0C is susceptible to creep and it needs to be taken into account upon using other methods.

6 RECOMMENDATION

Simplified methods can further be extended to 1-, 2-, or 3- sided fire exposures by taking into account fire induced bending's. Detailed modification for strength and stiffness reduction of concrete can also be added as there is limited information on temperature profiles. Use of software for determining these factors can be useful in the future. Further more, the effect of creep during fire can be studied, as slender column design is sensitive to long-term effect.

Properties like concrete strength and bi-eccentric loading can also be further studied.

Fire design is mandatory and should not be neglected. Risks should be minimized as to damage caused to human beings and buildings due to fire. Simplified methods can take into account the true behavior of the column during fire and allow freedom of design for the designer as compared to tables, which are prescriptive in nature. This paper can guide calculations for fire capacity of columns for the designer and I encourage the development of these methods with further studies in the future.

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