CHAPTER 2
LITERATURE REVIEW

CHAPTER 2
LITERATURE REVIEW

2.1 INTRODUCTION.

This chapter includes many previous researches which study the fire resistance of concrete and structural member's performance in a fire. Fire resistance is defined as the ability of the structure member to withstand exposure to afire without loss of load bearing function or ability to act as a barrier to spread a fire.

2.2Responses of Concrete to High Temperatures

2.2.1 Damage mechanisms of concrete under high temperatures

There are four types of major damage mechanisms responsible for deterioration of properties ofconcrete under high temperatures:

(1) Phase transformations taking place in cement paste.

(2) Phase transformations taking place in aggregate.

(3) Thermal incompatibility between the cement paste and aggregate.

(4) Spalling of concrete.

The firstthree damage mechanisms result in reduced strength and stiffness of concrete, while the last oneleads to reduced cross section of structural members and loss of structural integrity.

2.2.1.1 Phase transformations in cement paste.

Major products of hydration reactions of Portland cement are Calcium Silicate Hydrates ($C_3 S_2 H_3$) or simplified as C-S-H), Calcium Hydroxide (CH), and Ettringite ($C_6 AS_3 H_{32}$). All of thehydration products decompose under high temperatures. The decomposition processes of thehydration products at various temperature ranges are listed in **Table 2.1[Schneider 2002].**

Temperature range [°C]	Transformation or decomposition reaction	Heat of reaction or transformation [kJ/kg]	Heat of reaction or transformation [MJ/m^3 concrete]	Mass of reaction [kg/m^3 concrete]
30 - 120	Desorption or evaporation of physically adsorbed water	Heat of evaporation of water: 2258	290	130 kg water
30 - 300	gel destruction: 1. stage of dehydration	Heat of hydration: 250	< 20	< 78 kg hardened cement paste
120 - 600	Release of chemically adsorbed or zeolithically bonded water	Heat of evaporation of water: > 2258	> 135	60 kg water
450 - 550	Decomposition of portlandite $Ca(OH)_2 \rightarrow CaO + H_2O$	1000	< 40	< 40 kg CaO
570	Transformation of quartz $\alpha \rightarrow \beta$ SiO_2	5,9	8,8 1,2	1500 kg quartz 200 kg quartz
600 - 700	Decomposition of CSH-phases; formation of β-C_2S	Heat of hydration: 500	< 120	< 240 kg hardened cement paste
600 - 900	only calcite: dissociation of calcite	Heat of decomposition: 1637	2360	1600 kg limestone $CaCO_3$-content approx. 90 %
from 1100 - 1200	Melting of concrete, formation of glassy materials	Melting heat: 500 - 1000	quartzitic: 1575 calcitic: 1125	2100 kg concrete 1500 kg concrete

Table 2.1: Decomposition of cement paste at various temperatures ranges **[Schneider 2002].**

The chemical reactions for the decomposition processes can be described as following:

Decomposition of C-S-:$3.4CaO.2SiO_2.3H_2O \rightarrow 3.4CaO.2SiO_2+3H_2O\uparrow$

Decomposition of CH: $Ca(OH)_2 \rightarrow CaO+H_2O\uparrow$

The formation of $CaCO_3$ is due to accelerated carbonation reaction of CH,

$$Ca(OH)_2 + CO_2 + H_2O \rightarrow CaCO_3 + 2\,H_2O$$

Then, $CaCO_3$ decomposes at high temperatures.Water evaporates under high temperatures.Associated with the formation of new phases in the chemical reactions are changes in volume aswell as in stiffness of cement paste. There are cracks and voids formed in cement paste alongwith the decomposition of CH, which results in major damage of concrete. Similarly, thestiffness of the new products is different from the stiffness of the original phases, which leadsto a change in stiffness of concrete when temperature rises.

2.2.1.2 Phase transformations in aggregates.

For normal weight concrete, there are two common aggregate groups: siliceous aggregates suchas quartzite, gravel, granite and flint; calcareous aggregates such as limestone, dolomite andanorthosite. It is generally known that siliceous aggregates, especially quartzite, experiencephase transformation at approximately T = 570°C from α-quartz to β-quartz. This crystaltransformation is reversible and endothermic which involves a heat of transformation of 5.9kJ/kg of SiO_2. Assuming about 75 percent of the quartzitic aggregate in the concrete participatesin the transformation, the heat of transformation is found to be 8.8 MJ/m³ for quartzitic and about1.2 MJ/m³ for calcitic concrete.

2.2.1.3 Thermal incompatibility between cement paste and aggregate.

Concrete is a composite material with aggregates as inclusions and cement paste as matrix. Thetwo phases have different thermal and mechanical properties and thus respond differently upon atemperature rise. The aggregates in concrete expand with increasing temperature. Cement pastemay expand if the thermal expansion is dominant and may shrink if the moisture loss is dominant.

The combined effect of the deformation mechanisms of the two phases depends on many factorssuch as heating rate, holding period, and composition of the concrete. **Fig 2.1** illustrates theseeffects. The thermal incompatibility between the two phases causes very large mismatch in thedeformation between aggregates and cement paste, which results in cracks in the interfacetransition zone around aggregates. Subsequent heating, drying, and loading may causecoalescence of the cracks to form discrete large cracks leading to spalling of concrete and/orfailure of concrete structures.

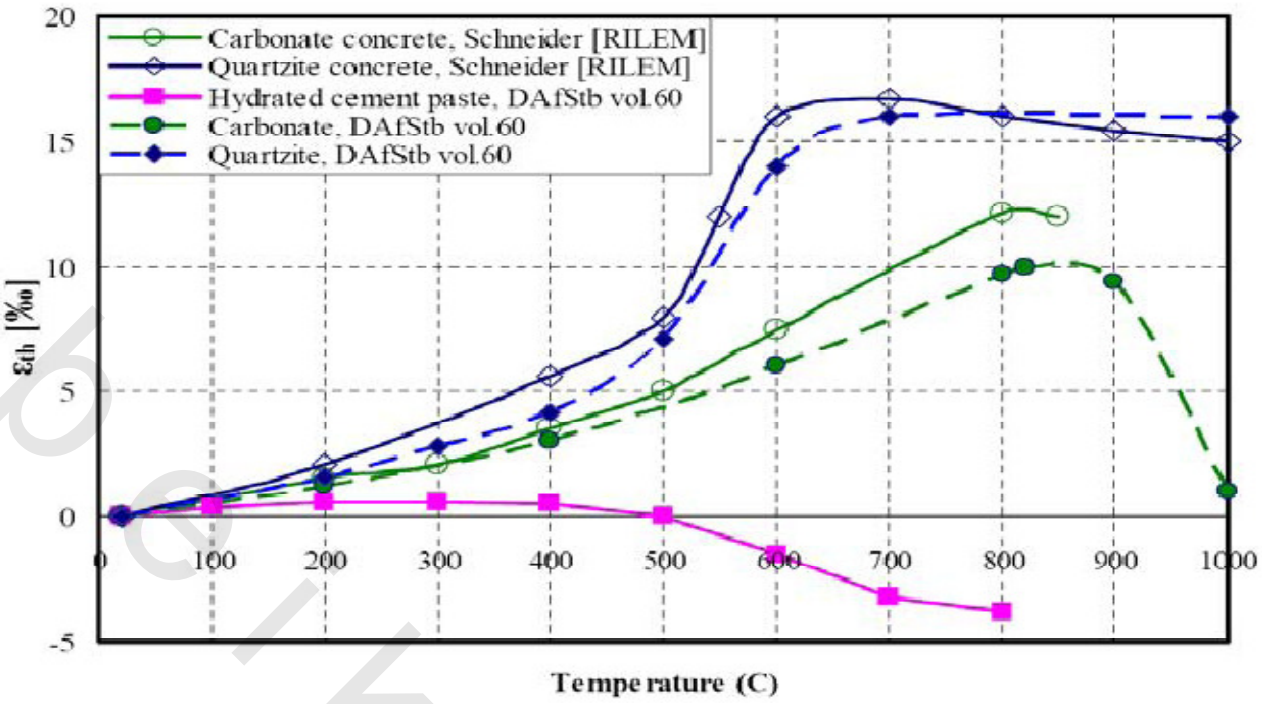


Fig. 2.1 Thermal strains for different concretes, aggregates and hydrated cement paste.

2.3 Actual State of the Codes on Fire Design in the Different Countries.

Many codes and countries produce standards for design and improve resistance the concrete to fire.

2.3.1 Actual State of the Codes on Fire Design in Japan.

In this paper, the state-of-the-art of fire resistance design of concrete in Japan is summarized intwo parts. First part corresponds with structure of building regulations on fire resistance ofbuildings. The second part is associated technical standards for reinforced concrete and compositestructures of steel and concrete.

2.3.1.1 Summary of performance-based changes in Building Standards Law of Japan

The building code of Japan (Building Standards Law of Japan, BSLJ, hereafter) was revised in 2000,including performance-based sentences for fire resistance. A simplified performance evaluationmethod was provided for various types of structures including concrete.

A- Requirements

The BSLJ was revised in 1998 to include functional requirements in place of detailed technicalspecifications of materials and constructions. Even though it is not perfect, the law has

shiftedtowards performance-based format. Following the changes in law, enforcement order (detaileditems of regulation) and notifications (technical standards) were revised in June 2000. Concerning with the structural fire resistance, performance evaluation framework and a set ofsimplified calculation formula have been added as Kenshoho (verification method) for fireresistance. By using verification method, it is possible to check the adequacy of fire resistance ofstructural elements easily and quickly.

B-Functional approach

After the revision, it is possible to adopt functional approach in fire resistance design. The objectiveimplied in BSLJ is to prevent;

(1) Collapse due to fires that are foreseeable to take place in the building,

(2) Fire spread to the buildings during fires that normally takes place around the building.

The functional requirements to satisfy the objective are;

(1) Load-bearing structural part shall sustain load throughout the complete process of fire.

(2) Building envelope (exterior walls and roofs) shall not create a gap that may penetrate flamefrom inside to outside

(3) Floors and internal firewalls shall not create a gap to penetrate flame nor transmit heat enoughto ignite combustibles in the opposite side of fire compartment in both directions.

(4) Exterior walls shall not transmit heat enough to ignite combustibles in the building.

The above requirements are summarized in **Fig.2.2**.

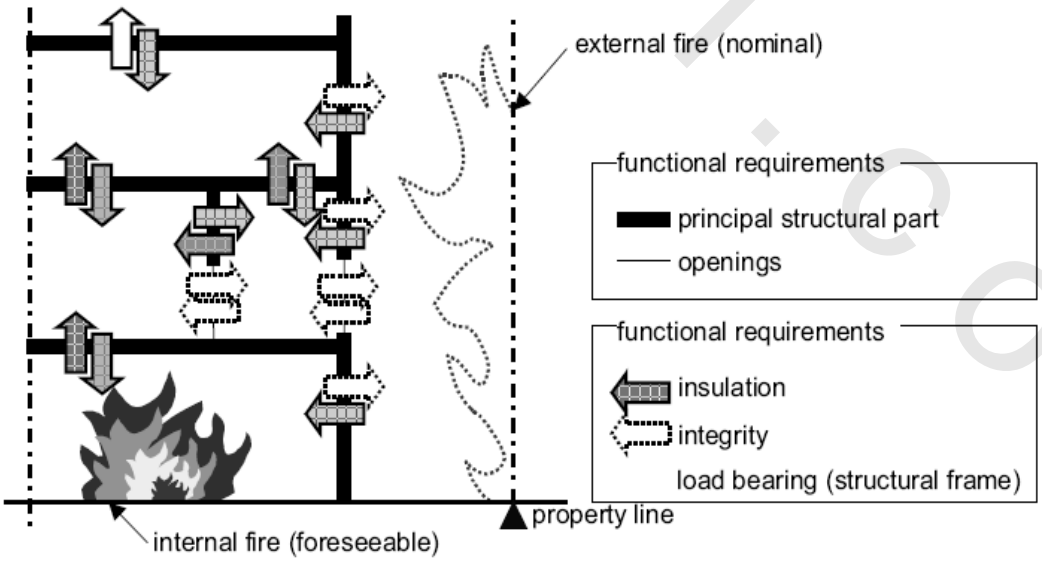


Fig. 2.2-Functional requirements for fire resistance**.**

C- Performance evaluation

To satisfy the requirement of BSLJ, planning body can choose among Route A, B and C. As shownin **Fig. 2.3**route A is a conventional method that follows prescriptions in the code. Code specifiesrequired fire resistance time of principal structural part depending on size (number of stories) ofbuildings. The principal part shall be made of fire resistive constructions listed in approvedconstructions. As to concrete-frame buildings, minimum dimensions (diameter and cover thickness)are prescribed as shown in **Table 2.2**.Performance-based routes were provided as in route B and C. Route B is to apply simplifieddesign formula specified in MoC's notification 1433(2000fy). The chance is at most increased ifthey choose Route C. The difference between route B and C are the degree of sophistication andcomplicity of design process, and the body that will review design solution. In the Route B, designprocess is simplified enough so that local building authority can review the design solutionsthemselves. In practice, it means that review and approval process is finished quickly, but at thesame time, the design would have to be conservative. In route C submittals, it is possible to adoptany design procedure as long as it follows the requirements of law and as long as it is correct inengineering sense. The appropriateness is judged by a peer- review body, followed an approval byMLIT (Minister of Land, Infrastructure and Transportation).

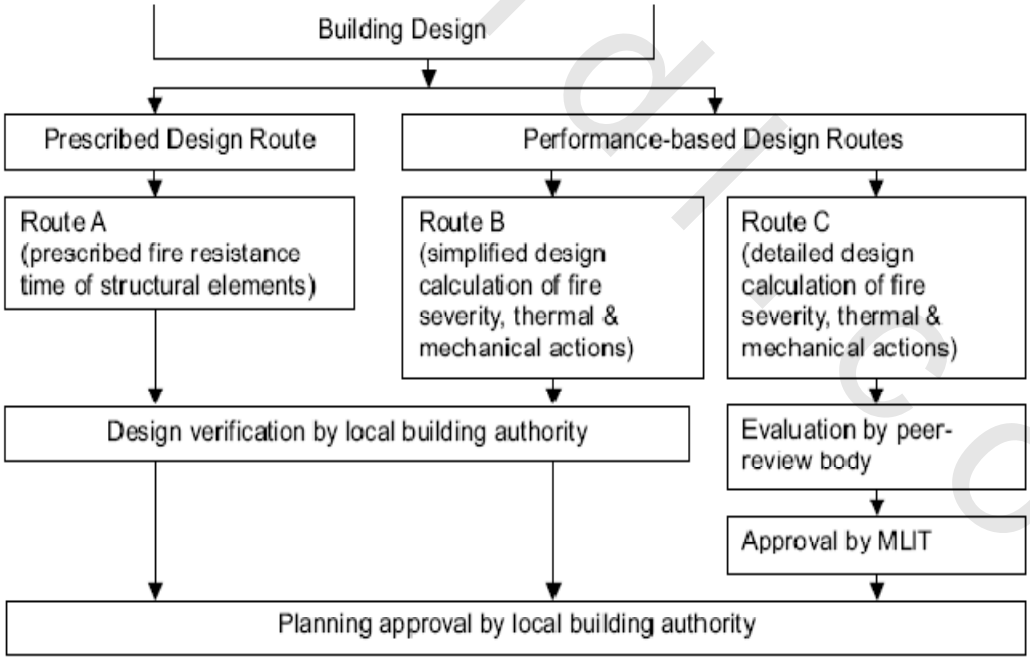


Fig. 2.3- Three Routes to Conform with Fire Resistance Requirements in BSLJ.

Table 2.2 Deemed-to-satisfy Specifications inBSLJ.

element of construction	fire resistance rating [min]	minimum dimension [mm]	minimum cover thickness [mm]
wall, floor	60	70	(not specified)*
column	120	100	30
	60	(not specified)	30
	120.	250	50
	180	400	60
beam	60	(not specified)	(not specified)*
	120	(not specified)	50
	180	(not specified)	60

* Minimum value are specified in terms of durability (20mm for floors, 30mm for columns and beams)

2.3.1.2 Design methods.

Depending on the type of concrete, design methods are selected as follows for most cases.

A- Reinforced concrete of ordinary strength.

For ordinary strength concrete (design strength Fc > 60N/mm2), simplified design method can beapplied.

1- General principle.

The general principle for structural fire resistance is to prevent the strength reduction of loadbearing elements. Namely the strength (resistance) R must be larger than the service load Sthroughout the fire process,

$R(t) > S(t)$, $t = 0 \sim \infty$ ----- Eq.(1)

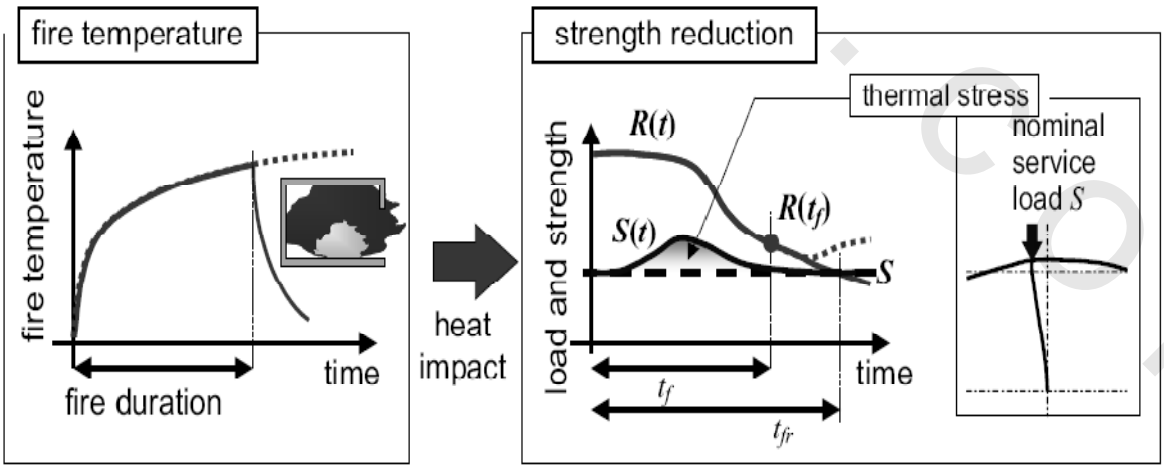


Fig.2.4 - Typical changes in load and strength of steel column during fire.

The typical changes in strength and service load are shown in **Fig.2.4**. Service load increases dueto the thermal stress in the early stage of fire. However, at the critical condition of structuralendpoint, thermal stress is negligible. This assumption is valid for ductile structures designedagainst wind and earthquake motion. As a result of seismic resistance design, structural frame isequipped with enough deformation capacity so that the frame is insensitive to perturbations causedby thermal stress in the early stage of fire.

Following above assumption, it is practical to check the strength at the fire duration (plussome post fire period) t = t_f. Equation (1) could be

$$M = t_{fr}(S) - t_f > 0 ,$$

Where S is the load applied by external force. It is more convenient to express by time margin, t_{fr} (S) is the critical time to failure under the service load S.

Calculation procedure consists of two parts. The first half is to calculate the fire severity of allthe potential fire rooms. The second half is to calculate the time to failure of structural element.

2- Calculation of time to structural failure.

Fig 2.5 shows the calculation procedure for time to structural end point. The procedure starts withcalculation of structural forces during normal condition to determine minimum cross sectional areafor load bearing. Then the time to critical thermal deterioration depth (ineffective section, usuallytaken by 500°C isothermal line) is calculated.

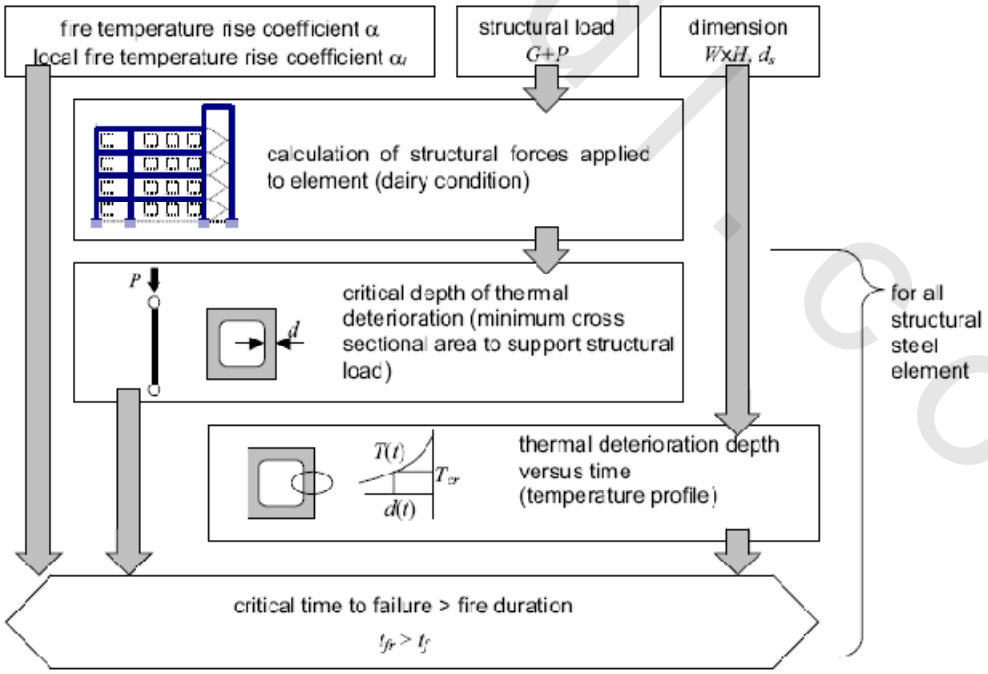


Fig. 2.5 - Calculation Procedure of Critical Time to Failure by Simplified Verification Method.

a) Design strength at high temperature

Fig. 2.6 shows existing results of compressive strength of concrete at high temperature. For mostcases, strength is reduced gradually in the temperature range of 300 and 800°C. At 500°C, strengthdecreases to about 2/3 of nominal design strength at normal temperature. To make conservativeestimate of strength at high temperature, it is assumed that

$$F_c(T) = \begin{cases} (2/3)F_c(20) & (20 \le T \le 500) \\ 0 & (500 < T) \end{cases}$$

where Fc(20) is nominal design compressive strength at normal temperature.

b) Temperature profiles

For a concrete members heated by ISO834 standard fires

$$T_f - T_0 = 345\log_{10}(8t+1) \approx 460t^{1/6}$$

The temperature profile can be approximated by

$$\frac{T(x,t) - T_0}{460t^{1/6}} = \exp(-\frac{cx}{\sqrt{t}})$$

Where c is a coefficient depending on the type of material (c=0.21 for normal weight concrete, 0.23for lightweight concrete). After mathematical approximations, the final form is

$$T(x,t) = 713(cx)^{1/3}\exp(-1.4\frac{cx}{\sqrt{t}}) + T_0,$$

Which is graphically shown in **Fig. 2.7.**

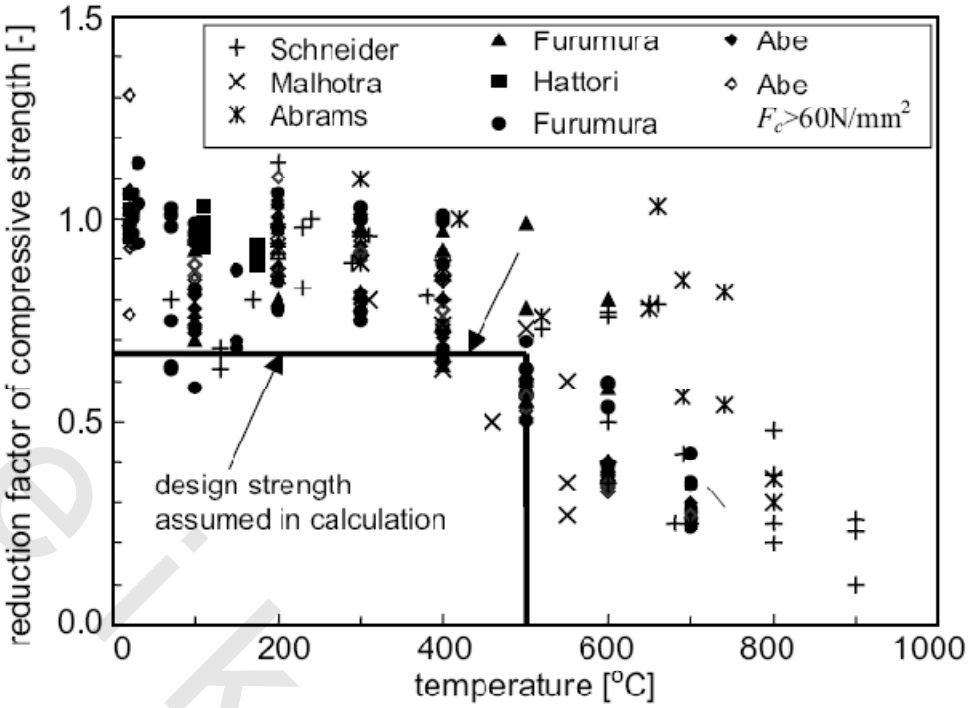


Fig. 2.6 Compressive strength of concrete at high temperatures and design strength (Abrams, 1971).

