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Structural Strengthening & Rehabilitation

Experimental Data on Fire-Resistance Behavior of Reinforced Concrete Structures with Example Calculations

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To my parents, professors,

colleagues and friends

Contents

Part One: Review and Analysis on Fire-Resistance Behavior of Reinforced Concrete

1. Introduction

1.1. Fire-Resistance Research in the World

With high-speed development of modern technology, specially perfect development of combustion science, computation science and experimental technology, lay the foundation for research and development of fire mechanisms and regularities. Professor Emmons of Harvard University first integrates mass conservation, moment conservation, energy conservation and chemical reactions into the research of building fire, he's called "father of fire technology".

In all over the world, Japan, America, Britain, France, Italy, Russia do the researches on extent of structural damage and fire-resistance rating. Japanese fire-resistance research begins from 1940', sets 9 research and testing organizations, area of fire-resistance testing building is 2581m^2,huge sized fire testing building 4963m^2 and 1 outdoor fire testing building(100m x 90m); in September of 1980, ACI structural fire safety committee convened the symposium[1] about extent of structural damage and fire-resistance rating; Russia does huge research on technology identification of fire damage building, and in 1985 published " Fire Damage Building Technology Identification ". Then in 1987 Russian national reinforced concrete institute established " Building after Fire Concrete Structure Identification Standard ", this standard ensures the principles and calculation methods after fire followed by structure identification[2].

European international standard organization and England also establish relative standards to fire-resistance analysis of steel structure and fire-resistance test of construction materials. Research on fire-resistance properties of reinforced concrete begins from 1950'[3]. America has an early start on fireproof research of reinforced concrete structure, American New York Construction Bureau's engineers Ira H Wool son and Rudolph P Miller first suggested to build one common standard testing procedure, be convenient to process structure test and increase comparability of test results. In 1905 American Society for Testing and Materials (ASTM) set up special commission to standardize test methods. In 1916, one united commission composted by 11 construction groups integrated test methods of each component, proposed standard temperature-time curve, that is ASTME119 curve. This curve becomes standard curve[4] of fire research for America. Since 1960, lots of American colleges and institutes have begun to research on fire-resistance performance of fire concrete structure. Research and Development Laboratory of Portland Cement Association since 1960' has begun to research on physical and mechanical properties of concrete materials under high temperature, at the same time research on fire-resistance performance of reinforced concrete and pre-stress concrete components. In 1981, ACI committee 216 established " Guide for Determining the Fire Endurance of Concrete Elements "[5], after three modified versions, the guide becomes standard for ensuring components fire-resistance performance. In 1997 ACI committee 216 published standard for ensuring design and analysis procedure of fire-resistance performance of concrete and masonry components, at the same time, test technology has also improved .

Since 1991, Europe has promulgated structure exposed to fire design standard Eurocode, let fireproof design have basis.

Since 1940, Japan has researched on fire test research and structure inner temperature presumption method to the reinforced concrete. In 1973 established " Fire-Resistance Building Structure Method ", introduced systematically building structure fire temperature presumption method and building material high temperature performance.

In terrorist attacks event of 9.11.2001, New York World Trade Center northern and southern buildings have undergone violent impact due to two aircrafts, explosion, and combustion, caused serious casualties and economic losses. Afterwards, U.S. Federal Emergency Management Agency and American Society of Civil

Engineers jointly published a research report. The report said, due to the unique design and construction features, the two buildings after violent impacts by two aircrafts did not collapse immediately; the reason of collapse of buildings is that the effects of aircrafts impacts and induced fire. After aircrafts impacts, two buildings supported the effects of impacts for a long time. The northern building of World Trade Center supported for 122 min, southern building supported just for 57 min. In this period of time, although external supports components of building are destroyed, the main structure success to separate strength to other support points. If there is no effect of fire, probably the structures of two buildings can be preserved for ever, beside effects of serious earthquake and storms. Therefore, high temperature fire damage is the direct reason for collapse.

After the terrorist attacks event, engineering designers and researchers of all over the world pay much more attentions to high temperature fire damage and destruction researches of large engineering structures. Under fire complexity of temperature changes and distribution, theoretically huge difficulties of obtaining different structures and mechanical properties and constitutive relationship of several materials under various high temperature environments, and huge expenses and complex technologies of test researches have great effects on research development. Recently many designers and researches do lots of researches on fire resistance properties and structural designs under fire. Therefore researching and summarizing the degeneration regularity of reinforced concrete structure material properties under or after fire are very necessary, it can lay foundation for deeply developing researches of new type reinforced concrete structural fire resistance properties and its damage assessment.

1.2. Fire-Resistance Research in China

Due to high-frequency occurrence and high-combustion of fire events (Fig.1.1 and Fig.1.2), fire-resistance researches become more and more important in China.

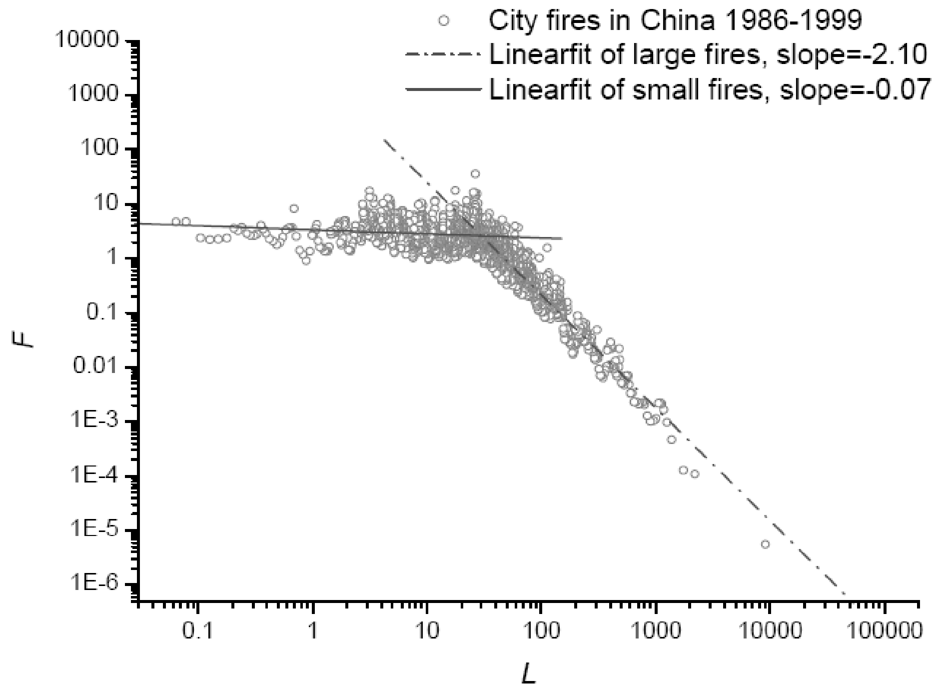

Fig.1.1 Statistical distribution of China fires[6]

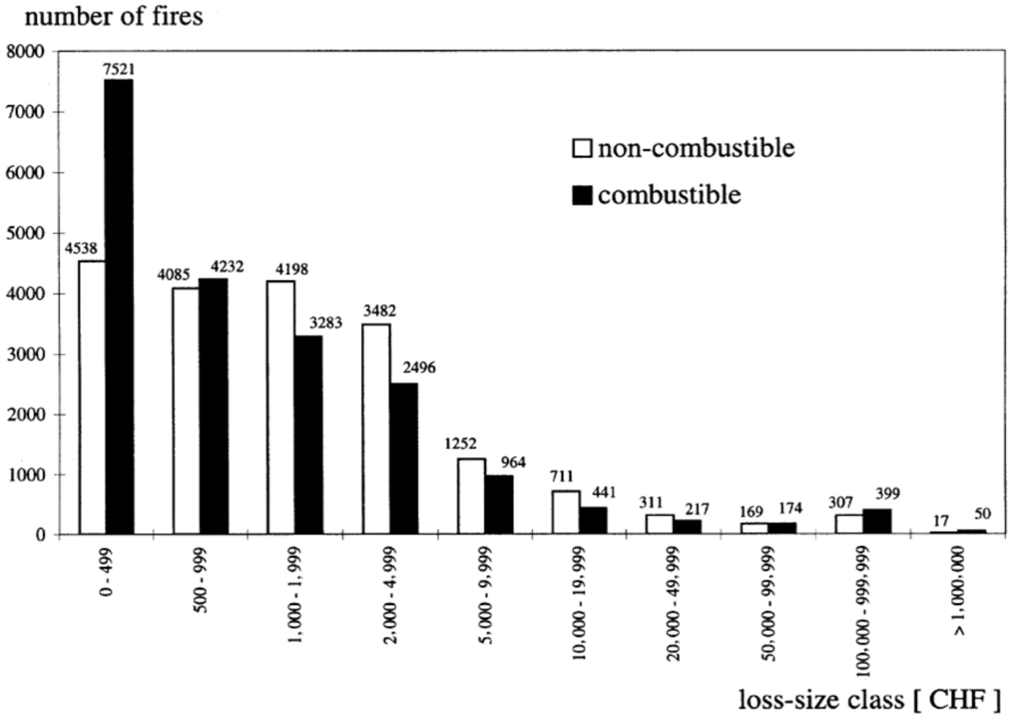

Fig.1.2 High-combustion of Fire Events[6]

Compared with other countries, China has a late start in research for fire-resistance behavior of reinforced concrete.

In 1960s,some institutes such as China Metallurgical Construction Research Institute did researches and tests on strength of concrete under high temperature, they researched and analyzed the effects of high temperature to plant structure and chimneys, and in 1970s compiled " Specification on Reinforced Concrete Structures of Metallurgical Industrial Plants Anti-Thermal Design " which indicates design methods and specifications for reinforced concrete structure between 60-200°C; four research institutes belonging to Ministry of Public Security had mainly done research on buildings for fire compartments, building products and fire resistance level for structural components, but less research on fire resistance for steel, concrete and other construction materials; in the late of 1980s, Tongji University, Tsinghua University, Southwest Jiaotong University and Harbin Architecture University had done lots of researches on materials performance for steel and concrete under and after high temperature, achieved fruitful results ,although the test data is relatively wide, the total data trend has certain regularity.

Concrete is composite construction material, composed by the cement (commonly the Portland cement) and other cementitious materials such as the fly ash and the slag cement, the aggregate (generally the coarse aggregate made by the gravel or crushed rocks such as the limestone, or the granite, plus the fine aggregate such as the sand), the water and the chemical admixtures. The concrete solidifies and hardens after mixing with the water and the placement due to the chemical process known as the hydration. The cement after the hydration is seen as composition of the gel mechanism and a number of capillaries. The gel gaps are narrow and interconnected, their maximum width is 18×10^{-10}m, it is five times of diameter of water molecules[7]. The water reacts with the cement, which bonds the other components together, eventually creating the robust stone-like material.

Inside the concrete there are mainly three kinds of the water[8]:

(1) Non-evaporated water due to role of chemical bond, it is one part of the cement gel formed during the cement hydration, it is amount of 5%, that is crystal water.

(2) It is evaporated water full of gaps and it is attached mainly by Van Der Waals force.

(3) It is the free water that exists in gaps and capillaries. Therefore, concrete actually is complex structure including solid ,liquid and gas.

Under high temperature the thermal properties, the mechanical properties of concrete and steel are able to change obviously. The main mechanical properties (compressive strength, tensile strength, elastic modulus) all reduce with increase of temperature.

For the concrete, when the temperature is 600°C, the compressive strength decreases about 50%; when the temperature is 800°C, the compressive strength decreases about 80% [9]. Even the steel strength under high temperature decreases greatly, and the plasticity increases. The tests data and a large number of fire examples verify that the bare steel structure under fire, the high temperature which lasts about 15 minutes, loses the capacity and occurs damage and destruction. The mechanical properties and the deformation under high temperature have effects on concrete structure, and particularly on fire-resistance properties of pre-stress concrete structure, it is the main factor of component control design under high temperature. Therefore, learning about high temperature properties of concrete and steel is very important.

The material properties of concrete and steel under high temperature are the basis of research on high temperature reinforced concrete structural behaviors.

This thesis selects reinforced concrete structure as research subjects, mainly commentary and discusses the property changes of steel and concrete materials under and after high temperature.

2. Concrete High Temperature Thermal Properties

The materials thermal properties are the premise of the structural heat conduction, the temperature response and the analysis. The concrete high temperature thermal parameters involve mainly heat conduction coefficient λ_c, heat capacity C_c, mass density ρ_c and thermal expansion coefficient α_c.

When concrete stays in non-uniform temperature environment or environment temperature changes, between internal materials and between structures and borders media will occur heat exchange. With the change of concrete internal temperature field, thermal properties indexes also change; and changes of thermal properties indexes will impact distribution of concrete internal temperature.