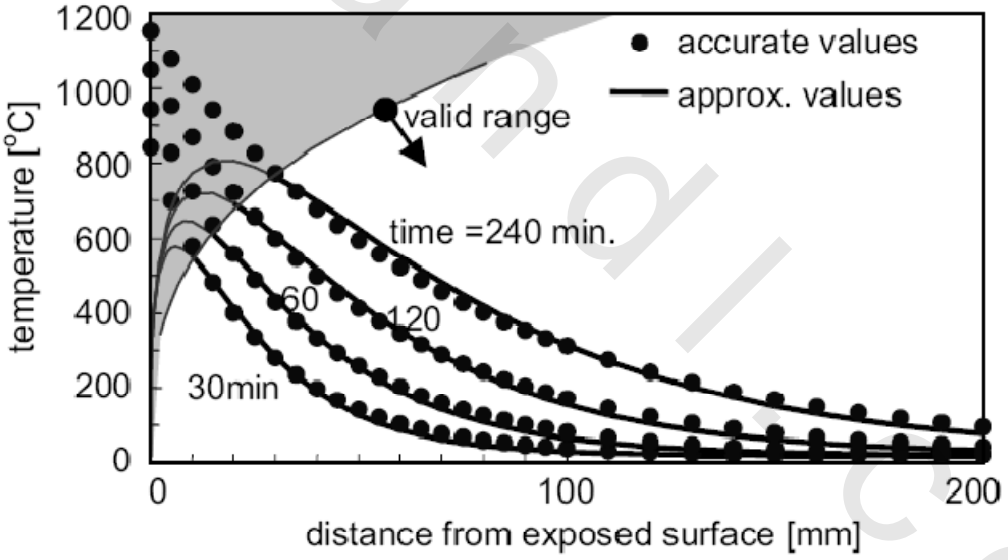


Fig. 2.7 - Temperature profiles against distance from fire-exposed surface as a function of Time.

2.3.2 Actual State of the Codes on Fire Design in Europe.

2.3.2.1 Status and background.

European standard EN 1992-1-2 Eurocode 2: Design of concrete structures – Part 1.2: Generalrules – Structural fire design was approved in Formal Vote in June 2004 and it is expected to bepublished by the end of 2004. European countries have to implement it as national standard. Somesafety related numerical values may be chosen or modified in National Annex which should bepublished in two years. Present national standards may coexist until year 2010. EN 1992-1-2 isrevised version of European prestandard ENV 1992-1-2, published in 1995 for experimental use.

Fire design rules in Eurocode 2 are based on CEB Bulletins on Fire Design of ConcreteStructures, latest edition N° 208 July 1991. Adaptation to the Eurocode system has been done,tabulated data on columns has been completely revised, high strength concrete has been added andsome other modifications have been made.

EN 1992-1-2 is intended to be used in conjunction with

- EN 1990 Basis of structural design
- EN 1991-series Actions on structures
- EN 1992-1-1 Design of concrete structures – General rules and rules for buildings background documentation is available on DIN Livelink for European Standard Bodies

2.3.2.2 Fire actions and load level.

Actions in fire situation are taken from EN 1991-1-2 Actions on structures exposed to fire.Mechanical actions are reduced by combination factors which depend on the type of load.Recommended partial factors for actions and materials are = 1.0.As a simplification the effects of actions may be obtained from a structural analysis for normaltemperature design as:

$$E_{d,fi} = \eta_{fi} \, E_d$$

Where:

Ed is the design value of the corresponding force or moment for normal temperature design, for afundamental combination of actions,

η_{fi} fi is the reduction factor for the design load level for the fire situation.

Reduction factor η_{fi} fi is explained in **Fig. 2.8**. A safe side estimation η_{fi} fi = 0.7 may be used.

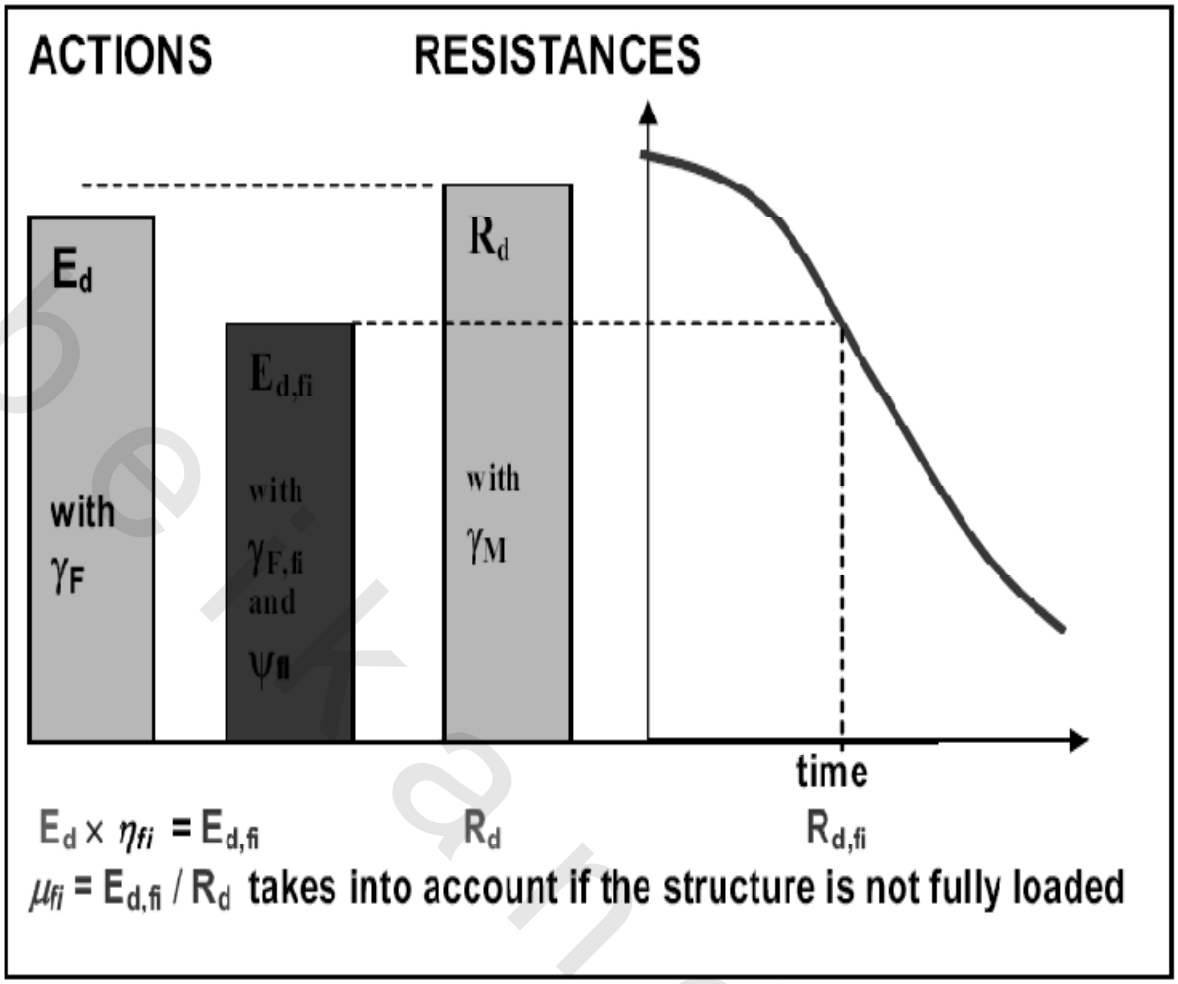


Fig. 2.8- Load level in fire design.

2.3.2.3 Design methods.

In principle there are three assessment methods: tabulated data, simplified calculation methods andadvanced calculation methods. For advanced calculation methods the principles only are given. Twooptional simplified calculation methods are included: 500°C isotherm method as in CEB N° 208 andzone method as in ENV 1992-1-2. In the first one concrete with temperature above 500°C isdisregarded and full strength is used for the rest of cross-section. In the second one the cross section isdivided into zones and more accurate strength value in each zone is used in calculations. Designmethods are illustrated in **Fig. 2.9**.

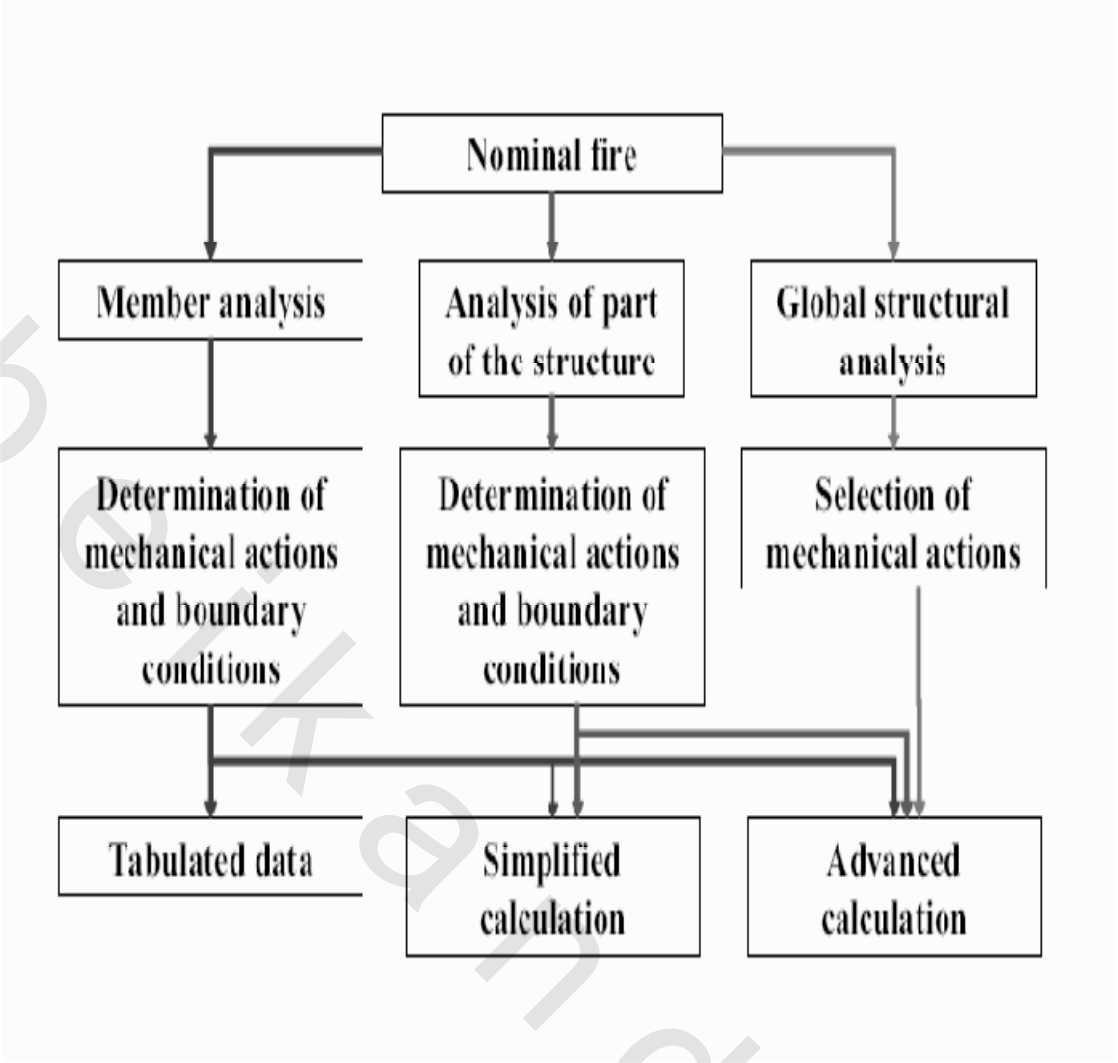


Fig. 2.9- Design methods for standard fire.

2.3.2.4High strength concrete.

Rules for high strength concrete have been included. Strength reduction at elevated temperaturesdepends on the composition and constituents of concrete. National Annex may choose strengthreduction from three recommended classes, see **Fig. 2.10**. There are four optional methods againstspalling, surface reinforcement mesh, tested type of concrete, protective layers or polypropylenefibers.

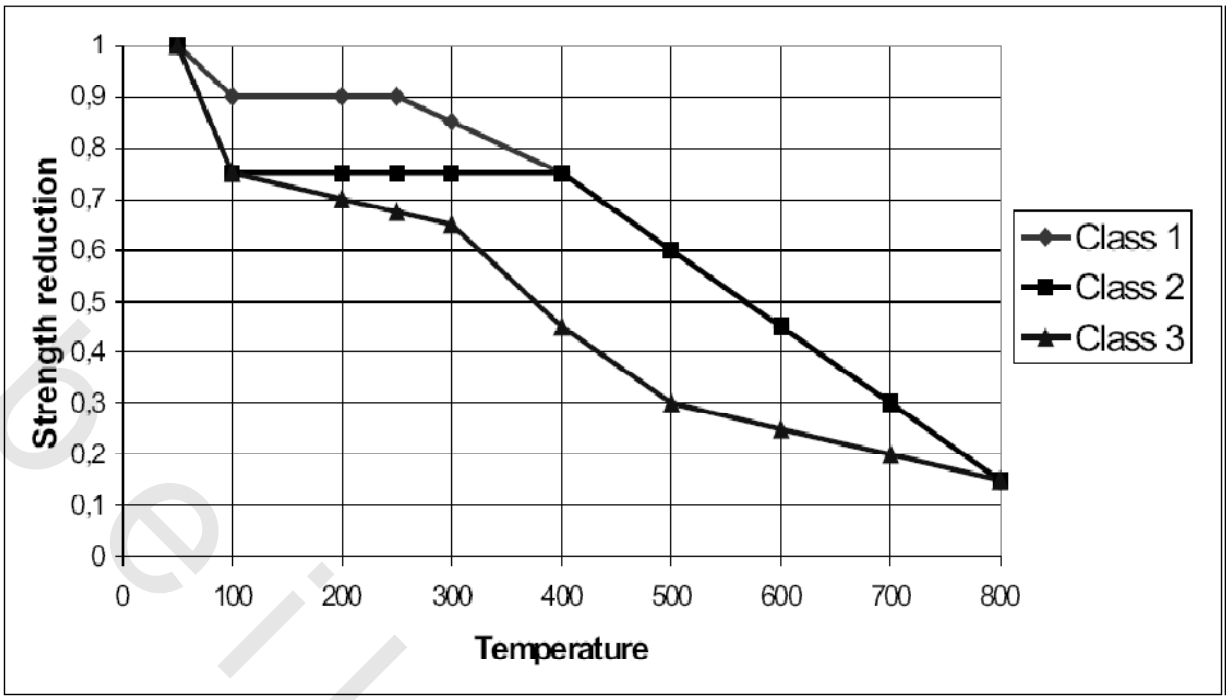


Fig. 2.10 - High strength concrete, strength reduction classes.

2.3.2.5 Shear, torsion and anchorages.

Simplified calculation rules for shear, torsion and anchorage are given in an informative annex. Theserules give guidance how to define the reference temperature of shear reinforcement. It is remindedthat non-linear temperature distributions may cause tensile stresses, reducing shear capacity for sometypes of cross sections. The reference temperature p should be evaluated at points P along the line 'a -a' for thecalculation of the shear resistance. The effective tension area A may be obtained from EN 1992-1(SLS of cracking), see **Fig. 2.11**.

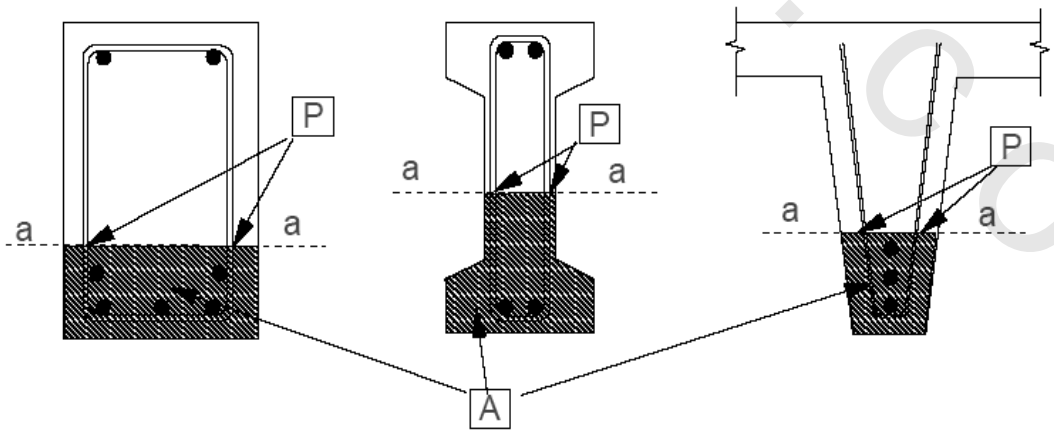


Fig. 2.11- Reference temperature for shear reinforcement.

2.3.3 Summary of current US Codes and Standards and their applicability.

Provisions for fire protection of structures prescribed by the model codes are adopted, in whole orin part, by the building codes of different states or local jurisdictions in the U.S. These fireprotection provisions are handled separately from the structural design procedures for other loadssuch as snow, wind, gravity, and earthquake loads. The sections below summarize the scope thefire protection provisions of the two current US model codes and the relevant standards that arereferenced by these codes.

A-IBC 2000

The objectives of **IBC 2000's** fire protection provisions are primarily to ensure life safety (ofbuilding occupants and emergency fire responders) and, to a lesser extent, to provide a measure ofproperty protection. These objectives can be achieved by using fire-resistance-rated-constructionswith fire resistance ratings that meet the prescribed minimum requirements and by complying withother requirements for active fire protection measures. IBC 2000 prescribes the following methodsfor determining fire resistance ratings of different building components or subassemblies:

-Qualification testing: based on standard fire exposure and test procedure set forth in **[ASTM E119Standard Test Methods for Fire Tests of Building Construction and Materials]**.

- Fire resistance designs documented in approved sources.
- Prescriptive designs of fire-resistance-rated building elements: based on tabulated dataprovided by the code for different structural parts
- Calculated fire resistance method: for concrete, the prescribed methods for calculated fireresistance ratings of different assemblies are those prescribed by **[ACI/TMS 216 StandardMethod for Determining Fire Resistance of Concrete and Masonry Assemblies]**.

In addition to these methods for determining fire resistance ratings of structural components,IBC 2000 also specifies prescriptive requirements for other fire protection measures such as thetypes and locations of fire protection systems (automatic sprinkler systems, standpipe systems, firealarm and protection systems, smoke control systems, smoke and heat vents, etc…) and the meansof egress. A brief description of ASTM E 119 and ACI/TMS 216, which are referenced by IBC2000, is given below.

1- ASTM E 119.

ASTM E 119 specifies laboratory procedure and criteria for determining fire resistance ratings ofbuilding components or assemblies exposed to a prescribed standard time-temperature history. Thisstandard time-temperature history does not represent real fire conditions, which can vary withcompartment size and configuration, ventilation, and fuel loads. The results of this standard test aremeant to provide a relative measure of the fire test response, in terms of fire resistance ratings, ofcomparative test components and assemblies, and are not meant to provide an assessment thestructural performance of the tested elements.Fire testing of concrete components and assemblies in accordance with ASTM E 119 is oftencontrolled by the following two acceptance criteria:

- Heat Transmission Criterion: which requires sufficient thickness of concrete is provided to limit unexposed surface temperature rise of walls, floors, and roofs.
- Load Carrying Ability Criterion: which requires sufficient thickness of concrete cover is provided so that the yield strength of steel reinforcement is at least 50% of that at ambient temperature.

2- ACI/TMS 216.

Standard ACI/TMS 216 is referenced by IBC 2000 as an approved method for calculating fireresistance ratings of concrete, concrete masonry, clay brick and tile masonry assemblies. Thisstandard does not apply to composite metal deck floor or roof assemblies. Except for continuousconcrete slabs and beams, where ACI/TMS 216 provides procedures for calculating fire resistanceratings, guidance for concrete columns and walls are strictly based on prescriptive tabulated data.

The fire resistance rating calculated or determined by methods prescribed by ACI/TMS 216 is alsobased on the standard fire exposure prescribed in **[ASTM E 119]**.The current version of ACI/TMS 216 does not specify a range of concrete compressivestrengths for which its provisions and calculation methods are applicable. In the calculationmethods prescribed for concrete slabs and beams, ACI/TMS 216 allows the concrete compressivestrength, f'_c, to vary as a function of temperature. However, this compressive strength-temperaturerelationship is mainly based on one set of experimental data obtained for normal strength concrete**[Abrams M.S., 1971]**and thus might not be applicable when concrete with higher strength grade is concerned.

a) ACI specifications on the strength of concrete under high temperatures

The current ACI code, ACI/TMS-216 was developed based primarily on the early test data byAbrams (1971), which showed the temperature dependence of the uniaxial concrete compressivestrength as shown in **Fig. 2.12**, **2.13**, and **2.14** are taken from ACI/TMS-216 on "FireResistance" of concretes with three different types of aggregates. The figures illustrate thereduction of compressive strength Offc'= 3900 psi concrete in the temperature range up to T =1600°F. The general trends show that axial pre-loading σ_{pre} = 0.4 fc' leads to higher strengthproperties when compared to unstressed conditions. Furthermore, residual strength after coolingleads to far greater degradation of strength than hot testing.

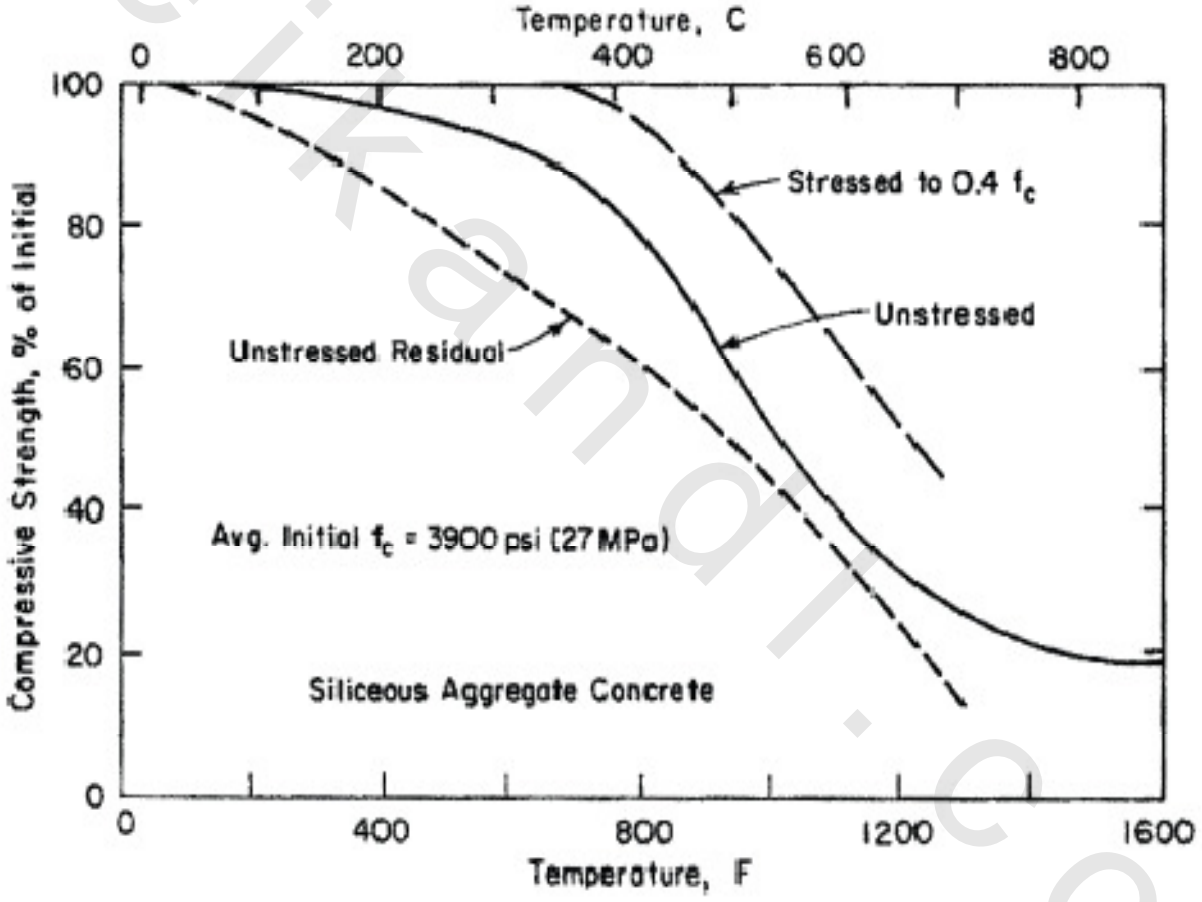


Fig. 2.12-Compressive strength of siliceous aggregate concrete: **[ACI-216]**.