2.1. Thermal Conductivity

Thermal conductivity is the property of the material's ability to the conduct heat. It appears primarily in the Fourier's Law for the heat conduction. Heat transfer across the materials of high thermal conductivity occurs at a faster rate than across the materials of low thermal conductivity. It is expressed by the units of $\frac{W}{m°\mathrm{C}}$.

Heat conduction coefficient λ_c function is mainly driven by aggregate type, moisture content, concrete mix rate, temperature and so on.

The tests indicate that when temperature is below 700℃, heat conduction coefficient decreases linearly with increase of temperature. The heat conduction coefficient of different kinds of concrete varies more than 50% as a function of composition.

For the concrete with certain fixed composition, its water content becomes the main effective factor for heat conduction coefficient: the effects of temperature below 100℃ are stronger than those of temperature over 100℃, and the higher the temperature, the weaker the effects[10]. This is why with increase of temperature free water inside concrete is vaporized. Structural buildings under fire reach several hundred degrees in short time[11]. So for the reinforced concrete structures under fire, concrete heat conduction coefficient can ignore effects due to water.

In Eurocode 3, relation[12] between concrete heat conduction coefficient and temperature for different aggregate types is shown in Fig.2.1:

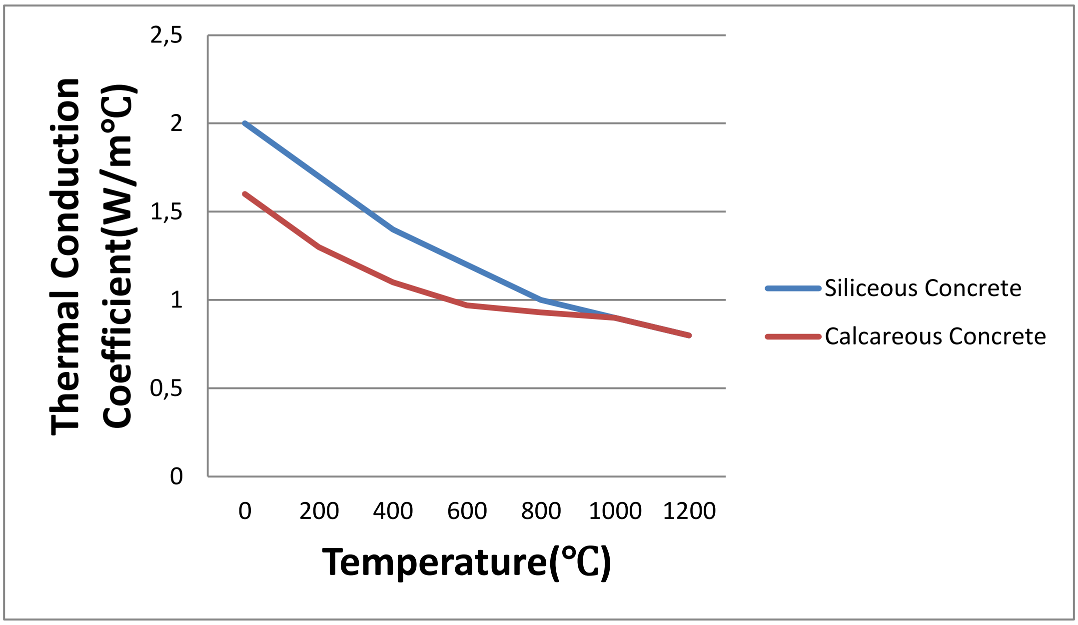

Fig.2.1 Relation between concrete conduction coefficient and temperature

Siliceous Concrete,

$$\lambda_c = 2 - 0.24\frac{T}{120} + 0.012 \times \left(\frac{T}{120}\right)^2$$

$$20°\text{C} \leq T \leq 1000°\text{C}$$

Calcareous Concrete,

$$\lambda_c = 1.6 - 0.16\frac{T}{120} + 0.008 \times \left(\frac{T}{120}\right)^2$$

$$20°\text{C} \leq T \leq 1000°\text{C}$$

As can be seen from Fig. 2.1, calcareous aggregates have better fire-resistance behavior than siliceous aggregates. When the temperature increases, the effects of aggregates reduce gradually.

Beside lightweight aggregates, the aggregates of general concrete have little effects on heat conductivity. The relation[13] between concrete heat conduction coefficient and temperature is following:

$$\lambda_c = 1.9 - 0.00085\text{T} \qquad (0 - 800^\circ\text{C})$$

$$\lambda_c = 1.22 \qquad (\geq 800^\circ\text{C})$$

Heat conduction coefficient of siliceous aggregate concrete is a little greater than that of calcium, but when the temperature increases, the effects reduce, except for lightweight aggregates, ignore the effects of general aggregates.

In 1989, according to Fourier Law, Lu zhou-dao[14] derived general concrete heat conduction coefficient by steady state protected hot plate test method:

$$\lambda_c = 1.6 - (0.6/850)\text{T} \qquad (\text{W/m}\ ^\circ\text{C})$$

2.2. Heat Capacity

In the formula of heat conduction control, heat capacity and mass density come out by the type of integration, therefore, they usually are handled as one term in the research. Heat capacity or thermal capacity, is measurable and physical quantity that characterizes the amount of the heat required to change the substance temperature by a given amount. Heat capacity is the heat energy accumulated in a unit mass of material when temperature increases 1 degree, and in fact it represents the ability to absorb heat. It is expressed in units of $\frac{J}{kg{\cdot}℃}$. Heat capacity increases gradually with increase of temperature(in Fig.2.2), increases with increase of heat capacity of aggregates, concrete density and moisture content rate[12] .

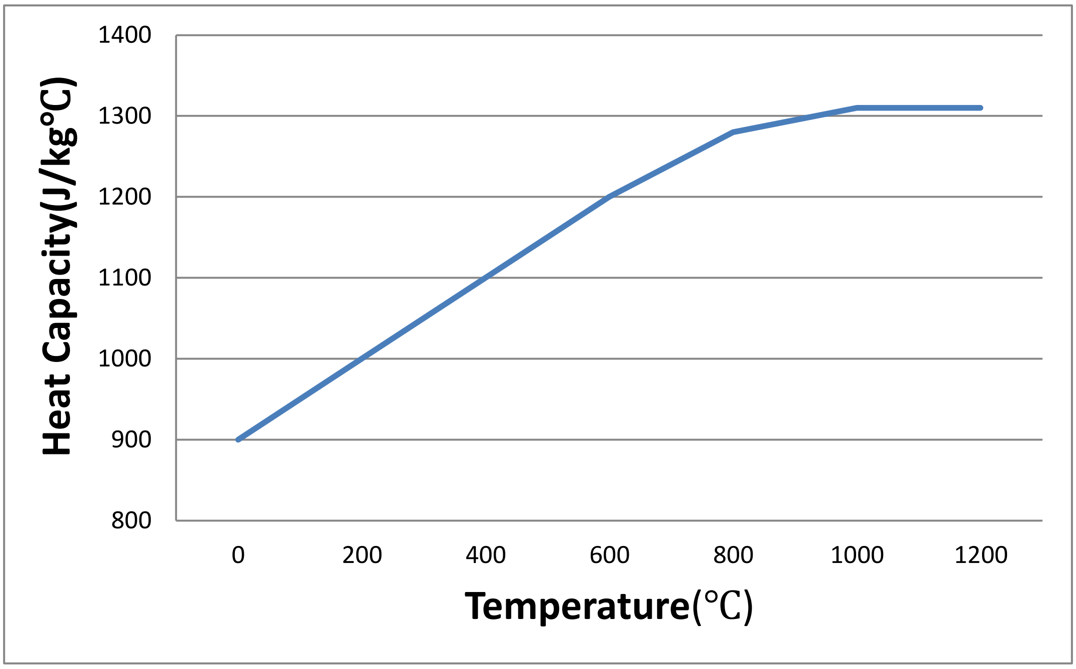

Fig. 2.2 Relation between concrete heat capacity and temperature

Overall, aggregate type and concrete moisture content rate have little effects on specific heat capacity, heat capacity of lightweight aggregates is relatively small; in general, heat capacity from literature doesn't consider their effects.

Several tests indicate that when temperature is below 800℃ , concrete aggregate types have little influence on heat capacity; when temperature is over 800℃, calcium aggregates accelerate the dehydration.

Concrete mix rate has effects on heat capacity. It is because that when inside concrete there is great amount of cement mortar, under high temperature it is easy to occur dehydration, therefore concrete with high water-cement ratio has high heat capacity. When temperature is below 200℃, concrete water content has greater effects on heat capacity, for concrete with high water content when temperature is over 100℃, its heat capacity is 2 times of dry concrete[15]. Concrete heat capacity formula given in literature usually just consider relation between concrete heat capacity and temperature. In Eurocode 3, the following formulae are introduced:

$$C_c = 900 + 80\frac{T}{120} - 4\left(\frac{T}{120}\right)^2$$

$$20℃ \leq T \leq 1200℃$$

In [14], the heat capacity is introduced in the formula with a linear relation:

$$C_c = 0.2 + \frac{0.1}{850}T \ [\frac{kcal}{kg}°C]$$

Or

$$C_c = 836 + 0.49T \ [\frac{J}{kg}°C]$$

And in [13] a connection among mass density ρ_c and heat capacity C_c is introduced in form of multiplication, to get relation between them and temperature with the function:

$$\rho_c C_c = (0.005T + 1.7) \times 10^6$$

$$0°C \leq T \leq 200°C$$

$$\rho_c C_c = 2.7 \times 10^6$$

$$200°C \leq T \leq 400°C$$

$$\rho_c C_c = (0.013T - 2.5) \times 10^6$$

$$400°C \leq T \leq 500°C$$

$$\rho_c C_c = (-0.013T + 10.5) \times 10^6$$

$$500℃ \leq T \leq 600℃$$

$$\rho_c C_c = 2.7 \times 10^6$$
$$T > 600℃$$

In 1986, according to heat balance principle Lu Zhou-dao[14] derived ordinary concrete heat capacity with magnetic stirred water calorimeter:

$$C_c = 0.2 + (0.1/850)\mathrm{T} \qquad (\mathrm{kcal/kg\ ^\circ C})$$

Or

$$C_c = 836 + 0.49\mathrm{T} \qquad (\mathrm{J/kg\ ^\circ C})$$

It has little change, usually $C_c = 724/\rho_c$.

2.3. Thermal Expansion Coefficient

Thermal expansion coefficient describes by how much material will expand for each degree of temperature increase. It is expressed by units of 1/°C.

Volume of concrete expands due to one heating procedure, its regularity of thermal strain is similar, but strain value is different due to different aggregates, temperature expansion deformation of concrete with lightweight aggregates is very small. When temperature is below 200℃, solid compositions inside concrete (coarse aggregates and cement mortar) occurs volume expansion, but it shrinks due to dehydration, the previous two phenomena cancels each other out, the deformation increases slowly; when temperature is over 300℃, solid compositions inside concrete continue to expand, occurring and development of internal cracks make the deformation develop fast; when temperature is over 600 ℃, temperature expansion deformation of several concrete becomes slower, even stops, probably crystalline mineral components of aggregates change, or accumulation of internal damage impede continue to expand[16]. Thermal expansion deformation actually is average expansion deformation.

In Eurocode 3, consider effects due to different aggregate types, carry out relation between heat expansion coefficient of concrete and temperature:

Siliceous Concrete,

$$\alpha_c = -1.8 \times 10^{-4} + 9 \times 10^{-6} T + 2.3 \times 10^{-11} T^3$$

$$20℃ \le T \le 700℃$$

$$\alpha_c = 14 \times 10^{-3}$$

$$700℃ \leq T \leq 1200℃$$

Calcareous Concrete,

$$\alpha_c = -1.2 \times 10^{-4} + 6 \times 10^{-6}T + 1.4 \times 10^{-11}T^3$$

$$20℃ \leq T \leq 805℃$$

$$\alpha_c = 12 \times 10^{-3}$$

$$805℃ \leq T \leq 1200℃$$

As can be seen in Fig.2.3, thermal expansion coefficient of concrete with different aggregates increases with increase of temperature, when temperature is over 800℃, its value approximates one constant.