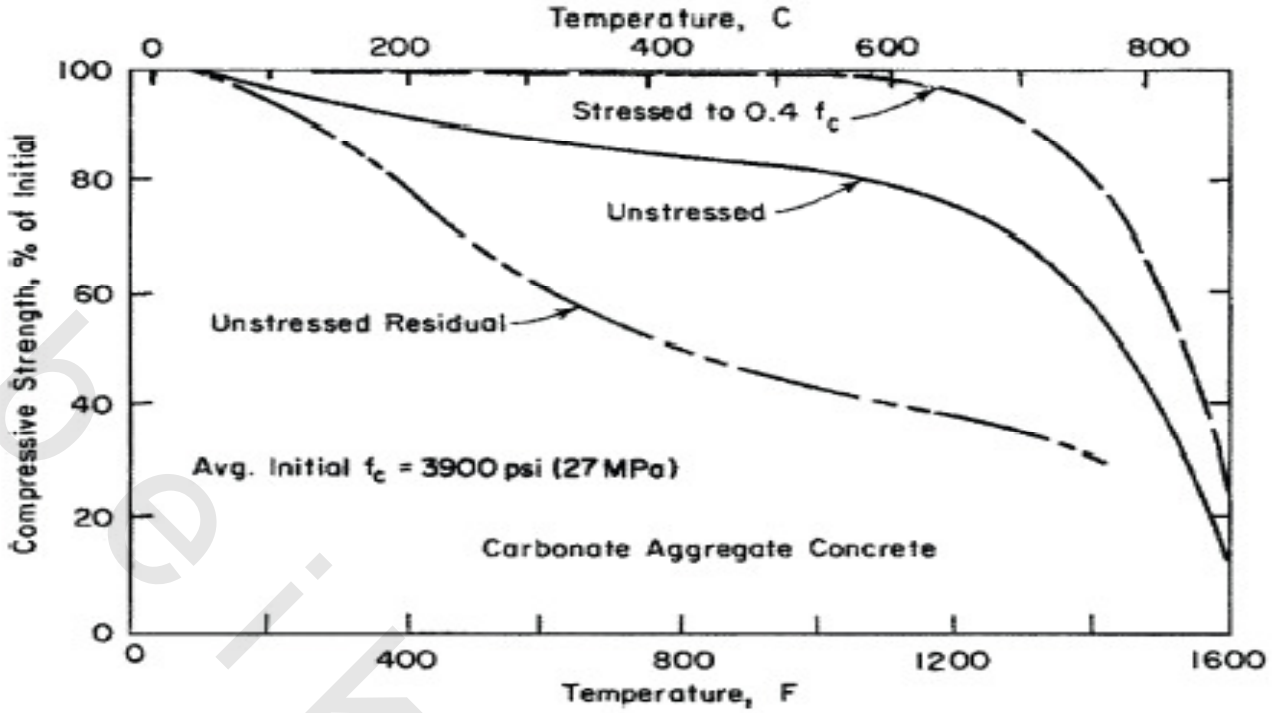


Fig. 2.13-Compressive strength of carbonate aggregate concrete: **[ACI-216].**

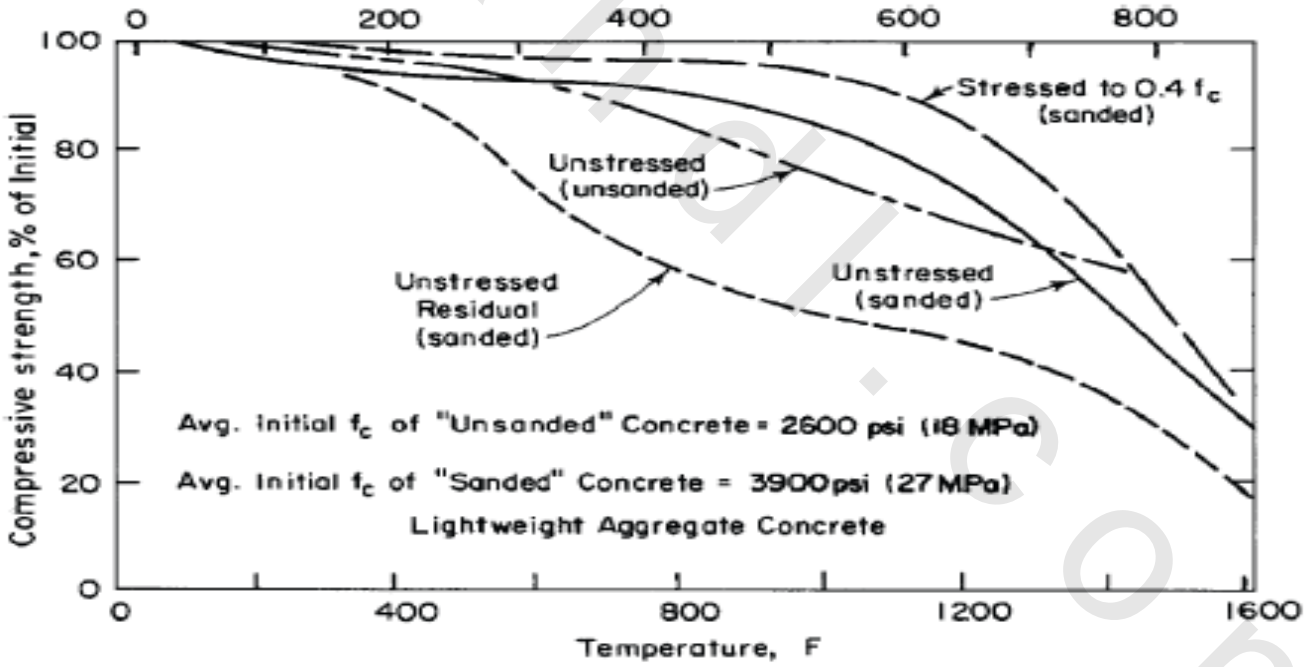


Fig. 2.14-Compressive strength of semi-lightweight concrete: **[ACI 216].**

From **Figs. 2.12, 2.13,** and **2.14** it is clear that ACI-216 provisions do not call for degradation ofthe temperature dependent strength properties when prestressing and hot test

conditions (stressedto 0.4 fc') are considered for all three types of aggregates. Whereas the hot strength of siliceousconcrete as seen in **Fig. 2.12** does not decrease up to 200°F = 93°C, the residual strength (unstressedresidual cold testing) exhibits less than 5% degradation below the reference unloaded concretecompression strength. Interestingly enough, both hot and residual carbonate aggregate concretesas shown in **Fig. 2.13** exhibit higher degradation properties than siliceous aggregate concrete up to 200°F= 93°C. However the reduction relative to the reference unloaded concrete compressive strengthis still less than about 5%. The main difference between siliceous and carbonate aggregateconcrete appears at higher temperatures. For the semi-lightweight concrete, the degradation ofthe concrete under hot test conditions started at about 100°C, but the degradation is notsignificant up to 500°C. It should be pointed out that caution must be used in the practice,because the properties of lightweight concrete really depend on the type of lightweight aggregateused in the concrete.

B- NFPA 5000.

In terms of fire protection, the objectives of model code NFPA 5000 are similar to IBC 2000's.NFPA 5000 provides prescriptive requirements for fire protection of fire walls, fire barrier walls,horizontal assemblies, exterior walls, smoke barrier, vertical openings, mezzanine, joints, concealedspaces, etc. and specifies the following methods for determining fire resistance ratings of fireresistivematerials and constructions:

- Qualification testing: based on standard fire exposure and test procedure set forth in **NFPA 251Standard Methods of Tests of Fire Endurance of Building Construction and Materials**.
- Analytical methods: as prescribed by **ASCE/SFPE 29 Standard Calculation Methods for Structural Fire Protection for Structural Elements or Assemblies** or **ACI/TMS 216 Standard Methods for Determining Fire Resistance of Concrete and Masonry Assemblies**.
- Other approved analytical methods based on the fire exposure and acceptance criteria specified in NFPA 251.

A brief summary of ASCE/SFPE 29 and NFPA 251 is provided below. ACI/TMS 216 is alsoreferenced by model code NFPA 5000 and its summary has been provided in the above section.

1- ASCE/SFPE 29.

ASCE/SFPE 29 provides calculation methods for determining equivalent fire resistance ratings forselected structural members and barrier assemblies made of steel, concrete, concrete masonry, claymasonry, and wood that would have been achieved in the ASTM E 119 standard fire test. Thus, asis the case for ACI/TMS 216 standard, the fire resistance ratings calculated using this standard donot necessarily describe performance for natural fires having a time-temperature relationshipdifferent from that prescribed by ASTM E 119. The concrete provisions of ASCE/SFPE 29 areapplicable to: (1) plain, reinforced, and prestressed concrete made with cementitious materials,aggregates (siliceous, carbonate, sand-lightweight, lightweight); (2) concrete with specifiedcompressive strength f'c not exceeding 69 MPa; (3) cast-in-place, precast, and slabs cast on stay-in placesteel forms where slab is designed to carry all superimposed loads including the slab deadload; (4) walls – single and multi-wythe; (5) floors – single and multi-layer, restrained andunrestrained; (6) roofs – with and without insulation, restrained and unrestrained; (7) beams –restrained and unrestrained; and (8) columns.

Except for the method for calculating the fire resistance ratings and concrete cover for flexuralmembers (provided by ACI/TMS 216 and not provided by ASCE/SFPE 29) and the range ofapplicable concrete compressive strength (limited to 69 MPa in ASCE/SFPE 29 and not defined inACI/TMS 216), the calculation methods for concrete components prescribed by ASCE/SFPE 29 aremostly identical to those prescribed by ACI/TMS 216.

2- NFPA 251.

NFPA 251 Standard Methods of Tests of Fire Endurance of Building Construction and Materialsprescribes laboratory procedures and methods for qualification testing of building materials andassemblies subjected to a standard time-temperature history. The standard time-temperature historyand most of the test procedure and requirements specified by NFPA 251 are identical to thosespecified by ASTM E 119. One minor difference between these two standards is that NFPArequires the furnace temperatures to be recorded at intervals not exceeding 1 min during the testperiod, while ASTM E 119 requires the furnace temperature to be recorded at intervals notexceeding 5 min during the first 2 h of the test, and thereafter at intervals not exceeding 10 min.

2.4 PERFORMANCE-BASED STRUCTURAL FIRE ENGINEERING (PBSFE).

While the field of Performance-Based Structural Fire Engineering is in the developmentalstage, the overall structure of the process has been well defined for some time.

[Grosshandler,2002]outlined the process in summarizing a recent fire resistance workshop. The processincludes both design and analysis components. The analysis components involve the definitionof the design fire exposure, the thermal/mechanical response of the structural assembly(including any fireproofing materials), and structural response of the structural system. Thebroader design processes are shown in **Fig.2.15**, including inputs from building coderequirements and inputs from assembly listings. Here we take a broad view of assembly listingsto include any engineering data that can be deduced from the testing involved in the developmentof the listing (despite the fact that such test data is not made public by the listing organization ortest sponsors at the current time) or fire resistance testing not associated directly with the listingprocess. The recommendations developed in this report are intended to provide additionalengineering information and data from the activity noted in **Fig. 2.15** as "Assembly Listing andData." These infrastructure components are shown above the dashed line, while the actualdesign portion of the process is shown below the dashed line. The design components includethe architectural and structural designs of the building, which form the basis for the fireengineering design.The fire engineering begins with the development of a design fire exposure to the structure.This normally takes the form of a time-temperature curve based upon the fire load, ventilation,and thermal properties of the bounding surfaces (walls, floor, and ceiling). Design fire loads aredependent upon the occupancy and other fire protection features of the building. Significantly,with respect to furnace testing, the performance of the boundaries to limit fire spread is theprimary component of defining the design fire area. Often the exposed fire area is defined byboundaries with sufficient fire resistance to prevent fire spread under the design fire load density.It is significant to note that the time-temperature curves developed in compartment fires mostoften exceed the time-temperature curves used in the test methods like **[ASTM E 119]**. As notedby **[Drysdale ,1999]**, this has been recognized but tacitly accepted since the 1920s in the settingof prescriptive fire resistance requirements for buildings.Based upon the architectural and structural designs, the design fire is used to develop thepassive fire protection design. This involves the selection of fire resistive assemblyconstructions for use as walls, columns, and floor/ceiling assemblies. The assemblies areselected to survive the design fire exposure, to be consistent with the architectural/structuraldesign, and to provide cost-effective protection. It would be normal to develop more than oneset of conceptual designs for further evaluation.

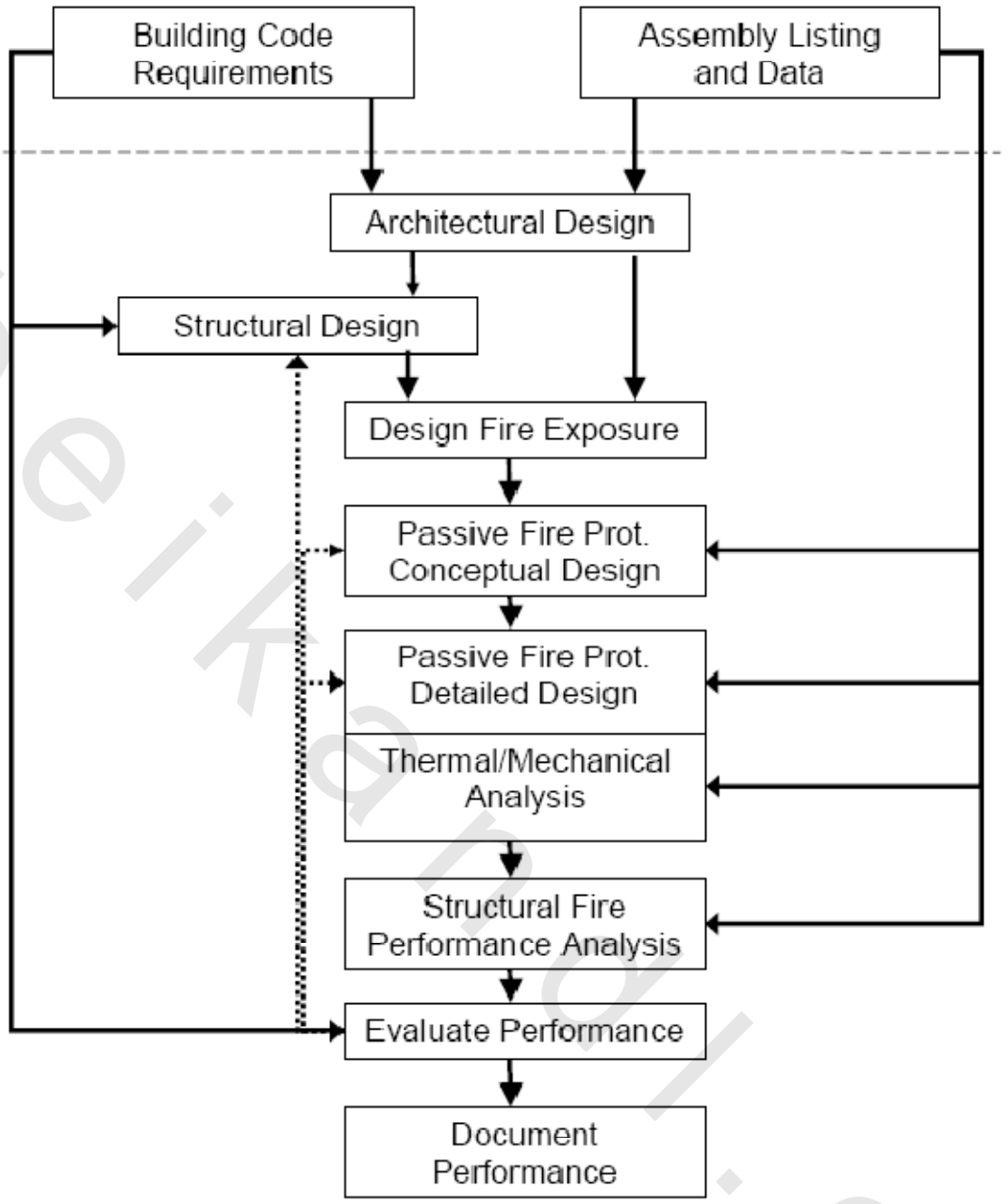


Fig. 2.15-Performance-based structural fire engineering (PBSFE) design process.

Detailed design involves the use of thermal/mechanical models to assess the performance of each conceptual design, resulting in trial protection thicknesses based upon tentative thermal failure criteria. It is typical to perform two-dimensional heat-transfer analyses, but three-dimensional analyses are sometimes required. It is significant that existing models cannot deal with the mechanical performance of the assembly in any substantive manner. Loss of physical integrity of a material or the assembly cannot be modeled at this time. The designer relies entirely upon the results of testing to assure that physical integrity is maintained over the design exposure

period. In most cases, the engineer will seek to use materials and assembliesthat can be relied upon to maintain integrity, or alternatively simple, and somewhat adhoc,assumptions about material loss are made in the design calculations.The final analysis process is the prediction of structural performance of the structure underdesign loads with the structural elements heated according to the heat-transfer analysis. Thisanalysis can be performed for individual elements, for the substructure in the fire area, or for thecomplete structural system. Typically, multiple analyses are performed with more detailedanalysis at the element level and more basic analysis at the structural system level.Based upon the performance of the system, redesign may be indicated. This could includechanges to the structural design (especially if changes here could allow removal of fireproofingaltogether), changes in the passive design concept (e.g., change insulating material), oralterations in the detailed design of the passive fire protection (modify the thicknesses of theinsulation). Other redesign aspects are possible, but these are the most common.

As indicated the assembly listing and data that is, or could be, included in thelisting documentation can contribute to the passive fire protection design, the thermal/mechanicalanalysis, and the structural fire performance analysis. It is important to note that the listingdocumentation (e.g., the test report) is not a public document under the current system so thatthese can only be used with the assistance of the owner of the listing. In addition, the currentlisting may not be directly supported by reported tests. Testing may have been performed with anold version of the protective material and the current material may be accepted under the listingbased upon the listing agency's engineering judgment. While this may be satisfactory forprescriptive use of the product, it has serious limitations with respect to PBSFE.Other data sources, not shown in **Fig. 2.15**, also contribute to these design and analysisprocesses. These include other published data concerning temperature dependent structuralproperties of materials and thermal properties of insulating materials. While some of this data isproduced using standard methods, other data is obtained via ad hoc testing methods.The analysis methods employed in the design process may vary from special purposesoftware to general heat-transfer or structural analysis software. Some software is developed bythe designer, some is developed by government laboratories, and some is commercial software.There is a specific need to address applicability, validation, and verification of these methods foruse in specific Performance-Based Structural Fire Engineering (PBSFE) designs.It is the vision of this report that a fire resistance test in support of PBSFE should be a part ofthe validation and verification (V&V) basis for the application of analysis tools to specific fireresistance designs. All needed data to support the analysis should be developed through tests6designed for that purpose

(e.g., thermal properties and structural properties). The furnace testshould be conducted and instrumented to provide high quality data and boundary conditions toform a data set that can be predicted using the analysis tools. The successful prediction of thetest would form a partial basis for demonstrating the applicability of the models to the particularfire resistance design. The test would further identify any mechanical behaviors such as erosion,cracking, spalling, shrinkage, fastener failures, warpage, and other behaviors that need to bemitigated in the design or accommodated in the design calculations.There is a wide range of testing and reporting aspects of standard fire test methods that arerequired to support PBSFE. These include simple characterization of the test article and theproperties of the component materials, as well as substantive measurements made and theconduct of the test itself. It has been recognized for many decades that realistic fire exposurescan exceed the exposure in ASTM E 119 and that the exposure conditions to the assembly varyamong furnaces operated in a manner consistent with existing test methods. There is also a needto develop and validate thermal properties of insulating materials and the methods andInstrumentation of standard test methods to support PBSFE. There are definite unresolved issuesconcerning the structural conduct of the test to assure that the results are applicable to longerspans and connections found in actual construction. This brings to the fore issues of structuralscaling laws, and the use of structural rather than thermal endpoints for the test. Issues also existwith the conduct of the test with respect to failure criteria. Valuable failure mode data can beprovided by the practice of "testing to failure." These and other issues have received varyinglevels of attention in the testing and research literature. There is no doubt that a new fireresistance test method can become a valuable tool in PBSFE design. The recommendationsincluded in the following sections are in support of this objective.

2.5 Compressive strength of concrete under high temperatures.

The compressive strength of concrete at high temperature is largely affected by the followingfactors:

1) Individual constituent of concrete.
2) Sealing and moisture conditions.
3) Loading level during heating period.
4) Testing under 'hot' or 'cold residual' conditions.
5) Rate of heating or cooling.
6) Duration under an elevated temperature (holding period).

7) Timemaintained in moist conditions after cooling before the strength test is carried out.

8)Number of thermal cycles **[Khoury, 2002].** Before discussing thermal responses of concreteunder high temperatures, it is important to distinguish several different loading scenarios forstrength testing. Three testing methods are commonly used to examine the strength and stiffnessof concrete under a high temperature. These testing methods are referred to as stressed tests,unstressed tests, and unstressed residual strength tests **[Phan, 1996].** The schematic of the threetest methods is shown in **Fig. 2.16**.

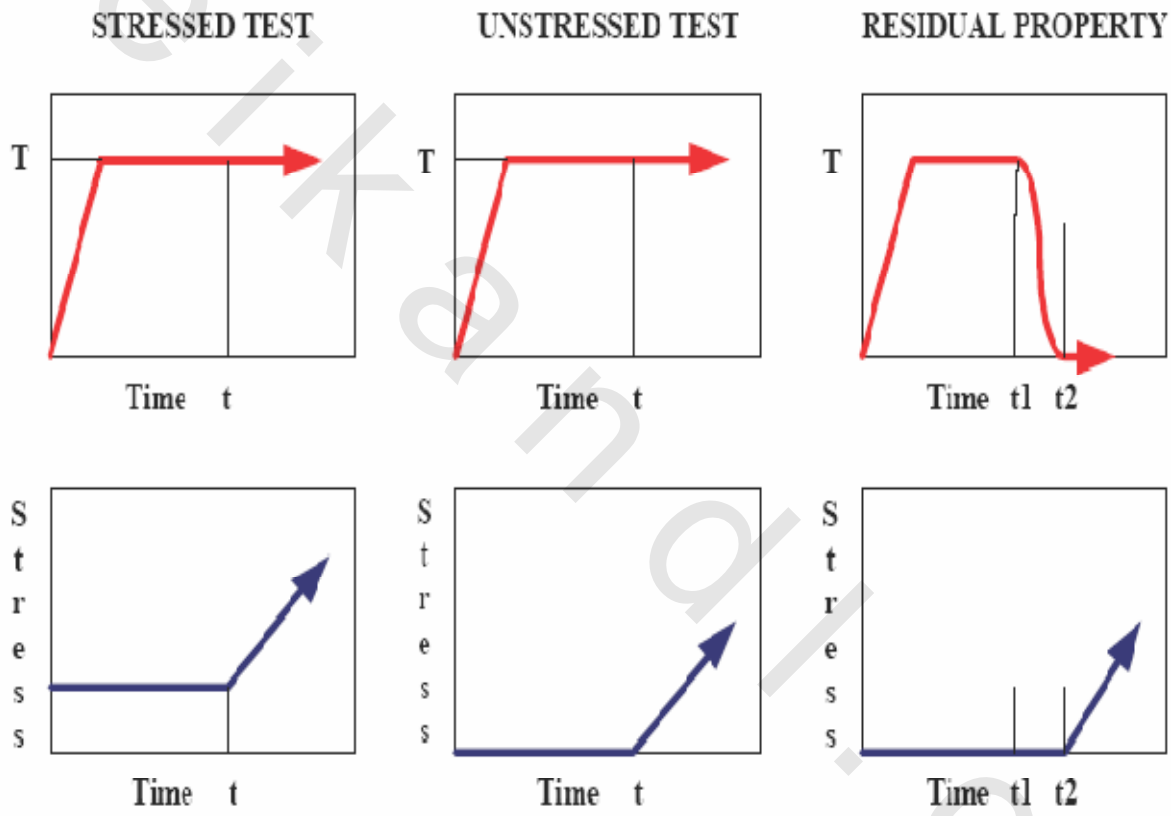


Fig. 2.16-Schematic of temperature and loading histories for the three test methods.

Stressed test is a test in which a preload, generally in the range of 20 percent to 40 percent of theultimate compression strength of concrete at room temperature, is applied to the concretespecimen prior to heating, and the load is sustained during the heating period. After the specimenreaches a steady state temperature condition, the stress or the strain is increased to a prescribedloading rate until the specimen fails. The test results of this test are most suitable for representingfire performance of concrete in a column or in the compression zone of beams and

slabs wherethe structural member is suddenly subjected to an overload during fire due to failure of otherstructural member.

Unstressed test is similar to the stressed test except that the preload level is zero. The concrete isunstressed prior to the final loading. The test results are most suitable for representing theperformance of concrete elements with low stress levels under service conditions and loadedunder high temperatures.

Unstressed residual strength test is a test in which the specimen is cooled to room temperatureafter one or several cycles of heating without preloading. The mechanical load is then applied(after the heat treatment) at room temperature under stress or strain control until the specimenfails. The results of this test are most suitable for assessing the post fire (or residual) propertiesof concrete.