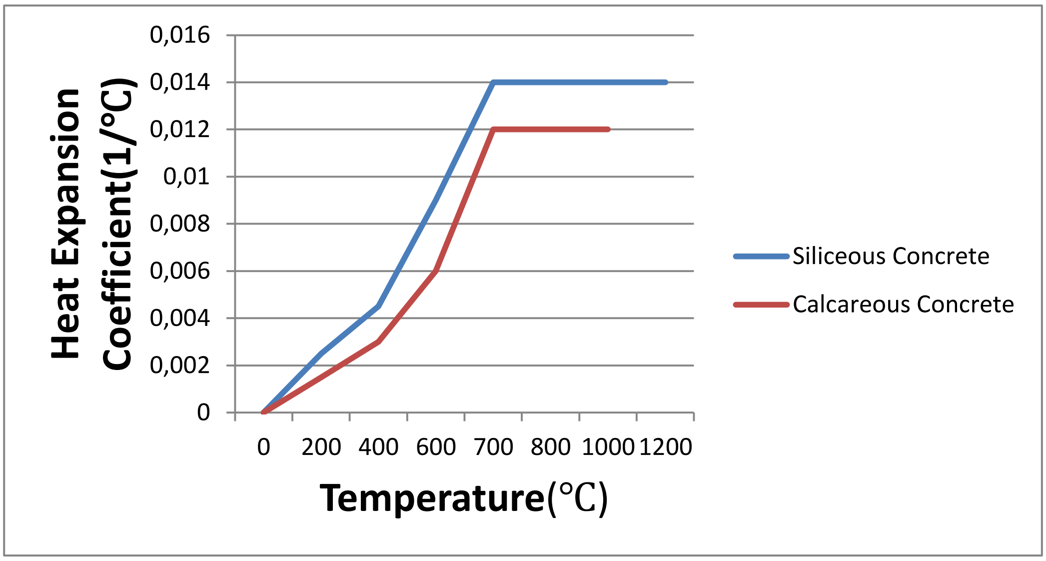

Fig. 2.3 Relation between concrete heat expansion coefficient and temperature

Thermal expansion coefficient α_c not only has relation with concrete ratio and age, but also with specimen size, heating rate, density, environment condition and so on, different specimens have different test results. There are many effect factors, generally to calculate simply just consider effects due to high temperature, then derive relationship.

Lu zhou-dao[14] derives two linear expressions:

$$\alpha_c = 1.0 \times \left(\frac{0.1}{850}\right) \mathrm{T} \qquad (0 - 400°\mathrm{C})$$

$$\alpha_c = 2.5 \times 10^{-5} T - 0.006 \qquad (400 - 700°\mathrm{C})$$

2.4. Mass Density

Concrete mass density ρ_c means object mass of unite volume, it can be expressed by $\frac{kg}{m^3}$.

Concrete under high temperature lets the water evaporate, so its mass density decreases. The decrease of mass density of concrete with general lightweight aggregates is a little greater than that of ordinary concrete. In spite that overall effects of temperature are great, when we calculate high temperature reactions, usually we consider mass density of concrete as constant.

3. Concrete High Temperature Mechanical Properties

Concrete mechanical properties include tensile strength, compressive strength, elastic modulus, Poisson's ratio and constitutive relation, among them compressive strength is the most basic and important one. In recent years, many fire resistance tests and strength tests of concrete materials were completed, as is summarized in Table 3.1.

Table 3.1 List of test mechanical behavior of concrete at high temperature

Investigator (a)	Test item (b)	Concrete class (c)	Test Method (d)	Concrete or aggregate type (e)	Specimen size/mm (f)
Lu Zhou-dao	C-E-SS	C28	28d-S-0.5h-U-N	525C-Si-B	10*32T(30)
Niu Hong	C-SS	C28	28d-S-0.5h-U-N	525C-Si-B	10*32T(30)
Tan Wei	C-SS	C28	210d-S2-0.5h-A-N	525C-Si-B	10*25T(28)
Ding Wei	C-SS	C28	210d-S8-0.5h-W-30d-N	525C-Si-B	10*25T(28)
Yao ya-xiong	C-E-SS	C40	28d-S5-0.5h-U-N	525C-Si-B	12*32C(25)
Yang Yan-ke	C-SS	C30	28d-S(F)-2h-A-N	425C-Co	7*21P(108)
Li Wei	C-SS-SP	C20,C40	28d-S8-6h-U-N,28d-S8-6h-S-1d-N	C-Si(Ca)	10*10C(173), 10*30P(40)
Jia Feng	C	C25,C35, C50	130d-S-0.75h-A-N	425,525C-Si-B	10*10P(45)
Hu Bei-lei	Biaxial compress	C25	28d-S-6h-A-1d-N	425C-Ca-B	10*5P(150)

	ive-SS				
Zhou Xin-gang	C-B-SS	C33	30d-S-2h-S-N	425C-B	10*10P
Zhou Xin-gang	C-FA	C28	43d-S-3h-S-N	325C-B	10*10P(45),10*30P(30)
Shen Lu-ming	C-SS	C28	210d-S8-0.5h-W-1d-N	525C-Si	10*30P(42)
Li Gu-hua	C-E-SS	C30	30d-S-2h-S(A)-N	425C-Ca-B	
Niu Hong	C-SS	C30	28d-S-0.5h-U-N	F-CE	10*32T(40)
Nan Jian-lin	SS	C30	28d-S5-U-Y		
Xie Gen-min	C-SP-B-SS	C28	28d-S-0.2h-S-90d-N	425C-Ca-B	10*10P(54)
Peng Gai-fei	Spalling	C47,C65,C78	90d-F-N	C-Si-B	10*10P
Wu Bo	Micro-analysis	C70,C85	28d-S10-3h-S-N	C-F-Si-B	10*10P,10*31.5P
Wu Bo	C-E-SS	C40,C60	60d-S10-3h-S-7d-N	425C-Si(Ca)	10*31.5P(44)
Xu Yu	C-SP	C50	35d-S-1h-S(W)-15d-N	C-Co	10*10P(104)
Zhang Yan-chun	C-SP-F-SH	C25	90d-120d-S-1h-A-N	425C-Ca-B-SF	10*10P,10*40P,10*30P
Yuan Jie	C	C75	28d-S10-3h-A-N	C-F-Si-B-PP	10*10P
Li Min	C	C40,C60,C70	28d-F-S-N	C-F-B	10*10P,15*15P
Xiao Jian-zhuang	C	C50	180d-S25-3h-A-7d-N	525C-Si-B-PP	10*10P,10*30P

Notes:

In column (b),

Abbreviation	**Interpretation**
C	compression

SP	splitting
F	flexure
SH	shearing
FA	fatigue
B	bond strength
E	elastic modulus
SS	stress-strain relationship of concrete

In column (c),

C20, C40 mean two kinds of concrete have been tested.

In column (d),

Abbreviation	**Interpretation**
28d	heated after curing 28days
S8(F)	put the specimens in furnace first then elevating at the rate of 8 ℃/min to required temperature (or heating furnace to required temperature before putting the specimens into the furnace)
1h	keep the temperature for 1h
S(W)	cooling in the furnace (cooling in the water)

S(A)	cooling in the furnace(cooling in air)
S(U)	cooling in the furnace (under high temperature)
15d	test mechanical behaviors 15days later
N(Y)	unstressed test method (stressed test method)

In column (e),

Abbreviation	**Interpretation**
425C	425# cement
F	425# cement mixing with fly ash
B	425# cement mixing with blast furnace slag
Si	425# cement mixing with silica fume
Si(Ca)	siliceous(calcareous)
Si (CE)	siliceous(ceramisite)
C(Co)	crushed(cobble)
PP	aggregates with polypropylene fiber

In column(f),

Abbreviation	**Interpretation**

10*25T(35)	35 T-type section specimens with the thickness of 40mm and the cross section of 100*250 mm
12*32C(104)	104 cross-type section specimens with the thickness of 40mm and the cross section of 120*320 mm
10*40P(42)	42 prism specimens with 400 mm in height, 100 mm in margin

3.1. Concrete Strength

A significant amount of tests and researches about concrete strength has been done. Due to effects of concrete strength classes, aggregate types, concrete mix rate, curing condition, heating and cooling mechanisms, the results are widespread.

Under high temperature condition, compressive strength changes nonlinearly with increase of temperature. When temperature is below 300°C, increase of temperature has little effects on concrete compressive strength, sometimes in this range it occurs that the compressive strength is higher than that at room temperature; when temperature is over 300°C, compressive strength decreases rapidly with increase of temperature; when temperature is over 400°C, deformation differences between coarse aggregates and cement mortar become gradually larger, cracks on surface continue to develop and extend; calcium hydroxide starts to dehydration, volume expands, compressive strength decreases sharply. When temperature is over 800℃, compressive strength is almost depleted, some specimens have occurred fragmentation just after high temperature.

The main reasons of concrete strength loss, deformation performance and deterioration due to high temperature are:

(1) Internal gaps and cracks formed after evaporation;

(2) Incoordination of thermal properties between coarse aggregates and surrounding cement mortar;

(3) Aggregates heated and expanded to rupture, development and accumulation of internal damage become worse with increase of temperature.

Based on tests and researches about concrete compressive strength under high temperature, more unified conclusions can be drawn:

(1) High temperature compressive strength of lightweight aggregates and calcareous aggregates is higher than that of siliceous aggregates. As can be seen from position of fire science, granite is the aggregate which mostly do not want to use, because in 650-800℃ occur irregular

expansion, and anorthite is a type of ideal aggregate, in 25-950℃ its physical and chemical properties are stable, at 950℃ its thermal expansion is just 1%;

(2) Under high temperature, compressive strength loss of low class concrete is less than that of high class concrete;

(3) Compressive strength of concrete with slow heating rate is lower than that of concrete with fast heating rate;

(4) The longer the time for which concrete stays under high temperature, the more the compressive strength decreases;

(5) Heating the concrete, then cooling concrete into room temperature and don't apply stress on concrete, its residual compressive strength is lower than compressive strength under high temperature;

(6) After lots of cycles of heating and cooling, concrete compressive strength decreases gradually.

Strength and temperature of concrete which reach one certain value from initial condition have many different ways, but just there are two basic ways:

(1) First heating then loading (Path T-σ);

(2) First loading then heating(Path σ-T).

Strength measured through Path σ-T is obviously higher than the one measured through Path T-σ , that is , initial stress can improve high temperature compressive strength of concrete[17]-[18].

The reason that why concrete compressive strength is higher under path σ-T is because the effect of initial compressive stress limits the free expansion deformation of concrete under high temperature, and relieves damage effects of bond between aggregates and cement mortar due to high temperature[18]-[19].

Upper limit of concrete high temperature compressive strength is derived from path σ-T , lower limit of cubic concrete high temperature compressive strength is derived from path T-σ, the following simplified formulae are given by literature[17](Fig.3.1):

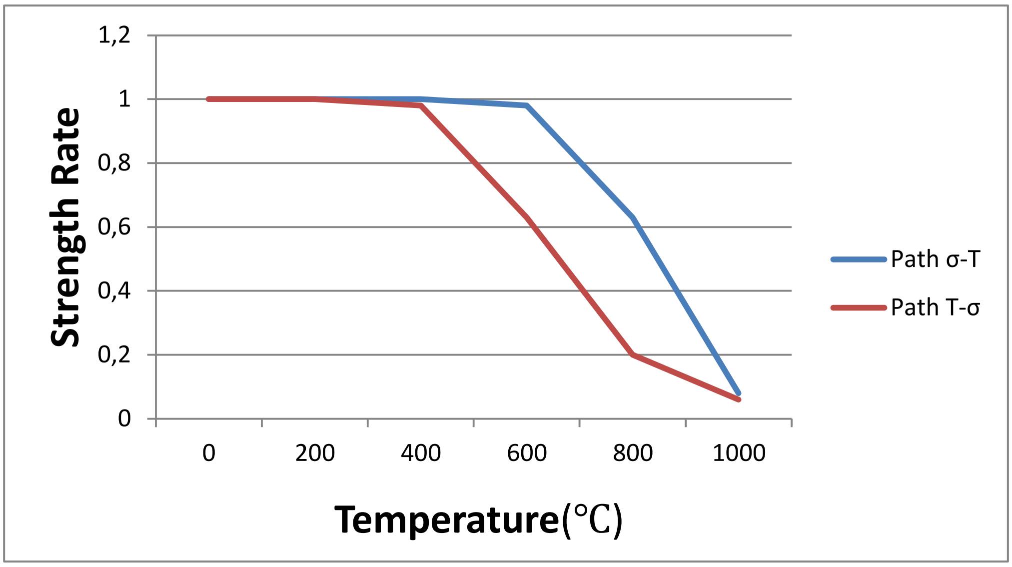

Fig. 3.1 Upper and lower limits of concrete high temperature compressive strength

Lower limit,

$$f_{cu,l}^{T} = \frac{f_{cu}}{1 + (0.154\gamma_T)^8}$$

Upper limit,

$$f_{cu,u}^{T} = \frac{f_{cu}}{1 + (0.128\gamma_T)^{12}}$$

Where $\gamma_T = \frac{T-20}{100}$

T is temperature(℃)

f_{cu}^{T} is cubic concrete compressive strength under high temperature

f_{cu} is cubic concrete compressive strength at room temperature

According to test data[20]-[21], calculated regression curves, a relation between cubic concrete compressive strength and temperature through path T-σ is derived in Fig. 3.2:

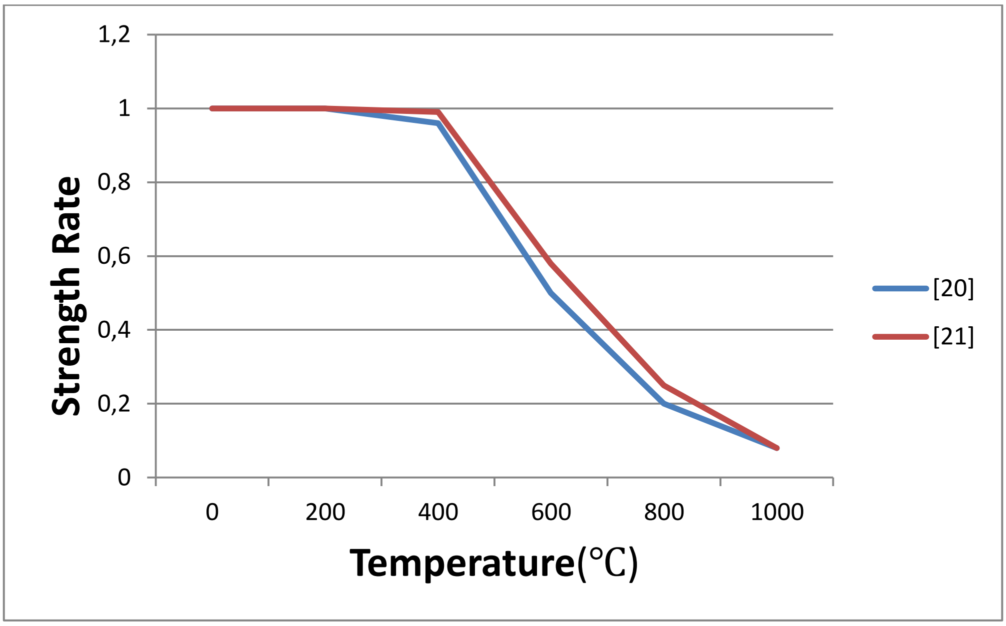

Fig. 3.2 lower limit of concrete high temperature compressive strength

$$f_{cu,l}^{T} = \frac{f_{cu}}{1 + 2.4(T-20)^{6} \times 10^{-17}}$$

$$f_{cu,l}^{T} = \frac{f_{cu}}{1 + 1.183(T-20)^{7} \times 10^{-20}}$$

The author will compare comprehensively main test results ,comparison of concrete compressive strength under high temperature is indicated in Fig. 3.3:

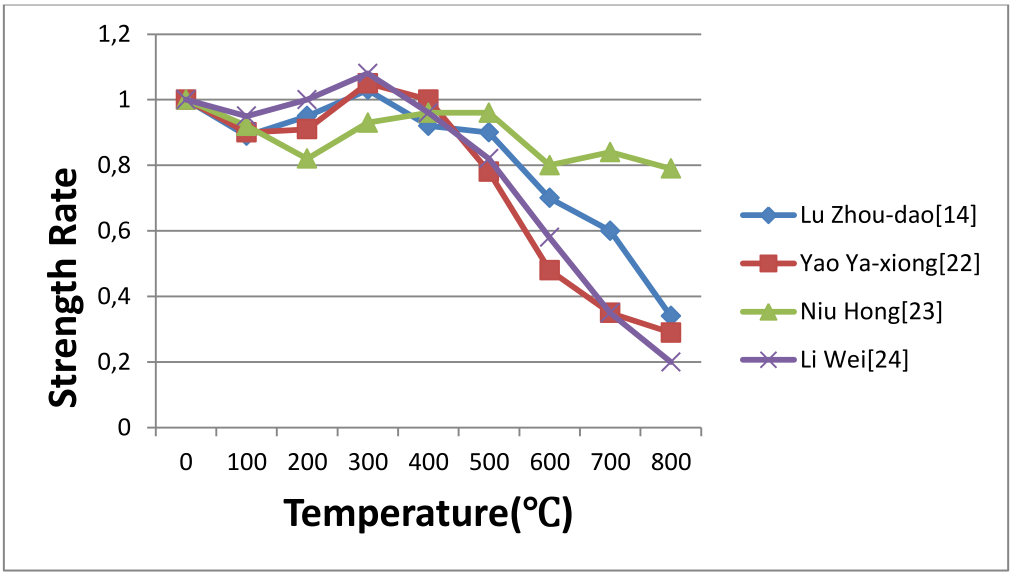

Fig.3.3 Compressive strength of concrete under high temperature[14]-[22]-[23]-[24]

The comparison of concrete evolving strength ratios after high temperature is indicated in Fig. 3.4:

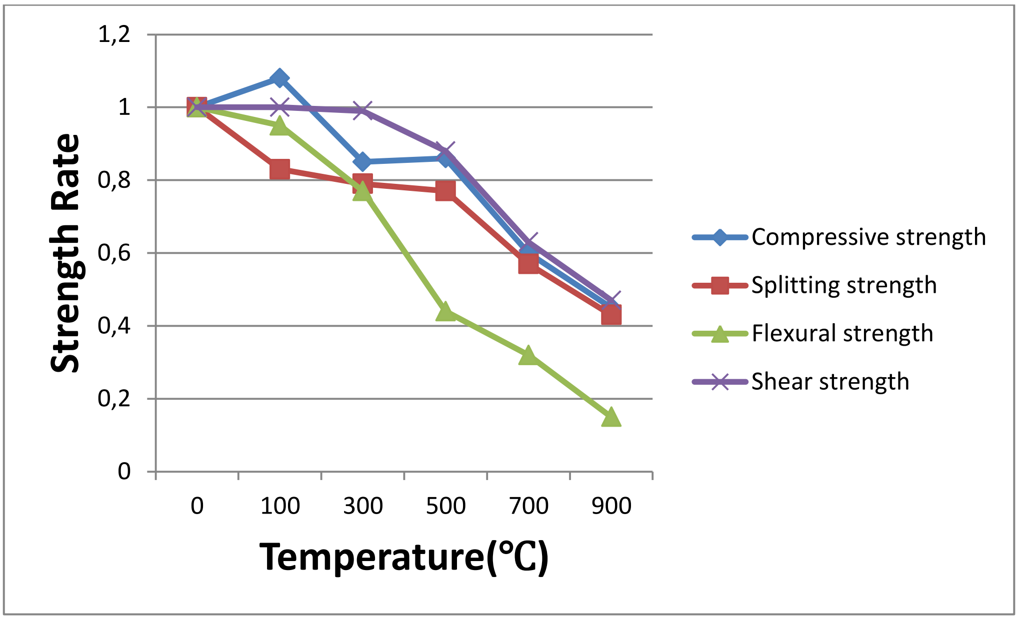

Fig.3.4 Mechanical behaviors of concrete after high temperature[22]

Comparison of concrete compressive strength under different cooling systems after high temperature is indicated in Fig. 3.5:

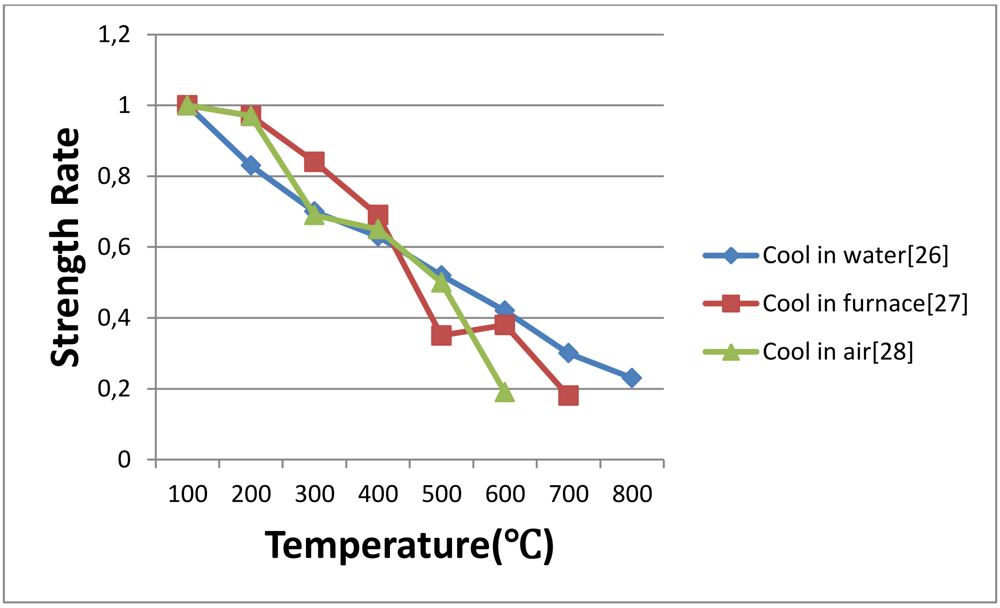

Fig.3.5 Effect of cool regimes on compressive strength of concrete[26]-[27]-[28]

The comparison of compressive strength of different concrete types after high temperature is indicated in Fig. 3.6:

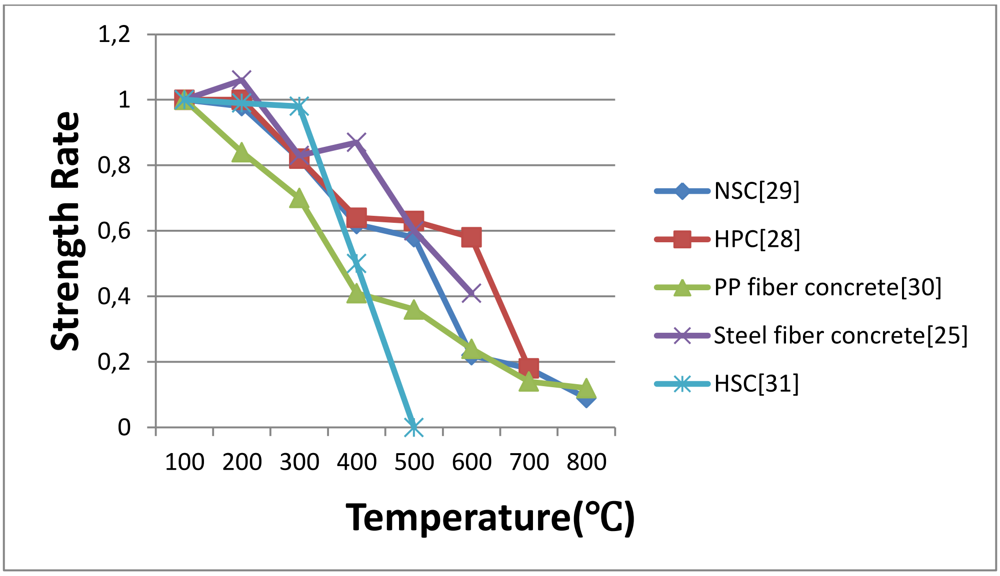

Fig.3.6 Compressive strength of various concrete after high temperature[25]-[28]-[29]-[30]-[31]

A number of conclusions can be achieved by Fig.3.3 to Fig. 3.6:

(1) Differences between specimens under high temperature and after high temperature:
under high temperature, in Fig.3.3 below 400℃ concrete prism compressive strength fluctuates at room temperature, first decreases then increases, generally it

can be considered as constant; over 800℃, it decreases obviously; at 800℃, its remaining is about 20% of compressive strength at room temperature. After high temperature, seeing from Fig. 3.4-3.6,compressive strength decreases gradually with increase of maximum fire temperature; under 400℃ it decreases slowly, over 400℃ it decreases fast. By comparing between Fig. 3.3 and Fig.3.5, residual compressive strength after high temperature is lower than compressive strength at same temperature under high temperature.

Li Wei[24] indicates that strength under high temperature decreases as increase of exposure time under high temperature, the decline increases with increase of temperature.

(2) Differences of concrete strength index: In Fig.3.4, after high temperature, concrete tensile strength decreases and is monotonic, the decline is greater than that of compressive strength, compressive strength first increases then decreases, in 300-500℃ the change is minimal; under the same conditions, bending strength is greater than splitting tensile strength; shear strength loss is similar with compressive strength loss.

(3) Effects of cooling system: In Fig. 3.5, after high temperature, below 400℃ water cooling strength loss is

obviously greater than cooling strength loss in the air, over 600°C it is almost the same between both. When water cooling is the concern, the faster the cooling, the faster the decline of strength. After watering, cement particles continue to be hydrated, residual strength increases rapidly for short time; but after cooling in the air, residual strength first decreases then increases slow[22-25].

(4) Effects of materials: because of different kinds of aggregates in concrete, the strength has different decline with increase of temperature, below 500°C siliceous and calcareous concrete are almost the same, the differences can be ignored. Regularity of strength change of carbonic concrete after high temperature is similar with that of ordinary concrete.