All three test methods can be used to determine the compressive strength (the maximum stress),the modulus of elasticity, the strain at the maximum stress, and the dissipated mechanical energyas function of temperatures.

It has been established that compressive strength of concrete decreases with increasingtemperature. However, determination of the temperature at which the reduction of strengthshould start is not an easy task. This is mainly because test data do not show the simplemonotonic decreasing trend for concrete strength as shown in ACI 216, instead, many test datashowed that the strength of concrete decreases first, and regains or even exceeds the originalstrength in the temperature range of 100 to 200 °C, and then decreases with further temperatureincrease. This phenomenon may be described as a first down, then up, and further down trend(or simply the down-up-down trend) in compressive strength of concrete under hightemperatures. There are some explanations on the physical mechanisms responsible for thedown-up-down trend, although there is no widely accepted theory validated by systematicexperimental studies. The current design codes did not consider such a variation in strength ofconcrete under high temperature.

[Bažant,Z., **et al. 1979]** as illustrated in **Figs 2.17 and 2.18** summarizes early test data fromdifferent researchers. They show large scatter of hot and cold (residual test) experiments whichresulted from a number of different mix designs and testing conditions. One can see from thefigures that some of the curves showed the variation of compressive strength in the lowtemperature range (the down-up-down trend). One should keep in mind that the tests wereperformed under unsealed and unstressed conditions, whereas testing of sealed specimens

wouldfurther reduce the strength values because of internal pore pressures generated by hightemperatures.

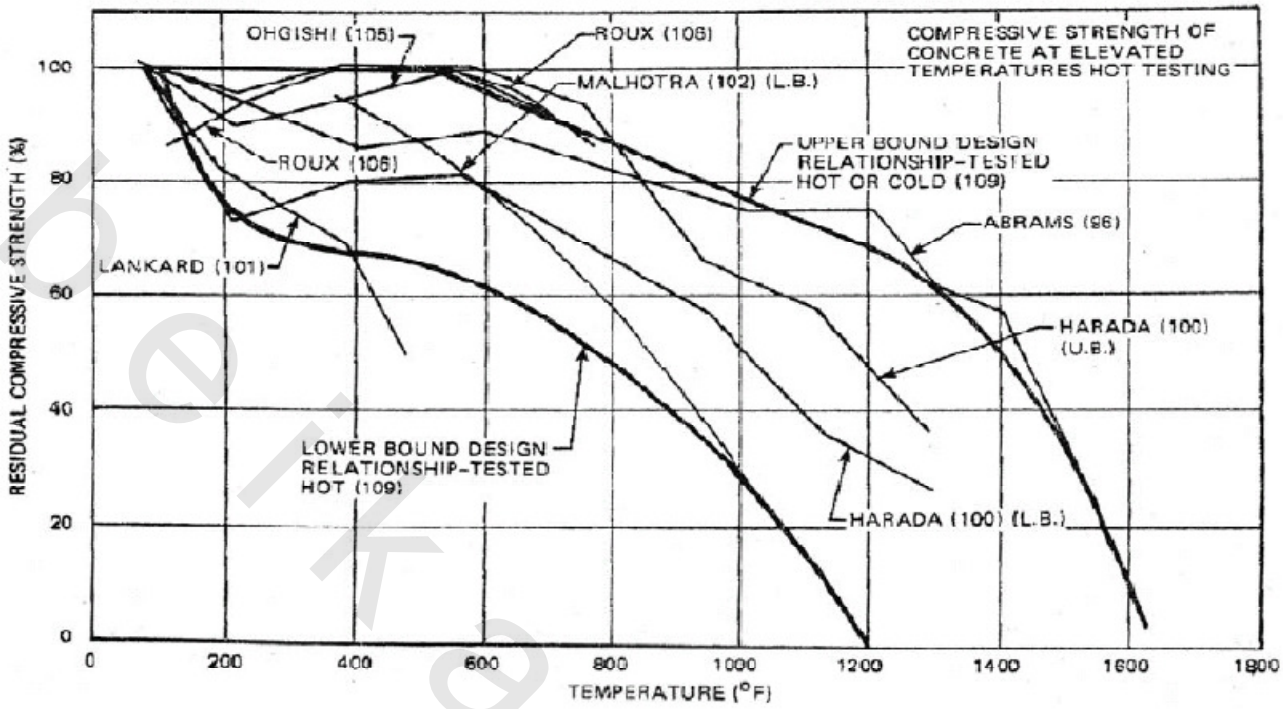


Fig. 2.17-Temperature sensitivity of compressive strength when tested at elevated temperature**[Bažant,Z.,et al. 1979].**

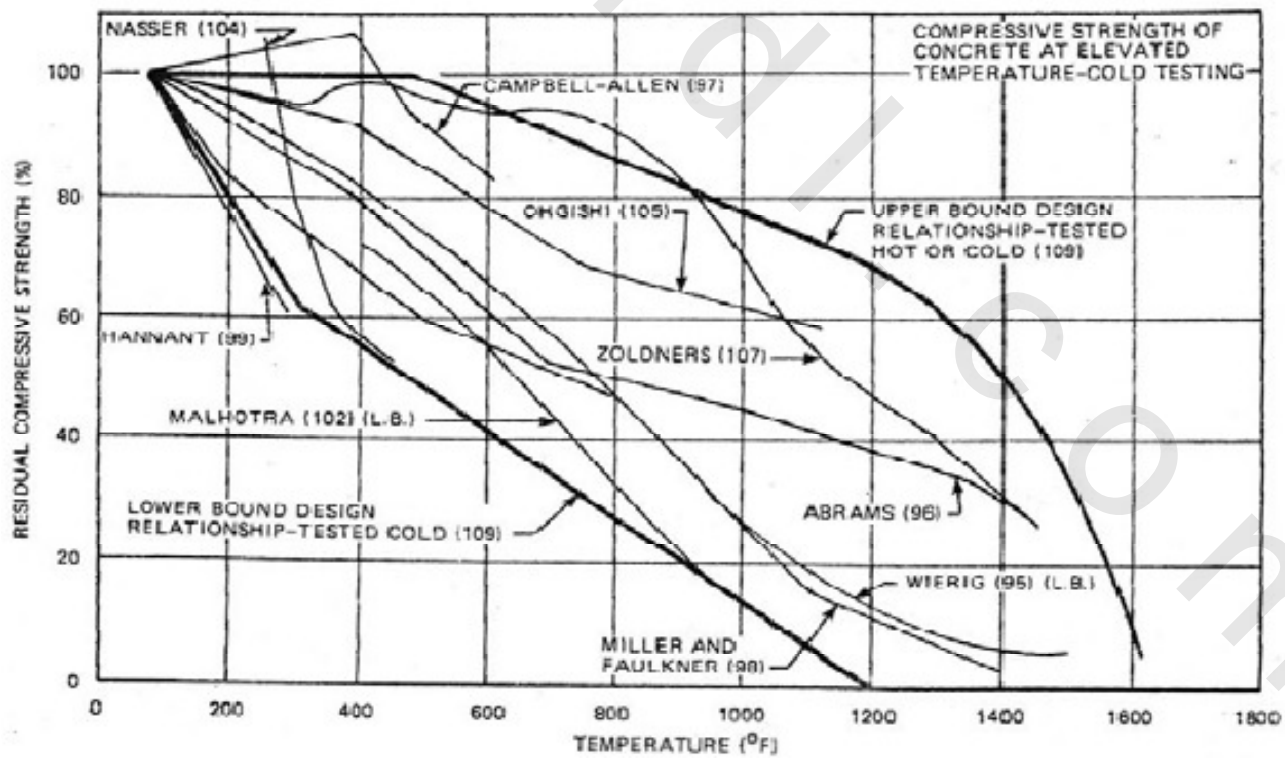


Fig. 2.18-Temperature sensitivity of compressive strength when tested cold (unsealed-unstressed), **[Bažant,Z.,et al. 1979].**

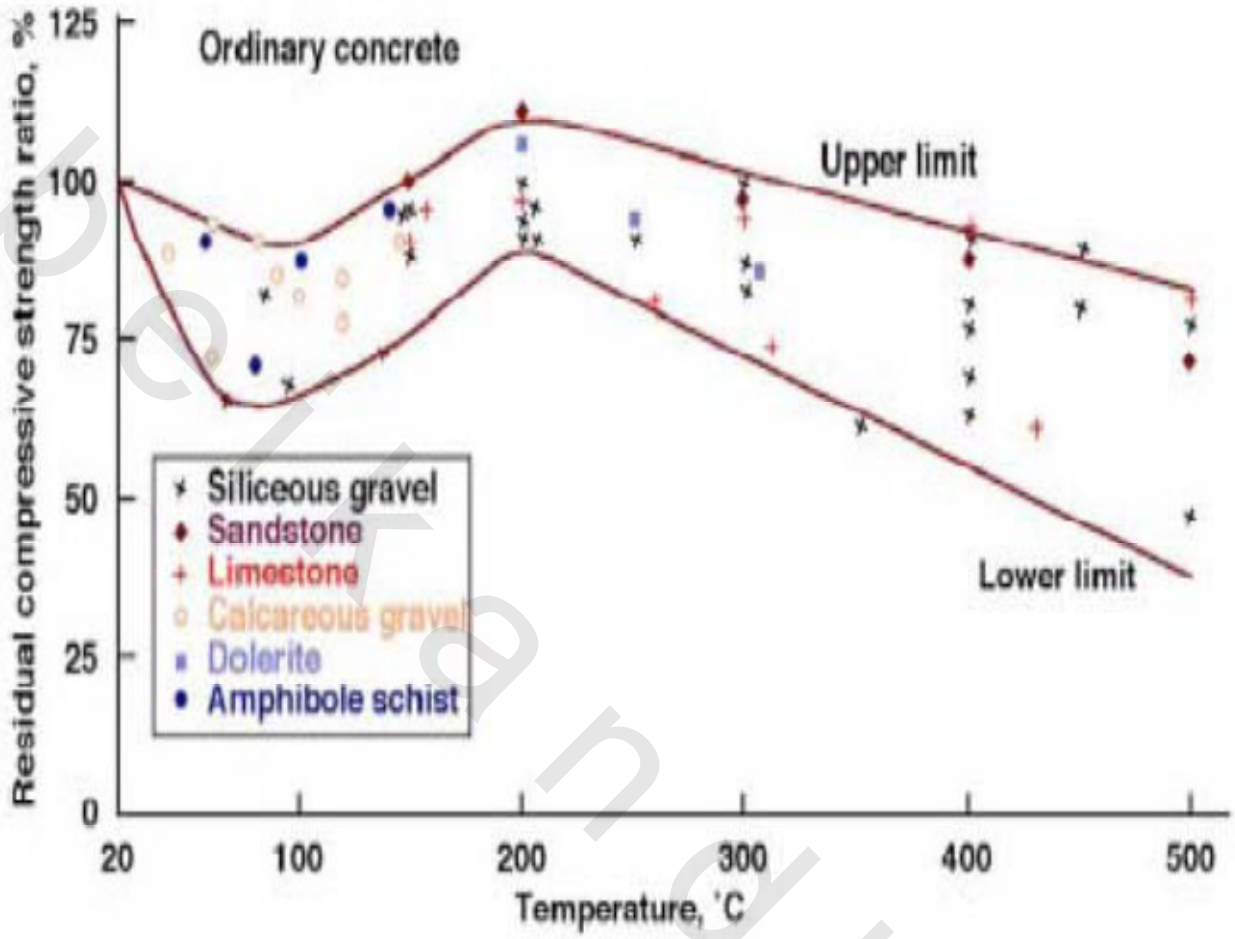


Fig. 2.19- Effect of aggregate on concrete strength materials exposed to high temperatures, **Blundell et al (1976).**

[Blundell, Diamond and Browne, 1976] published the early test data see **Fig. 2.19**, whichillustrates more clearly the down-up-down trend of concrete strength in the range of T = 50 ~200°C. The compression strength of concrete drops, regains and even exceeds at 200°C thereference strength at room temperature depending on the aggregate and moisture condition.Whereas these variations depend strongly on the aggregate type, they disappear at temperatureexposures above 200°C when the strength decreases progressively as expected in oven-dryconcrete materials.

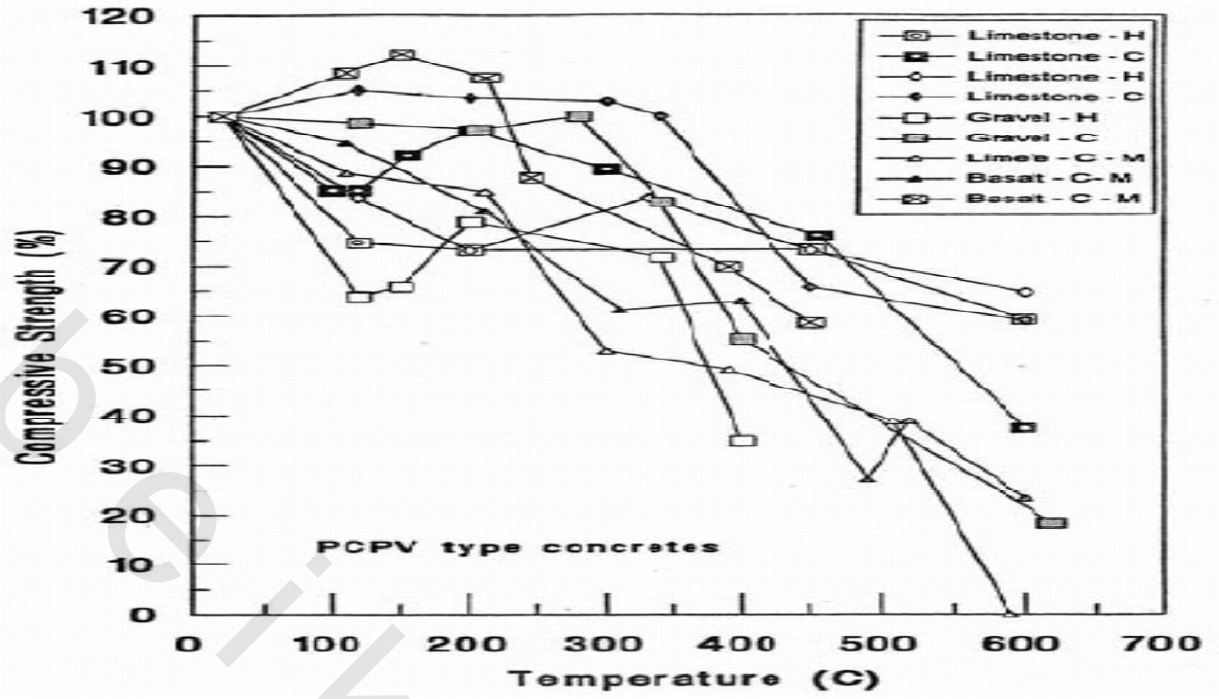


Fig. 2.20-Unsealed PCPV concrete specimens tested hot (H) and cold (C), **Khoury (1984).**

Similar observations to the early test data **[Blundell, Diamond and Browne 1976]** were made by**[Khoury, 1984]**which are reproduced in **Fig. 2.20**. The data demonstrate the strong influenceof aggregate type on the temperature sensitivity of strength of pressure vessel concrete whentested hot (H) and cold (C).

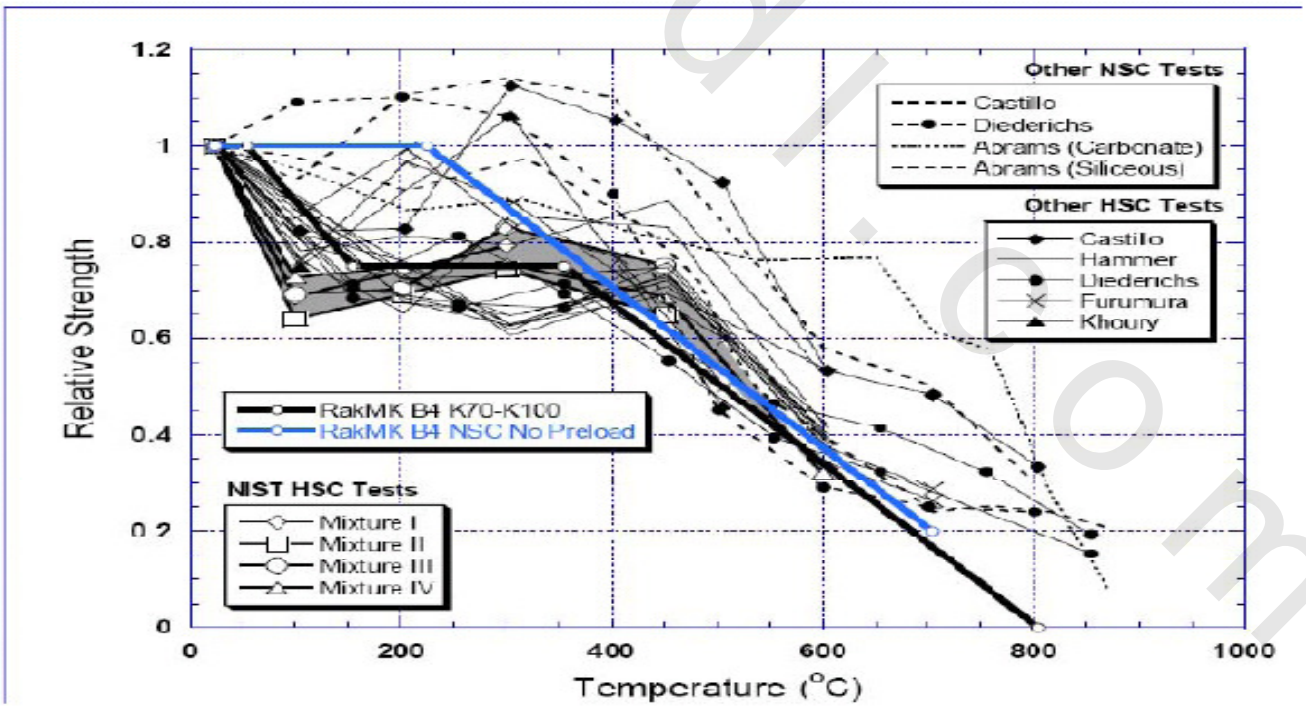


Fig. 2.21-Unstressed High Temperature Data of NSC vs HSC,**Phan and Carino (2003).**

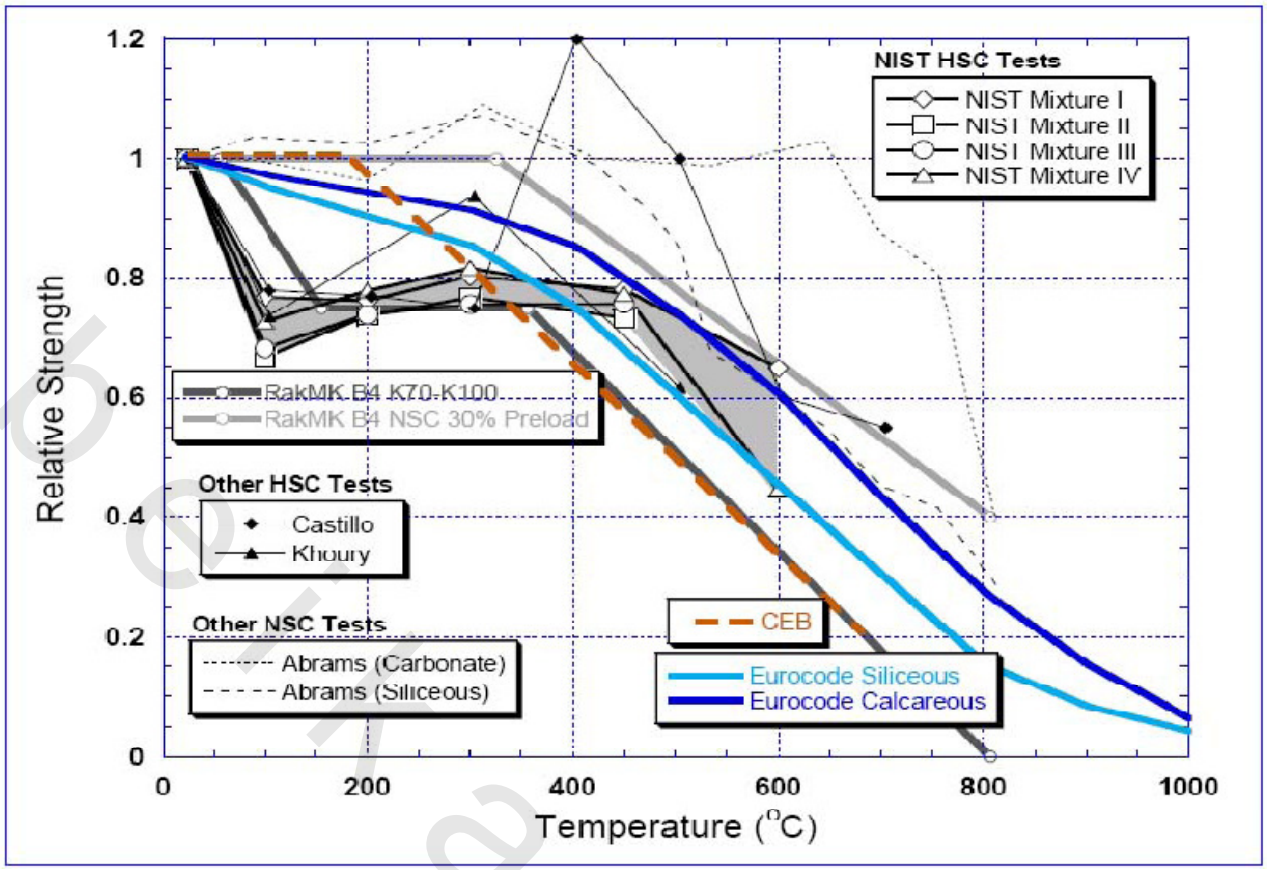


Fig. 2.22-Stressed High Temperature Data of NSC vs HSC, **Phan and Carino (2003).**

Evaluating temperature effects on HPC**, [Phan and Carino, 1998]** presented NIST data of a broadexperimental program using hot testing with and without preloading. They compared the testresults of different HPC mix designs with related design rules. **Figs. 2.21** and **2.22** compareNSC and HSC test data with the predictions of the Finnish Design Code RakMK B4 which wasat that time the only fire code separating temperature limits for HSC and NSC. Whereas there isno degradation of the compressive strength beyond 200°C in the case of normal strength concrete,both **Figs. 2.21** and **2.22** illustrate the early reduction of compressive strength of the highstrength concrete data at 100°C down to 72% of the reference strength at room temperature.On code specifications for the starting temperature for strength reduction, ACI 216 was based on**[Abrams ,1971]** test data, and the strength reduction started from very low temperature rangedepending on loading conditions and aggregate types, as shown in **Figs. 2.12, 2.13**, and **2.14**.Reviewing the provisions for temperature dependent strength values in ASCE Manual No 78(1992) and Eurocode EN 1992-1-2 (2004) we note that the temperature dependent compressivestrength is not reduced for temperatures T≤ 100°C, i.e. fc'T /fc' = 1. In contrast to normal strengthconcrete this temperature reduction factor diminishes to .fc'T /fc'= 0:75 for high

strength concretewhich was discussed by **[Phan and Carino, 1998],** and was further examined very recently by**[Kodur et al. 2008]** in the context of fire resistance of high strength concrete.A very recent study of high temperature compression data was presented by **[Kodur et al. 2008].Figs 2.23 and 2.24** compare test data on NSC and HSC tested under hot conditions with thepredictions of Eurocode EC2 for siliceous and carbonate concretes. Whereas there is nodegradation of the compressive strength up 200°F in the case of normal strength concrete, **Fig.2.24** illustrates the early reduction of compressive strength down to 75% when high strengthconcrete is tested at 200°F.