In Fig. 3.3, fire resistance of lightweight aggregate concrete is better than that of ordinary concrete, the strength decreases slowly; under high temperature the strength reduces slightly with decrease of concrete strength level.

In Fig. 3.4 and 3.6, mechanical properties of steel fiber concrete are better than those of ordinary concrete, after high temperature compressive strength ratio between high strength concrete with polypropylene and without polypropylene is close to a constant less than 1, there is no effect with temperature. Residual compressive strength of slag high performance after high temperature approaches that of ordinary concrete, and it is much better than high strength concrete with silica fume and fly ash[28] .High strength concrete has dense structure, at the same temperature the microstructure has small destruction and damage, macro-mechanics performance is good, but easy to burst under high temperature, the strength loss is large under high temperature[31].

(5) Effects of load path and way, Zhou Xin-gang[22] indicates that in 100-300°C under repeated loading the internal micro cracks in concrete expand further, it will reduce greatly axial compression fatigue performance.

And when normal stress is constant, the strength of ordinary concrete under biaxial compression after high temperature decreases with increase of temperature, from 150°C the strength decreases obviously, and its roughness decreases with increase of temperature; after high temperature, the strength of ordinary concrete under biaxial compression changes due to different α, it has maximum value when α is equal to about 0.5.

(6) Others: although the inner temperature and surface temperature of concrete are same, the quality of internal microstructure of concrete is still better than surface

quality of concrete, and inner concrete has higher strength.

And internal vapor pressure in high strength concrete is the main reason of its internal micro cracks expansion; after high temperature concrete tensile strength reduces proportionally to the compressive strength, but tensile stress is increasing due to water vapor pressure. Therefore cover detachment can happen as a consequence of micro crack coalescence.

In addition, in [14][22][24] test results relation between concrete cubic compressive strength f_{cu}^{T} and T under high temperature is derived in literature. Li Wei derives regularity of concrete prism compressive strength f_c^T with T, that is:

$$f_c^T = \frac{f_c}{1 + 2.4 \times (T - 20)^6 \times 10^{-17}}$$

And above f_c^T is reputed same with f_{cu}^{T}. In [21][31] simplified bilinear model of concrete prism's relationship between compressive strength and temperature is derived in literature.

Wu Bo[23] derives relationship between concrete prism compressive strength and time after high temperature. As relationship between tensile strength f_t^T and T after high temperature, test results indicate that tensile strength is discrete, and it has relation with concrete aggregate type, moisture content rate, test way, temperature and other factors.

Li Wei[24] derived simplified relationship from test results:

$$f_t^T = (1 - 0.01T)f_t \qquad (20 - 1000℃)$$

Xie Di-min[24] derives quadratic curve fitting model of tensile strength after high temperature:

$$f_t^T = [2.08\left(\frac{T}{100}\right)^2 - 2.666\left(\frac{T}{100}\right) + 104.792]f_t$$

And simplified broken line model:

$$f_t^T = \left[0.58\left(1.0 - \frac{T}{300}\right) + 0.42\right]f_t$$

$$(20 - 300℃)$$

$$f_t^T = \left[0.42\left(1.6 - \frac{T}{300}\right) + 0.42\right]f_t$$

$$(300 - 800℃)$$

Where f_t is concrete tensile strength at room temperature.

Hu Bei-lei[25] did test researches about deformation and strength characters in the bi-directional stress state, and derived strength criterion.

3.2. Concrete Constitutive Relationship

As indicated for path T-σ discussed before, total strain value under combined effects due to stress and high temperature is composed by three parts[17]:

(1) Strain $\varepsilon_{c,\sigma}$ due to stress at constant temperature;

(2) Strain ε_T due to temperature at constant temperature;

(3) Strain $\varepsilon_{c,cr}$ due to short time high temperature creep.

ε_T can be expressed:

$\varepsilon_{c,T} = \varepsilon_{c,th} - \varepsilon_{c,tr}$, $\varepsilon_{c,th}$ is concrete free thermal expansion strain, $\varepsilon_{c,tr}$ is transient thermal strain:

$$\varepsilon_c = -\varepsilon_{c,\sigma}(\sigma_c, T) + \varepsilon_{c,th}(T) - \varepsilon_{c,tr}(\sigma_c, T) - \varepsilon_{c,cr}(\sigma_c, T, t)$$

The elongation is positive to total strain, compression is positive to stress.

The coupling constitutive relationship is presented in previous formula, coupling relationship for stress, strain, temperature and time should be considered and integrated.

3.2.1. Concrete Stress-Strain Relationship at Constant Temperature

According to test results of [17], concrete stress-strain relation under Path T-σ is:

$$y = 2x - x^2 \qquad x \leq 1$$

$$y = \frac{x}{5(x-1)^2 + x} \qquad x \geq 1$$

Where $x = \frac{\varepsilon_{c,\sigma}}{\varepsilon_{c,T}}$

$y = \frac{\sigma_c}{f_{c,T}}$

σ_c is concrete stress

$\varepsilon_{c,\sigma}$ is concrete strain

$f_{c,T}$ is at temperature T concrete prism compressive strength

$\varepsilon_{c,T}$ is at temperature T concrete prism peak value strain

$$f_{c,T} = \frac{f_c}{1 + (0.17\gamma_T)^6}$$

$$\varepsilon_{c,T} = (1 + (0.09 + 0.046\gamma_T)\gamma_T)\varepsilon_p$$

Where

$$\gamma_T = \frac{T - 20}{100}$$

T is temperature(°C)

f_c, ε_p are at 20°C concrete prism compressive strength and peak value strain, respectively.

3.2.2. Thermal Expansion Strain and Transient Strain

The formula of concrete free thermal expansion strain is:

$$\varepsilon_{c,th} = \alpha_T = (0.008T + 6)T \times 10^{-6}$$

The formula of concrete transient strain[17] is:

$$\varepsilon_{c,tr} = \frac{\sigma_c}{f_c}(0.17 + 0.73\gamma_T)\gamma_T \times 10^{-3}$$

The algebra accumulation of thermal expansion strain and transient thermal strain is temperature strain under constant stress $\varepsilon_{c,T} = \varepsilon_{c,th} - \varepsilon_{c,tr}$,the relation between temperature under constant stress and temperature is shown in Fig. 3.7

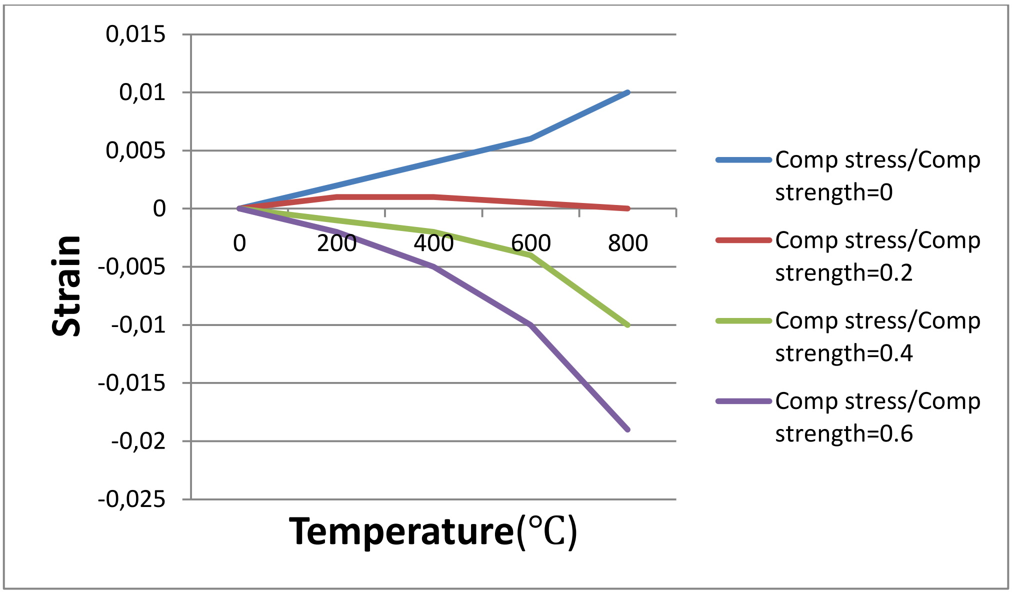

Fig. 3.7 Concrete Temperature Strain under Constant Stress

3.2.3. Short Term High Temperature Creep

Under constant temperature and constant stress, $\varepsilon_{c,cr}$ increases with time ,derive the following formula though tests in [17]:

$$\varepsilon_{c,cr} = a\frac{\sigma_c}{f_c}(T-20)^b t^c$$

Where t is time, the unit is second

a, b, c are parameters depend on tests

3.2.4. Tensile Strength

About concrete tensile strength under high temperature, lots of tests[26]-[27]-[28] were done, but results calculated are discrete. According test results[29], simplified linear relation formula is derived:

$$f_{t,T} = (1-0.001T)f_t \qquad (20-1200℃)$$

Where $f_{t,T}$ is concrete tensile strength under high temperature

f_t is concrete tensile strength at room temperature

3.2.5. Elastic Modulus

Concrete elastic modulus decreases obviously under high temperature, shows obviously plastic modulus, deformation increases. In the procedure of cooling of concrete, elastic modulus has little change[30], it is mainly because concrete internal damage cannot be recovered.

In [20] according to test results relation between elastic modulus and temperature can be expressed with linear relation:

$$E_{0T} = (0.83 - 0.0011T)E_0$$

Where E_{0T} is concrete initial elastic modulus under high temperature

E_0 is concrete elastic temperature at room temperature

In addition, concrete aggregate type, water mortar ratio and concrete level have effects on concrete high temperature elastic modulus.

Existing test results indicate that aggregate type, curing condition and test way have effects on concrete constitutive relationship under or after high temperature.

Main models about concrete elastic modulus under high temperature is showed in Fig. 3.8:

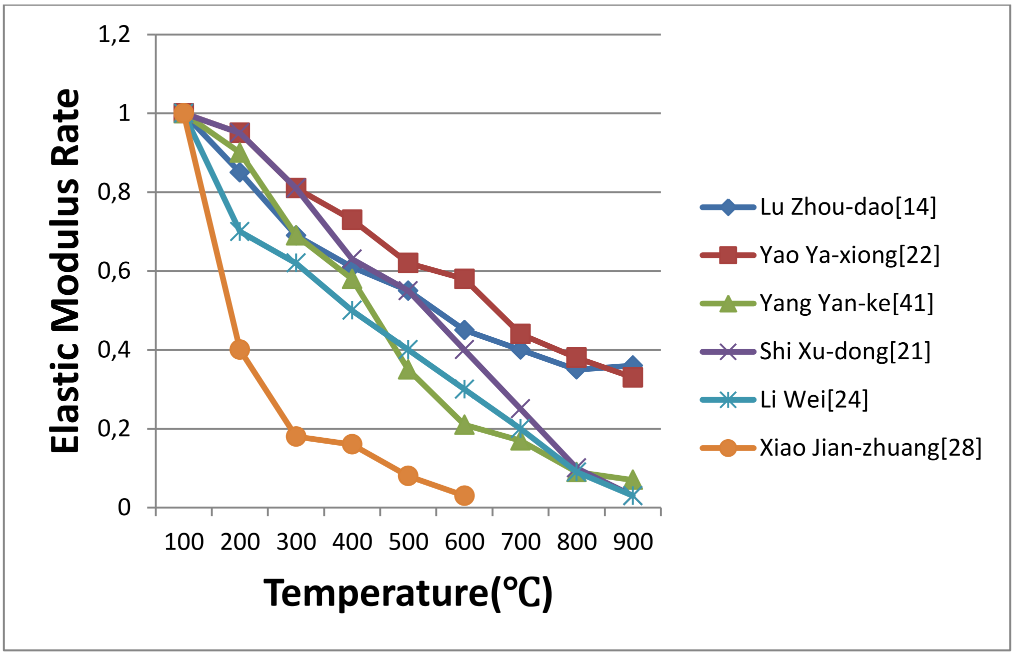

Fig.3.8 Elastic modulus of concrete at high temperature

In Fig.3.8, elastic modulus reduce fast. Elastic modulus after high temperature is lower than that under high temperature.

Lu Zhou-dao[14] used secant modulus in $0.4f_c^T$ to express change regularity of elastic modulus, with three broken line chart to derive relationship between E_c^T and T:

$$E_c^T = (1 - 0.00015\mathrm{T})E_c \qquad (20 - 200℃)$$

$$E_c^T = (0.87 - 0.0084T)E_c \qquad (200 - 700℃)$$

$$E_c^T = 0.28E_c \qquad (> 700°\text{C})$$

Li Wei[24] derives relationship between E_c and T with broken line:

$$E_c^T = E_c \qquad (20 - 60°\text{C})$$

$$E_c^T = (0.83 - 0.0011T)E_c \qquad (60 - 700°\text{C})$$

Where E_c is concrete elastic modulus in the room temperature

Xiao Jiao-zhuang's[28] researches indicate that elastic modulus of slag high performance concrete reduces with increase of temperature, below 400°C the decline rate is faster obviously than ordinary concrete, over 400°C the decline rate is almost the same with ordinary concrete.

Generally believed that aggregates have great effects on elastic modulus. As increase of temperature, the higher the water-cement ratio, the greater the decline; high temperature has stronger effects on low strength than high temperature; decline of high temperature elastic modulus is greater curing in wet condition than curing in air; it has less relationship with heating and cooling cycle times, it mainly depends on maximum temperature which had reached.

Regularity of maximum compressive strain ε_0 that changes with temperature under or after high temperature is shown in Fig. 3.9:

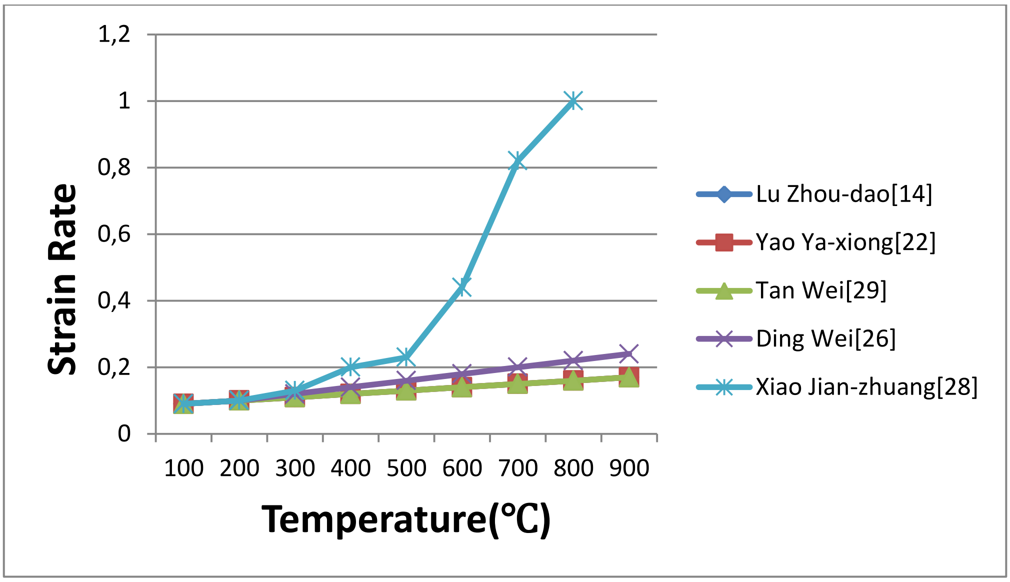

Fig.3.9 Peak compressive strain of concrete at high temperature

In Fig. 3.9, as increase of temperature concrete prism maximum strain increases gradually, it can be considered approximately as linear change, and this strain after high temperature has greater change than that under high temperature; this strain cooling by water after high temperature has greater change than that cooling in air, over 400°C strain increases greatly with time. But the research[28] about slag high performance concrete indicates that after high temperature concrete maximum compressive strain is almost the same with that of ordinary concrete below 400°C, and obviously greater than that of ordinary concrete over 400°C.

Wu Bo[31] indicates that in 20-400℃ for high strength concrete, Poisson's ratio ν= 0.11-0.25, over 400℃ Poisson's ratio increases with stress.

Xiao Jian-zhuang[28] indicates that when stress is not greater than half of maximum stress, Poisson's ratio decreases with increase of temperature, but when temperature is in 500-600°C, Poisson's ratio increases again.

In actual projects, structural components go through complex temperature-stress-strain history.

Lin Jian-nan[17] indicates temperature-stress coupled constitutive relationship. Every path that reaches certain constant σ-T value can be approached by level line which is composed by several increments of temperature and stress, and consider instantaneous heat creep changed with temperature under constant temperature-stress. Test researches indicate:

(1) Under constant loading and heating condition, strength is upper limit of high temperature strength, under constant temperature and increasing load condition, strength is lower limit of high temperature strength, other condition's results are between the previous two situations. In 600-800℃, upper limit and lower limit have large differences, upper limit is 1.4-2.5 times of lower limit;

(2) Subject to pre-stress, maximum value of stress-strain curve moves to left, strength and elastic modulus increase greatly;

(3) Pre-stress has large effects on temperature deformation, temperature deformation due to stress increases with heating, temperature deformation due to stress approaches constant with cooling.

The difference between first stress then heating and first heating then stress is mainly that the former generates instantaneous heat creep ε_{cr}. The tests indicate that:

(1) ε_{cr} increases in nonlinear with increase of temperature, but has linear relation with stress;

(2) When the increase rate of temperature is not high (0.5-5°C/min), instantaneous heat creep ε_{cr} of concrete specimens in small size is not large;

(3) Concrete age has less effects on instantaneous heat creep of concrete which is age greater than 60 days.

4. Steel High Temperature Mechanical Properties

The change of thermal properties of steel with increase of temperature is similar with that of concrete, generally coefficient increases linearly with increase of temperature, but average linear expansion coefficient α_s has little change; unit heat capacity c_s increases gradually; heat conductivity coefficient λ_s decreases linearly approximately, has larger magnitude of change; the change of mass density is little.

Current research about mechanical properties of steel under high temperature is mainly devoted to the following items:

(1) Strength, that is yield strength and ultimate strength;

(2) Deformation, that is stress-strain relation, elastic modulus and so on.

4.1. Steel Strength

Under high temperature steel softens and internal structure undergoes changes, steel strength continues to decrease with increase of temperature.

For hot rolled steel, when temperature is not over 300℃, tensile yield strength decreases little, when temperature is about 600℃, steel yield strength is just 50% of that at room temperature, yield level disappears gradually; for pre-stress steel wire, steel strand, when temperature is over 150℃, its tensile strength decreases greatly, when temperature is in 400-450℃, tensile strength is 50% of that at room temperature; for cold drawn steel, when heating, the internal structure distortion gradually returns to uniform distribution with increase of temperature, improved strength due to hardening process also decreases and is lost gradually, plasticity obtains a certain recovery.

Therefore, the decrease in strength of cold drawn steel is greatly more than hot rolled steel.

According to test results[31], relation between tensile strength and temperature in different steel levels is shown in Fig.4.1:

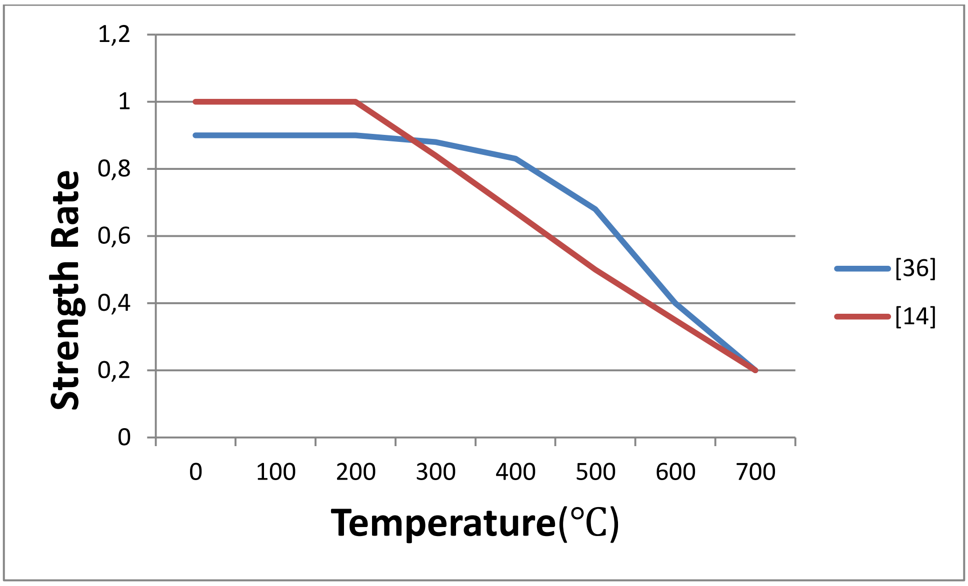

Fig. 4.1 Relation between steel high temperature tensile strength and temperature

$$\frac{f_y(T)}{f_y} = \frac{0.91}{1 + 3.6(T - 20)^6 \times 10^{-17}}$$

Where $f_y(T)$ is yield strength of different steel levels under high temperature

f_y is yield strength of different steel levels at room temperature

With linear fitting in literature,

$$f_y(T) = f_y \qquad (0 - 200°C)$$

$$f_y(T) = (1.33 - 1.64T \times 10^{-3})f_y \qquad (200 - 700°\mathrm{C})$$

Strength of ordinary low-carbon steel, low alloy steel, cold drawn steel under high temperature decreases in different degrees.

Lu Zhou-dao[14] and Yao Ya-xiong[15] derive test regression curve of steel yield strength, ultimate strength and temperature under high temperature, that is:

$$f_y^T = f_y \qquad (20 - 200°\mathrm{C})$$

$$f_y^T = (1.33 - 1.64 \times 10^{-3}T)f_y \qquad (200 - 700°\mathrm{C})$$

$$f_y^T = 0.182f_y \qquad (\geq 700°\mathrm{C})$$

Where f_y is steel yield strength at room temperature

Lv Tong-guang[31] does high temperature performance tests to several levels' steels, derives relationship between steel ultimate strength and temperature under high temperature. Generally believed that the yield level of ordinary low-carbon steel becomes gradually shorter with increase of temperature, in 300°C basicly disappears; below 400°C slightly increases, plasticity is reduced due to blue brittle phenomenon of steel in 200-350°C; over 400°C steel strength decreases gradually with increase of temperature, in 700℃ the strength decline is over 80%.

Hua Yi-jie[32] indicates that below 200℃ high strength mechanical steel wire has little change; in 200-300℃, the decline rate of nominal yield strength and elastic modulus increases, ultimate strength increases due to blue brittle phenomena, the increase is smaller than ordinary hot-rolled steel; over 300℃ strength and elastic modulus decrease greatly. He also derives two broken model of change of nominal yield strength and ultimate strength of pre-stress steel under high temperature.

The main relationship models of steel yield strength and temperature are shown in Fig. 4.2:

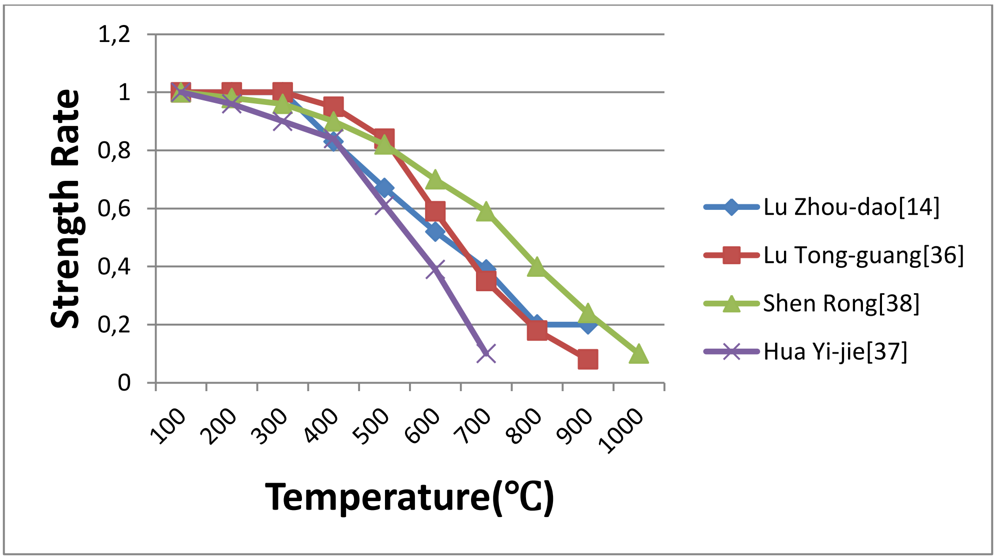

Fig.4.2 Yield strength of steel bar at high temperature

In Fig. 4.2 the strength loss of cold drawn steel and high performance steel is greater than low-carbon steel and low-alloy steel, loss under high temperature is greater than loss after high temperature. Compared to concrete, steel is more sensitive to high temperature, key point is temperature critical point, steel is 300℃, concrete is 400℃ .

4.2. Steel Elastic Modulus

The change trend of steel elastic strength modulus with increase of temperature is similar with the change of strength. When temperature is below 200℃, elastic modulus has limited drop, at 300-700℃ decrease sharply, at 800℃, elastic modulus is already very low, generally it is not over 10% of elastic modulus.