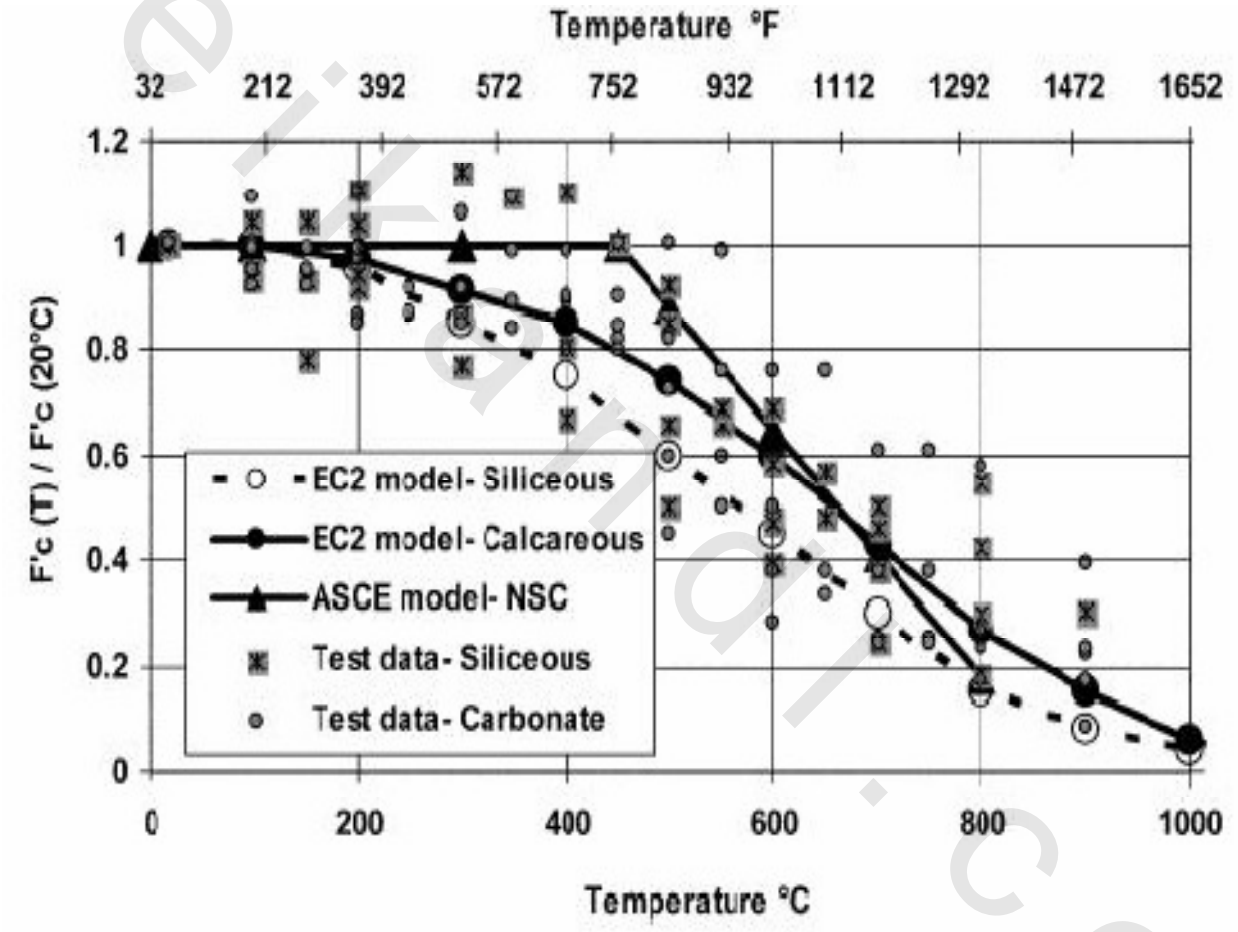


Fig. 2.23 High Temperature Models for NSC Concrete, **Kodur et al. (2008).**

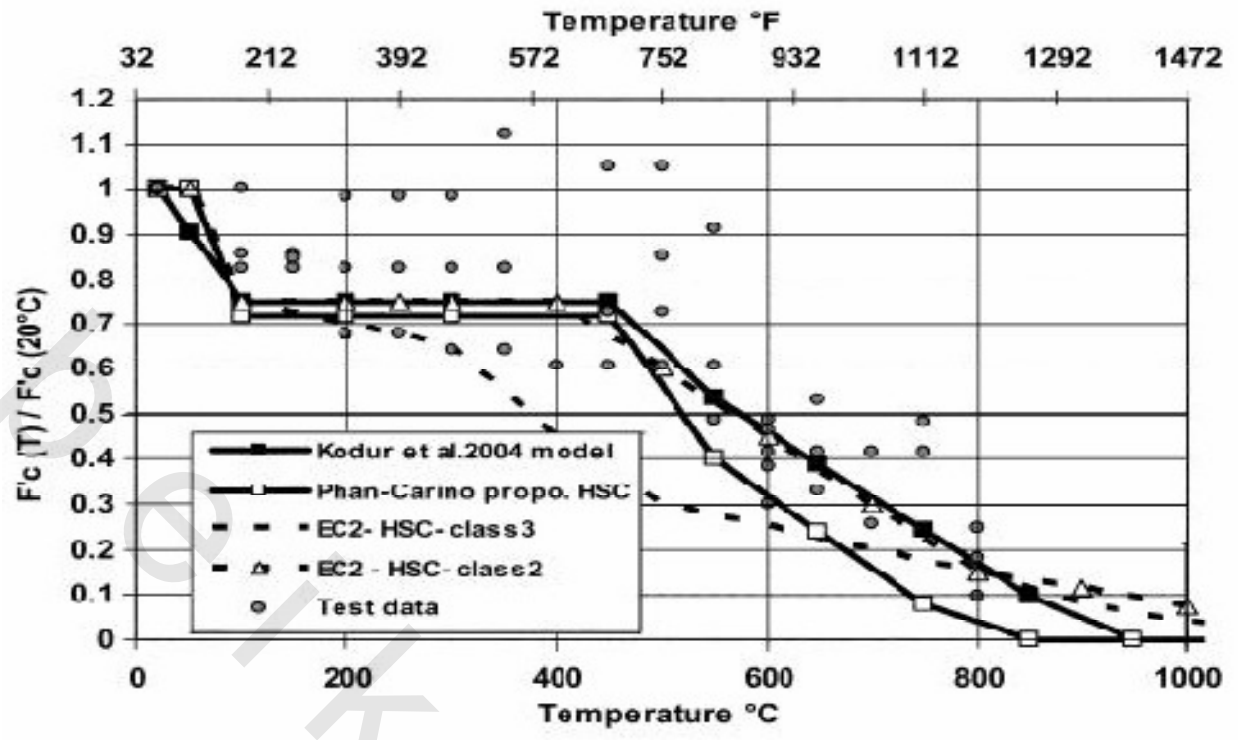
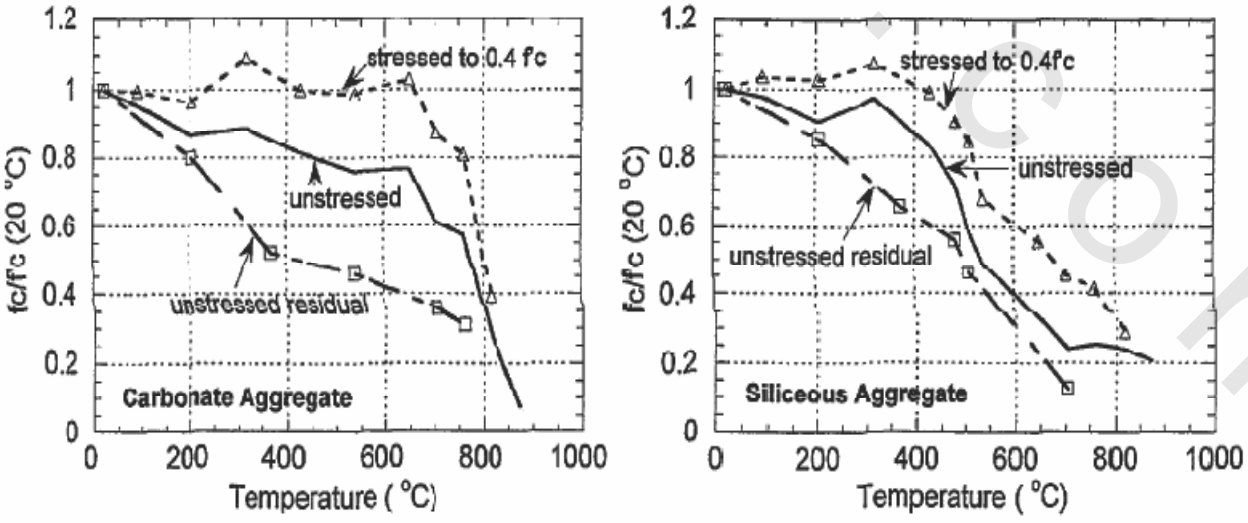


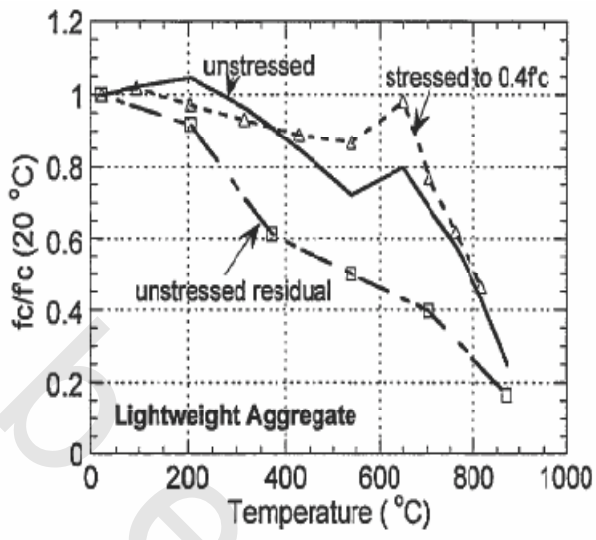
Fig. 2.24 High Temperature Models for HSC Concrete, **[Kodur et al. 2008].**

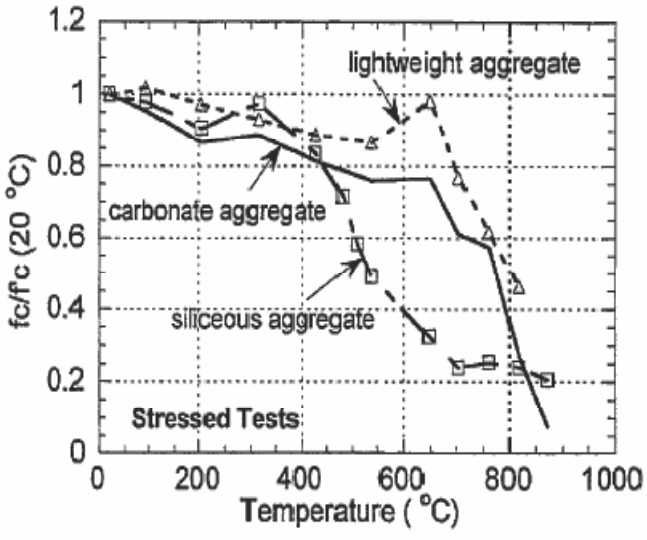
2.5.1 Experimental results on strength of concrete under high temperatures.

[Abrams, 1971] conducted a study of four variables including aggregate types (carbonatedolomite sand and gravel, siliceous, and expanded shale lightweight aggregates), testing methods(unstressed, stressed and unstressed residual experiments), concrete strengths (ranging from22.8 MPa to 44.8 MPa), and temperatures (from 93°C to 871°C). The specimens were75 mm×150 mm cylinders. Abrams' test data are important because they were used as a mainreference in the current ACI design code for concrete under high temperatures.

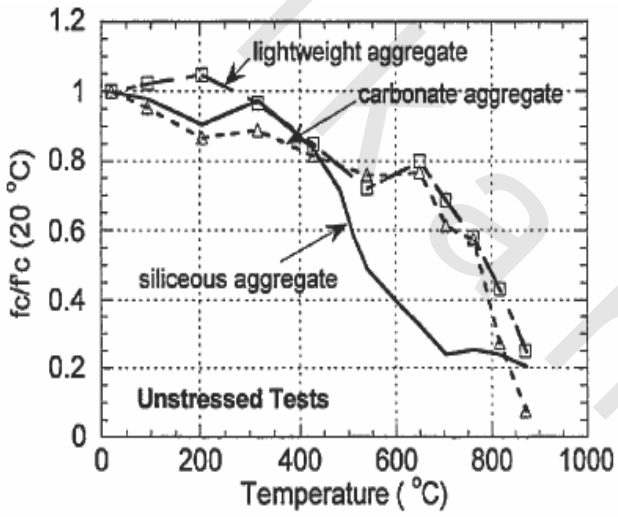


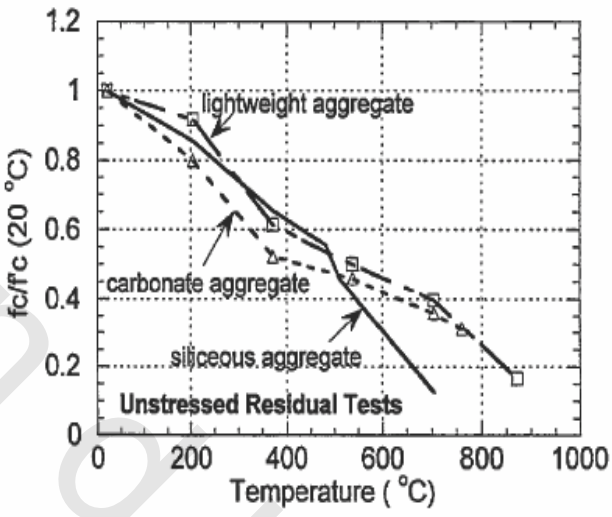
(a)Carbonate aggregate concrete.b) Siliceous aggregate concrete.





(c) Lightweight aggregate concrete.(d) Stressed tests.





(e) Unstressed tests.(f) Unstressed residual tests.

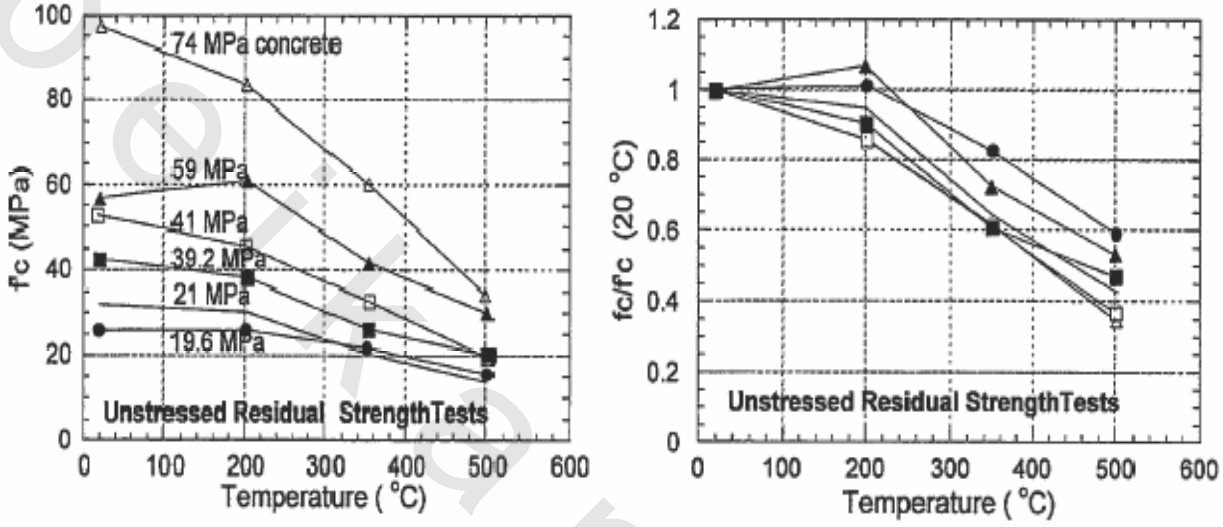
Fig. 2.25-Hot isothermal test results obtained by **[Abrams ,1971].**

Fig. 2.25 summarizes the test results. Up to about 480°C, all three concretes exhibited similarstrength loss characteristics under each test condition (stressed, unstressed, and unstressedresidual). Above 480°C, the siliceous aggregate concrete had greater strength loss and retainedless strength for all three test conditions. Specimens made of carbonate aggregates andlightweight aggregates behaved about the same over the entire temperature range and retainedmore than 75 percent of their original strength at temperatures up to 649°C in unstressed tests.

For the siliceous aggregate, the strength was 75 percent of the original strength at 430°C.Compressive strengths of specimens with preload (stressed tests) were generally 5 percent

to 25percent higher than those without preload (unstressed tests). This may be due to the possibilitythat the preload helps to close existing cracks in concrete. The unstressed residual specimens had the lowest strengthcompared with the stressed and unstressed specimens tested at high temperatures. This isbecause the cooling of concrete actually generates very significant damage in concrete, as shownrecently by **[Lee et al. 2008]**.

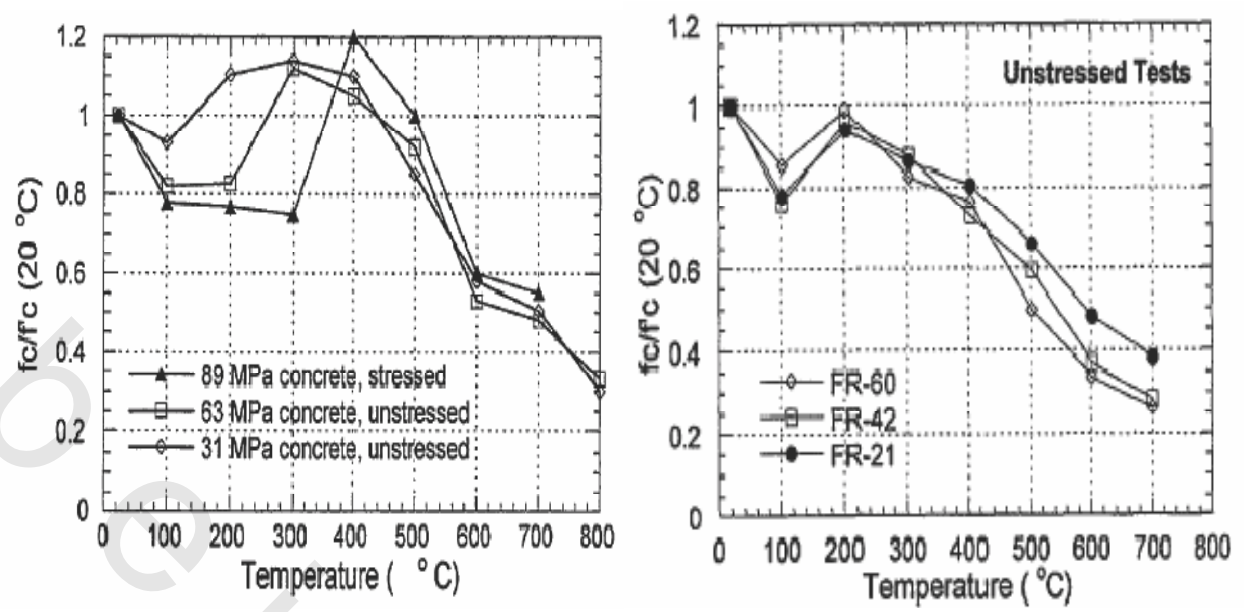
The test results of **[Abrams,1971]** indicate that strength recoverytook place only in a limited temperature range in the case of the stressed and unstressedexperiments.



(a) Compressive strength (b) Normalized strength

Fig. 2.26-Residual compressive strength obtained by **[Morita et al. 1992].**

[Morita et al. 1992] conducted unstressed residual strength tests as conducted in **Fig. 2.26**. The specimenswere 100 mm×200 mm cylinders. The heating and cooling rate were 1°C/min and targettemperatures were 200°C, 350°C, and 500°C. The holding time at target temperatures to allow asteady state was 60 min. The test data showed that high strength concrete has higher rate ofreduction in residual compressive strength.



(a)(b)

Fig. 2.27-Compressive strength vs. Temperature under hot isothermal conditions by (a) **Castillo and Durani, 1990** and (b) **Furumura et al. 1995**.

The effect of elevated temperatures on concrete strength and load-deformation behavior of HighStrength Concrete (HSC) and Normal Strength Concrete (NSC) were investigated by **[CastilloandDurani, 1990]**. Type I Portland cement with natural river sand and crushed limestone wereused for preparing the concrete specimens in the form of 51mm×102 mm cylinders. **Fig.2.27(a)** shows the test results. In the case of the stressed experiment, 40 percent of the ultimatecompressive strength at room temperature was applied to the specimens and sustained during theheating period. In the unstressed experiment, when exposed to temperatures in the range of100°C to 300°C, HSC showed a 15 percent to 20 percent loss of compressive strength, whereasthe NSC showed almost no strength loss in the temperature range. HSC recovered its strengthbetween 300°C and 400°C, reaching a maximum value of 8 percent to 13 percent above thestrength at room temperature. At temperature above 400°C, HSC progressively lost its compressive strength whichdropped to about 30 percent of the room temperature strength at 800 °C. The trend of NSC wassimilar to that of HSC.**[Furumura et al. 1995]** performed unstressed tests and unstressed residual tests on50 mm×100 mm concrete cylinders using three compressive strength levels: 21 MPa (normalstrength concrete FR-21), 42 MPa (intermediate strength concrete FR-42), and 60 MPa (highstrength concrete FR-60). The heating rate was 1°C/min and target temperatures were from100°C to 700°C with an increment of 100°C. The time at target temperatures to allow a steadystate was two hours. The concrete was made from ordinary

Portland cement. They observed that,for the unstressed tests, the compressive strength decreased at 100°C, recovered to roomtemperature strength at 200°C and then decreased monotonically with increasing temperaturebeyond 200°C as seen in**Fig. 2.27(b)**. For the unstressed residual tests, the compressive strengthdecreased gradually with increasing temperature for the entire temperature range without anyrecovery.

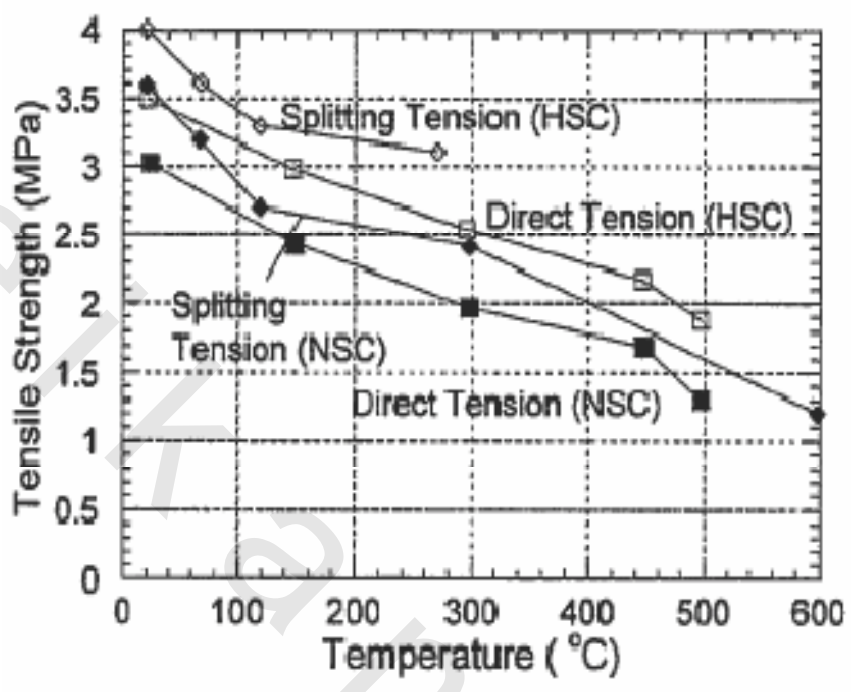


Fig. 2.28-Unstressed residual tensile strength obtained by **[Noumowe et al. 1996].**

[Noumowe et al ,1996] conducted unstressed residual strength tests to compare the performanceof HSC exposed to high temperatures with NSC. The specimens were 160 mm×320 mmcylinders and 100 mm×100 mm×400 mm prisms. Normal strength concrete (38.1 MPa) andhigh strength concrete (61.1 MPa) were used. The prismatic specimens had enlarged ends andwere used to measure tensile strength. Calcareous aggregates were used for both concretes. Thespecimens were heated at a rate of 1°C/min to target temperatures of 150°C, 300°C, 450°C,500°C, and 600°C, which was maintained for one hour, and then allowed to cool at 1°C/min toroom temperature. Uniaxial compressive tests, splitting tensile tests, and direct tensile tests wereperformed to obtain residual compressive strength, modulus of elasticity, and residual tensilestrength versus temperature relationships. **Fig. 2.28** shows the residual tensile strengthrelationship. Residual tensile strengths for NSC and HSC decreased similarly and almost linearlywith increase of temperature. Tensile strengths of HSC at all temperatures were 15

percenthigher than those of NSC. Also, the tensile strengths measured by splitting tension experimentswere higher than those obtained in direct tension.

2.5.2 The effect of pre-loading on strength of concrete under high temperatures.

Preloading during heating has positive effects on both compressive strength and modulus of elasticity of concrete. **Fig. 2.29** demonstrates the positive effects (measured in the 'hot' statefor unsealed CRT HITECO ultra-high performance concrete). Comparison of **Fig. 2.18 (a)** and**(b)** shows that the compressive strength and modulus of elasticity of the specimen undersustained loading is larger than those of specimens without the sustained loading. This aspect canbe explained from the fact that compressive preloading inhibits crack development – althoughthis explanation has not been fully validated.