With two broken lines model to analyze, derive the following formulae:

$$\frac{E_s(T)}{E_s} = 1 - 4.86T \times 10^{-4} \qquad (0 - 330℃)$$

$$\frac{E_s(T)}{E_s} = 1.515 - 1.879T \times 10^{-3} \qquad (330 - 700℃)$$

Where $E_s(T)$ is steel specimen elastic modulus under different temperature

E_s is steel specimen elastic modulus at room temperature

Regularity research of steel elastic modulus changing with temperature is shown in Fig. 4.3:

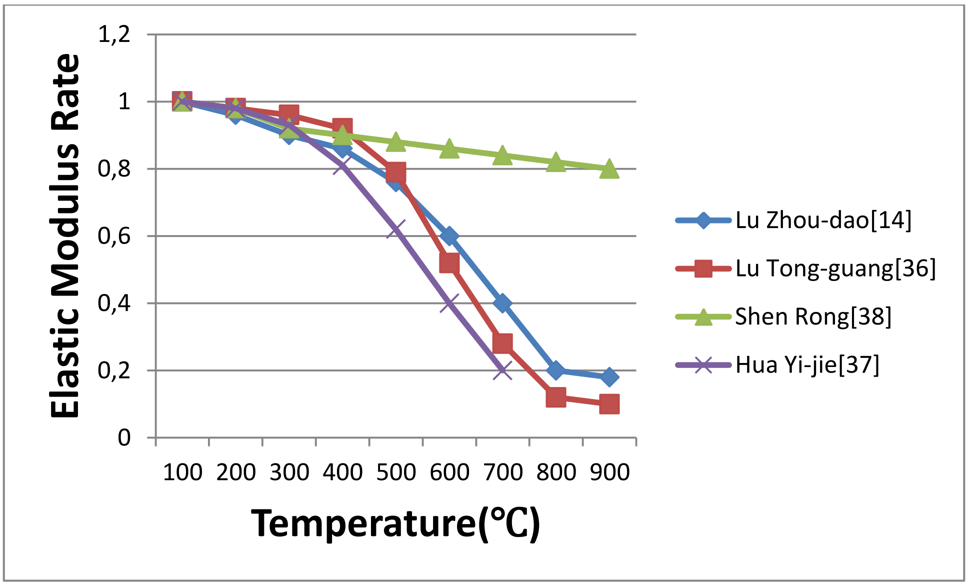

Fig.4.3 Elastic modulus of steel bar at high temperature

Steel elastic modulus decreases gradually with increase of temperature, similar to decline trend of strength, below 300°C decline is small, in 300-700°C decreases obviously.

Zhu Bo-long[8] derives relationship between steel elastic modulus and temperature with two broken lines under high temperature, in

[37] second order curve of pre-stress steel wire modulus under high temperature is derived;

Lv Tong-guang's[31] tests to each level steel indicates that steel elastic modulus under high temperature has not relation with steel types, and it can be expressed:

$$E_s^T = \frac{E_s}{1.03 + 7 \times (T - 20)^6 \times 10^{-17}}$$

After high temperature elastic modulus decreases approximately linearly with temperature, but decline is very small; he also indicates that after high temperature elastic modulus has not relation with steel types, and it can be expressed:

$$E_s^T = (1.00108 - \frac{0.024T}{100})E_s$$

Where E_s is steel elastic modulus in the room temperature.

4.3. Stress-Strain Relation

About stress-strain relation under high temperature, literatures suggest using simple geometric shapes, for example, when below 200℃ adopt perfectly elasto-plastic model, when in 300-700℃ , adopt elasto-plastic two broken lines model, and give out calculation formula of characteristic strength and deformation value.

Stress-strain relation usually is expressed by two broken lines.

Lu Zhou-dao[14] and Yao Ya-xiong derive two broken lines elasto-plastic formula σ-ε of level Ⅱ steel.

Lv Tong-guang[31] derives stress-strain relation curve of each level steel under high temperature, and indicates that under high temperature free line expansion has not relation with steel class, basically it increases linearly with increase of temperature; under stress temperature deformation has relation with steel class, the unified expression is :

$$\varepsilon(\sigma, T) = (a + \frac{b\sigma}{f_y}) \times (T - 20)^{15}$$

Where a, b have relation with steel class.

He also researches steel short time creep in different temperature.

Hua Yi-jie[32] derives strain hardening model of high strength pre-stress steel wire stress-strain relation:

$$f_{0.2}^{T} = [1 - 5.07 \times 10^{-4}(T - 20)]f_{0.2} \qquad (20 - 300℃)$$

$$f_{0.2}^{T} = [1.56 - 2.51 \times 10^{-3}(T - 20)]f_{0.2} \qquad (300 - 600℃)$$

Where $f_{0.2}$ is conditional yield strength of high strength pre-stress steel wire in the room temperature.

He also derives corresponding high temperature creep model, in the same time indicates that temperature has greater effects on creep than stress.

5. High Temperature Bond Properties Between Steel and Concrete

Concerning reinforced concrete, the bond strength between steel and concrete under high temperature has great effects on its performance. When reinforced concrete is heated, the steel expands, micro-cracks inside cement stone promote steel axial slip, and lead to decrease the bond strength between steel and concrete. Rebar surface is uneven, it has great mechanical bite force with concrete, so during heating procedure the bond strength decreases a little.

Under high temperature bond strength relative value between concrete and steel is shown in the Table 5.1[9]:

Table 5.1 Under high temperature bond strength relative value between concrete and steel

Temperature(℃)	100	200	300	400	500	600	700
Light Round Steel	0,7	0,55	0,4	0,32	0,05	-	-
Rebar	1	1	0,85	0,65	0,45	0,28	0,1

Table 5.2[8] is that after high temperature natural cooling, the bond strength between concrete and steel and the relative amount of slip reaching limit bond strength. As can be seen, bond strength has decline trend with increase of temperature, limit amount of slip

increases with increase of temperature, effects of light round steel bond strength due to temperature are stronger than those of rebar.

Table 5.2 Test Result of Bond and Slip in Natural Cooling

Steel	**Temperature(℃)**	**20**	**100**	**200**	**300**	**400**	**500**	**600**	**700**
Light Round Steel	Bond Strength(N/mm)	2,05	2,15	-	0,87	0,62	0,12	0	-
	limit Amount of Slip(N/mm)	0,32	0,21	-	0,55	0,6	-	-	-
Rebar	Bond Strength(N/mm)	6,27	-	6,16	-	4,93	-	-	1,19
	limit Amount of Slip(N/mm)	0,19	-	0,48	-	0,72	-	-	1,73

After fire high temperature bond strength loss between concrete and steel has relation with the following factors:

(1) Be proportional to fire high temperature. The higher the fire temperature, the greater the bond strength.

(2) Have relation with steel type. After fire bond strength loss of light round steel is more than that of rebar.

(3) Have relation with cooling way. Bond strength loss of water cooling is more than that of natural cooling.

(4) Cement type has little effects on bond strength loss, bond strength loss of limestone aggregate after fire is more than that of granite aggregate.

Bond properties for steel and concrete are the basis to ensure coordination between both, they have great effects on components' cracks, deformation and capacity. Bond capacity is composed by gluing, friction, mechanical occlusion. Under high temperature, because concrete heat expansion is smaller than steel, concrete squeezed in ring makes friction between concrete and steel increases, in the same time makes concrete tensile strength decrease, makes gluing strength decrease.

Research results about bond strength of reinforced concrete under high temperature are shown in Fig. 5.1:

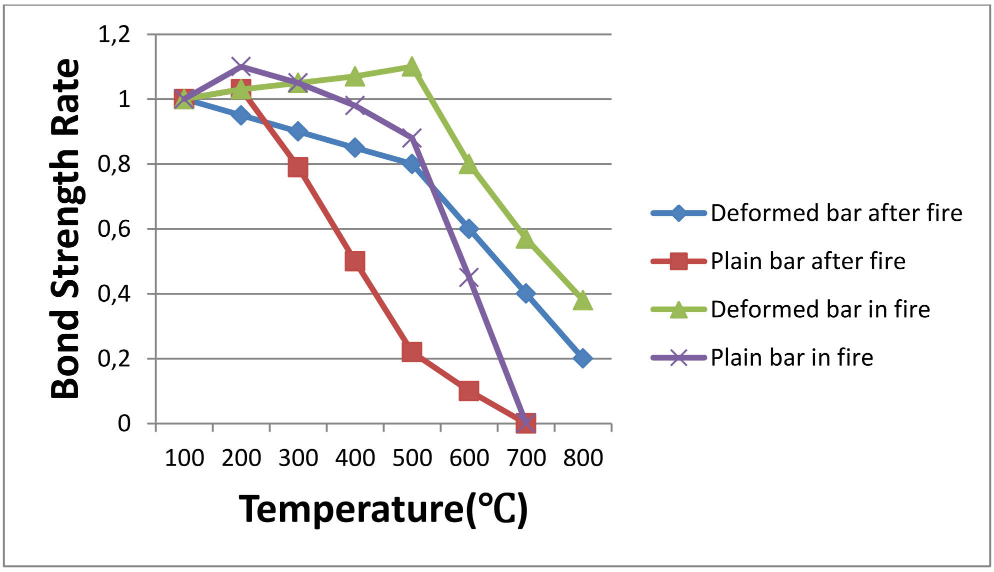

Fig. 5.1 Bonding strength between concrete and steel bars at high temperature[33]

In Fig. 5.1 bond between steel and concrete has varying degrees of damage and destruction under or after high temperature. Compared with concrete compressive strength, bond strength loss is much greater, it is almost the same with tensile strength loss. Temperature has strong effects on light round steel, less effects on screw thread steel. When temperature is low, temperature of screw thread steel is lower than 300℃, temperature of light round steel is lower than 200℃, bond strength increases. When temperature of screw thread steel is lower than 600℃, temperature of light round steel is lower than 400℃, bond strength decreases sharply. After cooling, bond strength does not increase any more, compared with bond strength in high strength the decline is obvious.

In addition, slip amount of bond limit for steel with rust is greater than that for steel without rust.

Zhou Xin-gang[34] does center pull test to different specimens at the room temperature and under high temperature, and measures corresponding bond slip relation curve, derives relation between bond strength, protective layer thickness and concrete tensile strength.

Xie Di-min[24] indicates that splitting bond stress which has certain slip amount of 0.25mm is proportional to concrete tensile strength:

$$\frac{\tau_u^T}{\tau_u} = \frac{f_t^T}{f_t} - 0.057 \times \frac{T}{100} \qquad (20 - 300℃)$$

$$\frac{\tau_u^T}{\tau_u} = \frac{f_t^T}{f_t} - 0.17 \qquad (300 - 700℃)$$

And derives fitting bond ultimate strength formula:

$$\frac{\tau_u^T}{\tau_u} = [2.743\left(\frac{T}{100}\right)^2 - 3.322\left(\frac{T}{100}\right) + 105.881] \times 10^{-2}$$

Where τ_u is bond ultimate strength between steel and concrete in the room temperature.

Part Two: Examples on Fire-Resistance Design for Structural Members

1.The example presents the procedure for design and verification of structural members.

The characteristics of one simple supported beam structural member of reinforce concrete structure is given:

beam span	l=6 m
dead load	G = 2.5 kPa
basic live load	Q = 2.6 kPa
concrete class	C35/45
steel type	B450C
weight of concrete	r=25kN/m3
beam cross section	500mmx300mm
thickness of concrete protected level	c=50mm

The load acting on the beam has to be calculated.

The weight of beam is:

$$W = 25x0.5x0.3x6 = 22.5kN/m$$

The load combination given by combinations of actions for accidental design situations in Eurocode:

$$\sum_{j\geq 1} G_{k,j}"+"P"+"A_d"+"(\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1}"+"\sum_{i>1}\psi_{2,i}Q_{k,i}$$

q=22.5+2.5+0.5×2.6+0.5×2.6=27.6kN·m

The maximum fire design bending moment:

$$M_{sd} = \frac{ql^2}{8} = 124.2kN \cdot \text{m}$$

$$M_{sd,fi} = M_{sd} \cdot \eta_{fi} = 124.2\text{x}0.70 = 86.94\text{kN·m}$$

Where

$$\eta_{fi} = \frac{E_{d,fi}}{E_d} = \frac{G_k+\varphi_{fi}Q_{k,1}}{r_G G_k + r_{Q,1}Q_{k,1}} = \frac{25+0.5x2.6}{1.35x25+1.5x2.6} = 0.70$$

The resistance moment $M_{rd,fi}$ for design of fire situation can be calculated:

$$M_{rd,fi} = \frac{\gamma_s}{\gamma_{s,fi}} \times k_s(\theta) \times M_{sd}\frac{A_{s,prov}}{A_{s,req}} = 1.15 \times 1 \times 124.2 \times \frac{508.68}{600} = 121.1kN \cdot \text{m}$$

Where

γ_s is the partial material factor for steel

$\gamma_{s,fi}$ is the partial material factor for steel under fire conditions

$A_{s,prov}$ is the area of tensile steel provided

$A_{s,req}$ is the area of tensile steel required for the design at ambient temperature

$\frac{A_{s,prov}}{A_{s,req}}$ should not be taken as greater than 1.3

$k_s(\theta)$ is a strength reduction factor of the steel for the given temperature θ under the required fire resistance

The following is specially for calculation of $k_s(\theta)$. Figure 1 shows how the temperature profiles represent the temperature in the cross-section of beams and columns taking symmetry into account.

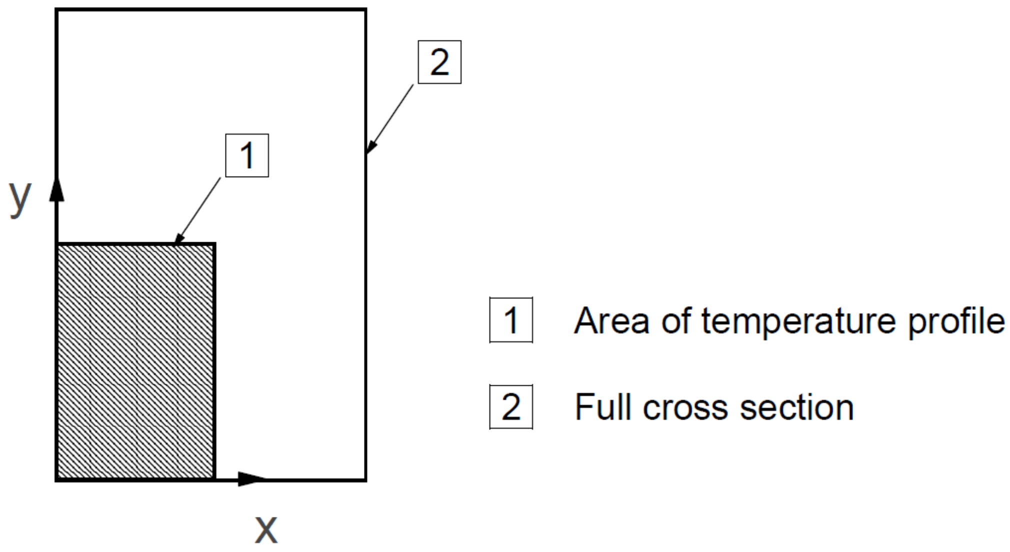

Fig.1 Area of Cross-section for which the Temperature Profiles are Presented

Thickness of concrete protected level c=50mm, figure 2 provides calculated temperature profiles for beams with cross-section 600mm×300mm under R90(fire resistance class for the load-bearing criterion for 90 minutes in standard fire exposure). From figure 2, check out the value of isotherm at the cross center of 50mm of horizontal axis and 50mm of vertical axis, it approximates 500℃.

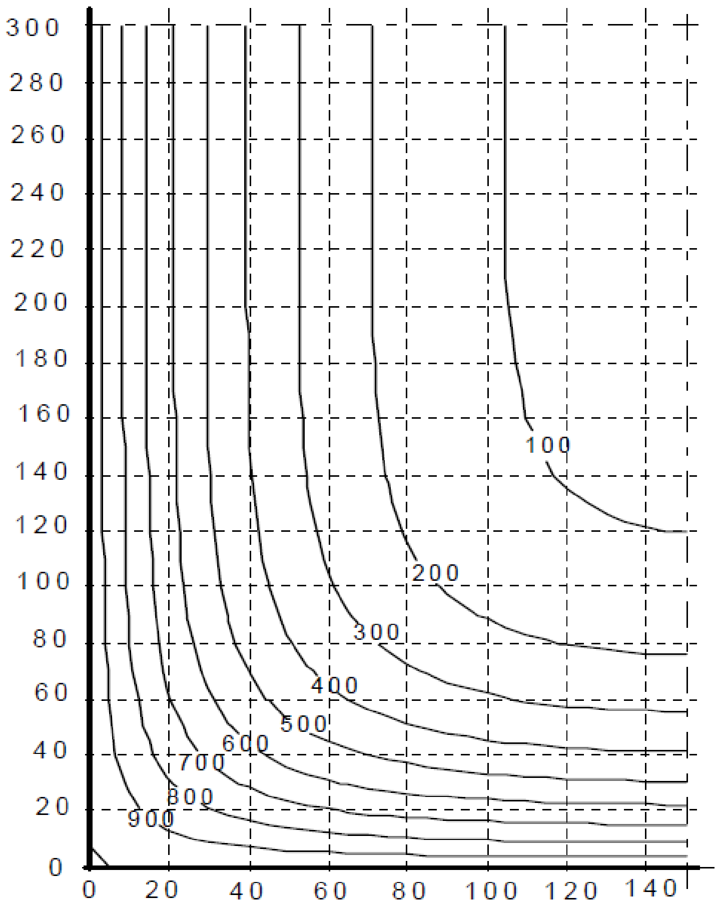

Fig.2 Temperature profiles (℃) for a beam *h* x *b* = 600 x 300 (R90)

After obtaining the temperature of reinforced concrete with 50mm of thickness of protected level, then check out relevant $k_s(500℃)$ by figure 3.

Obtain

$$k_s(500℃) = 0.79$$

Therefore it could be used to calculate the resistance moment $M_{rd,fi}$

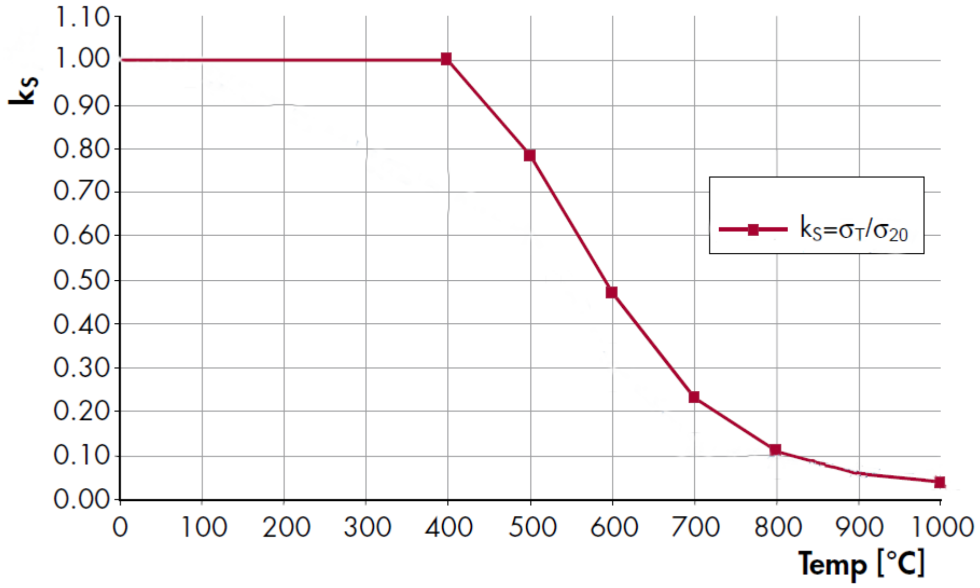

Fig.3 Thermo-mechanical Characteristics of Steel

It should verify

$$M_{sd,fi} \leq M_{rd,fi}$$

$$86.94\text{kN}\cdot\text{m} < 121.1kN\cdot\text{m}$$

Therefore, the beam is safe.

2.The example will present the applications that mentioned previously when a parametric temperature-time curve is used to determine temperature to be reached by steel members during fire. The parametric temperature-time curve could be used to determine steel member temperature with sufficient accuracy for structural design proposes. The parametric temperature-time curves used in the example are Eurocode parametric curve.

One of the most commonly used natural design fire curves is the Eurocode Parametric temperature–time curve, given in prEN1991-1-2.

The temperature-time curves in the heating phase are given by:

$$\theta_{max} = 20 + 1325(1 - 0.324e^{-0.2t^*_{max}} - 0.204e^{-1.7t^*_{max}} - 0.472e^{-19t^*_{max}})$$

Where

θ_{max} is the maximum temperature in the fire compartment

$$t^*_{max} = t_{max} \cdot \Gamma$$

with

t_{max} is time when θ_{max} happens

$$\Gamma = [\frac{O}{b}]^2/(\frac{0.04}{1160})^2$$

$b = \sqrt{(\rho c \lambda)}$ for the calculation of factor b, the density ρ, the specific heat c and the thermal conductivity λ of the boundary may be taken at ambient temperature with the following limits: $100 \leq b \leq 2200$

ρ density of boundary of enclosure

c specific heat of boundary of enclosure

λ thermal conductivity of boundary of enclosure

O opening factor: $A_v\sqrt{h_{eq}}/A_t$, with the following limits: $0.02 \leq O \leq 0.20$

A_v total area of vertical openings on all walls

h_{eq} weighted average of window heights on all walls

A_t total area of enclosure (walls, ceiling and floor, including openings)

$$t_{max} = \max[\left(0.2 \cdot 10^{-3} \cdot \frac{q_{t,d}}{O}\right); t_{lim}]$$

with

$q_{t,d}$ is the design value of the fire load density related to the total surface area A_t of the enclosure whereby $q_{t,d} = q_{f,d} \cdot A_f/A_t$. The following limits should be observed: $50 \leq q_{t,d} \leq 1000[MJ/m^2]$.

$q_{f,d}$ is the design value of the fire load density related to the surface area A_t of the floor $[MJ/m^2]$.

t_{lim} In case of slow fire growth rate, t_{lim} = 25 min; in case of medium fire growth rate, t_{lim}= 20 min and in case of fast fire growth rate, t_{lim} = 15 min.

Given a table for an enclosure with the following characteristics of one structure:

enclosure floor area (m^2)	A_f	69
enclosure height(m)		4
total enclosure surface area: walls, ceiling, floor and openings (m^2)	A_t	360
total area of vertical openings (m^2)	A_v	16
weighted mean height of vertical openings (m)	h	4
fire load energy density related to the floor area A_e (MJ/ m^2)	$q_{e,d}$	400
thermal characteristic	b	1700

Obtain

$$\theta_{max} = 20 + 1325\left(1 - 0.324e^{-0.2t^*_{max}} - 0.204e^{-1.7t^*_{max}} - 0.472e^{-19t^*_{max}}\right) = 901°\text{C}$$

Where

$$t^*_{max} = t_{max} \cdot \Gamma = 0.759$$

$$t_{max} = \max\left[\left(0.2 \cdot 10^{-3} \cdot \frac{q_{t,d}}{O}\right); t_{lim}\right] = \max[0.172; 0.33] = 0.33hour$$

$$q_{t,d} = q_{f,d} \cdot \frac{A_f}{A_t} = 76.7$$

$$O = A_v \sqrt{h_{eq}/A_t} = 0.089$$

b=1700

$$\Gamma = \frac{\left[\frac{O}{b}\right]^2}{\left(\frac{0.04}{1160}\right)^2} = 2.3$$

Once the fire temperature-time curve has been generated, the temperature rises in a steel member can be determined by the heat transfer method given in section 3 of Eurocode 1.

The use of “design fire” curves to represent the natural fire temperature-time condition in fire engineering design is increasing.

The two examples illustrate the applications of fire-resistance reinforced concrete structural members and introduce applications of parametric temperature-time curves, and lots of details are presented herein to allow their applications in design and verification of structural members. The properties of fire-resistance reinforced concrete can be used as reference of structural members design, verification and parameter temperature-time curves could be an effective design tool.

Part Three: Conclusion and Future Development

1. The researches about concrete high temperature thermal properties are the basis of concrete structural temperature field analysis, compositions and ratio of concrete materials have great effects on it. The effects of thermal properties due to high temperature, the thermal parameters and the cooling phases of performance concrete should be discussed in the future.

2. The effects of concrete mechanical properties due to high temperature have overall certain regularity, that is, as increase of temperature, concrete mechanical properties continue to decay. But the effects of different mechanical index due to high temperature exist discrepancies, in comparison, the effects of tensile strength due to high temperature are more conspicuous.

3. Decay effects of concrete mechanical properties due to different heating and cooling methods exist discrepancies. In the same conditions, cooling in water has more effects on concrete mechanical properties than cooling in air.

4. Compared with concrete, steel is more sensitive to high temperature, more obvious temperature limit point, for steel it is 300℃, for concrete it is 400℃.

5. Mechanical properties of slag high performance concrete after high temperature have discrepancies with that of ordinary concrete, overall high temperature makes more serious damages to high performance concrete, so it is not safe and rational that evaluate mechanical properties of high performance concrete after high temperature with mechanical properties regularity of ordinary concrete after high temperature.

6. Different steel types have great effects on high temperature bond properties between steel and concrete, and the effects of high temperature bond properties due to different concretes should be discussed in the future.

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