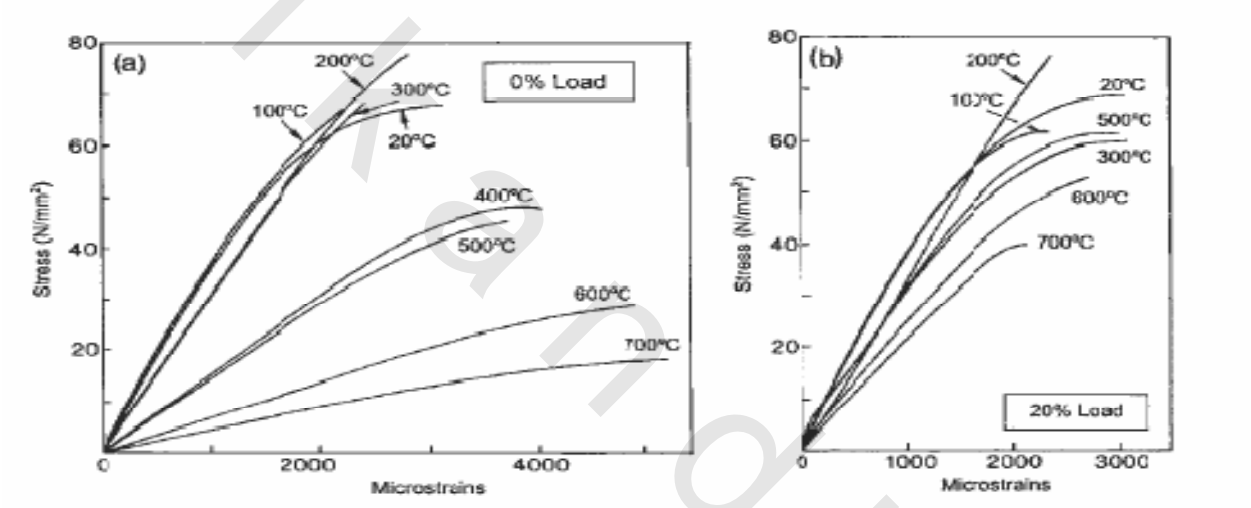


Fig. 2.29-The effects of loading and temperature during heating in uniaxial compression of unsealed concrete specimens **[Khoury, 2002].**

Fig. 2.30[Khoury, 2002] shows the positive effect of temperature upon the residual (aftercooling) compressive strength and the elastic modulus of unsealed C70 HITECO concretecontaining thermally stable Gabbro Finnish aggregate. The specimens were heat cycled at2°C/min under 0 percent and 20 percent of preload in compression, respectively. The results areshown in terms of percentage of strength and elastic modulus prior to heating. The figure showsthe down-up-down trend of compressive strength as well as the increase of compressive strengthwith 20% preloading. It also shows the enhancement of stiffness with the 20% preloading.

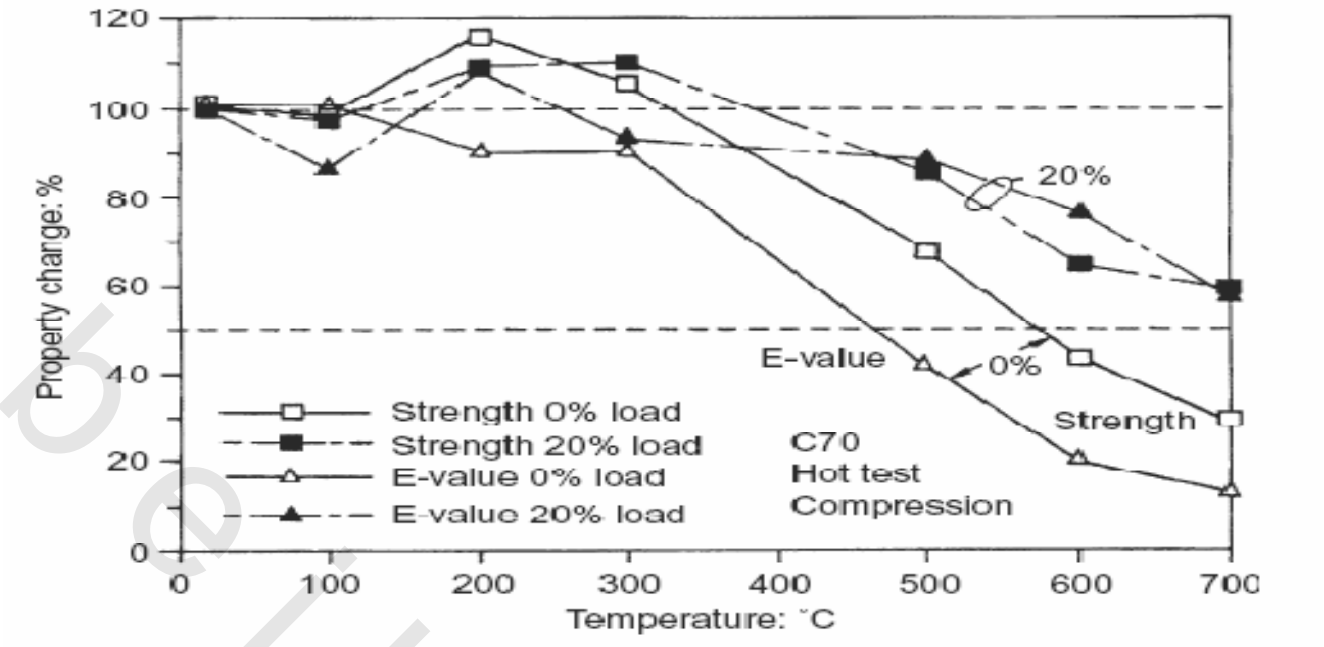


Fig. 2.30 Effect of temperature upon the residual (after cooling) compressive strength and elastic modulus of unsealed C70 HITECO Concrete -20 percent load: expressed as a percentage of strength prior to heating **[Khoury, 2002].**

2.6 Stiffness of concrete under high temperatures.

2.6.1 Experimental results on stiffness of concrete under high temperatures.

Similar to compressive strength of concrete, stiffness of concrete also decreases with increasingtemperature. **[CastilloandDurani, 1990]** conducted tests to determine modulus of elasticity ofHSC and NSC under hot condition where the modulus decreased by 5 to 10 percent in thetemperature range of 100°C to 300°C as seen **Fig. 2.31**. Beyond 300°C, the elastic modulusdecreased at a faster rate with increase in temperature. At 800°C, the elastic modulus was only20 to 25 percent of the value at room temperature.**[Morita et al. 1992]** observed that high strength concrete have higher rate of reduction inmodulus of elasticity than normal strength concrete after being exposed to temperatures up to500°C where the specimens of 100 mm×200 mm cylinders were tested under unstressed residualconditions **(Fig. 2.32 (a) and (b))**. **[Noumowe et al, 1996]** conducted residual modulus ofelasticity of NSC and HSC where the modulus remained approximately 10 percent to 25 percenthigher than those of NSC for entire temperature range **(Fig. 2.32(c)).[Furumura et al. 1995]**performed unstressed residual modulus of elasticity tests using three compressive strength levels.He observed the modulus of elasticity, in general, decreased gradually with increase oftemperature. **Fig. 2.32(d)** shows the test results.

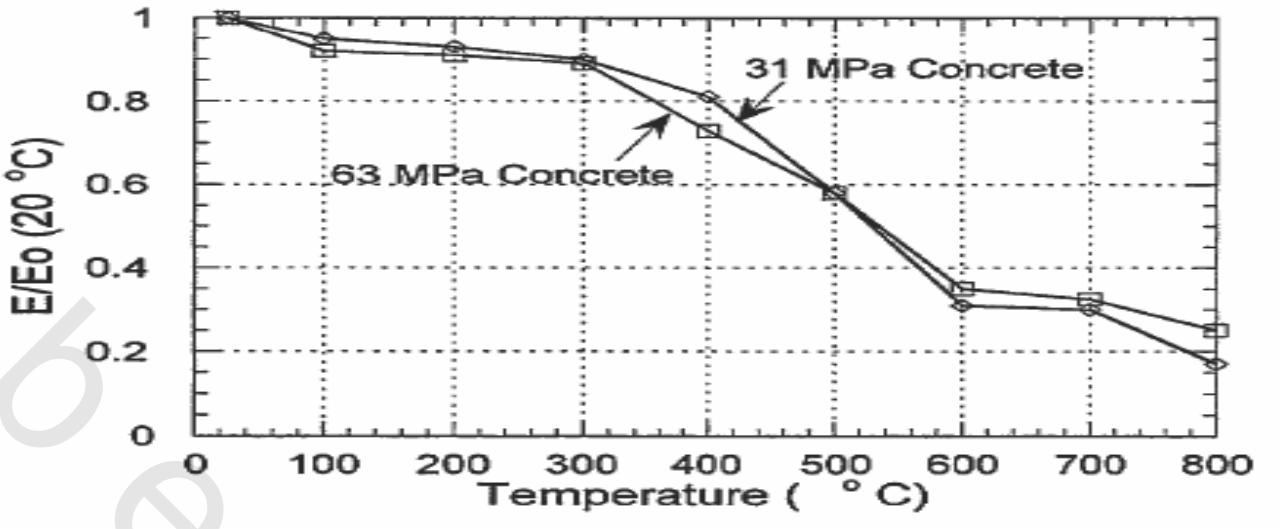
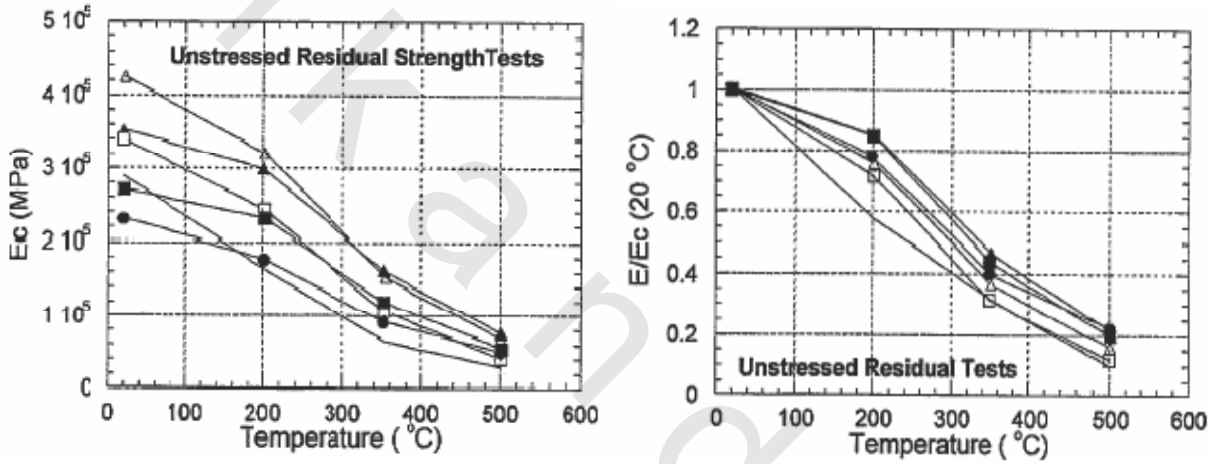
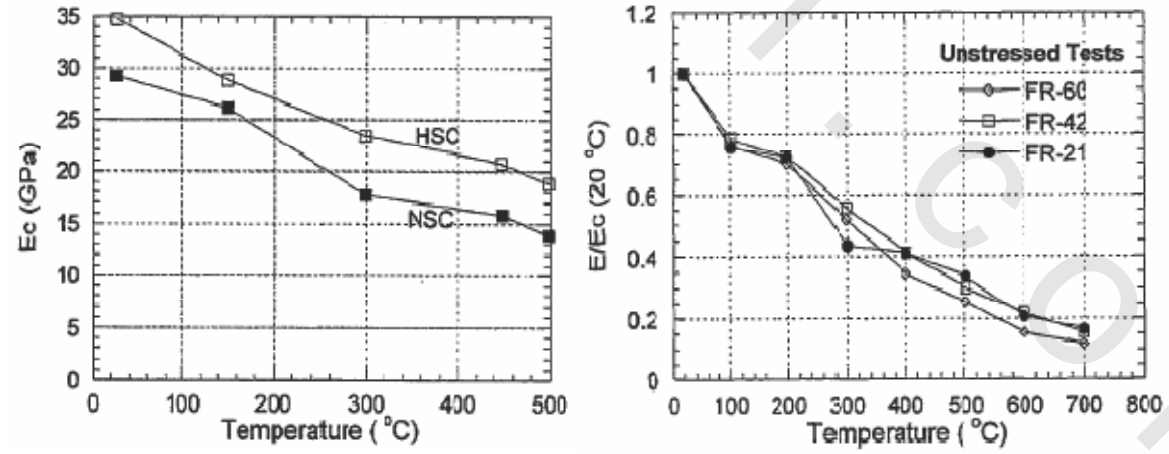


Fig. 2.31 Modulus of Elasticity under hot conditions by **[Castillo and Durani, 1990].**



(a) Residual elastic modulus.(b) Residual normalized elastic modulus.



(c) Residual elastic moduli for HSC and NSC. (d) Normalized elastic Modulus of HSC and NSC.

Fig. 2.32 Hot isothermal test results obtained by (a), (b) **Morita et al.,1992** and (c) **Noumowe et al., 1996, and (d) Furumura et al., 1995.**

Fig. 2.33 shows the effect of cooling methods on modulus of elasticity of concrete. The slow cooling, means cools the specimens in a chamber at the rate of 1°C/min. from the targetedtemperatures down to room temperature; the natural cooling means cooling the specimens underroom temperature; and the fast cooling stands for cooling of specimens in a water tank. Test datashowed that the water cooling (fast cooling) resulted in the largest reduction in the stiffness ofconcrete. This is due to the fact that there is a steep thermal gradient in the concrete during thefast cooling which causes more damage than the natural and slow cooling methods.

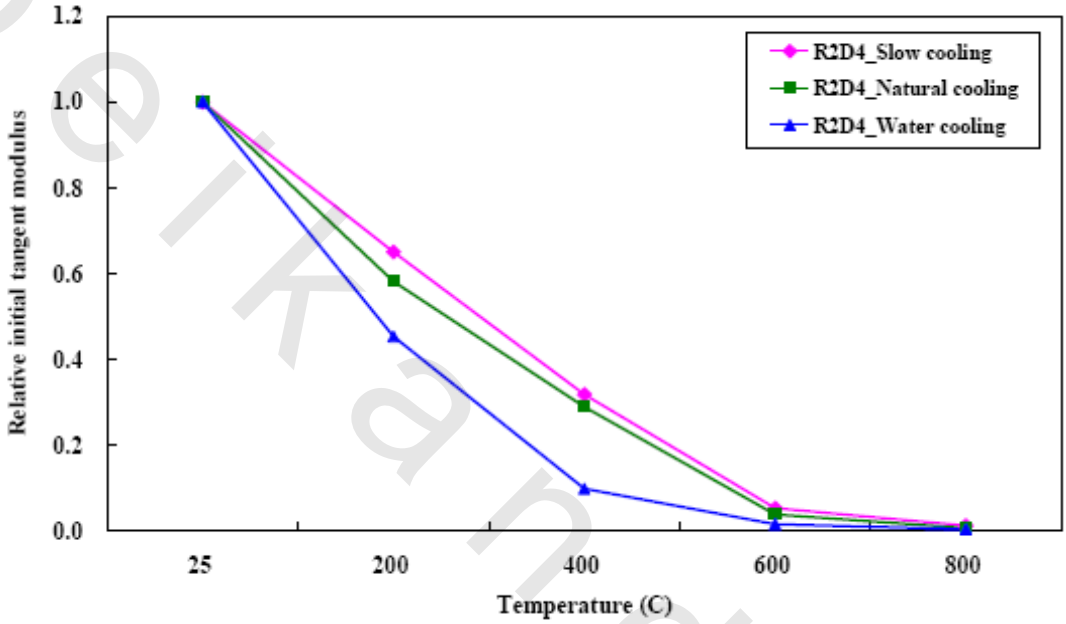
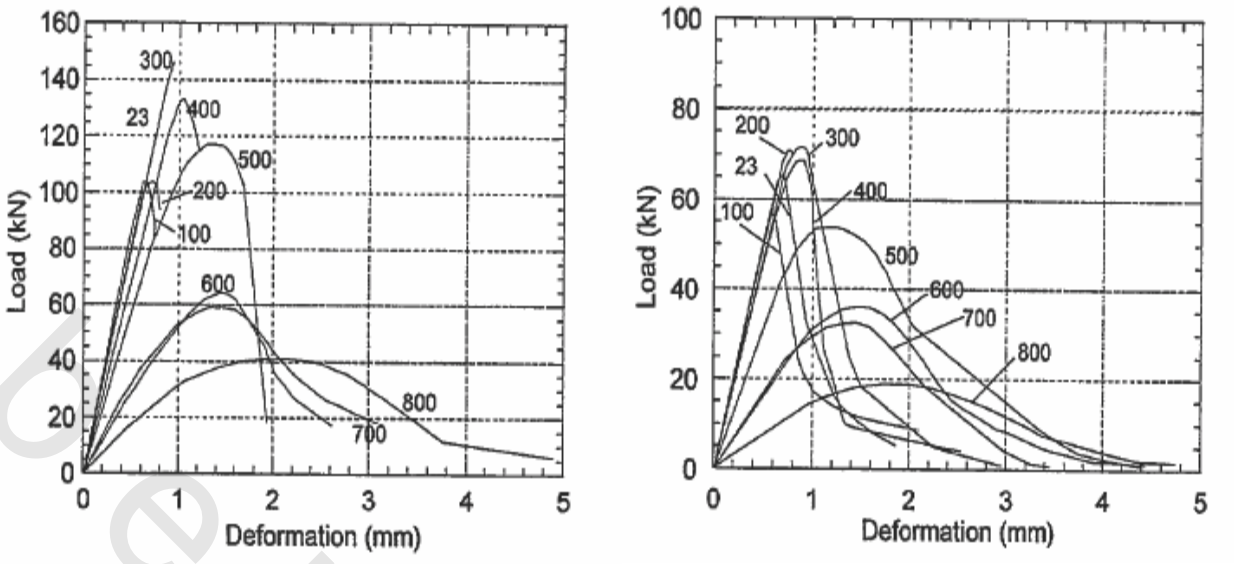


Fig. 2.33-The effect of cooling methods on modulus of elasticity of concrete **[Lee et al. 2008].**

2.7 Stress-strain relations of concrete under high temperatures

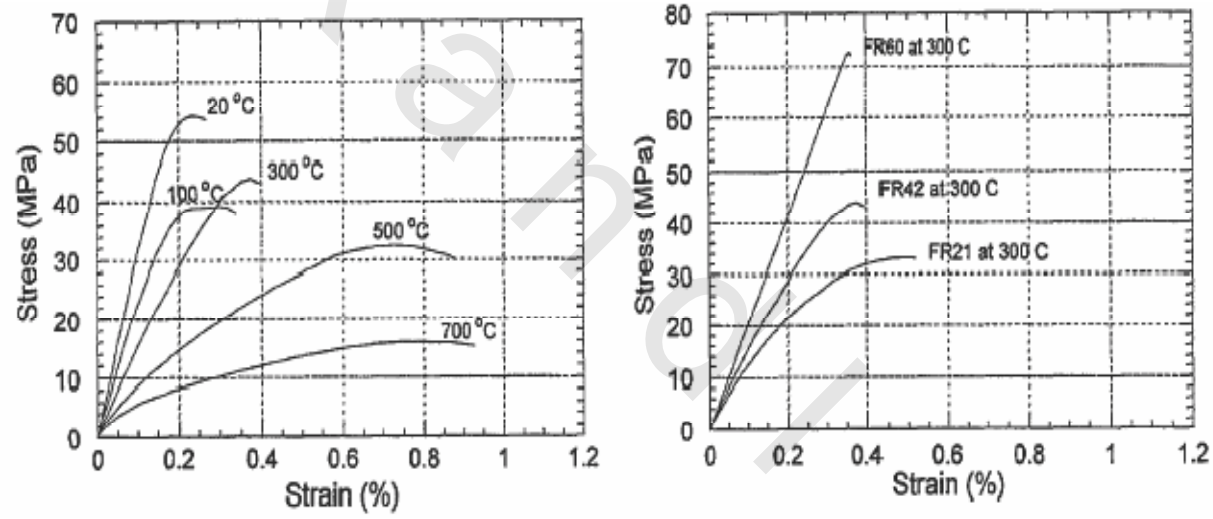
2.7.1 Stress-strain of concrete under high temperatures

The load-deformation plots for HSC and NSC are shown in **Fig. 2.34 (a)** and **(b)[Castillo andDurani, 1990]**. NSC specimens did exhibit ductile failure except for 200°C. Between 300°C and800°C, the NSC specimens were able to undergo large post-peak strains while the decrease instrength was more gradual. HSC showed brittle failure up to 300°C, and with further increasingtemperature, the HSC specimens began to exhibit a more ductile failure.**[Furumura etal., 1995]** obtained stress-strain curves for concretes with different strengths, as shown in **Fig.2.35 (a)** and **(b)**. HSC exhibited steeper slopes than the NSC at temperature up to 300°C to 400°Cin the unstressed test.



(a) Load-deformation of HST (b) Load-deformation of NSC

Fig. 2.34- Hot isothermal test results obtained by **[Castillo and Durani, 1990].**



(a)Stress-strain curves (FR-42)(b) Stress-strain curves at 300°C

Fig. 2.35-Hot isothermal test results obtained by **[Furumura et al. 1995].**

Fig. 2.36shows the effect of cooling methods on stress-strain curve of concrete **[Lee et al.2008]**. Comparing the test data of normal cooling and fast (water) cooling, one can clearly seethat both the peak stress (the strength) and the initial slope (the stiffness) of the curves of fastcooling method are lower than those obtained by normal cooling. Therefore, it can be concludedthat fast cooling has adverse effect on both strength and stiffness of the concrete.

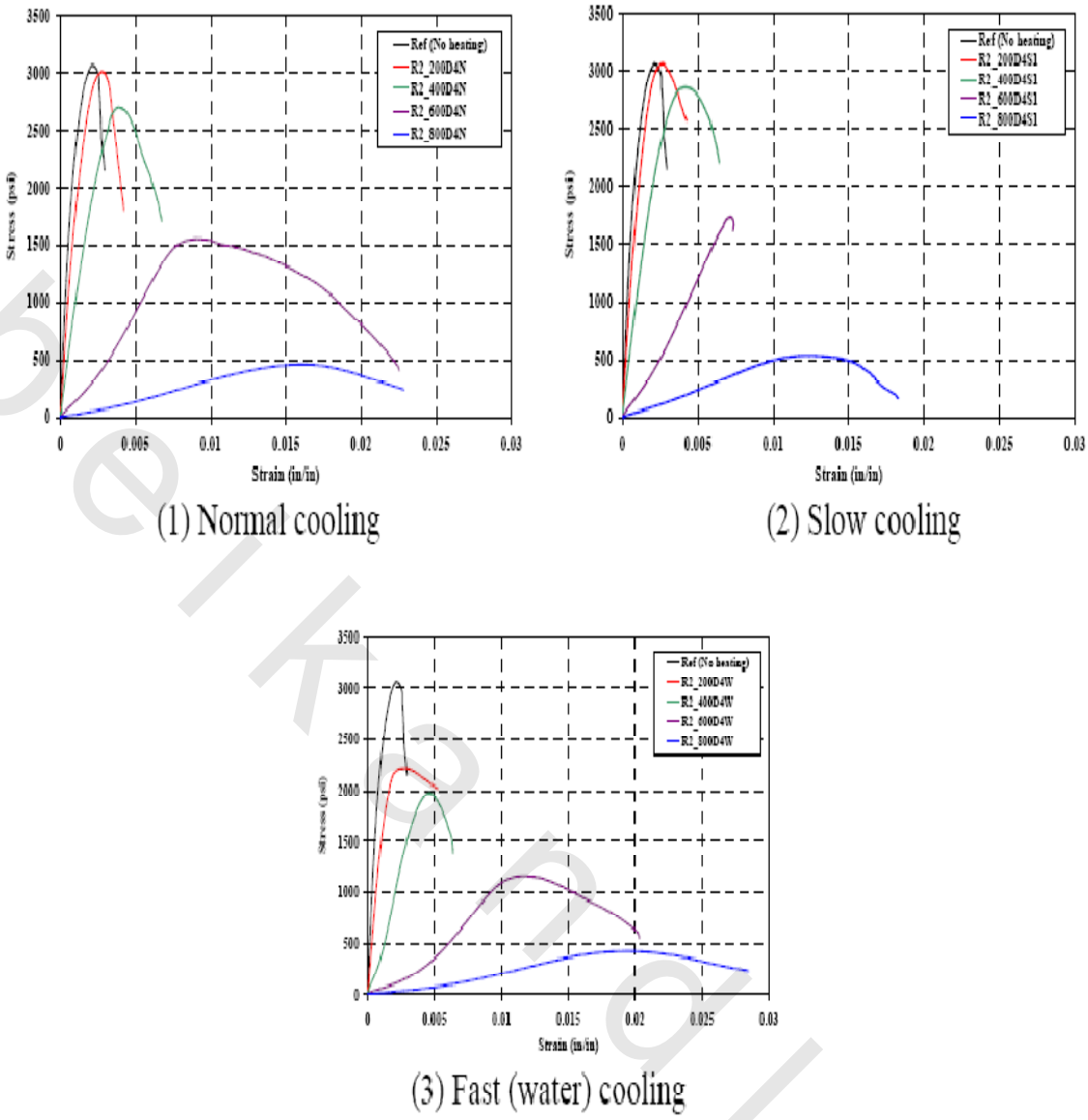


Fig. 2.36-Stress-strain curves of concrete specimens by (1) normal cooling, (2) slow cooling, and (3) fast (water) cooling after heating to 200, 400, 600, and 800 °C **[Lee et al. 2008]**.

2.8 Thermal expansion of concrete under high temperatures.

Free thermal strain of concrete is defined as the strain of concrete induced by heating withoutloading and drying. Free thermal expansion of concrete is usually measured under a saturatedstate to avoid the drying effect. In a typical test for thermal expansion of concrete, a concretespecimen is submerged in a water tank and the change in the length of the specimen is measuredwhile the water temperature rises to a target level (see AASHTO TP60-00). Under this specialcondition, both cement paste and aggregate expand with increasing temperature. However, thisis not a valid testing method when the environmental temperature is higher than 100 °C

sincewater starts to evaporate and cannot be used as the testing media. Furthermore, it is not theactual situation for a real concrete structure under service condition or under an accidentalcondition, where both heating and drying take place at the same time and thus the coupling effectmust be taken into account.

Thermal expansion of concrete is mainly affected by the type and the amount of aggregate. Anaverage value for the coefficient of thermal expansion of concrete is about 10 millionths perdegree Celsius (10 x 10-6/°C or 6 x 10-6/°F), although the value ranges from 6 x 10-6/°C to 13 x10-6/°C. **Table 2.3** shows some experimental results on the coefficient of thermal expansion ofconcretes made with aggregates of various types. These data were obtained from tests on smallconcrete specimens in which all factors were kept the same except aggregate type. In each case,the fine aggregate was made of the same material as the coarse aggregate.

Table 2.3 Coefficient thermal expansion of some types of aggregate.

Aggregate type	Coefficient of thermal expansion millionths per °C
Quartz	11.9
Sandstone	11.7
Gravel	10.8
Granite	9.5
Basalt	8.6
Limestone	6.8

Coefficient of thermal expansion of concrete is temperature dependent and its temperaturedependentbehavior is highly nonlinear. Assuming all other influential parameters remainconstants except temperature, the relationship between thermal strains and temperature is shownin **Fig. 2.37**, which was developed by **[Nielsen, Pearce and Bicanic] (2004).** The thermal strainincreases with temperature up to T = 620°C and beyond which it is assumed to remain aconstant. The coefficient of thermal expansion corresponding to **Fig. 2.37** is a function oftemperature as illustrated in **Fig. 2.38**. One can see from **Fig. 2.38** that under lowtemperature range (e.g. from 20 – 120 °C), the variation of α (coefficient of thermal expansion)is not large, and thus α can be considered as a constant under room temperature.

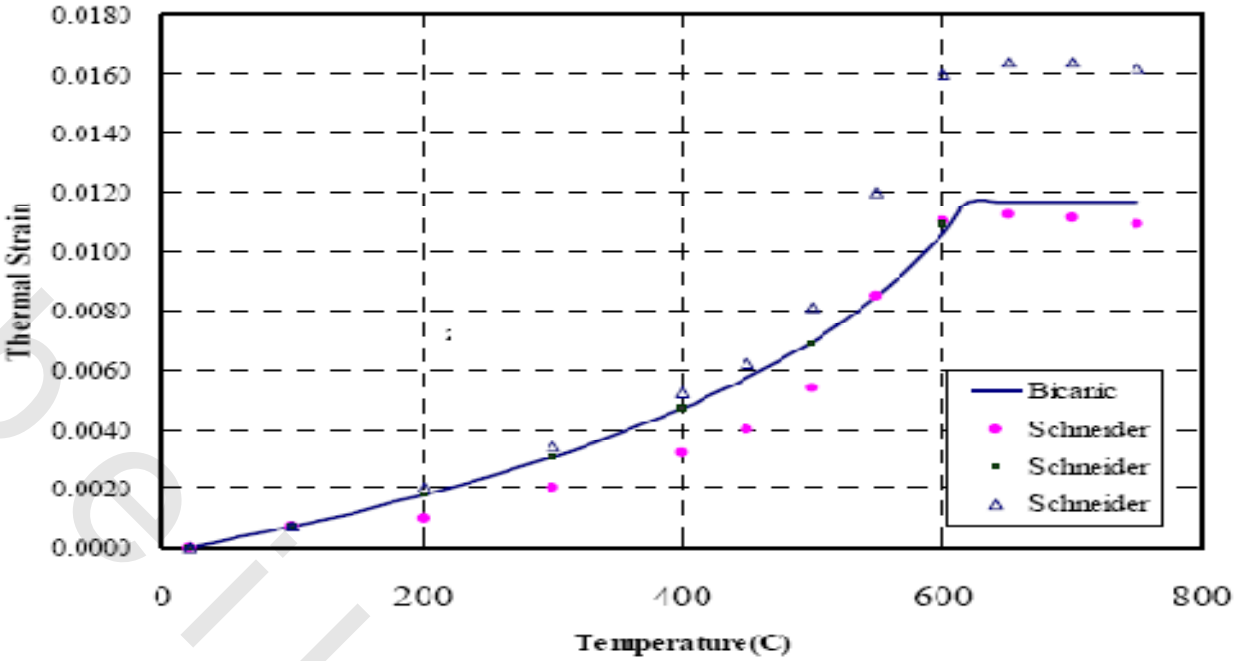


Fig. 2.37-Thermal strain of concrete vs.temperature.

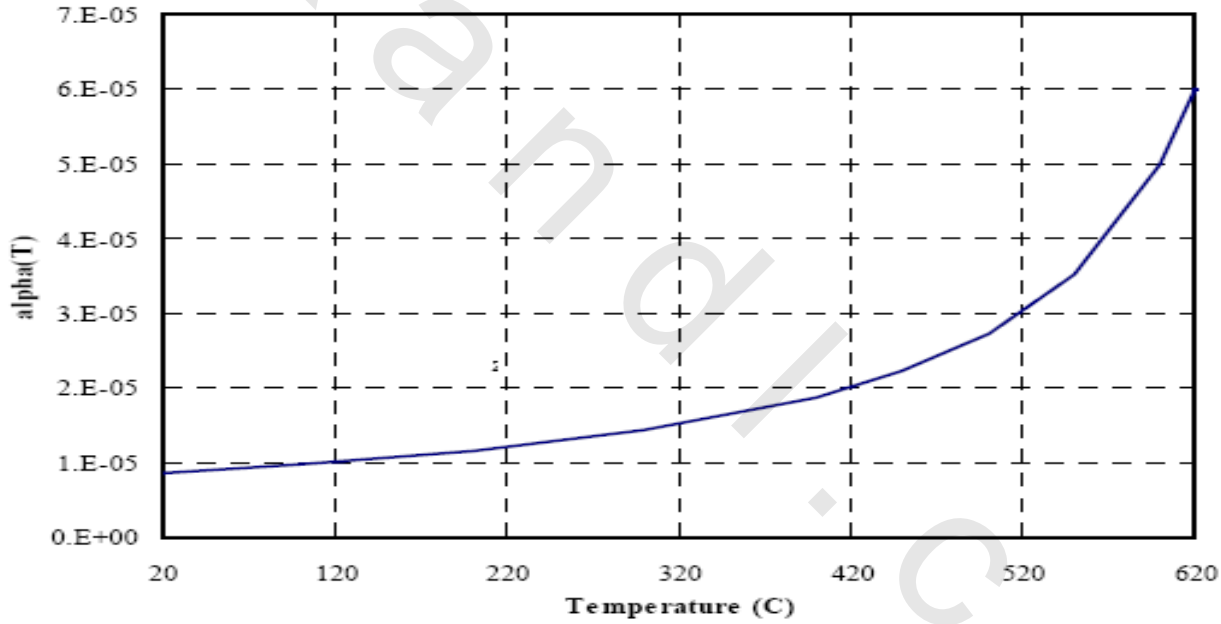


Fig. 2.38-Coefficient of thermalexpansion of concrete vs. temperature.

The expressions below are for the thermal strain and coefficient of thermal expansion shown in**Figs. 2.37** and **2.38**. They consider the temperature dependence of the thermal expansion in anincremental format (thermo-hypo-elasticity):

$$\dot{\varepsilon}_{th} = \alpha(T)\dot{T}$$

$$\varepsilon_{th} = -6\times10^{-3}\ln\left[1-\frac{\theta}{7}\right]$$

$$\alpha=\frac{6\times10^{-5}}{7-\theta} \quad 0\le\theta\le6, \quad \theta=\frac{T-T_0}{100}$$

2.9 Shrinkage of concrete under high temperatures.

Shrinkage of concrete is defined as long-term strain of concrete due to drying without loadingand heating. Shrinkage tests performed under different temperatures will result in differentresults. **Fig. 2.39** shows the test data from **[Schneider, 2002].** One can see that the shrinkageof concrete at 60°C is actually higher than those at 110°C and 140°C.

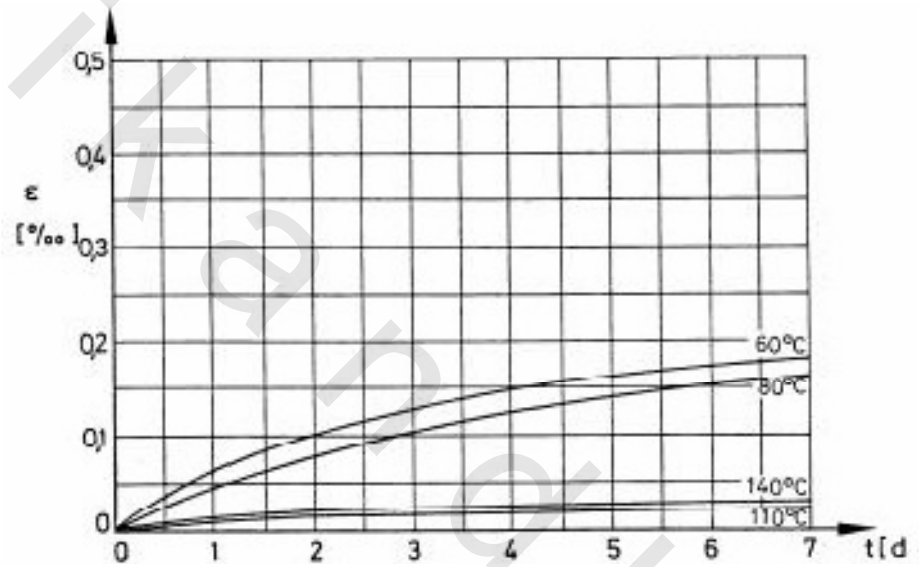


Fig 2.39-Shrinkage of concrete under different temperatures **[Schneider, 2002].**

There are many shrinkage models for concrete under room temperature. Since the shrinkage ofconcrete is induced by moisture loss, therefore, most of the models are related to diffusion theoryof moisture in concrete, which will not be described in this report. For the shrinkage of concreteunder high temperature, however, there has been not much research. Shrinkage of concrete iscoupled with creep and mechanical loading under elevated temperatures, which will be describedin the next section. Some empirical values for shrinkage of concrete were suggested by**[Schneider, 2002].**

- limestone concrete: 20-100 °C; e_s = 200 to 300·10^{-6}
- limestone concrete: ~ 80 °C; e_s = 300 to 400·10^{-6}
- quartzite concrete: 20-100 °C; e_s = 200 to 450·10^{-6}
- limestone concrete: ~150 °C; e_s = 500·10^{-6}
- limestone concrete:150-350 °C; e_s = 800·10^{-6}

2.10Spalling of concrete under high temperatures.

Spalling is the breaking off of pieces of concrete from the surface of the structurewhen it is heated due to thermal stresses or high pore pressures or both. The fire resistance of concrete assumesthat all concrete remains in place duringthe fire event. The beneficial properties ofconcrete in a fire are reduced if concretespalls during elevated temperatures. Thespalling phenomenon is not well understoodbut conventional theory states thatspalling is chiefly caused by the increasein water vapor during elevated temperatures.If the concrete cannot naturally dissipatethe pressure increase due to theincrease in water vapor, the pressureexceeds the tensile strength of the concreteand spalling occurs. It could berelated to a difference in the coefficient ofthermal expansion of the aggregate relativeto the cement paste. In that sense, carbonate aggregates are more compatiblewith cement paste than siliceous aggregate.In some cases, the chance of spallingcan be reduced based on the choice ofaggregate. High strength concrete tends tobe more susceptible to spalling due to itsreduced pore volume and low diffusion ofwater vapor during elevated temperatures.Lightweight concrete isless susceptible to spalling due to itsincreased permeability, which allowswater vapor to expand into voids, thusrelieving internal stresses.The behavior of spalling is very difficult to predict although large amount of researches have beencarried out into it.

[Connoly 1995] presented the categorization suggested by **[Gary, 2010]** that spalling can be grouped as follows:

a) Aggregate spalling, which is defined as a splitting of aggregates at the heatedsurface. This spalling occurs to areas on the heated surface within the first20minutes of exposure to the standard heating test with maximum depth of theirmoved section of about 5 to 10 mm. Aggregatespalling has minor effect on thefire resistance of concrete structures.

b) Corner spalling, which is defined as a gradual disintegration of parts of heatedconcrete at the corner of member such as beams and columns. This type of spallingoccurs after 30 minutes of exposure to the standard test. Corner spalling attributed tothe loss of tensile strength leading to bond failure. Fortunately, corner spalling canusually be repaired easily after fire.

c) Surface spalling, which is defined as the violent removal of sizeable lumps from theheated surface up to 100x100 mm. The loss of sizeable parts of concrete means theeffective area of concrete is reduced and the exposure of the steel-reinforcement tofire causes result in rapid rise of steel temperature. Therefore, the fire resistance ofconcrete structures decreases significantly.

d) Explosive spalling, which is described as a very violent bursting of large parts ofheated concrete. This type of spalling is extremely dangerous and may cause suddenand complete failure accompanied by a large release of energy and produces atypical explosive noise.

Concrete exposed to fire causes spalling of the concrete cover and exposure of thereinforcing steel. This exposure will definitely cause a reduction in the resistancestrength of the reinforcing steel, and thus reduce the ultimate capacity of the structuralelement. Theincorporation of synthetic fibers such as polypropylene fibers which melt atapproximately 160°C during the fire to produce channels where steam can escape isrecommended **[Connoly 1995]**. The permeability of concreteis considered as the main parameter influencing spalling. **Fig. 2.40-a**illustrates the situation in case the permeability is not sufficientto avoid spalling: water vapor, resulting from vaporization ofevaporable water at temperatures above the vaporizationtemperature, i.e., T>TVap (with TVap≥100 °C), flows towardsthe heated surface through the pore system of concretecharacterized by “dry” conditions.2 On the other hand, watervapor flows further into the concrete structure. In regionscharacterized by T≈TVap, the water vapor condenses again.This newly built water together with the evaporable waterpresent in this region forms a saturated layer, acting as an impermeablewall for the gas flow (“moisture clog”, see **Fig. 2.40-c**.If the permeability of the “dry” zone of the concrete member isnot sufficient to avoid a continuous pressure build-up inconsequence of vaporization of evaporable water, spalling asindicated in **Fig. 2.40-d** occurs.

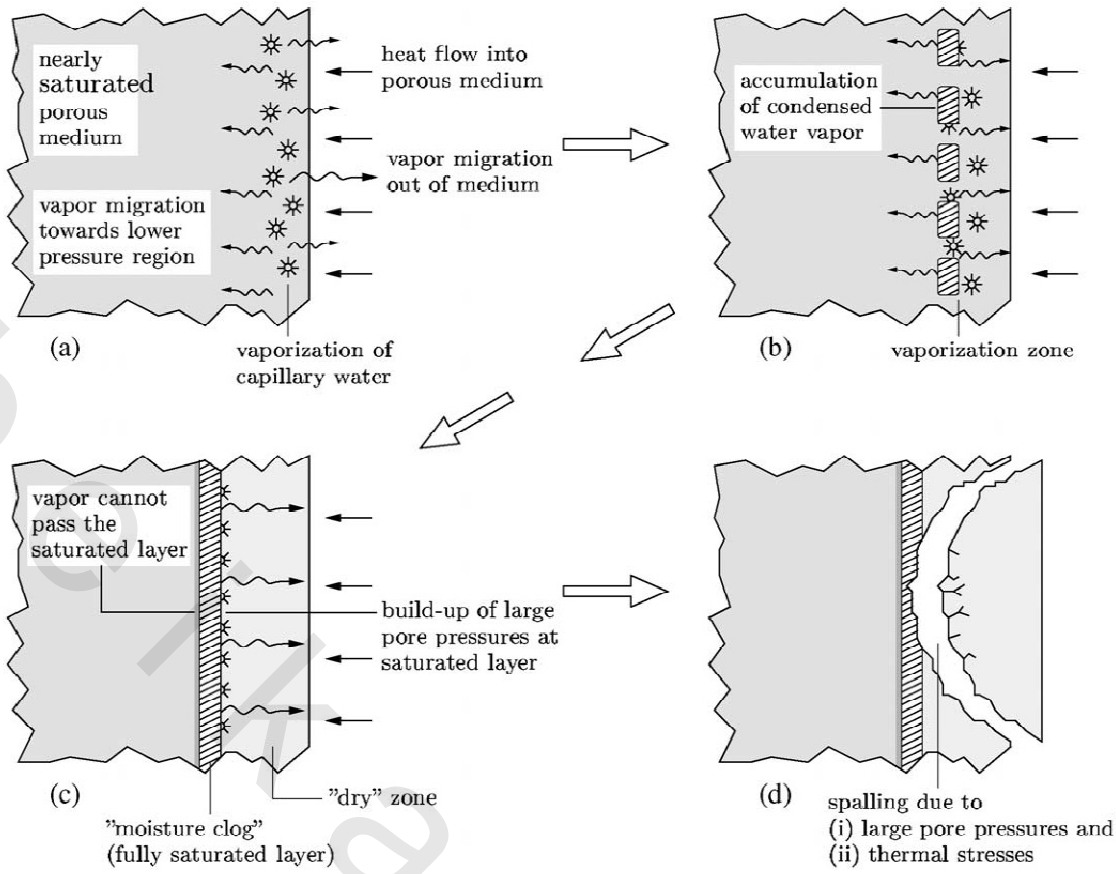


Fig. 2.40- Illustration of mechanism of spalling of concrete as a result of fire loading according to Refs.**[Consolazio 1997 and Schneider, 2002].**

2.11 Temperature Distribution Within the Concrete Members Exposed to a Standard Fire.

ACI-216 provides information on the temperature distribution in a number of concrete shapes during fire exposure. **Fig. 2.41 (a), (b),** and **(c)** show temperatures within concreteslabs during fire tests **[Abrams and Gustaferro 1968].**Slab thickness did not significantly affect the temperaturesexcept for very thin slabs or when the temperatures were lessthan about 400 F (200° C). Temperatures in slabs were obtainedfrom specimens 3 x 3 ft (0.9 x 0.9 m) in plan withprotected edges.

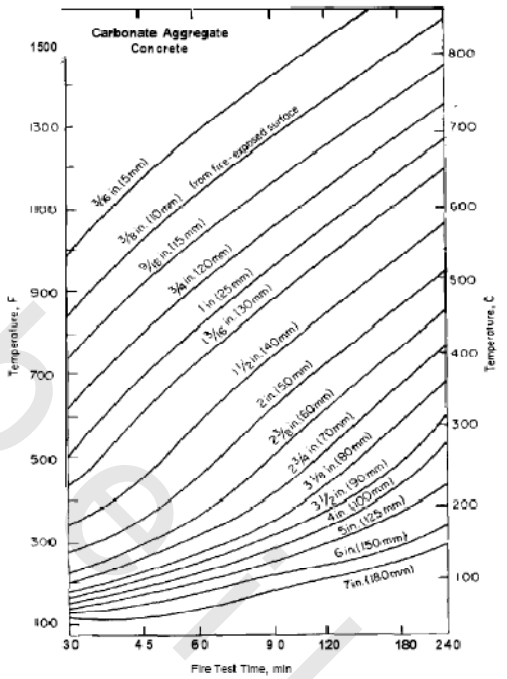


Fig. 2.41(a)-Temperature within slabs during, fire testscarbonate aggregate concrete

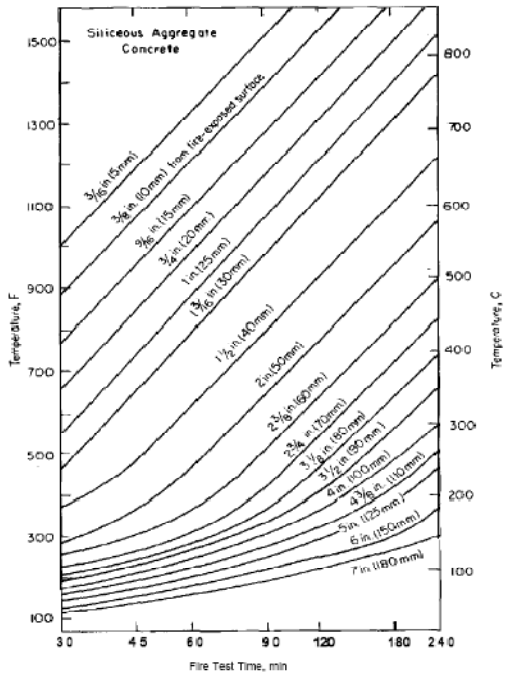


Fig. 2.41(b)-Temperatures within slabs during fire tests siliceous aggregate concrete

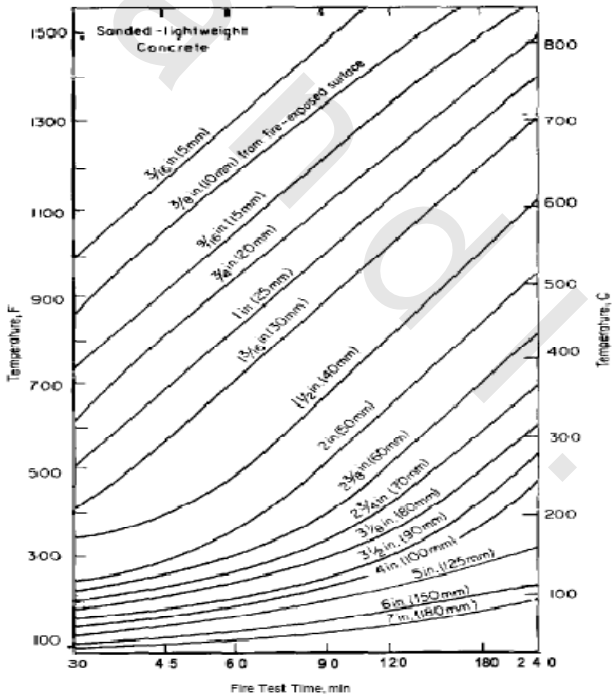


Fig. 2.41(c)-Temperatures within slabs during fire tests sanded lightweight concrete

Also **Fig.2.42 (a, b, c)** show temperature distributions in a 12in. (305 mm) wide rectangular carbonate aggregate concretebeam.Based on a numerical technique developed by **[Lie and Harmathy, 1972]** the temperature distribution in concrete-protectedsteel columns was analyzed, and an empirical formulawas derived for the calculation of the fire endurance of such

columns **[Lie and Harmathy, 1974].** Lie and Allen, in technical paper 378, studied the temperature distribution in solidconcrete columns during fire. Lie and Lin conducted a seriesof 38 fire tests of full-sized reinforced concrete columns inthe period from 1976 to 1986. These latter studies covered reinforcedconcrete beams as well.

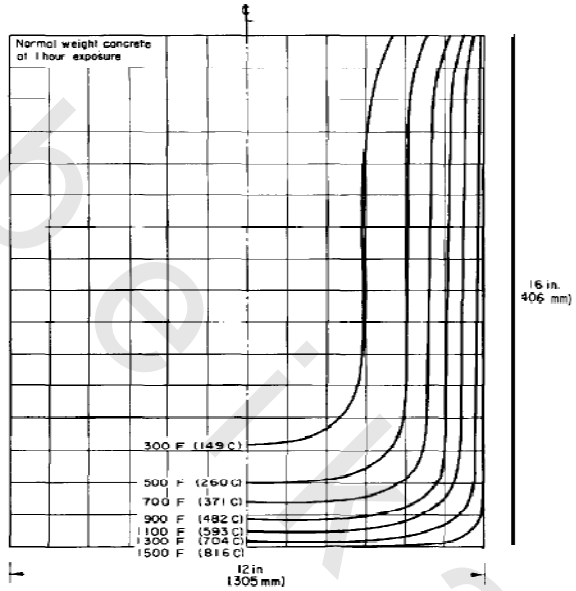


Fig. 2.42 (a) -Temperature distribution in normal weight concrete rectangular unit at 1 hr offire exposure

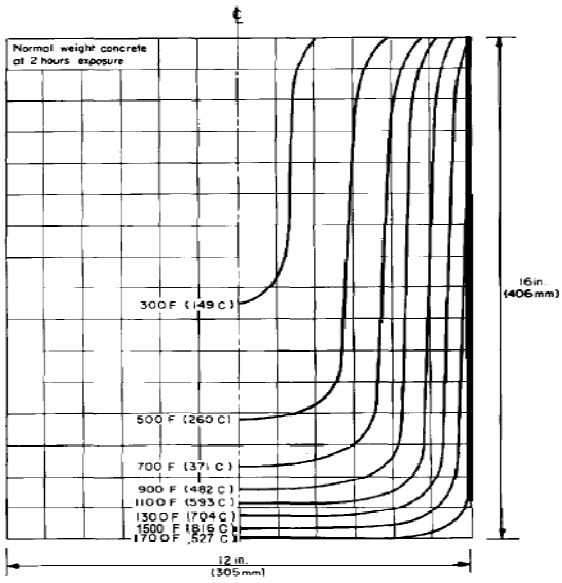


Fig. 2.42 (b)-Temperature distribution in normal weight rectangular unit at 2 hr of fire exposure

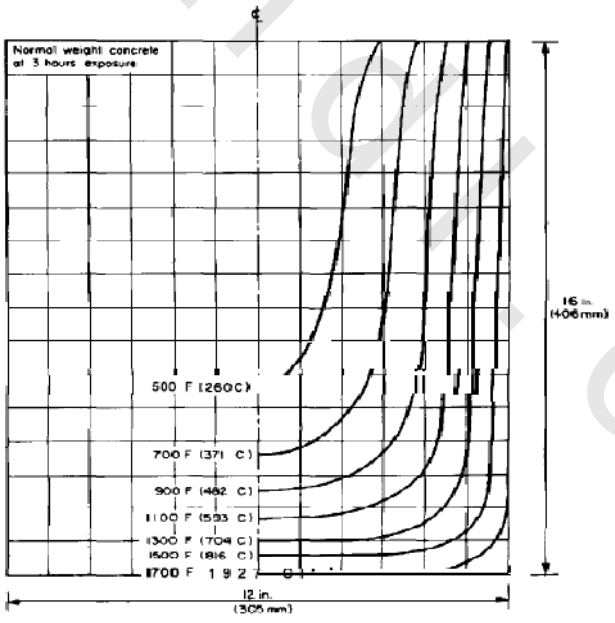
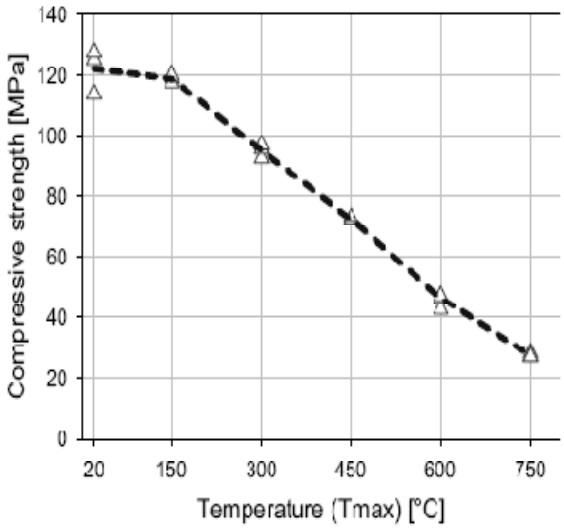


Fig. 2.42 (c)-Temperature distribution in normal weight rectangular unit at 3 hr of fire exposure

2.12 High-Temperature Performance of a HPFRC for Heavy-Duty Road Pavements.

A high-performance concrete reinforced with polymeric fibers (fc = 120 MPa) is being developedfor the construction of heavy-duty road pavements. All mechanical tests were performed in the laboratory of the structural engineering department of milanuniversity of technology.After the thermal cycles, each specimen was tested in uniaxial compression and in three-pointending, to evaluate the compressive strength, the elastic modulus (secant, static) and the indirecttensile strength. The plots of fc and Ec are shown in **Figs. 2.43** and **2.44**. Three tests were carried out for eachreference temperature.A limited number of tests was performed in "thermal-shock" conditions, i.e. by introducing thecylinders into the furnace, previously heated to 600°C, and by keeping them at high temperature for30'. No spalling occurred, but a net of hairlike cracks appeared on the surface of the specimens. Onthe contrary, two cylinders without fibers exploded 5'-10' after being put inside the furnace. Theexplosions broke the cylinders into three pieces, with the cracks localized in the cross sections,because of the combined effects of pore pressure and self-stresses.After being extracted from the furnace, the specimens were left to cool to room temperatureand then were tested in uniaxial compression. The residual strength fc* turned out to be very close tothat of the specimens slowly heated to 600°C (fc* = 47.5 MPa, see **Fig. 2.43**), this fact being totallyunexpected, since the mechanical decay appears to be the same for slowly-heated and suddenlyheatedspecimens. Should it be confirmed by further tests, the conclusion would be that the strengthdecay depends solely on the maximum temperature reached by the material, and not on the heatingrate.



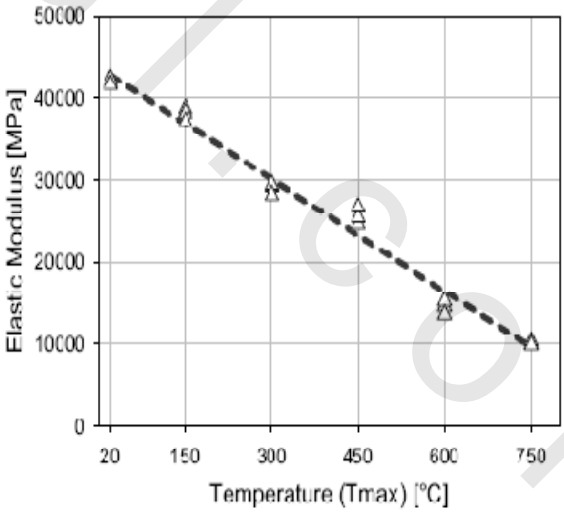
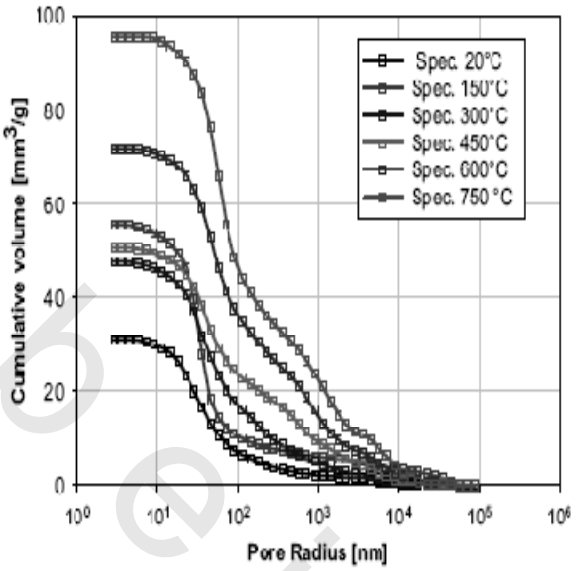


Fig. 2.43 - Residual compressive strength.**Fig. 2.44** - Residual elastic modulus.



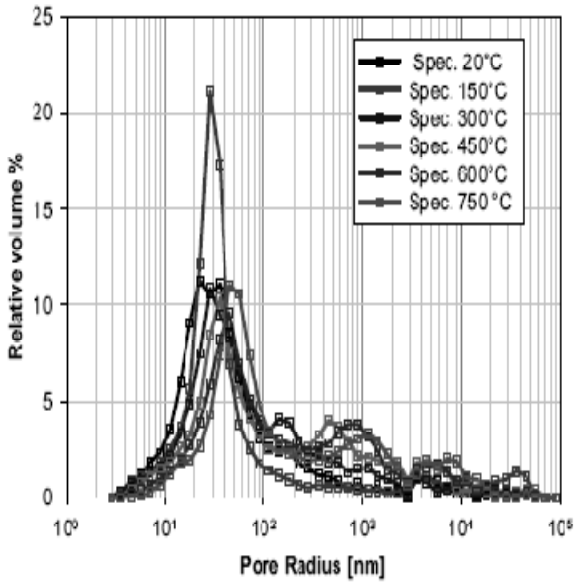


Fig. 2.45- Cumulative distribution of pore size.**Fig. 2.46**- Differential distribution of pore size.

Increasing with the maximum temperature as shown in **Fig. 2.45**, and the peak of the differential distributionwere moved towards larger pore dimensions as the maximum temperature increases as presented in **Fig. 2.46**. The physical state of the polymeric fibers was studied by means of electron microscopy. Above150°C, the fibers disappear, since they melt and vaporize, leaving an evident mark in the form of anoblong cavity **Fig. 2.47 (a, b).**

Scanning electron microscopy was used also to investigate the formation and propagation of themicro-cracks. As shown in **Fig.2.48**, discontinuous micro-cracks appear at 300°C; above thistemperature, the micro-cracks become more numerous and wider (Tmax = 450°C). After the transitionof quartz from theα- to the β-crystalline form (T = 573°C), the micro-cracks exhibit a markedincrease in both number and width, and some micro-cracks start crossing the aggregate particles. Finally, mercury porosimetry, and electron and optical microscopy give useful information onthe damaged microstructure.

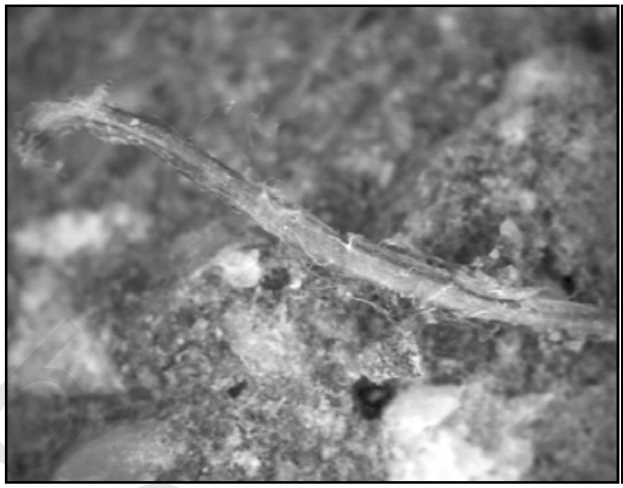


Fig. 2.47 (a)- After heating to 150°C, the fibers are still there (melting temp. ≈150-170°C).of the fibers is left.

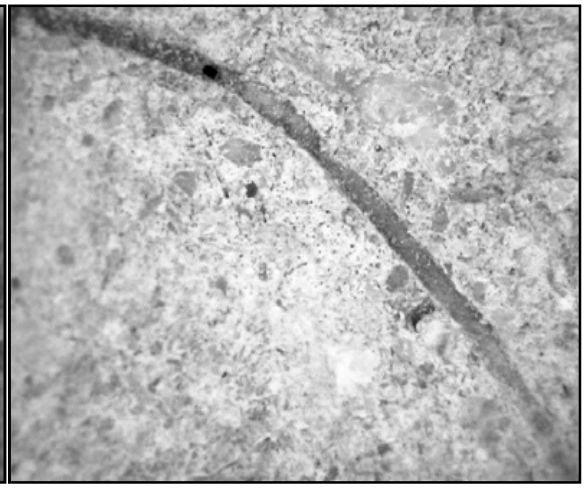
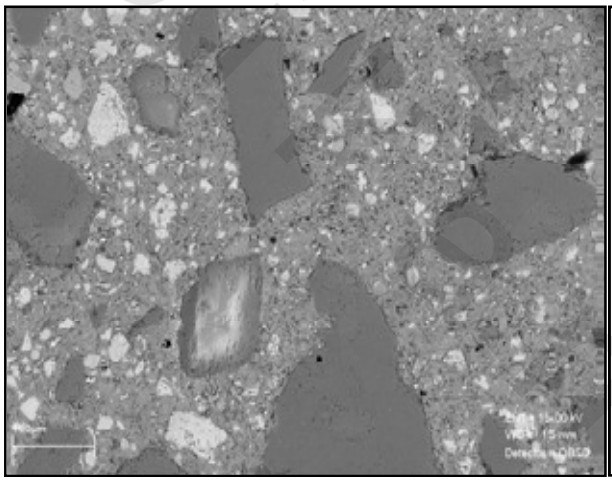
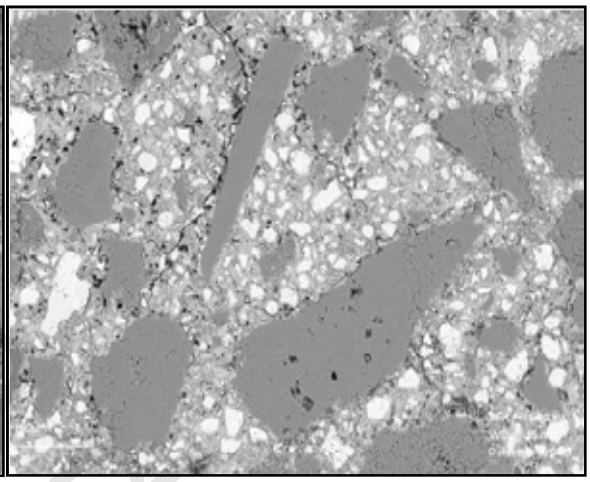


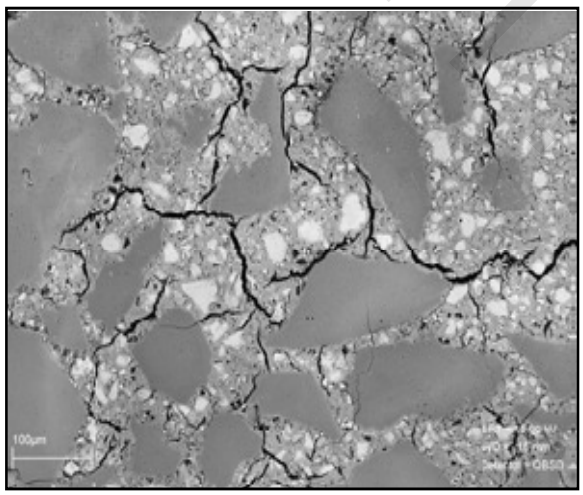
Fig. 2.47 (b) - After heating to 300°C, only the mark



Tmax = 20 °C



Tmax = 300 °C



Tmax = 600 °C

Fig. 2.48 - Evolution of the micro-cracks in the specimens first slowly heated to high temperature and then cooled to room temperature.

2.13Limestone powder.

Limestone, when ground for optimum fineness, can lower the water demand, reducebleeding, improve workability, and increase strength because it improves overall particle gradation of a cement(**Schmidt 1992).** In general the use of 15% to 20% limestone can result inoptimal packing density; however this is less pronounced in cements that are finer **[Schmidt et al.1994]**. When the clinker and limestone are interground, the limestone is normally easier to grindand tends to become the majority of the smaller particles (**Fig. 2.49**), thus broadening theparticle size distribution.When ground separately the limestone needs to be sufficiently fine since if it is toocoarse, an increase in particle spacing may occur along with an increase in the voids between theparticles. This can result in increased water absorption and reduced strength **[Cam and Neithalath2010].** Some researchers have suggested a potential benefit of intergrinding on thecarboaluminate reaction, for example to reduce porosity slightly **[Matschei et al. 2007a]** andthereby improve durability.

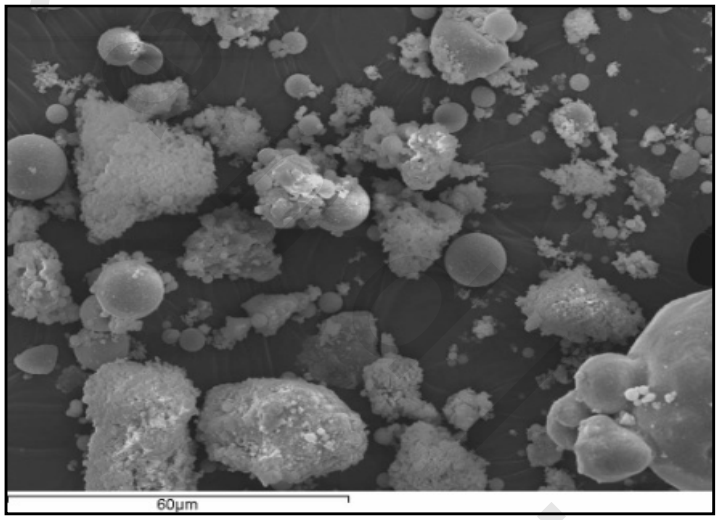


Fig. 2.49 Raw materials of limestone powder

2.14 Bentonite.

Bentonite is essentially highly plastic claycontaining not less than 85% clay mineral,montmorillonite. A great commercial importanceis attached with bentonitewhich possesses inherentbleaching properties like fuller's earth. Hence, it isknown as bleaching clay. There are two types ofbentonites; namely, swelling-type or sodium bentoniteand non-swelling-type or calcium bentonite. Sodiumbentonite is usually referred to as bentonite, whereascalcium bentonite is called Fuller's earth. The commercialimportance of bentonite depends more on its physicochemicalproperties rather than its chemical composition.Excellent plasticity and

lubricity, high dry-bondingstrength, high shear and compressive strength, lowperpermeability and low compressibility make bentonitecommercially viable.

Bentonite range in colour from black through to white but most frequently are bluish-green when fresh, weathering to a yellowish-brown colour at or near outcrop due to the oxidation of ferrous iron.

The composition of some bentonitemong the most contents is montmorillonite. Montmorrilonit is a mineral that contains compounds $Al_2O_34Si.H_2O$ **[Wijaya and J. D. S. Rohman, 2008]**, other minerals contained in the bentonite is Mg and Ca occasionally. Bentonite lattice structures composed of a single plate located between two Al_2O_3 SiO_2 plates. Since the structure is montmorillonite can expand and contract, and have thepower of water and caution adsorption higher.

2.15 How PP fibres inhibit explosive spalling

The addition of suitable polypropylene monofilament micro-fibersas shown in**Fig 2.50** to counteract explosive spalling in cast concrete **[Ali et al, 1996]** and in shotcrete has been accepted for many years, but to design an optimized micro-fiber to prevent explosive spalling, it is necessary to have an understanding of the detailed mechanism by which these fibers function. Since the spalling is caused by pressure created by a restriction on the movement of moisture or steam, then somehow the presence of the fibersmust relieve that pressure.

Fig 2.50-Polypropylene fibers (PPF)

As the temperature in the micro-fiber reinforced concrete rises the PP softens and begins to melt due to a progressive change of phase which starts at approximately 150°C when the crystallinity begins to break down into an amorphous polymer. It peaks at 165°C (the commonly quoted melting point), and is complete at approximately 175°C. It is this melting that is believed to facilitate the reduction in the internal stresses in the concrete that cause the explosive spalling. There are two main theories as to how the micro-fibers do this.

A)Mechanisms.

While recognizing the possibility of other mechanisms, **[Khoury, 2008],** advocates what he terms a PITS (Pressure Induced Tangential Space) theory in which the steam overrides the expansion of the PP as it melts, to squeeze between the micro-fibersand the concrete matrix and pass along the length of the fiber. He claims that the effectiveness of such a mechanism would be dependent upon the cumulative surface area of the micro-fiber and connectivity of the fibers, and is therefore favored by an ultrafine fiber with a diameter of around 18µm that provides a very high number of fibers. Since micro-fibers are dispersed throughout the concrete it is not clear how the connectivity of the fibers is created and how the steam pressure is alleviated. This theory also cannot explain why 32µm diameter micro-fibers - that provide only one third the number of fibers compared to 18µm diameter - have been proven to provide comparable and possibly slightly superior explosive spalling resistance **[Jansson et al, 2008].**

B) Microcracking mechanism.

An alternative theory presented by Sullivan (2001) contends that as an individual PP fiber melts, its much higher coefficient of thermal expansion compared to that of concrete (8.5x) creates a large number of micro-cracks. These newly created micro-cracks can then link with the micro-cracks created by the thermal expansion of neighboring micro-fibers, or from thermally induced stresses, to form an interconnecting network that can facilitate the movement of steam through the concrete. It is this permeability, which is created, most importantly, only should a fire event occur, that relieves the stresses created by the steam generation and counteracts the possibility of explosive spalling. **[Liu et al, 2008]** found from backscattering electron microscopy (BSE) and gas permeability testing that the melting of the PP fibers increased the connectivity of the isolated pores leading to an increase in permeability, with peak permeability occurring at approximately 200°C or soon after the melting point of the polypropylene. It was concluded that the creation of micro-cracks and their connectivity into a network as seen in **Fig 2.51** are major factors in determining the permeability of concrete upon exposure to high temperatures.

Fig 2.51-Microcracking network

From preliminary numerical simulation studies**, [Saka et al ,2009]** reports that a single PP fibersembedded in a mortar matrix and subjected to a temperature increase of 140°C creates a significant stress on the matrix because of the difference in the coefficients of thermal expansion. **[Khoury,2008]** also indicates a significant tensile stress is exerted on the surrounding matrix by the large difference in thermal expansion between concrete and PP polymer leading to the creation of micro-cracks.Micro-cracks are also created in concrete by thermal effects such as aggregate expansion, drying shrinkage and steam generation. However, it is the supplementary creation of micro-cracks provided by the melting of the PP micro-fibers that operates in a serial/parallel system with these pores and interfacial transition zones **[Kalifa et al, 2001]** that provides the superior level of protection of the concrete against explosive spalling. The larger the mass of the individual fiber, the higher will be the stress that the molten polymer can create and the increased tendency for micro-cracks to be formed as a result of this stress. Too large an individual fiberhowever, leads to a lower number of fibersdistributed throughout the concrete which reduces the network formation possibilities. Equally, at the other extreme, too small an individual fibersreduces the tendency for micro-cracks to be formed, restricting the network that can be created, which reduces the overall ability of the fibersto prevent explosive spalling**[Bostrom and Jansson, 2007]**. The optimum size of the fiber for the most efficient explosive spalling resistance lies between these two extremes.

2.16Scanning electron microscope.

The scanning electron microscope (SEM) is a powerful tool for imaging and chemicalanalysis in cement research. With a high resolution (down to 1nm) and a large depth of focus,it enables a detailed study of the rough surfaces of e.g. the formed calcium silicate hydrate(C-S-H). This is difficult to study using an optical microscope.The SEM is a microscope that uses electrons instead of light to form an image and it is usedunder vacuum conditions. A beam of electrons is generated by placing a high electricpotential difference between a tip and a plate with an aperture. The potential can range from afew hundred volts to several tens of kilovolts. The electron beam travels through themicroscope, where a series of magnetic lenses and apertures focus the electron beam. As theelectron beam hits the sample, it will scan the surface in a raster scan pattern, hence the nameof the technique.When the focused electron beam hits the sample, the beam electrons will interact with theatoms at or near the surface of the sample and a variety of signals is generated. There aremainly three signals measured in the SEM: secondary (SE) and backscattered (BSE) electrons(for imaging) and characteristic X-rays (for chemical analysis). They originate from differentdepths and volumes in the sample and provide different information about the sample. TheSEM images in this work are all taken in secondary electron detection mode.Using secondary electrons for imaging will show morphology and surface topography of asample. The secondary electrons originate from the outermost part of the surface and aregenerated when the beam electrons lose energy by inelastic collisions in the sample andknocks out secondary electrons from the atoms. It is primarily the incident beam electronsthat knock out the secondary electrons, although backscattered electrons can also contribute.

The energy of secondary electrons is typically 50 eV or less. It is only the secondary electronsgenerated close to the sample surface that will have sufficient energy to leave the sample.The emitted secondary electrons are attracted to the detector by a positive electric potentialand induce a signal. This results in a topological view of the surface that resembles a normalphotography.

2.17Repairtechniques.

Any concrete exposed to temperature more than 300° C should be removed and replaced. Below this temperature, concrete can be repaired by increasing the overall dimensions to take the design load. Based on the results of a thorough and well-planned damage assessment of a fire-damaged concrete structure, an appropriate repair technique can be selected from a range of

techniques including manually applied patching, pneumatically-applied patching, and composite strengthening. In cases where a fire has caused severe distortion to the structure and produced distress in both the concrete and the reinforcing steel, demolition and reconstruction may be required.

2.17.1 Manually applied patching.

Manually applied patching is the simplest, but is typically limited to small areas of repair where localized spall damage has occurred. Repair products include polymer and silica fume modified cementitious mortars and epoxies. These products typically provide high compressive strength and early strength gain for a fast repair cycle and are formulated for horizontal, vertical, and overhead applications. This type of application typically does not generate much noise and can be performed during normal business hours; however, surface preparation operations can be loud and will often have to be limited to off-peak hours if the structure remains occupied.

2.17.2 Pneumatically applied patching

When larger spalled areas must be repaired, pneumatic application of repair mortar (shotcreting) may be more cost effective because larger areas can be covered in less time than with manual applications. This process is loud and messy, and the repair is preferably performed when the building is not occupied.

2.17.3 Composite strengthening

When time is a critical factor or when space is limited, composite materials such as carbon fiber-reinforced polymer (CFRP) or glass fiber-reinforced polymer (GFRP) may be justified repair options. The composite materials typically come in pliable sheets that can conform to their substrate. They are also available in prefabricated solid strips that typically attach to the bottom of beams and slabs depending on the type of repair necessary and the structural element to be repaired. Because epoxies are used to provide adhesion to the substrate of these composite materials, additional fireproofing materials must be installed to protect the repair materials. This repair approach doesn't generate much noise after the surface has been prepared. In spite of the cost, this option becomes viable when the facility must remain open 24 hours a day and when noise is an issue.